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Evaluation of Alternative Shaft Sinking Techniques for High-Level Nuclear Waste (HLW) Deep Geologic Repositories

Final Report (Task 3)
June 1981 - July 1982

Prepared by L. Gonano, D. Findley, W. Wildanger,
R. Gates, S. Phillips

Golder Associates

Prepared for
U.S. Nuclear Regulatory
Commission

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ABSTRACT

This report represents the results of Task 3 of U.S. Nuclear Regulatory Commission (NRC) Contract NRC-02-81-037, "Technical Assistance for Repository Design." The purpose of the complete project is to provide NRC with technical assistance for the following reasons:

- To enable the focused, adequate review by NRC of aspects related to design and construction of an in situ test facility and final geologic repository, as presented in U.S. Department of Energy (DOE) Site Characterization Reports (SCR)

- To ascertain that the DOE site characterization program will provide, as far as possible, all the information necessary to permit a review to be conducted by NRC of a license application for construction authorization.

It is assumed that the Site Characterization Report and License Application will describe the exploratory shaft and concept designs for the repository shafts. This report provides a comparative evaluation of various shaft sinking techniques for production shafts for a repository. The primary comparative evaluation has been conducted for 14 ft internal diameter shafts developed in two composite media using four different methods of sinking/lining. The technical, cost and schedule comparisons draw a major distinction between shafts sunk blind and those which utilize bottom access. Based on the system of ranking introduced to grade the significant attributes of each method and the resulting design, it is concluded that for application to repository access, no one particular method of sinking exhibits a clear overall superiority.

When a specific site is made available for a study of the most suitable shaft sinking methods, it will be necessary to establish actual geological conditions and technological capabilities and the comparisons presented herein reviewed accordingly.

EXECUTIVE SUMMARY

Introduction

This report represents the results of Task 3 of U.S. Nuclear Regulatory Commission (NRC) Contract NRC-02-81-037, "Technical Assistance for Repository Design."

The purpose of the complete project is to provide NRC with technical assistance for the following reasons:

- To enable the focused, adequate review by NRC of aspects related to design and construction of an in situ test facility and final geologic repository, as presented in U.S. Department of Energy (DOE) Site Characterization Reports (SCR)

- To ascertain that the DOE site characterization program will provide, as far as possible, all the information necessary to permit a review to be conducted by NRC of a license application for construction authorization.

It is assumed that the Site Characterization Report will describe the design of the exploratory shaft and the concept designs of the repository shafts. Further, it is assumed that the License Application will describe the results of the exploratory shaft and refine the concept designs for the repository shafts.

Scope of Report

The objective of this report is to present a comparative evaluation of the various available shaft sinking techniques within the context of the particular short- and long-term engineering performance requirements of repository access structures. This report concentrates on production shafts for the repository, not on exploratory shafts or associated vent shafts.

The design and construction of the shafts for repository access are subject to two major constraints:

- The artificially induced potential for increased radionuclide migration during operation or particularly after decommissioning of the shaft as a result of disturbances to the rock or groundwater regime during construction, operation or backfilling of the shafts

- The paramount importance of the shaft sinking schedule on the development time for repository commissioning.

The first constraint encompasses such matters as the effect of different construction techniques on rock disturbance, impact on groundwater condition, the application of grouting, freezing and sealing techniques and the relationship of the construction method to retrievability of nuclear waste. The second constraint is concerned with the lead time for licensing, the ease of acquisition of geological and geotechnical data during construction and other factors such as construction time, cost, safety, and reliability and predictability of construction.

Study Approach

With this perspective, a comparative evaluation of various shaft sinking techniques has been conducted with specific reference to the particular requirements of repository shafts as opposed to shafts with different functional roles in civil and mining engineering applications. Both technical and nontechnical aspects of shaft sinking are compared.

Two sets of assumptions namely, the geological conditions and the technological basis for each method of construction have an important bearing on the outcome of any comparisons. So as to make these comparisons more definitive and quantitative, the approach adopted has been to base the study on a limited but typical geological package and selected optimum available construction techniques. The resulting comparisons and conclusions are regarded, in consequence, to be more meaningful and more directly applicable to the project objectives than a broader general review of the discipline of shaft sinking technology.

"Composite media" are defined to reflect two general situations: "Hard rock" which incorporates the sites and media proposed by DOE to include basalt, tuff, and granite; and "salt" which incorporates the sites and media proposed by DOE to include bedded salt and domal salt.

Following an introduction and study activities (Sections 1 and 2) the report defines shaft boundary conditions for repository applications (Section 3), to include circular vertical shafts with concrete linings with an inside diameter of 14 ft. The depth for hard rock is assumed to be 4000 ft. and for salt to be 3000 ft. Other repository unique aspects are also covered. Section 4 then describes shaft sinking methods and categorizes them as drill-and-blast, blind rotary, ream-and-slash, and large-diameter raising. Section 5 describes representative geology/geohydrology for all five media proposed by DOE and the two "composite media." Section 6 provides detailed shaft designs for the methods of Section 4 in the geology of Section 5. Hydrostatic linings to restrict groundwater inflows to 100 gpm or less are adopted for all designs. Shaft construction techniques adopted are restricted to those technically feasible but not necessarily proven in 1982.

Section 7 provides the evaluation of shaft sinking methods for repository applications. Detailed cost and schedule comparison for each method considered feasible in the two composite media are presented. In addition, a comprehensive evaluation of the technical pro's and con's for each method is addressed.

Evaluation

Some significant differences in technical and nontechnical characteristics between the various sinking methods have been identified; namely:

- For the stipulated shaft requirements, drill-and-blast, ream-and-slash, and rotary drilling using top drive equipment are considered to be technically feasible. Back-reaming is regarded as marginally feasible while blind boring using in-hole equipment is discounted as currently not viable.

- Of the blind sinking methods, rotary drilling offers distinct advantages in terms of safety, minimum construction duration and least damage to the rock and groundwater regime. It is considerably more expensive than all other methods. However, this disadvantage is often more than compensated for by the considerable savings in capitalized costs for the repository project which accrue from the attendant short construction time. Blind rotary drilling is particularly favorable for the construction of shafts in the size range less than 18 ft diameter and located in unfavorable geological conditions.

- The ream-and-slash and drill-and-blast methods of construction both impart considerable damage to the shaft walls, are unfavorable from the safety aspect and generally involve long construction times. Their main advantage is low direct cost. Because of the torque limitations on rotary sinking methods and the subsequent impact on advance rates, both ream-and-slash and drill-and-blast methods are generally more advantageous for the deeper larger shafts in good geological conditions. In fact, the ream-and-slash method is very competitive in terms of both cost and schedule if good ground conditions exist.

- Back-reaming methods of shaft construction are only marginally feasible at the upper range of shaft geometries being considered here. Although raise-drilling technology from which it derives is well developed, there are definite inherent mechanical limits to the scale of operations. When geological conditions are good and for the smaller range of shaft diameters, back-reaming combines the safety, schedule and minimum disturbance advantages of mechanical construction methods with the cost advantage of drill-and-blast sinking.

Conclusions

The report concludes in Section 8 that with the general assumptions outlined above that there is no clear "winner" in shaft sinking techniques and that bottom access for other than the first shaft is not necessarily a significant advantage.

It is further concluded that for the application of the four feasible methods of shaft construction to the repository determined conditions cited earlier, schedule and cost variations are relatively insensitive to shaft diameter. In short, shaft diameter per se does not affect the choice of sinking method within the range of conditions considered.

Finally, the shaft designs presented in this evaluation illustrate the scope of a shaft sinking operation and the level of design considered appropriate for vertical repository access facilities. These designs represent the practical interpretations of the currently envisaged design conditions and criteria for the construction of shafts for high level nuclear waste deep geologic repositories.

When a specific site is made available for a study of the most suitable shaft sinking methods, it will be necessary to establish actual geological conditions and technological capabilities and the comparisons presented herein reviewed accordingly. Recommendations for extension and application of the study is given in Section 8.

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NUREG/CR-2959	3/83	4	Relationship of an In Situ Test Facility to a Deep Geologic Repository for High-Level Nuclear Waste

PLANNED REPORT

Report Number	Issuance Date	Contract Task	Title
		5	Evaluation of Engineering Aspects of Backfill Placement for High Level Nuclear Waste (HLW) Deep Geologic Repositories

ABBREVIATIONS AND SYMBOLS

The following symbols, abbreviations and units have been adopted in this report. As a general rule, the foot-pound-second system is used except where other derived units are widely used and more readily recognized. Conversion factors with other units in common use are also supplied.

PARAMETER	UNIT	ABBREVIATION	CONVERSION
Shaft			
Depth, Diameter	feet	ft	=0.3048 meters
Lining Thickness	inches	in.	=25.4 millimeters
Hoisting Speed	feet per minute	ft/min	=0.00508 meters per second
Drilling/Sinking Rate	feet per hour	ft/hr	=0.3048 meters per hour
Drilling/Sinking Rate	feet per week	ft/wk	=0.3048 meters per week
Equipment Weight	pounds	lbs	=0.4536 kilograms
Water Inflow	gallons per minute	gpm	=0.063 liters per second
Geology			
Drill Hole Depth	feet	ft	=0.3048 meters
Regional Area	square miles	sq miles	=2.59 square kilometers
Bed Thickness, Joint Length	feet	ft	=0.3048 meters
Joint thickness	inches	in.	=25.4 mm
Bedding Dip	degrees	°	------
Joint Orientation	degrees	°	------
Geohydrology			
Hydraulic Conductivity	centimeters per second	cm/sec	=2834 feet per day
Hydraulic Permeability	centimeters per second	cm/sec	=2834 feet per day
Hydraulic gradient	dimensionless	unit	------
Porosity	dimensionless	%	------
Specific Storage	per foot	ft-1	=3.28/meter

ABBREVIATIONS AND SYMBOLS

Geoengineering			
Density	grams per cubic centimeter	g/cc	=64.42 pounds per cubic foot
Young's Modulus	Kips per square inch	ksi	=6.895x10-3 Giga Pascals
Strength (Tensile, Cohesive, Unconfined Compressive)	pounds per square inch	psi	=6.895x10-3 Mega Pascals
Angle of Friction	degrees	°	------
In Situ Stress	pounds per square inch	psi	=6.895x10-3 Mega Pascals
Poisson's Ratio	dimensionless	unit	------

1. INTRODUCTION

This report represents the results of Task 3 of U.S. Nuclear Regulatory Commission (NRC) Contract NRC-02-81-037, "Technical Assistance for Repository Design."

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- To ascertain that the DOE site characterization program will provide, as far as possible, all the information necessary to permit a review to be conducted by NRC of a license application for construction authorization.

It is assumed that the Site Characterization Report will describe the design of the exploratory shaft and the concept designs of the repository shafts. Further, it is assumed that the License Application will describe the results of the exploratory shaft and refine the conceptual designs for the repository shafts. This report concentrates on shaft sinking methods for repository production shafts. While it is anticipated that exploratory shafts and associated vent shafts would be designed and constructed on much the same technical bases as for production shafts, those design considerations of specific relevance to exploratory shafts are not addressed in this report.

The design and construction of the shafts for repository access are subject to two major constraints:

- The artificially induced potential for increased radionuclide migration during operation or particularly after decommissioning of the shaft as a result of disturbances to the rock or groundwater regime during construction, operation or backfilling of the shafts.

- The paramount importance of the shaft sinking schedule to the development time for repository licensing.

The first constraint encompasses such matters as the effect of different construction techniques on rock disturbance, impact on groundwater condition, the application of grouting, freezing and sealing techniques and the relationship of the construction method to retrievability of nuclear waste. The second constraint is concerned with the lead time for licensing, the ease of acquisition of geological and geotechnical data during construction and other factors such as construction time, cost and safety.

With this perspective, a comparative evaluation of various shaft sinking techniques has been conducted with specific reference to the particular requirements of repository shafts as opposed to shafts with different functional roles in civil and mining engineering applications. Both technical and nontechnical aspects of shaft sinking are reviewed.

Two sets of assumptions namely, the geological conditions and the technological basis for each method of construction have an important bearing on the outcome of any comparisons. So as to make these comparisons more definitive and quantitative, specific and narrow assumptions have been established. The approach therefore has been to base the study on a limited but typical geological package and selected optimum available construction techniques. The resulting comparisons and conclusions are regarded, in consequence, to be more meaningful and more directly applicable to the project objectives than a more general broader review of the discipline of shaft sinking.

These assumptions which form the basis of the evaluation necessarily impose limitations on the applicability of the results. Thus, when a specific site is made available for a study of the most suitable shaft sinking methods, it will be necessary to establish actual geological conditions and technological capabilities and the comparisons presented herein reviewed accordingly.

2. STUDY ACTIVITIES

The study activities that make up this report are designed to meet the scope of work of Task 3 that evolved early in the study to meet the technical requirements of NRC. The report is a definitive statement on the subject of shaft sinking for repositories. The objective of this report is to help the NRC in their review of Site Characterization Reports and License Applications. The interrelationship of the various study activities designed to achieve this objective are described below and outlined in Figure 2-1. Basic assumptions and major constraints are established here for some of the sections to follow.

The boundary conditions and constraints on shaft construction are defined in Section 3. The possible exclusion on technical grounds, of certain promising trends in the development of technical variations and extension of existing shaft sinking methods from possible repository applications is considered too limiting and inappropriate. Thus, it is assumed that the shaft sinking technique does not have to be proven, although it is acknowledged that where a technological extrapolation has taken place, some debugging of the technique at the specific site may be required.

It is customary and desirable to conduct geological and geotechnical investigations during shaft sinking for the purposes of finalizing shaft design. Such investigations may be carried out from within the shaft, from boreholes drilled from the surface or both. For the first or exploratory shaft it is preferable to have direct inspection of the shaft walls during construction, all other factors being equal. This report concentrates on production shafts for the repository, not on exploratory shafts. The issue of inspection is not exclusionary and is evaluated with other factors in Section 7.

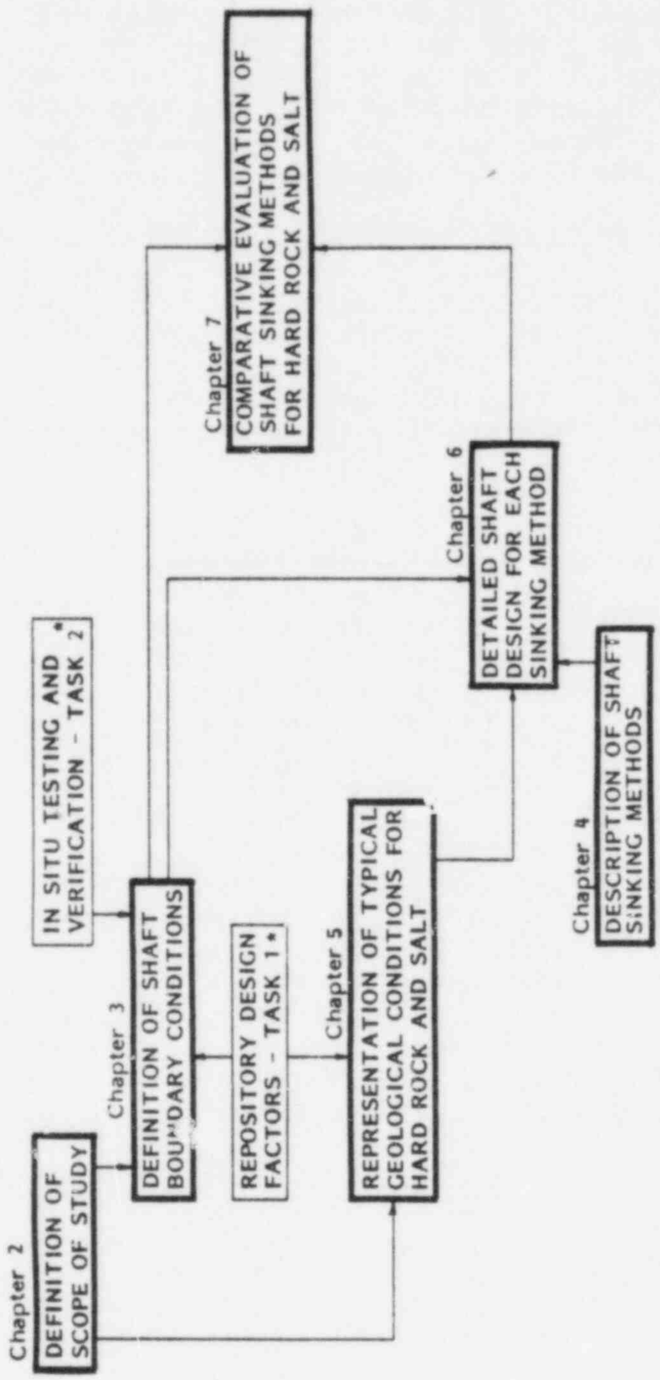
Absolute shaft diameter has a significant effect on the relative cost and feasibility of shaft sinking methods. In this study, all engineering factors are evaluated and compared for one typical diameter chosen from existing conceptual designs. This diameter reflects the operational requirements of the shaft and the desire to keep shaft diameters to a minimum for repository containment and sealing objectives. However, the influence of shaft diameter on relative construction costs and times is specifically evaluated in Section 7.

Section 4 describes in general terms the types of shaft sinking methods that will be considered in the study. In recent years, a combination of drill-and-blast and rotary methods, here called the "ream-and-slash method," has been gaining wider acceptance. The potential savings in cost and construction time for follow-on shafts, when bottom access is available, warrant its inclusion in the study. Thus, three general methods of shaft sinking are studied:

- Drill-and-Blast Methods
- Rotary Methods
- Ream-and-Slash Methods.

ACTIVITY FLOW CHART

Figure 2-1



HARD ROCK = BASALT-GRANITE-TUFF
SALT = BEDDED-DOMAL/SALT

* See "Previous Reports in Series"

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Section 5 describes the derivation of typical geological and geohydrological conditions to be used as the basic platform for the study. First, the geological and geohydrological conditions for the sites and media proposed by DOE to include basalt, tuff, and granite are provided. A composite geological profile is described that incorporates these conditions. This composite medium is called "hard rock." In a similar manner, the geological and geohydrological conditions for bedded salt and domal salt are described and a composite medium defined as "salt."

In Section 6, the shaft sinking methods are combined with the composite geological and boundary conditions to formulate a detailed shaft design for each method and composite medium. The associated shaft design is formulated with specific reference to the unusual construction and long-term requirements of a repository shaft.

Section 7 contains the comparative evaluations of the technical and economic factors for the three methods of shaft sinking. The approach adopted for this evaluation is to appraise in detail the implications of the various characteristics of each shaft sinking method to the factors noted below, using the method of shaft sinking as the prime variable:

- Technology/geometry limitations
- Geological conditions
- Investigation/preparation requirements
- Impact on formation/groundwater/sealability
- Facility for investigations
- Construction Factors (total construction time/cost/safety/alignment/quality control).

Construction schedule and cost comparisons complement this appraisal to arrive at a general evaluation for a preferred shaft sinking method. The level of accuracy of the time and cost comparisons has been chosen to be commensurate with the geological detail available. The use of "composite media" for this study does not limit the usefulness of the comparisons. It is essential that the geological and geohydrological conditions selected for each composite media are accurate since they influence the procedures and techniques of shaft sinking which would in all probability be used.

Section 8 presents a summary of the evaluations and conclusions reached and recommendations for further study.

3. SHAFT BOUNDARY CONDITIONS AND CONSTRAINTS FOR REPOSITORY APPLICATIONS

3.1 INTRODUCTION

To place the comparative evaluations of the various shaft sinking methods in the proper context of the most likely construction and operational scenarios, known and anticipated boundary conditions need to be defined. Four types of conditions are defined for each of the two composite media:

- Geometrical conditions of the shafts
- Construction related conditions
- Service/operational constraints
- Geological/geohydrological conditions.

The geometrical, construction and service conditions are based largely on preconceptual and conceptual design reports prepared for basalt and bedded salt (Kaiser 1978, 1981). These boundary conditions are essentially related to the generic requirements of construction and/or storage and isolation of waste in an underground waste repository. Thus, they are not considered to be substantially affected by changes to the design of the repository itself or the specific site chosen.

The general design features such as shaft diameter and shape configuration have been the subject of detailed studies by others with regard to efficiency of transportation of air, men, materials and equipment (Kaiser, 1981). These can be considered as reliably fixed. Shaft depth and the details of the shaft construction and design e.g., liner design, can be expected to be media-dependent.

The construction and service/operational conditions and constraints described below can be considered to apply to all media and sinking methods.

This chapter appraises and defines the first three sets of conditions and constraints. Geological/geohydrological conditions are described in Chapter 5.

3.2 GEOMETRICAL CONDITIONS

The design shaft depth is a function of optimum repository elevation within the host medium. The site specific design for basalt identifies a repository depth of about 3700 ft while the conceptual design for bedded salt - not site specific - assumes a depth of 2000 ft. The nature of the geological regions for hard rock and salt are such that repository horizons for salt are most likely to be less deep. Thus, to properly reflect these conditions, shaft depths of 4000 and 3000 ft for hard rock and salt, respectively, have been chosen. Only vertical shafts will be considered since economic analyses associated with the concept designs by DOE contractors eliminated other options (Hardy and

Heley, 1981; Dravo, 1974). Golder Associates concurs with these conclusions.

Shaft size is a prime variable which will exercise a significant impact on the relative construction costs and feasibility for each sinking method. The importance of minimizing the effect of shaft access to the repository on the geological and geohydrological integrity of the medium has forced a serious consideration of the minimum number and size of shafts commensurate with efficient operation. Four shafts in the size range 10 to 22 ft for bedded salt and five shafts in the size range 10 to 18 ft for basalt have been proposed (Kaiser 1978, 1981). As mentioned previously, exploratory shafts or associated vent shafts are excluded from this discussion. A reasonable basis for all studies is to assume, therefore, one size shaft of 14 ft. The effect of variations in shaft diameter have been considered as a parametric substudy of the primary comparison of shaft sinking methods.

The trend in shaft construction is towards the greater use of circular shafts with concrete linings. All bored shafts and 75 percent of drill-and-blast shafts sunk in the United States today are of this type (Dravo, 1974). Currently, all major shafts sunk in South Africa, the acknowledged world leader in sinking techniques, are circular concrete lined. Their use as waste repository access is particularly advantageous, e.g.:

- Structurally efficient especially at great depths
- Maintenance is low
- Good ventilation flow is possible
- Fast sinking using mechanized methods is possible
- Fire hazard is very low
- The shape is ideal for efficient sealing by grouting and freezing.

Because of the mechanized construction methods employed, techniques for decreasing cost and for expediting the construction schedule are more likely to be developed in the future. Thus, all studies described herein will be based on the use of circular concrete lined shafts with an allowance for the possible use of steel or iron linings where dictated by the ground conditions.

3.3 CONSTRUCTION CONDITIONS

The minimization of disturbance to the rock traversed by the shaft and the sealing of the shaft during commissioning of the repository and after decommissioning constitute very important constraints (Webster, 1980). Besides affecting ultimately the choice of the construction method, these constraints dictate that for any given method of sinking, the excavation, lining, and sealing processes should not compromise the ability to engineer suitable barriers to waste migration. Thus, lining and grouting designs will be more conservative than normal and a preventative approach to groundwater and stability control will be assumed in the design of each sinking method. Every effort should be made to adopt construction techniques which eliminate or at least control the disturbed zones. At the Near Surface Test Facility constructed at the Hanford Site, the disturbed zone extended about 8 ft into the basalt (Burns, 1981). As a guide, the grouting, excavation and lining procedures will be developed to reduce groundwater inflow rates for each shaft to less than 100 gpm, irrespective of the geological conditions presented as typical for each composite media or whether larger flows into the shaft can be handled during sinking.

Construction of the shafts is a critical path item in the commissioning of the repository. It is likely therefore that high-speed modern mechanized shaft sinking methods will be adopted with only secondary regard to cost implications. In structuring the time and cost estimates for comparative purposes, appropriate shift arrangements will also be assumed for the drill-and-blast and ream-and-slash sinking methods.

The construction schedule is also affected by the setting-up time for headframes, hoists and other equipment and the scope for the collective utilization of surface facilities between two or more shafts. Optimization of these schedule related factors will be incorporated so as to develop a realistic shaft sinking program.

3.4 SERVICE/OPERATIONAL REQUIREMENTS

The adoption of circular concrete-lined shafts is consistent with the design life expectancy of 50 to 100 years. This service duration will also affect the grouting requirements and the design of the lining against corrosion. Lining surfaces must be designed to facilitate decontamination, backfilling and backfill excavation.

Service requirements for alignment tolerances are related to the hoisting speeds. These may vary between 500 and 2500 ft/min depending on shaft design and associated MSHA requirements. No problems are expected in meeting the tolerance requirements if modern construction control techniques are utilized.

The implications of all these various constrains are:

- A prudent level of presinking investigations will be required i.e., the geological conditions will need to be established with sufficient confidence to reduce the possibility of unacceptable construction delays or inferior quality of final completed product to a very low level

- A reliable method of sinking not susceptible to uncontrollable or severe construction delays would be preferred

- A continuous watertight and hydrostatic lining with special bulkhead and seal design features will be required. The use of a watertight lining has also been suggested by Heley and Hardy (1981).

4. DESCRIPTION OF SHAFT SINKING METHODS

4.1 INTRODUCTION

The shaft sinking methods that are currently in use encompass a wide variety of techniques and variations and sometimes employ hybrid methods. In addition, new sinking techniques are constantly being developed. The categorization of all these methods is necessarily subjective. The division into three basic types for the present purposes is based on the following operational criteria.

1) Conventional drill-and-blast methods. This includes all those methods where the shaft is sunk blind using a drilling, blasting and mucking-through-the-shaft cycle. The shaft may be unlined, or lined either concurrently with sinking or on completion.

2) Rotary methods. These methods employ either down-hole (bore) or surface (drill) rotary equipment to excavate the shaft from the collar downwards at the full diameter. Material is removed through the shaft collar using one of the various circulation/muck handling systems developed. For completeness, a discussion of several variations of raise drilling is included in this category.

3) Ream-and-slash methods. This is a two stage operation; the first being the preparation of a smaller diameter shaft, usually by raise drilling, the second being the slashing to full diameter by drilling and blasting. In the first stage, the raise may be drilled or else reverse-reamed from the surface if bottom access is not initially available. However, bottom access must exist for mucking etc. during the second stage.

The actual method employed in a particular application depends primarily on rock conditions, shaft diameter and depth, schedule constraints, equipment availability and bottom access. With few exceptions, the facility of mucking through a smaller predrilled raise shaft is used to considerable advantage, when this is available.

In this chapter, the three methods defined above are briefly described in terms of equipment, performance characteristics, typical problems, and technological and practical limits. Only those methods directly applicable to circular concrete-lined shafts in the size range 8 to 22 ft diameter and beyond 3000 ft deep have been considered. Ground improvement techniques such as pregrouting, freezing, and grout sealing are also briefly discussed.

The shaft sinking methods as described below form the basis of the detailed shaft designs developed in Chapter 6 and serve to place the shaft design and sinking recommendations in proper perspective with respect to the wide range of potential options.

4.2 DRILL-AND-BLAST METHODS

4.2.1 Introduction

The majority of shafts sunk today are still excavated by drilling and blasting. While the basic technique of drilling, blasting, mucking and lining has remained the same, shaft sinking and lining has undergone many changes. Significant improvements include the use of mechanical muck handling equipment by the South Africans resulting in higher rates of advance; the use of circular shafts with concrete and steel linings; and the development of more efficient excavation, hoisting and lining equipment. This section describes the basic method of blind shaft sinking using drill-and-blast where both muck and water are removed through the shaft itself. Optional techniques and variations appropriate to shafts in the 10 to 22 ft diameter and 2000 to 4000 ft deep range are of prime concern.

4.2.2 Drilling-Blasting-Mucking

Most circular shafts are mined full face and drilled with jumbos containing 4 or 5 booms. Full face rounds are best if water inflow is not a problem or where concrete forms must rest on the muckpile. The bench method is favored where water inflow is excessive or where hand-held drills are used. The length of the round depends primarily on the lining support method, ground conditions, shaft size and drill-muck cycle. The common round length is about 10 ft. The proper cycling of drilling, mucking and lining is extremely important in obtaining the most favorable sinking costs.

Gelatin dynamites or water gels are usually used in shafts because of the presence of water in the holes. Normal practice is to use regular delay electric blasting caps. Powder factors usually vary from 3 to 7 lb/cubic yd.

One of the dangers in shaft sinking is the possibility of drilling into a misfired hole. The presence of muck and water in the shaft bottom makes it extremely difficult to locate misfired holes. The choice of method and the care used in blasting should take this factor into account.

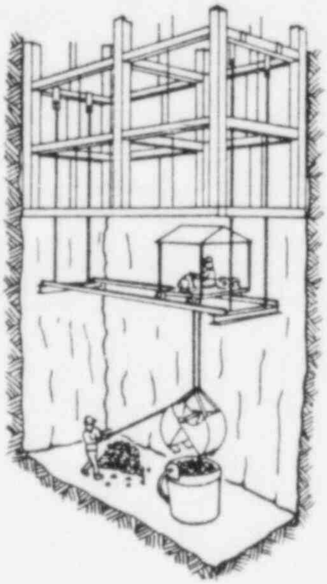
Full-scale investigations in which the rock is fragmented using an impact breaker instead of drill-and-blast have indicated promising results (Beus and Phillips, 1981).

Numerous types of mucking equipment are in use, the most common being (Figure 4-1):

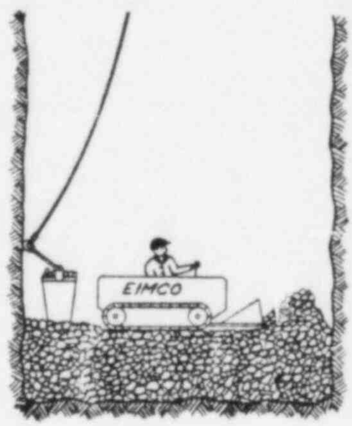
- Clamshell, e.g., Riddell mucker. Generally slower than others but simple to operate.

MUCK HANDLING EQUIPMENT
FOR CONVENTIONAL SHAFT SINKING

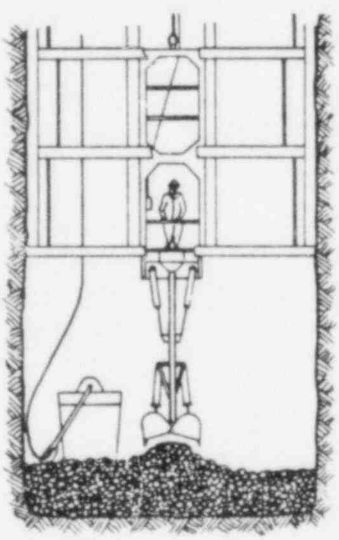
Figure 4-1



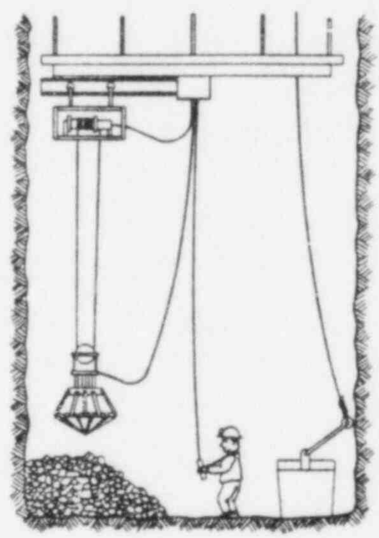
RIDDELL MUCKER



EIMCO 630 MUCKER



CRYDERMAN MUCKER
(suspended from the
galloway stage)



CACTUS GRAB MUCKER

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- Cryderman. This has a bucket similar to the clamshell but the cables are replaced by cylinders giving it a positive action allowing it to clean the bottom very thoroughly and quickly. It is suspended from an operator's cage and is hoisted clear prior to blasting. Two muckers can be used in large shafts.

- Eimco 630. This crawler-mounted unit requires at least 16 ft of clearance to operate efficiently, and is not practical in a benched shaft. The unit is best suited for shaft station excavation, but is not good in wet, soft ground. The mucker is kept on the surface and lowered to the bottom on a hoist rope when needed.

- Cactus Grab or "Orange Peel." This design is better known in South Africa as the Priestman Grab, where it has been highly developed. The world's records in sinking rates have been established with this design. The unit is centrally mounted under the stage, and revolves about the shaft with a cantilever beam suspending the grab. A heavy galloway is required.

- Backhoe. A backhoe type of mucker that is mounted under the stage, similar to the grab, has been developed in Sweden. As yet, this arrangement has not gained significant popularity over other methods. Backhoes have been used in the shafts of the Dinorwic project and in Australia with good results. There is no fogging in the shaft bottom. The system is not extensively proven in North America.

In most cases, muck is dumped into a bucket which is hoisted to the surface. The bucket is also used to transport men and small supplies. Special skips have been designed for shaft sinking but these are seldom used in North America.

The bucket is aligned in the shaft either on rope guides or fixed guides. If rope guides are used, the common practice is to use the galloway suspension cables as guides. When using fixed guides, the crosshead may be equipped with safety dogs. In deep shafts, three or four buckets are commonly used; one being filled, one being hoisted and one being lowered in balance. Sinking hoists for deep shafts are usually double-drum clutched hoists. In deep shafts, it is sometimes necessary to reduce the bucket capacity at depth to maintain the minimum hoist rope safety factor.

4.2.3 Lining Systems

Shaft linings are designed to perform one or more of the following functions:

- Support the shaft walls
- Restrict water inflow

- Provide for support of shaft structural members
- Lower the resistance to airflow.

Linings may be formed from concrete, wood, shotcrete, cast iron tubbing, glass reinforced fiber cements or steel liner, sometimes in combination. In some cases, shafts in competent rock are left unlined.

The typical concrete lining is formed using special cylindrical steel forms which last for the life of the shaft sinking project. These forms consist of a steel skin with structural steel stiffeners. Concrete is placed in the forms through doors.

There are two common systems for setting these forms for pouring:

- On the muckpile
- On hanging rods.

If the form is to be set on the muckpile, the muckpile is first leveled, then the form is lowered to its new location and leveled. After the concrete is poured, mucking can continue.

When hanging rods are used, a curb ring is lowered to the location of the bottom of the next pour and is suspended on hanging rods and aligned. A batch of quick setting concrete is placed in the curb ring to bring the pour up about 2 ft. The remaining forms are then broken loose from the previous pour and are lowered into place, using the curb ring as a bottom. Concrete is vibrated into place to achieve a 25-day strength in the range 3000 to 3500 psi. Much higher strengths are used for deep shafts.

As the concrete rises in the forms, the pour doors are closed and the concrete distribution hoses are raised. A grout ring is often placed at the top of the new pour to seal the concrete to the previous lift.

Concrete is usually batched on surface and delivered either in a bucket or through a slickline. At the galloway stage, the slickline enters a pot to remix the concrete components after their fall from surface. Distribution to the pour doors is through flexible hoses known as elephant trunks.

Work stages have a number of decks, depending on the height of pour and the travel time that can be allowed to move workers to different locations. The South Africans use as many as 8 decks, while 3 or 4 decks are considered adequate in the United States.

For the most part, concreting is not done simultaneously with drilling or mucking in North American shaft sinking operations. Many straightforward jobs have a simple one-deck stage, which is moved several times during the concreting cycle. Multiple decks are recommended when the mucking machine is suspended underneath the stage.

Shaft structures may be fastened to the walls by brackets or plates cast into the walls, or by expansion bolts drilled into the walls after the concrete is poured.

The lining concrete is usually unreinforced. However, reinforcement is frequently used in zones of high expected pressure and is often used around stations and at shaft collars.

Shotcrete is sometimes used as a temporary support system or as a permanent lining in vent shafts. The development of remotely controlled automatic shotcreting systems for shafts is likely to boost the use of shotcrete linings (Valencia and Breeds, 1981).

The use of cast iron tubbing has been restricted to zones of high pressure in concrete lined shafts. Each tubbing segment weighs about 10,000 lbs and is bolted in place. The joints between the segments are sealed using lead gaskets.

4.2.4 Water Control

Water is always present in shafts, sometimes in disastrous quantities. Pumping is invariably made available for emergencies with second-line systems in readiness. It is normal to investigate or to have available extensive data on groundwater conditions prior to the commencement of shaft sinking (e.g., Swaisgood and Versaw, 1973). If large quantities of water are anticipated, pregrouting prior to sinking may be advantageous. If a localized aquifer exists, this pregrouting may be carried out from the surface or from the shaft bottom. This latter approach is usually more thorough. The approach adopted depends on the distribution of aquifers.

In South Africa, almost all shafts are pregrouted from the surface using 3 to 5 drill holes and the holes are grouted with cement in stages of approximately 1000 ft (Nel, 1981). The holes are drilled outside the proposed shaft circumference. This method is very successful in many ground conditions in the United States and other parts of the world. In some cases, chemical grouting is used in conjunction with cement grouting.

If there is considerable water, and the ground conditions are extremely incompetent, such as with quicksand, gravels, etc., then it may be necessary to resort to freezing. The disadvantage is the time required to achieve freezing of the ground ahead of sinking and the extreme cost involved. There are, however, some ground conditions where freezing is the only known method of achieving the ground stabilization necessary for shaft penetration.

Using these preventative measures for groundwater control, the resulting water inflows of the order of 150 gpm can be handled by conventional pumping techniques.

4.2.5 Shaft Equipment

Shaft equipment consists of the sets, the guides and the utility systems that are installed in the shaft.

In a circular concrete-lined shaft, the shaft dividers are usually made of structural steel shapes. Usually they are spaced from 5 to 15 ft apart. The purpose of the steel is to divide the shaft into compartments and to provide support for conveyance guides, ladderways and utilities. The steel is not required to resist ground pressure. In shafts used for ventilation, the buntons are sometimes streamlined to reduce their resistance to airflow.

Guides are made of steel or wire rope. Steel guides may take the form of a crane rail, a special guide section or a hollow rectangular section. In special instances, pipes have been used as conveyance guides and to convey compressed air or water along the shaft. In high speed hoisting, guide alignment tolerances are extremely tight (+ or -1/16 in.) and any misalignment can cause rapid wear and possibly result in shaft wrecks.

Tensioned wire rope guides are fastened only at the shaft top and bottom. The cable is usually of the flattened strand or locked coil variety to reduce the wear rate on the outside wires. For skip hoisting with wire rope guides it is necessary for the conveyance to enter tapered, rigid guides called spears in the tailshaft and when entering the dump scrolls. Wire rope guides have some disadvantages:

- They cannot be used in very deep shafts or where ventilation velocities are high because of the sway problem
- Conveyances cannot be equipped with effective safety dogs
- When the guide is worn, the entire rope must be replaced.

4.2.6 Shaft Sinking Plant

Shaft sinking plant includes surface facilities and sinking plant; namely, headframe, hoist, compressors, ventilation, shaft doors, pumping and grouting plant, galloway stage and stage hoists. Deep large shafts, such as used in South Africa and contemplated in this study, justify elaborate surface facilities with large capacity hoists.

In shafts deeper than about 2000 feet, the hoists usually consist of two drum clutched hoists which may be equipped with staged braking and controlled deceleration systems. Often, the permanent shaft service/operating permanent hoist will be used for sinking deep shafts due to the need for powerful, specialized equipment.

The usual practice for the ventilation of blind shafts is to install a high pressure, reversible vent fan at the collar and to blow the air through rigid vent tubing to the shaft bottom. The ventilation air serves several functions:

- Provides fresh air for the crew
- Dilutes blasting fumes and gases released from the rock
- Reduces temperatures in the working area.

Galloway stages are usually constructed of light structural members and have expanded metal decking. Where multiple part lines are used for suspending the stage, a sheave deck is usually included one deck below the top deck.

Some examples of galloway stages for deep large shafts are:

Kidd Creek No. 2 Shaft. Five deck galloway weighing 60 tons and suspended by four 1 in. locked coil ropes in double purchase. The stage was 40 ft in length and 23 ft in diameter. Two Cryderman muckers were housed in the stage (McKay, 1981).

Mt. Taylor 24 ft Diameter Shaft. The galloway was a 5 deck stage, 45 ft in length and supported on four 1 3/8 in. ropes.

South Vaal. In this South African operation, 60-ft long 6-deck stages were used. The weight of the galloway and muckers was 70 tons. The stage was suspended on two - 2 part lines 1 1/2 in. in diameter. Due to the depth of the shafts, (8000 ft) a Blair type stage hoist was used.

President S. Teyu No. 4. This shaft is nearly circular in cross section being 33 1/2 x 36 ft inside the lining. The total depth is 7760 ft. A multideck Galloway, 72 ft in height and weighing 70 tons was suspended on 12 falls of rope.

From the foregoing it can be seen that very long ropes are involved in suspending a heavy galloway in deep shafts. A special winding system has been invented for these cases called the Blair system. The Blair system consists of a friction drum in which one or more wraps of rope are wound on the driving drum. The live end goes down the shaft to the galloway and the other end goes over a tensioning tower thence to a storage drum. The storage drum has a small motor which is capable of maintaining the tension in the rope to the driving drum.

Other schemes for handling large amounts of rope include lowering and refastening the dead end to the shaft walls and use of a magazine drum on the dead end of the stage rope.

Typical details of drilling and mucking operations on a number of recent North American shaft sinking projects are given in Table 4-1. Excellent accounts of conventional shaft sinking concepts are given by Nel (1981) and Wilson (1976).

EXAMPLES OF DRILL-AND-BLAST SHAFT SINKING   Table 4-1

SHAFT	INSIDE SHAFT DIAMETER	DRILLING METHOD	DEPTH OF BLAST HOLES	TYPE OF ROUND
Silver Shaft	18 ft.	Hand Held Sinkers	8 - 10 ft.	Bench
Mt. Taylor	14 ft. (1) 24 ft.	Hand Held Sinkers	8 ft.	Bench
Kidd Creek No. 2 Lower 2300 ft.	25 ft.	Hand Held Sinkers	8 - 10 ft.	Bench
Gnome	10 ft.	Hand Held Sinkers	5 - 7 ft.	Full face, pyramid cut.
Pea Ridge	19 ft.	Jumbo	?	Full face.
Homer-Wauseca	20 ft.	Jumbo	8 - 10 ft.	Full face, pyramid cut.
FHC No. 6 &7	22 ft.	Jumbo	7 ft.	Full face, pyramid cut.
Fecunis Lake No. 1	13'11" x 19'9"(2)	Hand Held Sinkers	7'9" to 10'5"	Bench
Giant Yellowknife "C"	12'2" x 17'10"(2)	Hand Held Sinkers	?	Bench
Kerr Addison No. 3	11'9" x 16'3"(2)	Hand Held Sinkers	?	Bench
Creighton No. 9	21 ft. diameter	Hand Held Sinkers	?	Bench

NOTES:

1. Probe hole is drilled 70 ft. ahead of shaft bottom to test for water or methane under pressure. Drill used is heavy duty rotary on airtrack frame.

2. Dimension of rock excavation

4.3 ROTARY METHODS

4.3.1 Introduction

Shaft sinking using drill-and-blasting is one of the most time consuming, costly and hazardous forms of underground excavation. The accident rate for shaft sinking is twice that for coal mines. Frequently, shaft construction is the critical path for the development of the mine or underground facility and schedule reductions have a substantial impact on the economics of exploitation. Conventional sinking is labor intensive and the poor working conditions make it increasingly difficult to find miners willing to work in shafts.

Rotary methods of shaft sinking were initially developed to improve the safety and to reduce the time for sinking along lines similar to that experienced with the introduction of boring machines to tunneling. In fact, the similarity between tunnel and shaft construction has encouraged an extensive cross-application of the technology. Another major impetus to the development of shaft drilling techniques was the difficulty of handling soft, water-bearing formations with conventional sinking.

A variety of rotary methods of shaft sinking have been developed. These can be categorized in the first instance as:

- Surface drilling
- Down-hole boring
- Raise drilling.

Drilling involves the rotation of a cutter head using top drive equipment while in boring, the shaft is advanced by a mole type machine in the hole. Raise drilling is the up-hole equivalent of down-hole drilling. All three basic methods may excavate the shaft using full-face or staged reaming techniques. The down-hole drilling has been developed from the petroleum drilling industry. Only those methods where the material is removed through the shaft collar can be considered true blind boring or blind drilling techniques.

The categorization of the various methods is shown in Table 4-2. The main sources of variation to each method are the mud circulation system, and the sequencing of enlargement. Not all the available methods will be described in detail. Only those methods of sinking of large-diameter shafts to great depths are of relevance here. Methods such as clam shell excavation, bucket augers, and various shields and caissons are either suitable only for shallow shafts or for shafts in overburden.

This section describes the current state-of-the-art of rotary shaft sinking by reviewing only the most promising applications for each method. Limitations, current trends and future capabilities have been established, sometimes by direct contact with some of the contractors who have drilled, bored or furnished equipment for large-diameter

DESIGNATION OF ROTARY SHAFT SINKING METHODS — Table 4-2

1. <u>DOWN-HOLE DRILLING</u>

   1A Full face using top drive equipment

   1A1 Direct mud circulation
   1A2 Reverse mud circulation
   1A3 Reverse air circulation
   1A4 Reverse air and mud circulation
   1A5 Reverse air and direct mud circulation

   1B Staged down hole drilling
   1C Calyx drilling

2. <u>DOWN-HOLE BORING</u>

   2A Full face using mole

   2B Down-hole reaming using mole

3. <u>RAISE DRILLING</u>

   3A Up-hole reaming using top drive equipment

<u>NOTES:</u> (1) Refer to text and figures for descriptions.

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shafts. This has been necessary because of the recent rapid changes in technology.

The results of one survey (Dravo, 1974) illustrate the emerging significance of rotary shaft sinking methods. For the period 1963 to 1973, and for the category of mine shaft development in the United States, a total of about 221,000 ft of shaft were constructed of which 25 percent was by rotary methods. Because of the relatively greater advances in mechanical drilling and boring techniques, the corresponding percentages for the period 1973 to 1982 and far beyond 1982 are expected to be much higher. The greater part of these bored or drilled shafts were excavated using raise and blind drilling utilizing direct and reverse mud circulation systems. These are methods 1A and 3A in Table 4-2.

The main perspective on rotary shaft sinking methods can be summarized as follows. Cutting bit technology has advanced to the point where hard rock conditions do not usually preclude the use of rotary drilling and boring heads for excavation of shafts. The main barrier has been the development of equipment to drill holes large enough and the efficient removal of material from the cutting face. Technology and equipment are available today to drill or bore shafts up to and greater than 20 ft in diameter and over 3000 ft deep. The tremendous potential savings in cost and time offered by rotary over conventional methods continue to offer huge incentives for the continued development of rotary shaft sinking technology.

4.3.2 Full-Face Drilling Using Top Drive Equipment (Method 1A)

This technique has evolved rapidly since 1950 when large-hole drilling was introduced for emplacement of nuclear devices underground. Mining companies soon began using this method for shaft sinking where ground or water conditions made shaft sinking using conventional techniques difficult.

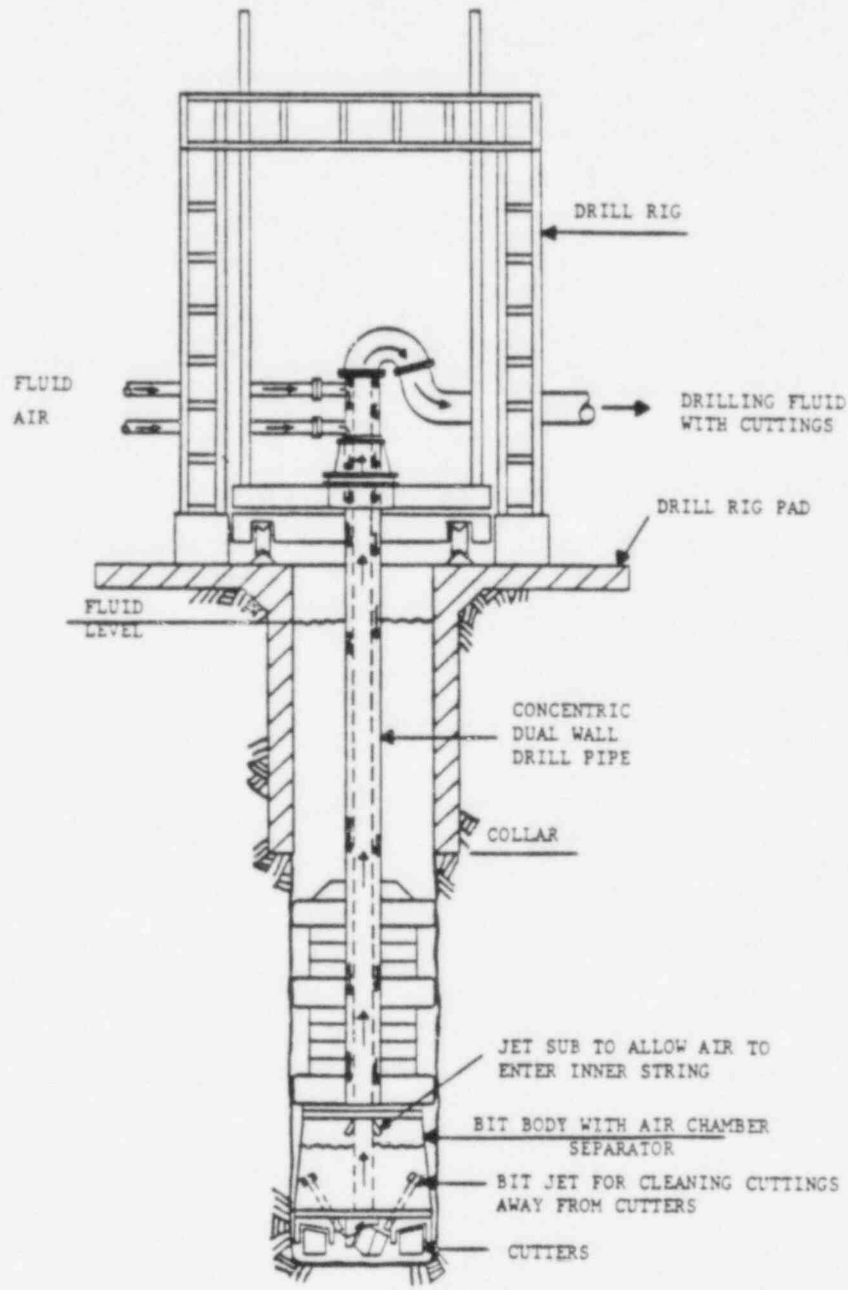
The equipment being used has evolved from the large oil and gas well rigs to specialty shaft drilling rigs. A typical rig consists of a rotary table which drives the drill pipe, and a draw works which raises and lowers the drill pipe and is used to place the shaft liner (Figure 4-2).

The drill pipe is used to transmit torque from the kelley to the drill bit. Present practice is to use 13 5/8 in. O.D. pipe, however, the pipe on the latest specially developed rig is 20 in. O.D. The drill pipe may consist of two concentric pipes, an outer pipe to transmit the torque and an inner pipe to return the drill cuttings and fluid to the surface. The inner pipe is held concentric to the outer pipe by spiders which allow compressed air to travel through the annulus.

Special muds are used to support the shaft walls, to prevent water incursion and to provide a medium for flushing the drill cuttings. Mud

LARGE DIAMETER DRILLING ARRANGEMENT
(Reverse Air-Mud Circulation)

Figure 4-2



After Carone and Whitley, 1980

Rev. ____ Dwg. No. 82-1164D Date 3-82 Eng. LG components depend upon the type of rock being drilled and hydrological conditions. Two general configurations for circulation of mud in deep shafts are available, Reverse Air Assist Circulation and Jet Circulation. In the Reverse Air Assist Circulation system, an air pipe is suspended in the drill pipe to about 300 ft to act as an air lift. The cuttings and mud rise through the drill pipe as a result of the pressure differential between the mud outside of the drill pipe and the slurry/air mixture inside of the drill pipe.

The second system, Jet Circulation, uses injection of drilling mud and compressed air in the annulus between the drill pipe and the inner tube to pick up the cuttings and lift them to the surface through the inner tube.

The Jet Circulation system may take one of two forms: 1) the air/mud mixture may be pushed to the bit using the energy of the compressed air to overcome the pressure of the mud column in the hole; 2) the mud may be circulated down through the annulus and back up the inner tube using an air-lift arrangement. The problem with the first system is the high horsepower requirements for compressing the air. The problem with the second Jet Circulation system is the need to handle two pipe strings. These circulation systems are presented diagrammatically in Figure 4-3.

The drill bit is a flat-faced circular plate which supports the cutters. The cutters are cylindro-conical rollers which are faced with either steel teeth or tungsten carbide inserts. These cutters may be removed and replaced or sharpened as they wear. The bit body is usually used for the life of the project.

Above the bit body, donut weights are mounted on the drilling string to allow for application of pressure to the bit face and to keep the drill string operation in a true plumb line. Stabilizers may be added to the drill string between donut weights or above the weights. Stabilizers may be rotating or nonrotating.

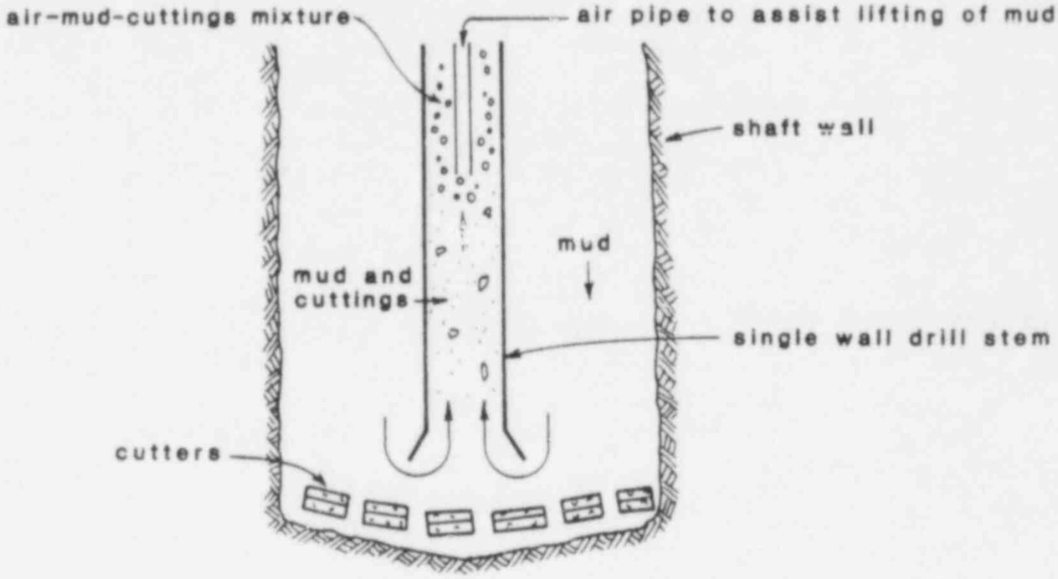
Surface ancillary facilities consist of a bank of compressors, a diesel/electric power plant if no utility power is available, pits for mud storage and cuttings disposal, a mud treatment plant for cuttings removal, and water supply system. The shaft is usually collared oversize and a slab is poured to support the rig.

The shaft liner typically consists of a special rolled steel section with stiffener rings to keep the pipe in a cylindrical shape and to hold it off the side of the hole. Liner plates often are as thick as 2 in. at the bottom sections. The cost of this type of liner is often equal to or greater than the cost of drilling the shaft. Research is underway to attempt to make these liners less expensive.

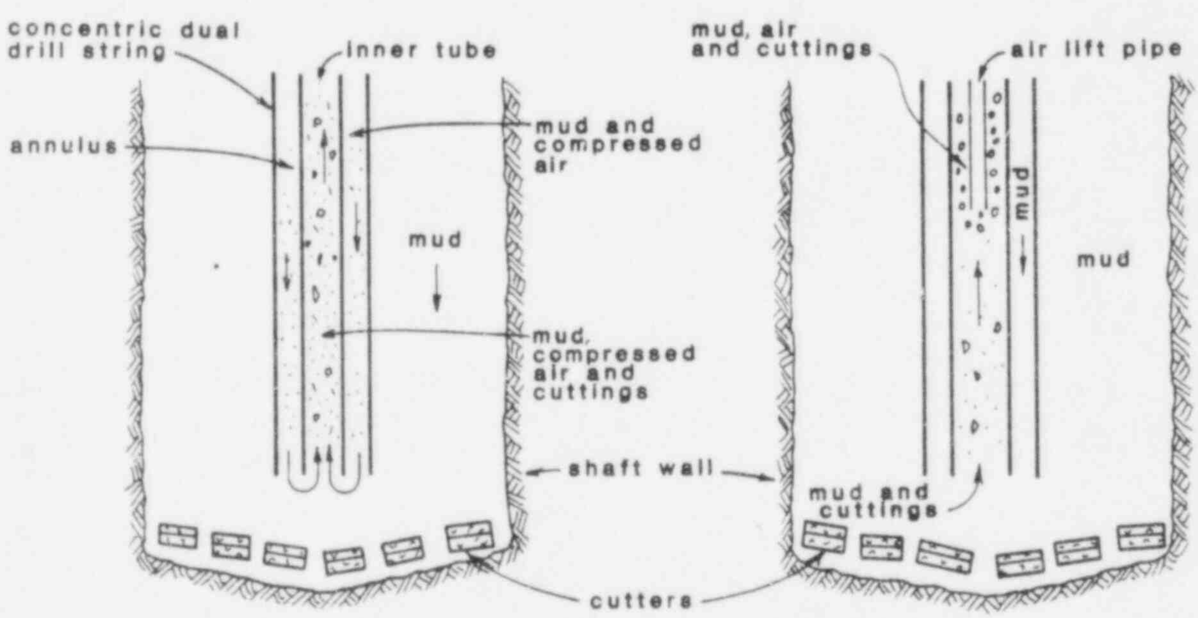
The liner is floated in the shaft, using the rig drawworks to lift the prefabricated sections into place for welding. When the bottom section of the liner reaches the bottom of the hole, grouting of the liner commences from the bottom with the cement grout displacing the mud in

CIRCULATION SYSTEMS FOR LARGE DIAMETER DRILLING

Figure 4-3



REVERSE AIR ASSIST CIRCULATION



REVERSE AIR MUD CIRCULATION

JET CIRCULATION AIR LIFT

JET CIRCULATION SYSTEMS

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the annulus between the shaft walls and the liner. When grouting is completed, the shaft is ready for equipping. Precast concrete liners have also been proposed for achieving a dry and hydrostatically lined shaft. The concrete sections cast and cured at the surface are then stacked into the shaft from the bottom up while the shaft remains full of fluid. The lining is grouted-in immediately behind the ascending column (Skonberg, 1980). A slip-form method of concrete lining large-diameter shafts has been tested full scale in a preliminary manner (Maser, 1981).

A good example of a large blind drilled shaft is the 96 in. by 2352 ft deep shaft drilled in the Piceance Creek Basin in 1977 (Utter, 1980). This shaft was sunk for the U.S. Bureau of Mines to conduct research on the environmental effects of underground oil shale mining. Project supervision was by Fenix and Scisson, the rig was supplied by Rowan Drilling Company.

The presence of two troublesome aquifers and methane played a part in reaching the decision to drill the shaft rather than to use conventional sinking. The rig was a new oil well type rig especially built for blind shaft drilling. The 120 in. diameter hole was lined with 96 in. diameter steel liner grouted into place. Shaft bottom misalingment amounted to 19 in. The resulting cost per foot of unequipped shaft was $3551.

The Hughes Tool Company has constructed a rig for blind drilling shafts 20 ft in diameter to 3000 ft. The rig (called SD-300) is currently at Agnew Nickel Mine, Australia drilling a number of shafts of this size. Large diameter drilling was chosen as the most suitable method of shaft sinking after detailed study by the Australian Mineral Industries Research Organization. This rig which uses air-assisted reverse circulation and is capable of exerting 500,000 ft lbs of torque to the cutting head is considered to be the most advanced shaft drilling rig available. The average drilling rate is believed to be in the range one to two ft/hr with an approximate drilling and lining cost of $2300/ft (Fenix and Scisson, 1981). This compares with $3100/ft and one quarter the progress rate for conventional sinking.

This method of shaft sinking is most competitive with conventional sinking when running ground, extensive water or methane are likely to be encountered. Risk of collapse of shaft walls and maintaining verticality are two serious constraints which need further attention. Other recent examples of notable shaft drillings are:

- A 660 ft deep and 11 ft diameter shaft at Emerald No. 3 mine in West Virginia by McKinney Shafts Inc., using a reverse circulation rig

- The 2000 ft deep Conoco pilot shaft at Crown point in New Mexico drilled to 4.5 ft diameter by Challenger Drilling in 1980. A fully hydrostatic steel casing was installed to meet a very tight construction schedule.

4.3.3 Staged Down-Hole Drilling (Method 1B)

This method is very similar to Method 1A except that the shaft is enlarged in stages to the full diameter. The most significant operation of this type was the sinking of two shafts at the Beatrix Mine in Holland in 1952. Current technology has obviated the need for staged drilling using top drive equipment.

At Beatrix, two shafts were drilled to 25 ft diameter and to 1700 ft depth by initially blind drilling to 6 1/2 ft diameter followed by several enlargements with different bits. The hole was kept filled with mud-flush during the entire drilling and lining operation. This was effective in controlling flows and preventing cave-ins in the 1600 ft of soft water-bearing strata traversed. Caving problems were checked by increasing the soda content of the mud (Fenix and Scisson, 1981).

Special procedures were used to measure and correct alignment, and good accuracy was achieved. The lining was constructed of two concentric steel shells filled with 8000 psi concrete, reducing the inside diameter to 18 ft 4 1/2 in. A bitumen outer membrane provided elasticity against damage to the shaft lining from mining subsidence movements. Spot drilling rates varied from 1 in. to 3 ft/hr depending on the formation being drilled and the particular stage of the reaming process (Fenix and Scisson, 1981).

4.3.4 Calyx Drilling (Method 1C)

This concept bears mentioning in that it is a pioneer shaft boring method and it is ideal in terms of energy efficiency. The calyx drill is a coring machine in which the operator station is in the machine which is down in the hole. The electric motors drive the calyx barrel which has a serrated bottom edge. Steel shot and grit are used as grinding media and these are fed from the operator's station above the barrel. Jacks are set into the shaft walls to resist rotation.

After one coring cycle is completed, either due to filling of the barrel or due to blocking of the core, the machine is lifted to surface by a crane and a core lifter is lowered. If the core cannot be broken free using the crane, it is necessary to place a small explosive charge on one side in the kerf. Once the core if lifted, the machine is again lowered, the jacks are set and boring begins again.

The Calyx drill was first used in the United States at the Brunswick Mine in Grass Valley, California in 1936. A 50 in. hole was drilled to a depth of 1125 ft. The machine was subsequently used in Minnesota in 1938 and in Wisconsin in 1944 to bore 66 in. holes to depths of 1208 and 2487 ft, respectively. A 66 in. by 650 ft hole was drilled by American Zinc using the calyx drill in 1959.

The most serious limitations of the calyx drilling method are the types of formations which can be drilled and the diameter size which can be effectively handled using the kerf-cutting method. For large-diameter shafts, the blind boring machine essentially supercedes the calyx drill.

4.3.5 Full Face Boring Using Mole (Method 2A)

This is perhaps the ideal shaft sinking method but it has not been possible to date to develop the hardware necessary to make this a viable shaft sinking technique.

The most significant development of this method is the construction of a shaft at Oak Grove mine near Birmingham in Alabama (Sands and Little, 1979). This machine called the Blind Shaft Borer (BSB) weighs 300 tons and bores a 24 ft - 5 in. diameter shaft. The machine resembles a tunnel boring machine in a vertical position and functions in much the same manner (Figure 4-4). The machine is equipped with a mechanical mucking system to raise the cut material by bucket elevator to deposit it in the shaft skip hoisting system.

The machine development by The Robbins Co., and the shaft construction project were funded by the Department of Energy to develop a system for sinking coal mine shafts at a rate of 25 ft/day. Some of the operational concepts were developed from earlier experiences with the V mole built by Wirth. The V mole is a 2B type machine and is described below. The latest version, the V mole 4, is capable of operating either as a pure blind shaft borer or as with its predecessors, using a pilot hole with underground access for mucking.

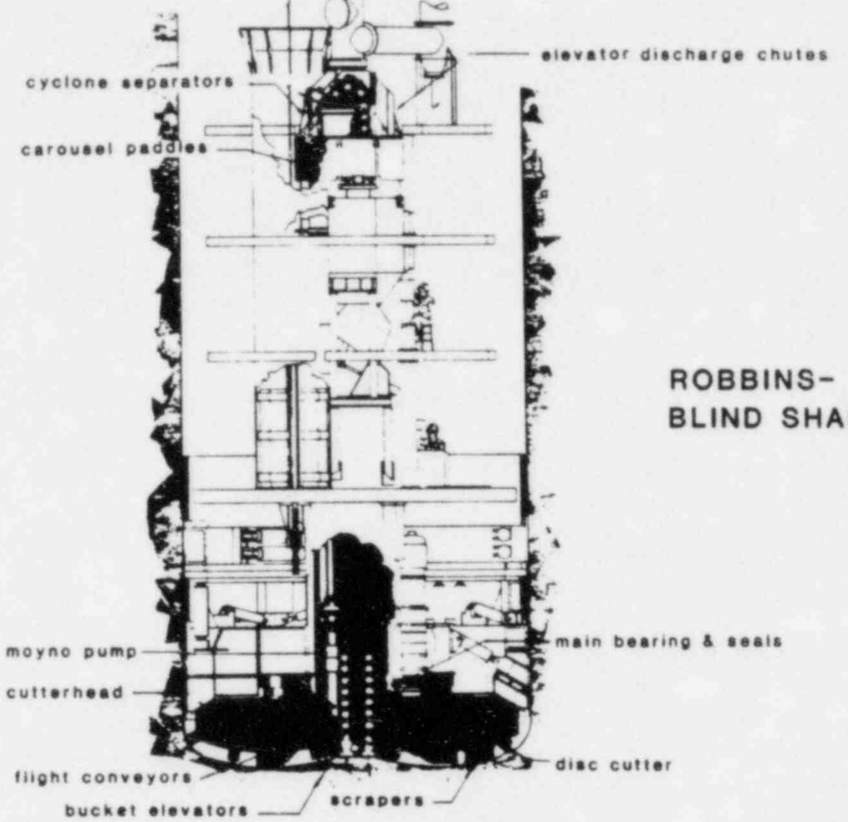
The V mole 4 borer built by Thysson is being used to construct a series of 8 shafts to a 2000 ft depth and concrete lined to 22 ft diameter. Sinking rates of 60 ft/day of finished lined shaft are being achieved (Grieves, 1979). A pilot hole is used for muck removal.

The key to the successful operation of blind boring machines is the muck handling system. The Robbins BSB at Oak Grove was withdrawn after completing 600 ft of the 1132 ft shaft. Minor amounts of water in the shaft bottom created a gumso which could not be handled by the muck pickup system. Ruby and Sands (1979) reports the design and testing of a superior pneumatic hoisting system recently developed for the BSB machine.

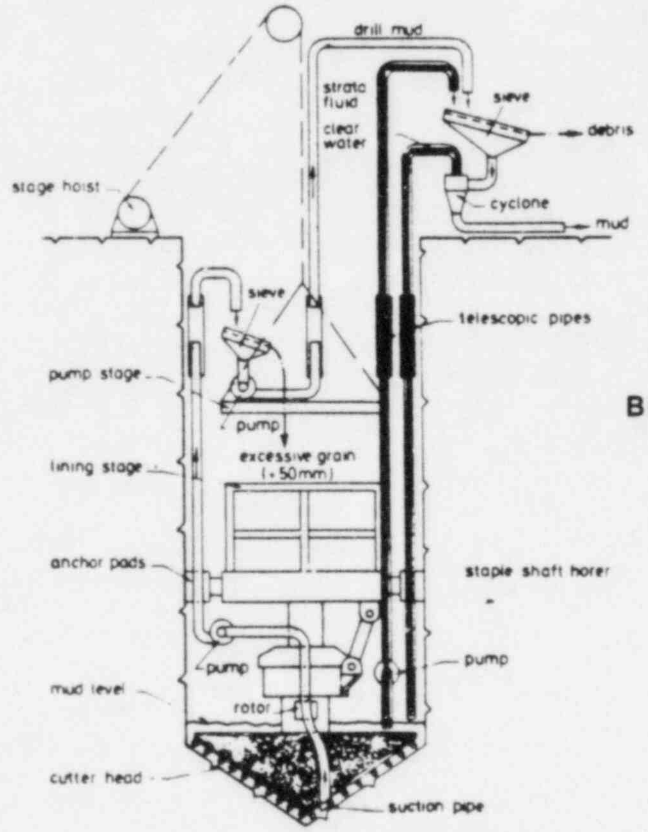
The BSB machine was notably successful in a number of respects. During testing, it was possible to achieve advance rates of 16 ft in 24 hours. The steering system was able to operate with remarkable accuracy. The machine was built for gassy mine conditions to stringent permissible or intrinsically safe electrical standards. Although the BSB machine was not a commercial success due to mechanical problems, it does contain many excellent features. It cannot be considered a proven method at present. A continuous concrete lining device designed for use with blind shaft boring machines has been invented and patented by Battelle Memorial Institute and The Robbins Company (CIM Bulletin, 1982).

BLIND DOWN-HOLE BORING SYSTEMS

Figure 4-4



**ROBBINS-
BLIND SHAFT BORER**



**V MOLE 4-
BLIND SHAFT BORER**

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The main limitations of the two down-hole blind boring machines is that they will not work in running ground or where heavy water inflows occur. The use of these machines is also restricted by the range of diameters that can be bored with any one machine.

4.3.6 Down-Hole Reaming Using Moles (Method 2B)

This method involves the staged enlargement of a shaft using downward boring with in-hole equipment. At least three types of machines have been developed and applied:

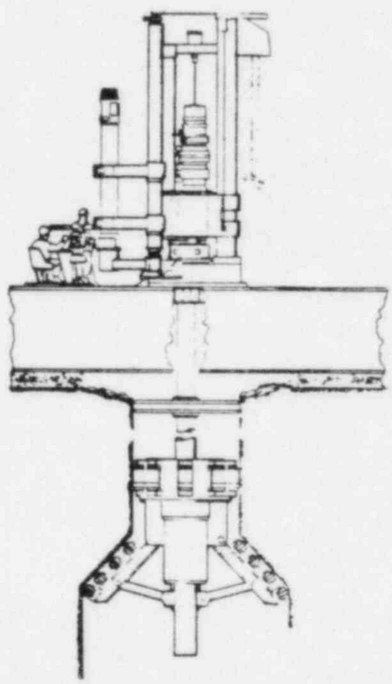
1. V Moles (Wirth System)
2. Robbins 1211 SR Shaft Reamer
3. MMBW Shaft Reamer.

The concept developed in the German coal fields in the early 1970's where an alternative was sought to the labor intensive conventional method of shaft sinking. Because of the limitations of normal raise drilling on alignment accuracy, and the depth, diameter and torque limits imposed by the drill string, German contractors opted for a down-the-hole shaft boring machine using a pilot hole to remove the muck (Bruemmer and Wollers, 1976). For the intended applications in the Ruhr coal fields, bottom access was readily available and since large inclined shafts had been raise-bored in Switzerland and Austria, the technology was readily available.

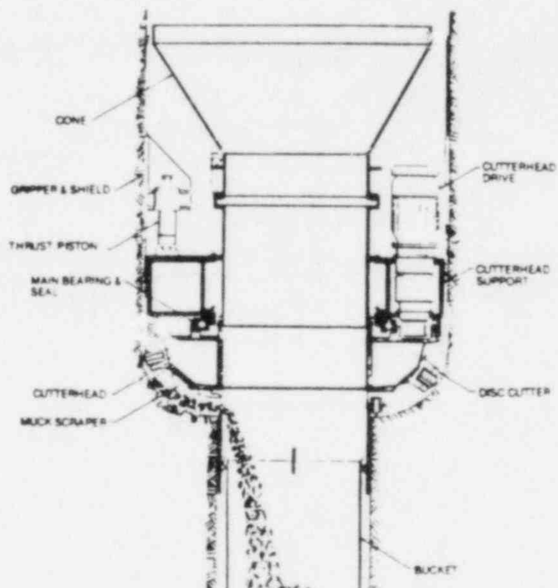
The first mole, called V Mole 1 bored a shaft to 16 ft in diameter and was so successful, 9 shafts were drilled between 1971 and 1977 with this machine. Advance rates averaged 44 ft/day. In 1977 this machine was replaced by V Mole 2 capable of boring 16 to 22 ft in diameter. This machine bored a 1500 ft deep shaft at Preussag's Ibbenleveren mine in 44 days. The machine was operated by remote control to traverse highly pressurized gaseous beds.

V Mole 3 has been constructed to drill shafts to 26 ft in diameter in the Saar Region of West Germany. Depths up to 4000 ft are planned. The shafts will be concrete lined to 24 ft. Alternative lining techniques have been developed for use with the V mole in which thin pre-cast glass-fiber reinforced concrete panels are set and grouted into place behind the machine as it progresses (Grieves, 1979).

The Robbins 1211 SR Shaft Reamer was used on the Chicago water storage project to enlarge a previously drilled 6 ft bore to 12 ft (Figure 4-5). It is remotely controlled and capable of boring in hard rock. Although fully shielded, it does not included facilities for lining (Friant, 1980). The cuttings are moved by a combination of scrapers and gravity to the center of the shaft and dropped to a lower level for removal. Three shafts have been drilled in limestone to depths of about 300 ft. Peak advance rates up to 55 ft in 10 hours have been recorded.



ROBBINS 121BR EXPANDABLE RAISE REAMER



ROBBINS 1211SR SHAFT REAMER

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The MMBW (Melbourne Metropolitan Board of Works) machine was developed to construct two 360 ft deep and 19.5 ft diameter shafts for the Thompson Dam project in Australia in 1977. In concept, the machine is very similar to the V moles and the final design comprised a hybrid of Robbins, Jarva, Mitsubishi, Memco and Calweld components. The 25 degree face angle ensures efficient removal of muck to the center hole. The geology is folded and faulted pyritic siltstone with strengths in the range 6,000 to 21,000 psi. The presence of persistent bedding planes, crushed seams and clay coated joints of 2 to 24 in. spacing resulted in potentially unstable wall areas.

Rock support included mesh, bolts and strapping. The rock bolting turret fixed to the machine body entailed bolting to be carried out to within 20 ft of the face during the boring cycle.

Using two lasers for survey control and good directional maneuverability, the initial misalignment of 6 ft was reduced considerably. Concrete pads were placed over the full length of the shaft to provide good gripper reaction.

The mechanical excavation of the shafts caused only minor disturbance to the rock mass. This contributed considerably to the resultant stability of the excavation and the minimal overbreak despite persistent rock mass defects. Each shaft was excavated in approximately 8 weeks of which time about 15 percent represented actual boring (Callow, 1981).

One major advantage of the down-hole reaming machine is that minor deviations in the pilot/mucking shaft can be corrected to produce a truly vertical shaft. The necessity for a pilot hole has two implications:

1. The need for bottom access
2. The machine can be used only in fairly competent ground.

The lining/support systems developed are designed to prevent ravelling rock from entering the shaft. The problem of installing a hydrostatic lining system limits the use of the machine to reasonably dry rock. This main disadvantage is most easily overcome by using the V Mole 4 concept described earlier which incorporates a hydraulic muck disposal system and a concrete lining stage (Figure 4-4).

4.3.7 Up-Hole Reaming (Method 3A)

This method is essentially the raise drilling or reaming of a shaft from the bottom using top-drive equipment (e.g., Figure 4-5). At the present time, raise drilling is the most widely used technique for mechanically excavating shafts and in excess of 30 machines are operating in the United States alone (Wilson, 1976). The current world inventory of raise drilling machines stands at more than 300 (Eng. Min Jr., 1981).

One of the more notable accomplishments in raise drilling was the opening of two, 30 ft, 3 in. diameter 310 ft deep shafts at the Monterey Coal Company's No. 1 Mine near Carlinville, Illinois. A 13 3/4 in. pilot hole was drilled from the surface and a cutting head attached to back-ream to the full diameter in one pass by Frontier-Kemper Constructors using a Robbins RBM-211 machine. Boring rates average 2.7 ft/hr in sandstone shale and limestone. The shafts were concrete lined by slip-forming to a final diameter of 21 ft (Mining Engineering, 1978).

In 1975/76, a 12-ft diameter shaft 2300 ft deep was raise drilled in a single pass at the Cargil Inc., salt mine in the Finger Lakes district of New York. Holing through was achieved 78 days after rig-up of the Robbins 81R drill had begun (Eng. Min. Jr., 1981) giving an average rate of 3 ft/hr. The compressive strength of the rock through which the shaft was drilled varied from 15,000 psi to 25,000 psi (Min. Mag., 1976).

Atlas Copco has recently built a raise drilling head for use in German coal fields and civil engineering which is modular in design, enabling it to ream 12.5 ft, 17.4 ft and 20.6 ft diameter shafts (Min. Jr., 1982). This versatility reduces capital investment costs and improves equipment utilization.

The major advantages of the raise drilling method are safety of personnel, minimal damage to the sidewall and a high rate of advance. The main disadvantages are:

- Practical limits to the combination of shaft diameter and length as imposed by the torque and axial load capabilities of the drive rod. It is currently impractical to design a raise drilling machine capable of finishing a shaft to 20 ft diameter and 3000 ft deep.

- There is some problem in attaining acceptable standards of alignment accuracy due to the thrust being supplied by the pilot drill during the first pass of down-hole drilling.

- An inability to cope successfully with running ground or high inflow of water except with prior grouting or freezing. Current technology requires that the shaft wall stand unsupported after raising is complete until the liner is placed.

4.4 REAM-AND-SLASH METHODS

4.4.1 Introduction

Ream-and-slash is a method of adding shafts to existing subsurface facilities which has gained wide usage. Compared to the conventional drill-and-blast method, significant time and cost savings can be effected. They are considered here because of their important potential role in sinking shafts, other than the first at a site, to the repository level.

Some notable examples of recent applications of the ream-and-slash method are given in Section 4.4.3 to illustrate the available technology and recent trends.

4.4.2 Sinking Methods

The method of sinking consists basically of drilling a small pilot hole, raise drilling to a larger size and then slashing to the required diameter using conventional drill-and-blast techniques. The use of the center raise as a glory hole to remove muck results in a considerable increase in productivity in the muck handling cycle.

This central pilot shaft may also help in solving a number of other difficult problems. It acts as a relief hole during blasting, thereby improving blasting efficiency and allowing better control of profile and rock disturbance. Pumping of water away from the active shaft bottom is unnecessary, working conditions are generally drier and if the bore is kept open, it may be used as a ventilation duct. Raises are usually 4 to 10 ft in diameter.

Various methods of pilot hole drilling and raising are available.

- Standard raise - the pilot hole is drilled from the top then backreamed from the bottom
- Reversible raise - the main shaft is reamed from the top down
- Blind drilling or raising - The raise is drilled full diameter up or down without the benefit of a pilot hole.
- Excavation by raise climber e.g., Alimak.

If there is sufficient access, the standard raise drilling method is often the best because the equipment is less complex and easier to handle.

In normal operations,a raise drill is operated by a two-man crew. The types of drive systems in common use include fixed speed electric, variable speed electric and hydraulic. Raise drill machine characteristics are described by Monroe (1980).

Following completion of the raise, the slashing and lining operation is carried out from the top down. Drilling of blast holes is mostly done with stopers instead of a jumbo and the muck is extracted from the bottom of the bore using a choked draw.

Two problems sometimes encountered with the ream-and-slash method are the difficulty of maintaining alignment of the pilot hole and the difficulties of preventing blockages when mucking down the raise.

4.4.3 Case Histories

The approach adopted at Crown Point, New Mexico illustrates the effort undertaken to maximize the length of shaft development by ream-and-slash. A 10 ft diameter hole was drilled to a depth of 2190 ft using a top drive rotary drill and reverse circulation. While the hole was still full of mud, a steel hydrostatic liner 85 in. in inside diameter was lowered into the hole and grouted in place (Eng. Min. Jr., 1980a; Hawes, 1982).

The rig was then moved to the second hole and a 4 ft diameter hole was drilled and lined. A third hole was completed in a similar manner.

It was then planned to install a pump station on the 1490 ft level and to develop a station on the 2130 ft level in the 85 in. shaft. Following that, the second and third shafts would be slashed and lined to 14 ft. Muck and water would be dropped to the 2130 ft level and raised in the 85 in. shaft.

This project has now been curtailed due to weak uranium prices. The 85 in. shaft is now complete and the second and third boreholes have been completed to the lining stage.

The Northfield Mountain pumped storage scheme contains two shafts, a 15 ft diameter 630 ft deep vent shaft and a 31 ft diameter 1080 ft high pressure shaft. These were excavated using 12 in. pilot holes, 6 ft raise reamers and then slashing and mucking down the hole. Problems were experienced in excessive pilot hole deviation and eventually gyroscopic sensors were used. Slashing was by both jackdrills in 10 ft lifts and a drill jumbo (Missel, 1975).

The high pressure shaft for the Dinorwic Scheme in Wales was excavated through grit and slate by first raise drilling an 8 ft bore for the full 1430 ft using a Robbins 71R machine. Average drilling rate was 1.5 ft/hr. This was followed by slashing to 33 ft diameter and mucking down the bore. Problems were initially experienced with blockages (Ellis et al, 1979; Tunnels and Tunnelling, 1981).

The F58 shaft at Mount Isa Mines, Australia is 3780 ft deep and concrete lined to 20 ft diameter. A 6 ft diameter Robbins 61R was used in a series of lifts using access at a number of levels from the existing mine. This permitted simultaneous operations with considerable advantage to the schedule. Some problems with hole deviation (up to 8 ft) necessitated hand drilling. Otherwise, slashing was carried out using a deep stage, a 6-boom drill jumbo and 12 ft rounds. Some problems with hangups were also experience.

It was concluded that the use of pilot raising with slashing and concreting is comparatively rapid, economic and safe. The considerable amount of underground development required specifically to enable raise drilling to be carried out was more than justified by time and cost savings.

The cable and pressure shafts of the Raccoon Mountain pumped storage scheme were excavated using ream-and-slash. The cable shaft is 23 ft in diameter, 1050 ft deep and lined with mesh and shotcrete. The pressure shaft is 38 ft in diameter and 890 ft deep and lined with 18 in. concrete (Kimmons, 1972).

Two surge shafts and an elevator shaft of the Helms pumped storage scheme were sunk by back-reaming to 8 or 10 ft and then slashing to the final diameter. This involved the use of a jumbo on hoist ropes and mucking down the raise. The shafts vary in diameter from 10 to 50 ft and from 600 to 1000 ft in length. The linings were slip formed using concrete hauled from the shaft bottom. Steel sets and spiling were used for temporary support in difficult ground conditions (Andersen, 1981).

In the development of the Kidd Creek No. 2 Shaft, advantage was taken of an existing nearby shaft 3000 ft high for use in raise drilling and mucking. The top 2800 ft of the 25 ft diameter, 5100 ft deep shaft in dacite/basalt was slashed to a 6 ft diameter raise bore. The sinking of the lower section using drill-and-blast methods was considerably slower. Benches were used to handle excessive water problems. Rockbolts and strapping were used as temporary support and the shaft was then lined with 12 in. thick 4000 psi concrete in 20 ft pours (McKay, 1981).

The 250 ft deep surge shaft of the Foyers Scheme in Scotland was excavated from the bottom to 9 ft diameter using an Alimak raise climber and a choked drawn. It was then enlarged to 28 ft diameter again by raising to the full height. Slashing to 64 ft diameter was carried out using two Eimco 21 overload muckers, and an access platform suspended from a winch. The chamber was then slipform concrete lined. The slashing/trimming operation allowed greater accuracy and better finish to the rock surfaces (Land and Hitchings, 1978).

The No. 3 shaft at Brunswick Mining and Smelting, Bathurst, Canada was excavated using an adjacent 7 ft diameter raise drilled shaft to hoist the muck. Extreme difficulty was experienced in drilling straight holes because of the dip of the formation weakness planes. Misalignments of 20 ft were reduced to 8 ft during the slashing process. Raise bores were used to muck the 26 ft diameter shaft for its full 4525 ft depth (Dengler and Brown, 1976).

A record breaking shaft was sunk at the Elliot Lake Mine,, Ontario in 1973 and 1974. A pilot raise 10 x 10 ft and 8 x 8 ft. was driven 1805 ft to the surface. The raise was then slashed to 25 ft diameter for a ventilation raise.

The lift, busbar and ventilation shafts of the Drakensberg Scheme in South Africa, were excavated by ream-and-slash (Int. Const., 1980). They were back-reamed using a 6 ft diameter Robbins machine. The shafts were then slashed to full diameter and concrete lined from the collar down. The 1200 ft pressure shaft was sunk by conventional methods as schedule constraints prevented timely bottom access.

4.5 GROUND IMPROVEMENT

4.5.1 Introduction

Ground improvement methods are often used for decreasing permeability and increasing the strength/stiffness characteristics of the ground around shafts. Ground freezing provides temporary ground control during construction. Grouting may be used either for temporary improvement during construction or as a method of sealing around shafts so as to permanently improve the characteristics of the ground. Grouting to seal shafts is regarded as a very important aspect of shaft design and construction especially for repository shafts. Pregrouting is one of the contributory factors that has led to the improvement of sinking speeds (Lambert, 1968).

In some types of ground such as sand, silt, soft clay, or weak sandstone, it may not be possible to construct a shaft without special procedures because the ground may squeeze or be washed into the shaft faster than it can be excavated. In other cases, freezing or grouting may allow much more rapid and safe construction than would otherwise be possible.

Grouting is used to fill cracks and voids behind the final lining of shafts. This seals the rock and greatly reduces the amount of seepage behind the lining so that moisture-sensitive rock units are protected. The stiffness of the rock and liner is also improved so that more load can be transmitted to the rock and the liner can be designed for lower capacity loads.

Particulate grout including clay, cement, and hydraulic lime were first used in the 19th century. Chemical grouts were first used in early 20th century (Lenzini and Bruss, 1975).

4.5.2 Grouting

4.5.2.1 Methodology

The design of a grouting program includes the pattern of grout holes, the sequence of injecting the different horizons, the method of injection into each hole, and the type of grout.

There are several hole patterns that may be used for grouting of soil and rock. Curtain grouting uses an aligned array of deep holes to construct a curtain or barrier. Blanket or area grouting consists of shallow holes drilled and grouted on a grid pattern to reduce the permeability of a layer of rock, such as in the wall of a tunnel or shaft. Closure or split spacing grouting consists of first grouting widely spaced primary holes then drilling and grouting additional intermediate holes at successively closer ("split") spacings until no additional grout can be injected.

There are several methods used to inject grout into the individual holes:

- In stage grouting, the holes are drilled to a shallow depth and grout is injected to refusal. The holes are then cleaned out and drilled deeper and additional grout is injected. The cycle is repeated until the desired depth is attained and the grout takes are sufficiently low.

- In series grouting, successively deeper holes are drilled and grouted until the desired depth is attained. There is no washing and reusing of holes in series grouting.

- In packer grouting, the grout holes are drilled to full depth and packers are used to isolate sections of the hole during grouting. The benefit of packer grouting is that higher pressures can be used deep in the hole with less chance of causing surface heave.

- In circuit grouting, the injection pipe extends to the bottom of the hole and the excess grout returns through the annular space between the pipe and the hole wall. The benefit of circuit grouting is that there is less chance of solids settling out and causing premature clogging of the smallest voids and fractures.

4.5.2.2 Particulate Grouts

The type of grout that is used depends on the size of the voids or fractures that must be filled, as shown on Figure 4-6. Generally, particulate grouts can be used only in coarse sand or gravel with an effective grain size of greater than about 0.03 in. Chemical grouts may be used in silt or other materials with effective grain sizes greater than about 0.0004 to 0.0008 in. (Corps of Engineers, 1973).

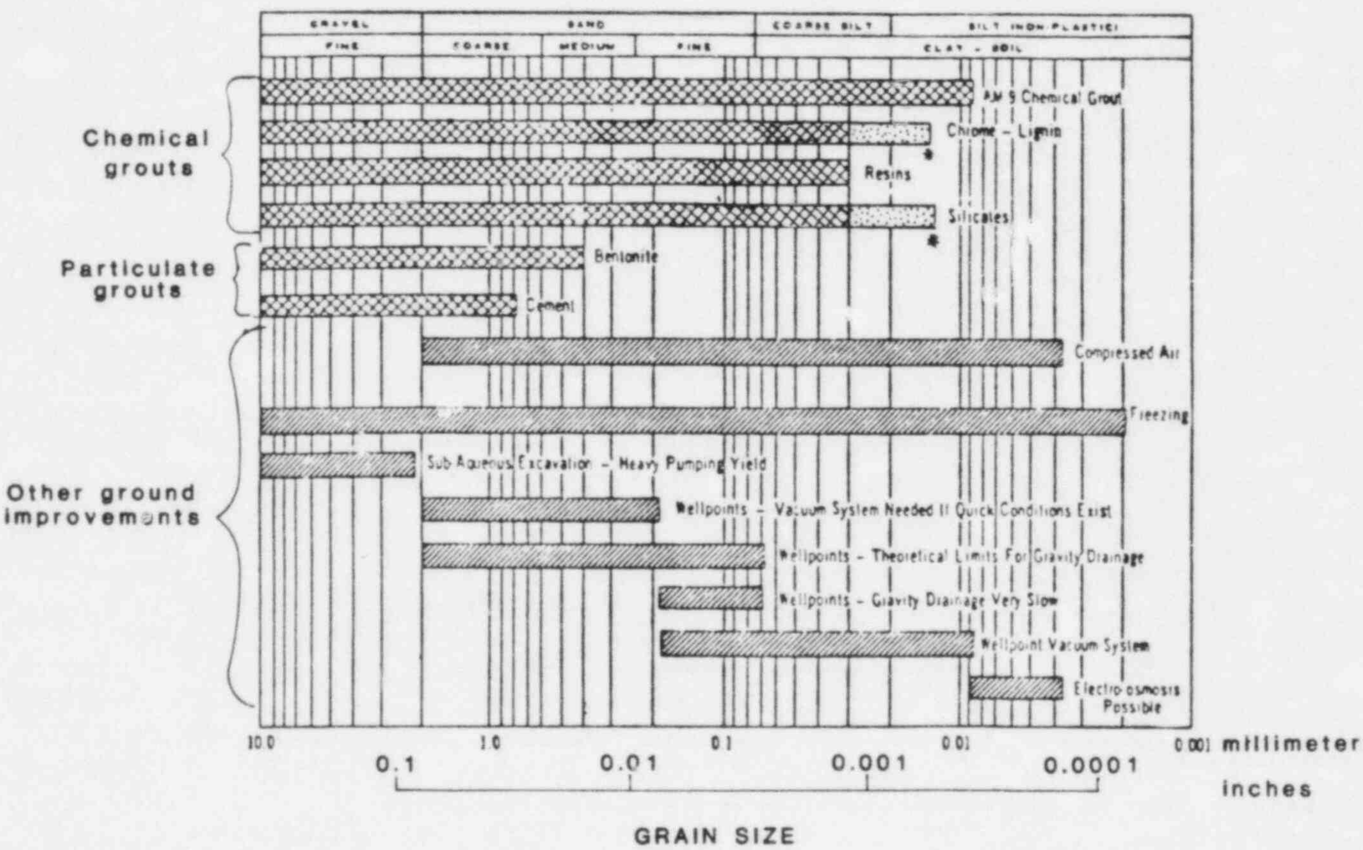
Cement grout is commonly used during shaft construction for reducing water inflows through fissures in rock. Chemical grouts are used for decreasing the permeability of relatively thin (less than about 10 ft) zones of sands and coarse silt.

Cement and clay are commonly used as particulate grouts. Fly ash, diatomaceous earth, or ground volcanic ash are often added to extend the grout. Aluminum powder may be added to cement grout to cause expansion of the grout and reduce segregation. Calcium chloride may be added to accelerate the set time of the grout. Calcium lignosulfonate may be added to fluidify the grout and retard setting.

Particulate grouts may either be directly injected into the grout holes or chemicals may be injected first to "prime the hole." Sodium silicate, colloidal silica, sodium alkyl sulfonate, and sodium laurel sulfonate can be used to prime the hole because they help lower surface tension, lubricate and they have a lyophilic effect that helps to increase the penetration of the grout.

EFFECTIVE GRAIN SIZE LIMITS FOR GROUT INJECTION

Figure 4-6



*Clarification required to extend grout to this particle size range

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4.5.2.3 Chemical Grouts

Chemical grouts were developed because of the need to stabilize and impermeabilize materials that were too fine for the use of clay or cement grouts. The oldest most commonly used chemical grout is sodium silicate-calcium chloride. The calcium and silicate ions combine to form a gel. In the two solution method, a solution of sodium silicate is first injected into the ground and followed by a solution of calcium chloride. In the one solution method, sodium silicate and calcium chloride are mixed with various retardants which delay gel time. The concentration of the solutions and the additives affect the gel strength, shrinkage, and solubility in groundwater. Gels containing a high proportion of silica have the highest strength, are longest lasting, and shrink less than those with a lower proportion of silica or those containing retardants. Sodium silicate grout has the advantages of being the least expensive, safest, and longest lasting of the chemical grouts. The main disadvantage with silicate grout is that on exposure to air, the gel starts to segregate into macromolecules resulting in shrinkage and an increase in permeability. This process is called syneresis and it occurs most rapidly in the grout containing retardants.

The acrylamide system can be used to penetrate soils with effective grain sizes as small as 0.0008 to 0.0004 in. In this system, a powder mixture of acrylamide and methylene-bisacrylamide organic monomers is dissolved in water and is polymerized using a catalyst activator and oxidizer initiator. The material is sold under the trade name AM 9. The grout is mixed as it is injected and the gel time can be adjusted by changing the proportions of the components. Acrylamide grouts are mainly used to decrease the permeability of soil or rock and it has been used to reduce the permeability of a sand from 10^{-3} cm/sec to 10^{-10} cm/sec. The greatest disadvantage of the acrylamide system is that many of the components are very hazardous to health and must be handled carefully. After the ground has been grouted, it may continue to emit fumes into adjacent underground openings. Some of the grout components may dissolve in the groundwater and present a hazard to nearby groundwater users.

Lignochrome grouts can be used for materials with an effective grain size of greater than 0.002 in. (fine sand). Lignosulfonate is oxidized with sodium dichromate to form a heavy metal precipitate that reduces the permeability of the material into which it is injected. The set time may be adjusted using ferric chloride and the pH must be adjusted using various acids. The dichromate is toxic and care must be used when handling it.

Resin grouts include epoxy and polyester. These grouts may be injected into material with an effective grain size of greater than about 0.002 in. Resin grouts increase the tensile and compressive strength of soil and rock as well as reduce the permeability.

Polyurethane foam has been used to increase strength and decrease permeability. The polyurethane is first injected into the ground and then water or carboxylic acid are injected to induce foaming. This is the most toxic of the chemical grouts and gas masks must be worn where it is used. It must not be used in confined spaces.

4.5.2.4 Grouting Program Design

All of the grouts described above have been used for construction of shafts and other underground openings. There are 12 factors that are generally considered when a grouting program is designed (Dempsey and Moller, 1970):

1) The reliability and completeness of the soils information available

2) The most practical method of introducing grout into the ground

3) The degree of permanence required of the grout

4) The possible effects on existing structures of ground movement as a result of grouting

5) The degree of saturation of the material and the erosive potential of groundwater movement

6) The chemical composition of the groundwater and/or soil which might inhibit the reaction or set of the grout constituents

7) The risk and effect of grout drying out upon exposure

8) The extent of the treatment and the spacing of injection points in order to produce the desired effect of impermeability or imparted strength

9) The toxicity of the grout components and their possible effect on groundwater or underground operations

10) The working environment in which the grouting materials have to be stored, mixed and injected

11) The justification and economics of providing intensive supervision for the more sophisticated processes

12) The availability of grouting materials, both to begin and to sustain an operation where the total requirements are unknown.

There will generally be two stages of grouting of shafts constructed for HLW repositories: grouting of the rock prior to disturbance and grouting after disturbance. Known zones of high permeability must be grouted ahead of shaft excavation to reduce the permeability of the rock adjacent to the shaft and control groundwater inflows. This may be carried out either from the surface or from the shaft bottom. After excavation, the walls of the shaft may be grouted to seal fractures that were induced by the excavation to reduce the permeability of the disturbed rock and to fill voids between the lining and rock. The requirements for these two grouting operations are different. One is designed to reduce the natural permeability and the other is designed to reduce the induced permeability. Reduction of the natural permeability of the rock is not important for the long-term performance of the repository. However, it is very important to reduce the permeability of the disturbed zone because this is one of the potential migration paths for radionuclides. Such grouting is frequently carried out as part of the construction of a watertight lining.

Grouting prior to disturbance may be performed with any grout that is compatible with the size of voids or fractures and permeability of the material that must be penetrated. The grout must also be compatible with the groundwater chemistry, ground temperature, and must be stable. Grouts, once installed, cannot be withdrawn and replaced. Therefore, grouts used during construction must also satisfy the long-term requirements of shaft sealing in terms of durability and sealing efficiency.

Grouting may be performed either systematically or as required during construction to strengthen the ground and reduce groundwater inflows. Table 4-3 summarizes typical application of grouting to shaft construction. The table shows that in general, particulate grouts were used to reduce secondary permeability (through fractures). Chemical grouts were used to reduce primary permeability (through pores). Generally, units that required grouting were identified using cored borings and water pressure tests that were drilled during the site investigation for the shaft and from pilot holes within the shaft. In most cases, the shaft was excavated to a level a few tens of feet above the unit to be grouted. A grout curtain was then formed around the shaft by inclining one or more rows of holes out from the shaft.

Grouting must be an integral part of the entire shaft construction. This is especially true for repository shafts designed and constructed under the constraints given in Chapter 3. Shaft design and the schedule for sinking and liner emplacement must accommodate the specific requirements of the grouting procedure. Design considerations can only be completed as grouting procedures and results are established.

Research is currently being performed on various materials to use for grouts in the disturbed zone (Ellison et al, 1981). These materials include cement, clay, zeolites, and organic polymers which will reduce permeability and also provide good sorptive characteristics for a long period of time. These grouts would probably be injected in a blanket pattern, using holes drilled about one shaft diameter deep. Packer grouting starting at the end of the hole, working towards the top of the hole is commonly used. Further, research is needed to determine the optimum hole pattern, hole depth, and grout injection sequence.

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EXAMPLES OF SHAFT GROUTING EXPERIENCES

Table 4-3

PROJECT	GEOLOGY	REASON	GROUT	REFERENCE
Falconbridge Mine Ontario, Canada 6.0 x 4.3 m 349 m deep	Norite Granite	Seal water-bearing joints	Cement	Gilje, 1957
Project Gnome Carlsbad, New Mexico 3.0 m Ø	Sandstone, Siltstone, Dolomite, Anhydrite, Salt	Seal water-bearing fractured dolomite	Cement	Howes, 1962
Cleveland Mine Ohio 5.5 m Ø 578 m deep	Shale Limestone Sandstone Evaporites	Seal sandstone	Sodium silicate-Calcium chloride	Bleimeister, 1964
Gasciogne Wood Selby Project England	Limestone Sandstone Coal measures	Seal fissures and vugs in limestone, Seal water-bearing sandstone	Sodium silicate and cement	Black et al, 1982
Mulga Coal Mine Alabama 5.5 m Ø 70 m deep	Shale Sandstone	Seal sandstone	Cement and rock dust	Smith and Foust, 1964
Nose Rock Mine New Mexico 4.9 to 5.5 m Ø 474 to 633 m deep	Shale Sandstone	Seal sandstone	Polymer resin	Greenslade et al, 1981
South African Gold Mine	Dolomite Lava	Seal water-bearing fissures and vugs	Cement and Bentonite	Nel, 1981
Mt. Taylor Mine New Mexico	Shale and Siltstone	Seal fractures	"Geoseal" Formaldehyde-Urea base	Engineering and Mining Journal 1980
San Manual, Arizona 3-D shaft	Conglomerate Volcanics	Reduce water inflow	Cement and Chemical grout	Hynd et al. 1976

During shaft drilling, the drilling mud will penetrate into the walls of the shaft and reduce the permeability of the rock. This mud will penetrate both natural and drilling induced fractures and will prevent effective grouting. Therefore, the muds that are used for drilling must have characteristics that will ensure long-term reduction of permeability, or be designed to rapidly degrade so that the fractures can later be treated with a more permanent grout.

4.5.3 Ground Freezing

Ground freezing was first developed in 1883 to sink shallow shafts through water-bearing overburden in Germany and England. It is used mainly for the temporary stabilization of weak, saturated ground where the material is too fine-grained for efficient grouting or where grouting is not possible because of other restrictions (problems with ground water contamination, toxicity etc.). The method involves freezing a cylinder of ground extending from the surface to below the water-bearing formation and then excavating inside the cylinder. The permanent shaft lining is then placed and the ice wall allowed to melt. During the thawing stage the freeze tubes must be sealed to prevent water migration. Heat is removed from the ground using a refrigerated liquid that is circulated from the surface through pipes installed in boreholes. The critical factors that must be considered in the design of a freezing program are the size of the holes and freeze pipes, type of refrigeration system, spacing between the pipes, and temperature of the refrigerant. These are determined by the required thickness of the frozen zone, time allowed for freezing, and the time that the frozen zone must be maintained. Ground freezing affects the design of the linings and supports because of loads that are imposed by the frozen ground and because of the thawing characteristics. The advantage of ground freezing is that it can be used in any type of material to eliminate groundwater inflows. The disadvantage of the system is that it is often very expensive and it can cause significant disturbance to the ground and also hinder geotechnical studies in the shaft walls.

Tne freeze holes are normally drilled about 4 to 6 in. in diameter and 2 to 4 in. diameter casings are installed in the holes. A plastic pipe is inserted into the casing. The circulating fluid is pumped down the plastic pipe and it returns up through the annulus between the plastic pipe and the inside of the casing.

The most common system is a freon or ammonia refrigeration system with calcium chloride brine as a circulation fluid. Methanol is occasionally used as a circulation fluid. Other liquids with lower freezing temperatures may also be used for faster freezing. Liquid nitrogen can be used without a refrigeration system by pumping it into the freeze pipes and allowing it to boil.

The hole spacing is a function of the time allowed for freezing the depth of freezing and the temperature of the circulation fluid. Closer hole spacing results in more rapid freezing. Hole spacings are typically 2 to 4 ft but not greater than 13 diameters of the freeze pipes (Schuster, 1981).

The freeze pipes are normally installed from the surface and they extend into an impermeable unit below the zone that needs stabilization. In some cases, the upper portion of the hole is insulated so that freezing occurs only in the lower part of the hole. Freezing has been performed in holes more than 2000 ft deep (Hegemann, 1981). Good hole alignment is necessary to avoid leaving unfrozen "windows." The design of the system must include determining the tolerances for hole drift and installation must include surveying the holes to ensure that they are within tolerance.

The design of the freezing system must include allowances for groundwater movement. Moving groundwater may add heat faster than it can be removed by the circulating fluid. Heat generated during construction may also induce thawing.

The design of the ice wall and initial support system is based on the strength of the frozen ground and the pressures that must be resisted before a lining is installed. The strength of frozen ground is time-dependent because ice readily creeps under a load. Strength values from short-term unconfined compressive strength tests are normally reduced by a factor of three for design. The pressures that must be resisted include hydrostatic pressure, rock pressure, and stresses induced by the freezing and resulting expansion of the porewater. The design is usually based on methods which assume a plastic yielded zone on the inside of the freeze wall. Design aspects of ground freezing for potash shaft mining in Saskatchewan presented by Ostrowski (1967) aptly illustrate the capabilities of freezing methods.

The design of the final lining is based on the stresses that must be withstood before and after thawing of the ground and on anticipated changes in the properties of the ground due to the freeze-thaw process. When the final lining is first installed, the ground is still being frozen and loads may be imposed on the lining due to the expansion of the porewater on freezing and the continual creep of the ice wall. (Braithwaite, 1970). When the ice wall is thawed, hydrostatic pressure will be imposed on the lining. In addition, uneven thawing of the ice wall could induce bending moments in the lining.

The changes in characteristics of the ground are due to formation of ice lenses and the expansion of the pore water during freezing. Silts and fine sands are most susceptible to formation of large ice lenses during freezing because they possess high capillarity with moderate permeability. When the ice lenses thaw, the silt or fine sand is supersaturated and has very little strength. Less disturbance occurs in coarse-grained materials.

Fine-grained, plastic soils with natural moisture contents above the plastic limit may also be damaged by freezing and thawing. Freezing causes consolidation of the soil and formation of small ice lenses parallel to the freeze pipes. When the soil is thawed, the water drains off and the soil has a water content lower than the initial moisture content, resulting in an overconsolidation of the soil. This can result in significant settlement. In addition, the vertical and horizontal permeability of the soil is permanently increased by several orders of magnitude (Chamberlain, 1981). Shaft linings must be designed for this consolidation and the resulting downdrag. In addition, structures adjacent to the shaft may be damaged by the differential settlements.

4.6 SUMMARY

Of the various shaft sinking methods introduced in this Chapter, five methods are considered to be the most promising in terms of application to repository development. There are:

- Drill-and-blast
- Ream-and-slash
- Blind rotary (boring)
- Blind rotary (drilling)
- Back-reaming.

The technical characteristics of each of these methods are summarized in Tables 4-4 to 4-8. The significance of each of these factors cannot be included on these tables without reference to a set of specific geological conditions and shaft design criteria. Such on evaluation is presented in Chapter 7.

SUMMARY REVIEW OF TECHNICAL FACTORS DRILL-AND-BLAST

Table 4-4

ADVANTAGES	DISADVANTAGES
Technology/Geometry Limitations	
• Proven technology • Unlimited depth or diamater • Except for certain geologies, outcome fairly predictable • Can be sunk blind	• Future advances and improvements likely to be minimal
Geological Conditions	
• Can cope with hardest rocks • Can handle most reasonable water inflows • Can deal efficiently with support requirements for most fractured, blocky, swelling rocks	• Difficult to cope effectively with large aquifers
Investigation/Preparation Requirements	
• Possible to probe and selectively grout during sinking	
Impact on Formation/Groundwater/Sealability	
	• Considerable damage to formation, leaving disturbed/stressed zone which may be difficult to grout/seal.
Facility for Investigations	
• Good scope for inspection and testing using cut-outs, boreholes, etc. prior to lining.	
Construction Factors	
• Alignment control not a problem • Station breakouts simple • Lining concurrent with sinking • Lining design, grouting arrangements more flexible, impliment as necessary • Good quality control possible • Simultaneous equiping of shaft	• Induces overbreak in formation requiring more grout and concrete and muck disposal • Advance rates slowest of all methods
Safety	
	• Hazard exposure to men in shaft bottom a serious problem
Miscellaneous	
• Costs predictable	• Labor intensive and difficult to obtain competent miners • Future cost trends appear unfavorable • High cost per foot

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SUMMARY REVIEW OF TECHNICAL FACTORS REAM-AND-SLASH

Table 4-5

ADVANTAGES	DISADVANTAGES
Technology/Geometry Limitations	
• Proven technology • Unlimited diameter • Feasability generally predictable	• Future advances and improvements minimal • Requires bottom access for mucking • Depth limited by length of raise bore
Geological Conditions	
• Can cope with most hard rocks	• Serious limitations on rock and groundwater conditions because raise must stand unsupported. • Could lead to high water inflows into repository unless grouted or frozen prior to sinking
Investigation/Preparation Requirements	
	• Requirements for pre-sinking design/feasibility greater than for drill-and-blast and large diameter drilling. • Need for grouting or freezing often greater than for blind methods
Impact on Formation/Groundwater/Sealability	
	• Damage to formation making it necessary to be grouted and sealed later.
Facility for Investigations	
• Mapping, inspection, testing and design adjustments readily carried out during sinking	
Construction Factors	
• Alignment deviation less of a problem than with back-reaming • Station breakouts simple • Design-as-you-go possible • Good construction control possible • Simultaneous equiping of shaft • Line concurrent with sinking	• Overbreak requires more haulage, grout, concrete. • Advance rates slower than mechanical methods but generally faster than drill-and-blast
Safety	
	• Hazard exposure to men in shaft a serious problem
Miscellaneous	
• Costs predictable	• May interfere with underground operations • Labor intensive and difficult to obtain competent miners

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SUMMARY REVIEW OF TECHNICAL FACTORS BLIND ROTARY (BORING)

Table 4-6

ADVANTAGES	DISADVANTAGES
Technology/Geometry Limitations	
• Theoretically no limits on shaft diameter and depth • Can be sunk blind • Major technological advances expected in the far future	• Technology very limited and not proven
Geological Conditions	
	• Prediction of feasibility difficult • Of all methods except back-ream, least able to cope with adverse geology
Investigation/Preparation Requirements	
	• Requires thorough investigation to check beforehand on potential problems (Diggability, muck, water inflow, stability) • Complete groundwater control necessary
Impact on Formation/Groundwater/Sealability	
• Minimum formation damage and destressing • Good sealability	• Difficult pregrouting ahead of the face
Facility for Investigations	
• Reasonable access to shaft wall for inspection, testing and design validation	
Construction Factors	
• Potentially very high penetration rates (unlimited torque and thrust) • Good alignment control • Lining concurrent with advance of machine • Improved stability as a result of smooth surface • Reduced need for shaft and aquifer seals	• Muck handling problem unresolved • Station breakouts difficult • Inclined beds or sand and gravel interbeds may present alignment and machine stability problems
Safety	
• Limited hazard exposure. In difficult ground, may use remote control	
Miscellaneous	
• Costs per foot potentially low	• Very high initial capital cost

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SUMMARY REVIEW OF TECHNICAL FACTORS BLIND ROTARY (DRILLING)

Table 4-7

ADVANTAGES	DISADVANTAGES
Technology/Geometry Limitations	
• Technology available for proposed sha. diameters • Significant advances in technology, costs, etc. expected in near future • Cost and schedule outcome reasonable predictable • Shaft can be drilled blind.	• Technology not proven for very deep shafts
Geological Conditions	
• Can cope with almost all rock conditions (hardness, fracturing) • Readily handles large high pressure or artesian aquifers	
Investigation/Preparation Requirements	
• Pregrouting dewatering or freezing unnecessary except at collar	
Impact on Formation/Groundwater/Sealability	
• Minimal formation damage (also improves long term functioning) • Excellent control of groundwater • Good sealability	• Influence of mud on sealability needs further study
Facility for Investigations	
	• No inspection for design purposes during sinking or lining • Investigations only possible after lining using boreholes
Construction Factors	
• High penetration rate-up to 3 times drill-and-blast • Generally no stability problems during construction • Fewer shaft and aquifer seals required	• Lining not concurrent with sinking • Long lead time on equipment • Control of verticality marginal • Slow progress likely in very hard rock • Shaft equiping possible only after lining • Must have uniform rock excavation diameter
Safety	
• Workers not in shaft before structural completion	
Miscellaneous	
	• High lining costs for currently available technology • High initial capital cost for equipment, mud

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SUMMARY REVIEW OF TECHNICAL FACTORS
BACK-REAMING

Table 4-8

ADVANTAGES	DISADVANTAGES
Technology/Geometry Limitations	
• Technology proven • Modest improvements in near future • Feasibility generally predictable	• Depth limited to torque shaft requirements • No previous experience with deep shafts • Requires bottom access
Geological Conditions	
• Rock hardness usually not a problem	• Serious limitations on rock mass and groundwater conditions because raise must stand unsupported • Could lead to uncontrollable inflows unless extensive grouting or freezing used.
Investigation/Preparation Requirements	
	• Cautious verifications using pre-sinking investigatoin required • Often requires extensive grouting or freezing
Impact on Formation/Groundwater/Sealability	
• Minimum formation damage/disturbance • Good sealability	• Preventative control of groundwater difficult or expensive
Facility for Investigations	
	• Testing may be affected by frozen ground • Inspection and testing prior to lining difficult in practice
Construction Factors	
• Relative good penetration rates • Possible to stage ream to vary excavation diamater with depth	• Torque factor limits penetration rates at large diameters • Lining not concurrent with advance • Verticality requires very close control (may be problematical) • Station breakouts difficult
Safety	
• No hazard exposure in shaft	
Miscellaneous	
• Very favorable for good formations in small depth/diameter combination • Lining costs relatively low	• Costs unpredictable

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5.0 REPRESENTATION OF GEOLOGY/GEOHYDROLOGY

5.1 INTRODUCTION

As noted earler, the definition and comparative evaluation of the three shaft sinking methods will be performed on the basis of typical geological profiles, one for each of the two composite media. The synthesis of these geological profiles and conditions is carried out in this chapter. The approach adopted is as follows.

The geological and geohydrological characteristics of each of the five media (basalt, granite, tuff, and bedded and domal salt) are reviewed with particular emphasis on those properties influencing the design and construction of the shaft in both the repository host rock and the overburden region through which the shaft will pass. The characteristics are broadly categorized as:

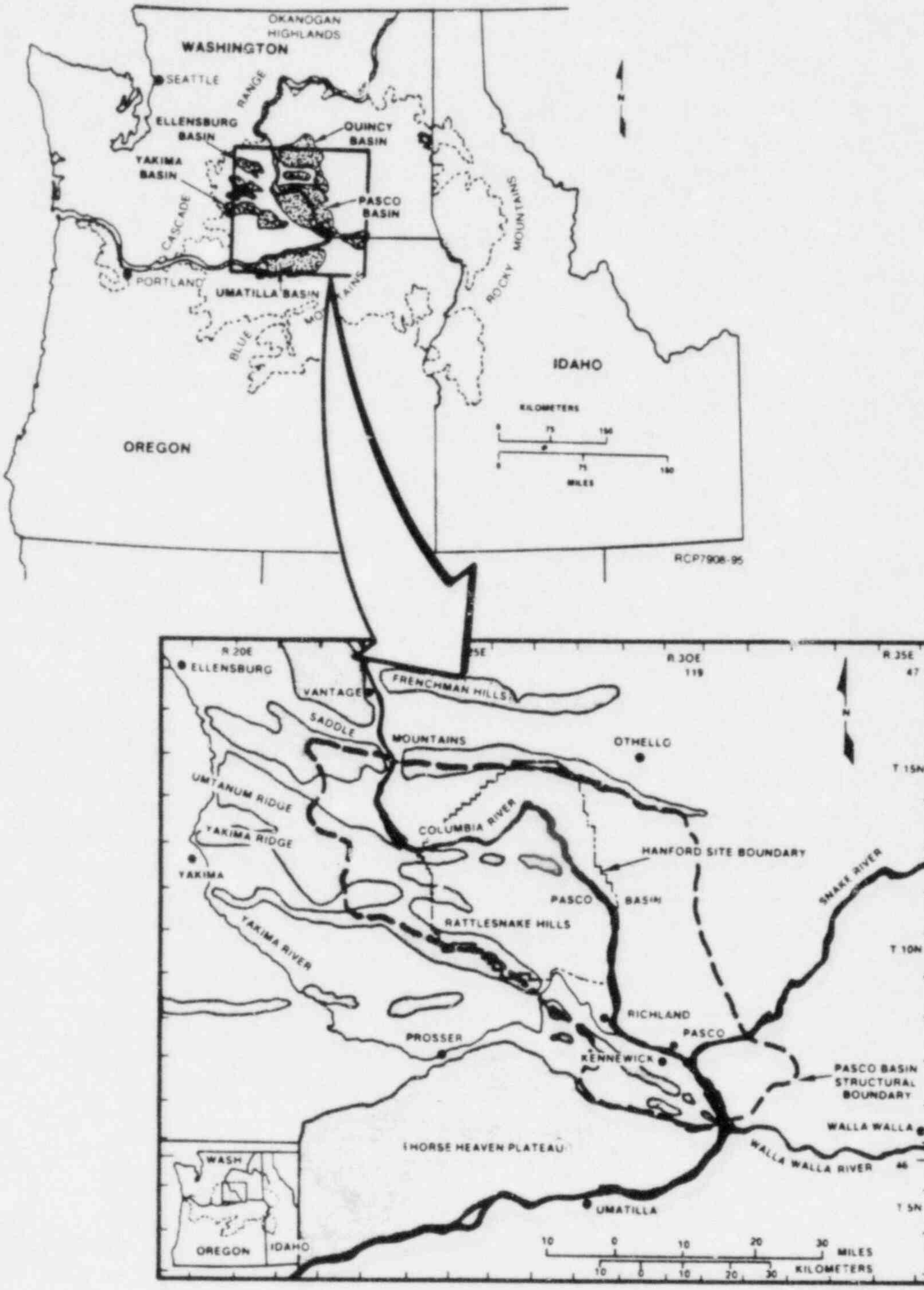
- Regional geology
- Stratigraphy
- Structural features
- In situ stresses
- Groundwater flow and permeability
- Mechanical properties including hardness.

The geological conditions chosen as the basis for the study exercise a controlling influence on the shaft design and consequently on the cost and favorability of using any particular method of shaft sinking. For this reason, detailed attention has been given to the review of geological conditions and to the synthesis of the composite media profiles for hard rock and salt. These various reviews are presented in summary form together with the derived profiles for hard rock and salt in Sections 5.2 and 5.3. While these derived profiles are considered the most appropriate for use in the present study, it must be cautioned that a considerable degree of uncertainty exists with regard to actual conditions likely to be encountered at any one site. Thus, the conclusions reached in this study will need to be revised after site investigation studies of a proposed shaft site have been completed.

It is considered essential that because of the wide variety of geological conditions possible for any one medium, the data obtained should be as site specific as possible. Thus, the data for basalt is based on the geology of the Hanford site while that for tuff is based on the Nevada Test Site. Only a specific region is available for bedded salt, i.e., the Palo Duro Basin, while for domal salt and granite, potential host regions are relied upon for information.

LOCATION MAP, COLUMBIA RIVER BASALT
PASCO BASIN, HANFORD SITE

Figure 5-1



After Myers and Price, 1979

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For the less specific areas, the quantity and quality of relevant data is much reduced and commensurately, these media, namely, granite, and domal salt are of secondary influence on the synthesis of the composite media geological profiles and conditions presented in the following sections.

From the review of geological conditions for each of the five media: basalt, granite, tuff and bedded and domal salt, typical geological profiles are drawn up which are considered to be representative of conditions for "hard rock" and "salt." It is emphasized that these two composite media, designed to be practically meaningful in terms of construction implications, are essentially vehicles on which to base the comparative study of the various sinking methods.

The implications of the results of the geological reviews with respect to shaft design and construction, are discussed in Chapter 6 as part of the detailed shaft design. These implications are:

- Required presinking investigations and preparations
- eneral competency and permissible construction techniques
- robable inflow rates and freezing and grouting requirements
- Construction support requirements
- Final lining design
- Sealing measures.

5.2 HARD ROCK (BASALT - GRANITE - TUFF) COMPOSITE MEDIUM

5.2.1 BASALT

5.2.1.1 Introduction

Basalt is one of several geologic media being considered for placement of high level nuclear waste. Particularly, the flood basalts of the Columia Plateau in eastern Washington are being evaluated by the Department of Energy (Figure 5-1). The following review is based upon investigative work conducted in the basalts at the Hanford Reservation within the Pasco Basin for Rockwell Hanford Operations organization and on previous reports by Golder Associates as referred herein. Currently, the Umtanum flow, which lies from 2950 to 3780 ft beneath the surface in the Cold Creek Syncline area is considered the leading candidate repository horizon (Myers and Price, 1981).

5.2.1.2 Stratigraphy

The Columbia River Basalt Group covers southeastern Washington and adjoining portions of northern Oregon and western Idaho comprising an area of approximately 78,000 sq miles and having an estimated volume of 41,000 cubic miles.

The flows of the Columbia River Basalt Group are interbedded with and overlapped by Miocene-Pliocene epiclastic and volcaniclastic sediments, especially along the margin of the province. The youngest suprabasalt sedimentary units on the plateau are fluvial, lacustrine, glaciofluvial, and eolian deposits of Pliocene to Holocene age. However, localized accumulations of Pliocene to Pleistocene lavas are also present within the western and southern portions of the province. As may be seen in Figure 5-2, the Umtanum flow is a subunit of the Grande Ronde Basalt. The Grande Ronde is overlain by the Wanapum Basalt which consists of three members in the Pasco Basin: the Frenchman Springs, Roza, and Priest Rapids. The Vantage Sandstone separates this formation from the underlying Grande Ronde Basalt. Overlying the Wanapum Basalt is the Saddle Mountain Basalt, consisting of seven members in the Pasco Basin: the Umatilla, Wilbur Creek, Asotin, Esquatzel, Romona, Elephant Mountain, and Ice Harbor. With the exception of the Wilbur Creek Member, all members are present in the Cold Creek syncline area.

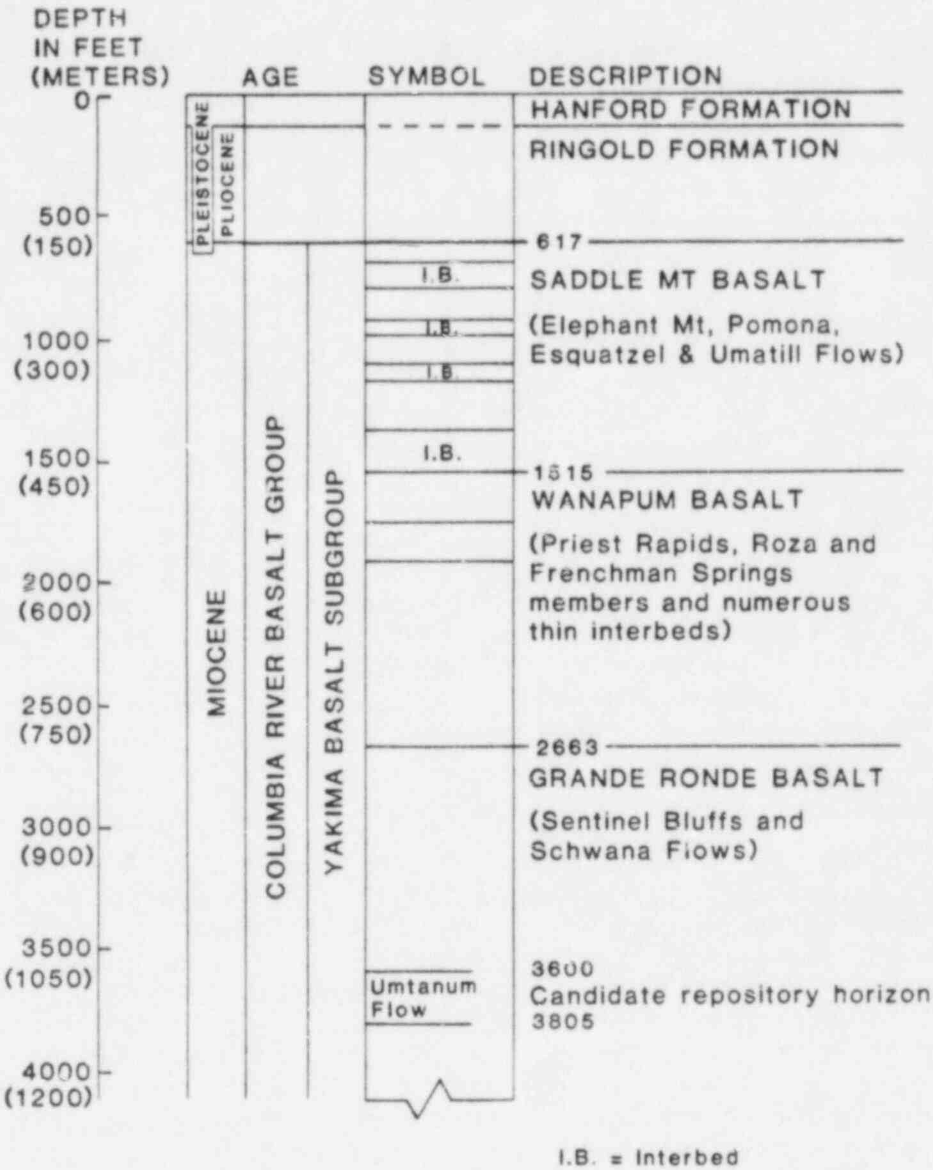
Intercalated with and overlying the flows of the Columbia River Basalt Group are sedimentary beds of the Ellensburg Formation. Within the Pasco Basin, Ellensburg sediments are interbedded in the Wanapum and Saddle Mountain Basalts. The lateral extent and thickness of the sediments generally increase upward in the section. These interbeds are generally fine-grained sandy, clayey, tuffaceous, or diatomaceous rocks.

The post-Columbia River Basalt Group sediments of the Cold Creek syncline are composed of two major units: 1) the Ringold Formation, a Miocene-Pliocene fluvial unit with some lacustrine facies, and 2) the Pleistocene glaciofluvial sediments, informally termed the Hanford Formation. The Ringold Formation in the vicinity of the proposed repository is approximately 1150 ft thick. The Ringold formation consists of a basal, lower, middle, and upper units. The basalt unit (150 ft thick) is primarily a gravel supported by a coarse to fine sand matrix. The lower Ringold (350 ft thick) is a silty, coarse to medium sand to silty sand. The middle Ringold (350 ft thick) is composed of well-rounded pebbles and small cobbles. The unit is moderately to well indurated. The upper Ringold is largely eroded in the vicinity of the proposed repository site, but some unknown thickness has been reported in drill holes near the 200 W Area of Hanford (Myers and Price, 1979).

Overlying the Ringold Formation is the Hanford Formation composed mainly of gravel and sand which may vary in thickness from 110 to 200 ft. These deposits are generally poorly consolidated.

TYPICAL STRATIGRAPHIC COLUMN NEAR THE CANDIDATE REPOSITORY SITE IN BASALT AT HANFORD

Figure 5-2



Modified from Myers and Price, 1979

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5.2.1.3 Structural Properties of Basalt

Structural features which could have an influence on the design and construction of a repository shaft include:

- Bedding (flow edges)
- Discontinuities
  - joints and fractures
  - faults/shears.

Bedding

The basalts of the Pasco Basin region consists of a complex of interlayered basalt flows and interbed sediments of clay, silt, sand, and gravel. Individual basalt flows may range in thickness from a few inches to nearly 300 ft with an average thickness of 90 to 120 ft. Sedimentary interbeds may vary in thickness from 30 to 80 ft.

Discontinuities

Little data is available on jointing and fracturing in the basalts overlying the Grande Ronde Basalt. Work to date has concentrated on the Grande Ronde, as it contains the primary candidate repository horizon, the Umtanum flow (Myers and Price, 1981). Joint and fracture data for the basalts overlying the Grande Ronde is expected to be similar to that within the Grande Ronde Basalt.

Jointing/Fracturing

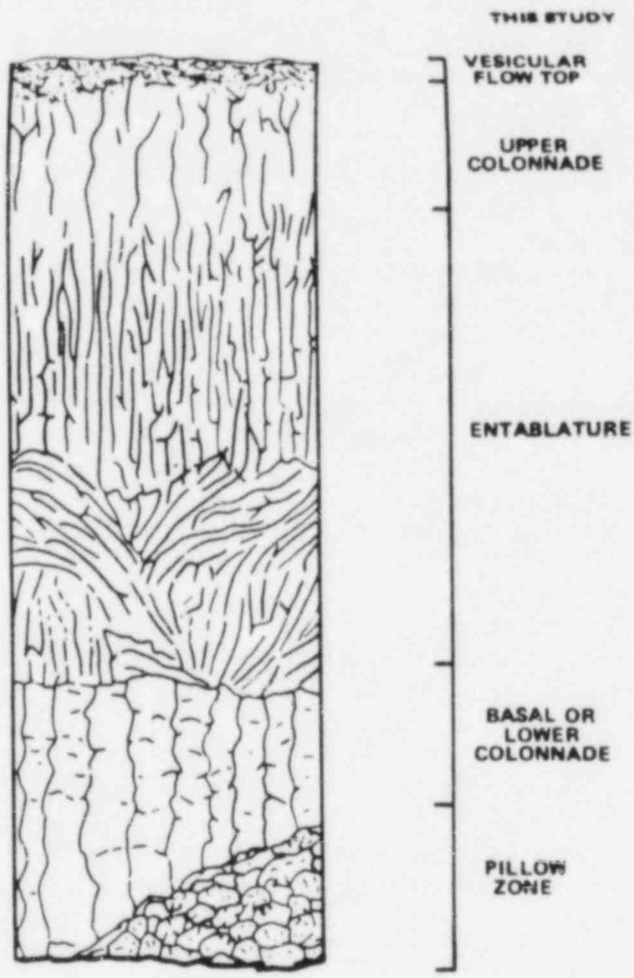
The cooling of an individual flow inward from both the top and bottom surfaces produces two distinct layers. The lower layer, called the colonnade, is characterized by columnar joints which develop by shrinkage as the lava cools and solidifies. The upper layer, termed the entablature, consists of irregular columns, 2 ft or less in diameter. Columns within the colonnade are typically 2 ft or more in diameter. In some flows, columns within the entablature may be inclined, horizontal, or radiate downward in peculiar fan-like structures (Figure 5-3). Overlying the entablature is the flow top, which is typically vesicular in nature due to the migration of dissolved gases within the lava to the top of the flow. The flow top may be brecciated from migration of the still molten lava beneath the rapidly cooled crust. Originally the vesicles may not have been interconnected, but brecciation from continued movement may have created connected openings.

The orientations of fractures in the Grande Ronde Basalt are directly related to the type of intraflow structure. Typical intraflow structures as described by Myers and Price (1981) include:

- A ropy to brecciated vesicular flow top

TYPICAL INTRAFLOW STRUCTURES PRESENT IN A BASALT FLOW

Figure 5-3



After Myers and Price, 1981

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- Upper colonnade with relatively large (2 to 7 ft in diameter) irregular columns, with or without vesicles
- Entablature consisting of relatively small, hackly to regular (0.6 to 3 ft in diameter) columns
- Lower colonnade consisting of well-formed to irregular or massive, large (1.5 to 5 ft in diameter) columns
- A glassy basal zone that varies greatly in thickness and may be highly fractured, vesicular, or pillowed.

Flow tops are characterized by randomly oriented fractures. Upper colonnade zones consist of vertical fractures, generally forming irregular polygons up to 7 ft in diameter. Entablature zones are characterized by polygonal columns in various orientations from horizontal to fanning arrays. Randomly oriented, hackly fracture zones are also typical of the entablature. Lower colonnade zones are defined by generally vertical fractures forming regular polygons. Basal zones are typically cut by numerous, randomly oriented, discontinuous fractures. The dominant joint orientation is vertical.

Horizontal and subhorizontal fractures in the colonnade are generally confined to individual columns and thus, should be less than 3 ft in length.

In situ fracture apertures for both filled and unfilled fractures vary from less than 0.004 to 0.008 in. (Myers and Price, 1981). Unfilled fracture aperture widths do not exceed 0.012 in.

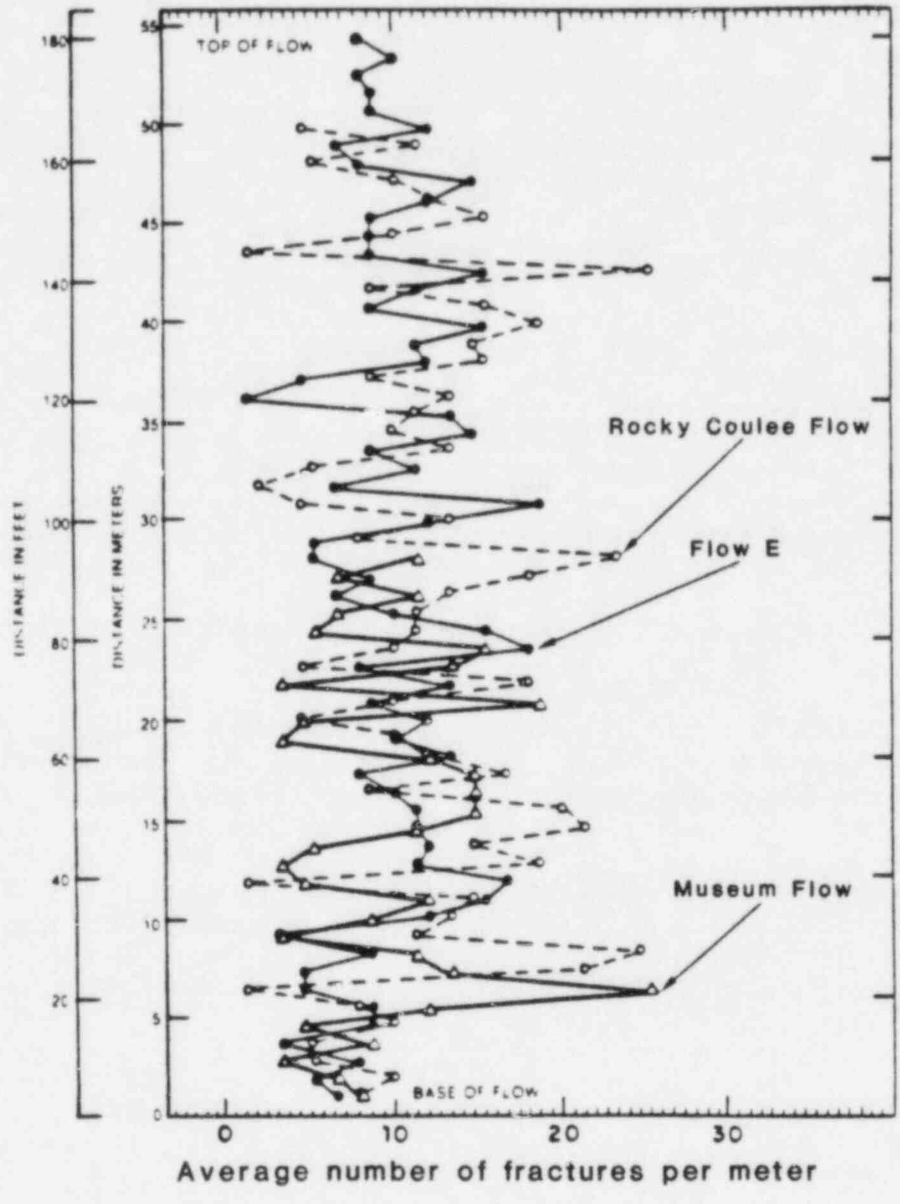
Fracture frequency of the Grande Ronde Basalt has been studied by Myers and Price (1981). Structural logging data for a typical Grande Ronde Basalt flow are shown in Figure 5-4. A variation of fracture density with the type of intraflow structure is evident. Within the entablature zone, averages fracture frequencies are 3 fractures per ft and the colonnade zone averages approximately 2 to 5 fractures per ft.

The continuity of fractures and joints within the Grande Ronde Basalt is related to the intraflow structures. Intraflow structures are generally traceable for distances of up to one mile (Myers and Price, 1981) and found to be unpredictable on the order of tens of miles.

Abrupt lateral changes in intraflow structures as well as zones of anomalous fracturing crossing flow boundaries have been noted. Fracturing and jointing patterns which characterize the intraflow structures are discontinuous. Studies done during the construction of the Near Surface Test Facility in the Pomona basalt flow (Moak and Wintczak, 1980) best describe the continuity of joints and fractures.

FRACTURE FREQUENCY -
GRANDE RONDE BASALT FLOW

Figure 5-4



Note: Data taken from core hole DH-105

After Myers and Price, 1979

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Faulting/Shearing

Two primary types of faulting or shearing are generally recognized in the Pasco Basin; one is nontectonic or emplacement features and the second are tectonic features.

From studies of thousands of feet of cored holes drilled in the Cold Creek syncline area, tectonic fractures were found to be infrequent. The breccia zones that were identified are generally intact and less than 4 in. thick. They appear in all deep core holes within the Hanford Site and are principally in the Grande Ronde and Wanapum Basalts. None of the tectonic breccias examined were judged as being associated with large displacements (Myers and Price, 1981).

No major structural features have been identified in the immediate vicinity of the proposed repository site.

Tectonic breccia zones less than 4 in. thick have been encountered in all deep drill holes in the Pasco Basin. These are particularly ubiquitous in the Grande Ronde Basalt. These faults are interpreted as strain features typical of folded basalts, and should be expected within the limbs of the Cold Creek Syncline.

Tension fractures 1 in. wide and 50 ft long have been observed in the Umtanum flow (Myers and Price, 1979).

The lengths of most known faults in the basin are less than or equal to the length of the anticlines in which they occur. Anticlinal structures in the Pasco Basin are on the order of tens of miles in length.

The type of minor faults expected to occur in the vicinity of the reference repository site are small shear zones related to fold flexures. Shear zones of this type are not expected to be continuous for distances over 300 ft.

5.2.1.4 Geochemistry

The importance of mineralogy to the design and construction of a repository shaft lies primarily with the secondary minerals. Diagenesis of the Grande Ronde Basalt has produced secondary minerals along fractures, in vugs, and in relatively porous, vesicular rock. These secondary minerals are predominantly smectite clays, zeolites (clinoptilolite), and quartz (Myers and Price, 1981).

Smectite clays are more abundant in fractures than in vugs. The abundance of secondary minerals in fractures relative to the total rock volume is between 0.3 and 0.4 percent. The volume percent of vesicle fillings is nil in the interior of the flow, but increases to as much as 20 percent by volume in the flow top and vesicular base. These expansive clays are of special significance to the design and construction of shafts.

5.2.1.5 Geohydrology

The hydrologic units have been defined as the suprabasalt sediments (unconfined aquifer), the Saddle Mountain Basalt (including the Mabton interbed), and the Grande Ronde Basalt (Figure 5-2). Each basalt formation is characterized by two types of zones, one being the dense interior of the flows and the other the interbed and interflow portions of the formation.

Hydraulic Conductivity

Values of horizontal hydraulic conductivity (Kh) determined are not representative of a rock mass and the values utilized to arrive at equivalent homogenous permeabilities herein may not be representative of the bulk system behavior. Table 5-1 lists the preliminary best estimates of equivalent homogenous horizontal hydraulic conductivities.

Hydraulic conductivity of the basalt units is expected to be more uniform than the conductivity of the overlying unconfined aquifer.

No field measurements of vertical hydraulic conductivity (Kv) of the Columbia Plateau basalts are available. Vertical fracturing of basalt typically results in a Kv which may be higher than Kh. For example, Rockwell Hanford Operations assumed Kv to be ten times greater than Kh for some of their groundwater modeling studies. Sedimentary units are typically characterized by lower Kv than Kh with a typical Kh to Kv ratio being 5 to 1.

Specific Yield

No published values for the specific yield of the unconfined aquifer have been found. Deju and Fecht (1979) report porosity values of the overburden as: 0.05 to 0.20 for the glaciofluvial sediments which are usually positioned above the water table; less than 0.12 for the unconfined aquifer (Ringold sediments) and 0.05 to 0.20 for the confined aquifer (Lower Ringold). As the upper and middle Ringold constitute the bulk of the unconfined aquifer with a porosity of something less than 0.12 (assume 0.10) and assuming a specific retention of 0.05 an estimate of 0.05 is made for specific yield of the unconfined aquifer.

Specific Storage

Most of the literature reports storage coefficients without mention of the thickness of the zone considered. In dense, rigid, basalt formations, the measured values of specific storage are subject to large errors. For this reason the selected values of specific storage (Table 5-2) are based on the generic and computed data reported in Summers et al, (1978) Gephart et al, (1979) and Golder Associates (1981).

PRELIMINARY BEST ESTIMATES OF EQUIVALENT HOMOGENEOUS HORIZONTAL HYDRAULIC CONDUCTIVITY

Table 5-1

Hydrologic Unit	K (cm/sec)
Unconfined Aquifer	2×10^{-2}
Saddle Mountains (Dense Zones)	2×10^{-5}
Saddle Mountains (Interbed and Interflow Zones)	3×10^{-3}
Wanapum (Dense Zones)	5×10^{-7}
Wanapum (Interbed and Interflow Zones)	2×10^{-3}
Grande Ronde (Dense Zones)	1×10^{-7}
Grande Ronde (Interbed and Interflow Zones)	3×10^{-5}

after Golder Associates, 1981

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BEST ESTIMATE FOR STORAGE PARAMETERS OF HYDROLOGIC UNITS IN THE PASCO BASIN

Table 5-2

Unit	Specific Yield	Specific Storage
Unconfined Aquifer	0.10	(cms^{-1})
Saddle Mountain		
(Interbed-Interflow)	--	5×10^{-8}
(Dense Zones)		9×10^{-9}
Wanapum		
(Interbed-Interflow)	--	4×10^{-8}
(Dense Zones)		6×10^{-9}
Grande Ronde		
(Interbed-Interflow	--	3×10^{-8}
(Dense Zones)		3×10^{-9}

after Summers, et al, 1978 ; Gephart, et al, 1979 ; Golder, 1981

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Total Porosity/Effective Porosity

The best estimates for porosity of the hydrologic units in the Pasco Basin are given in Table 5-3 based on Deju and Fecht (1979) and La Sala and Doty (1971) as discussed in Golder Associates (1981). Permeability measurements indicate porosity decreases with depth. Hence, a range of average total porosity was selected for each subunit type (i.e., interflow-interbed, dense) and a linear decrease of porosity with depth was assumed.

Permeability

La Sala and Doty (1971), estimated permeabilities assuming a correlation between geophysical log and hydraulic properties. Values for typical lithologic units given in Table 5-4 indicate that intact basalt is for present practical purposes, impermeable. However, breccias at basalt floor boundaries will almost certainly have much higher permeabilities. Significant groundwater inflow problems may be encountered at these boundaries and should be allowed for in-shaft design. The additional occurrence of interbed sediments at these boundaries could present a complication to shaft stability.

Hydraulic Gradient

Regional and basin scale data have been used to define the horizontal hydraulic gradients (Golder Associates, 1981). Regional scale data for the Wanapum and Grande Ronde Basalts are speculative, due to the limited number of measurements near the proposed repository site. The horizontal gradients for the various hydrologic units have been estimated as:

Hydrologic Unit	Horizontal Hydraulic Gradient (Ih)
Unconfined	1 to 3 x 10-3
Saddle Mtns. Basalt	3 to 5 x 10-3
Wanapum and Grande Ronde Basalts	5 x 10-3

The flow direction is to the northeast.

Vertical distributions in hydraulic head (I_v) have been measured in several boreholes near the proposed repository site (see Golder Associates, 1981). Based on this data, the following vertical hydraulic gradients are estimated as follows:

BEST ESTIMATES FOR POROSITY OF HYDROLOGIC UNITS IN THE PASCO BASIN

Table 5-3

Unit	Average Total Porosity (%)	Average Effective Porosity (%)
Unconfined Aquifer	10.0	5.0
Saddle Mountains		
(interbed-Interflow)	15.0	4.0
(Dense)	3.0	0.1
Wanapum		
(Interbed-Interflow)	13.0	3.0
(Dense)	2.0	0.05
Grande Ronde		
(Interbed-Interflow)	10.0	2.5
(Dense)	1.0	0.01
(Dense Umtanum)	< 1.0	0.008

after Deju and Fecht, 1979; La Sala and Doty, 1971 ; and Golder, 1981

Rev. Dwg. No. 83-1164D Date 4-82 Eng. LG

PERMEABILITY VALUES FOR TYPICAL LITHOLOGIC UNITS

Table 5-4

Description	Permeability (cm/sec)	Estimated Effective Porosity
Dense basalt above 300 m	1×10^{-5}	1%
Dense basalt below 300 m	1×10^{-6} to 5×10^{-7}	1%
Vesicular Basalt	3×10^{-6} to ? $\times 10^{-7}$	5%
Fractured or weathered	2.4×10^{-3} to 1×10^{-5}	10%

after La Sala and Doty, 1971

Rev. ______ Dwg. No. 83-1164D 042 Date 4-82 Eng. L.G.

Hydrologic Unit	Vertical Hydraulic Gradient (Iv)
Basalt units above Umtanum unit:	1 x 10-3
Umtanum unit:	3 x 10-3
Basalt units below Umtanum unit:	7 x 10-4

Vertical hydraulic gradients indicate upward flow in all basalt units.

5.2.1.6 Mechanical Properties

Little data are available on the mechanical properties of the Columbia Plateau Basalts overlying the Grand Ronde Basalt in the Pasco Basin. Discussions of the mechanical properties of the Grande Ronde basalt are found in Myers and Price (1981) and Golder Associates (1981).

Table 5-5 summarizes the expected range of values for various mechanical characteristics of Columbia Plateau Basalts. The rock mass properties of basalt, particularly the strength, are dominated by the joints and fractures because these are the weakest part of the rock mass.

Data on in situ stresses other than at the repository level are not available. Vertical stresses are estimated to be approximately equal to the theoretical overburden pressure i.e., 4200 to 4640 psi at the repository level of 3700 ft below the ground surface. A qualitative estimate of horizontal in situ stresses at this depth is approximate only and is based on the occurrence of discing. Horizontal/vertical stress ratios in the range 1 to 3 are indicated (Golder Associates, 1981). Certainly, stress ratios of the order of 2 should be considered in the design of the shaft sinking operations.

5.2.2 GRANITE

5.2.2.1 Introduction

The majority of repository siting studies to date in the United States have concentrated on media other than granite. Little specific information on geological conditions for granite is available. Furthermore, because no particular region has been chosen as a possible location, it has been necessary to base the data on a number of large granitic intrusions within the United States, including the Precambrian Shield, Appalachian, Sierra Nevada and Pikes Peak regions. The information obtained from these areas is thus essentially generic. Geological conditions for the shaft section within the granite are well documented and can be fairly well defined. However, the variability of the overburden rocks and the limited data render a corresponding description of conditions in the upper portion of the shaft more difficult.

MECHANICAL PROPERTIES OF THE COLUMBIA PLATEAU BASALTS

Table 5-5

Property	Average Value
Density	2.8 g/cc
Young's Modulus	9830 ksi
Poisson's Ratio	0.26
Cohesion	4640 psi
Uniaxial Compressive Strength	29000 psi
Angle of Internal Friction	55°
Tensile Strength	2030 psi

after Kaiser Engineers, 1978

Rev. Dwg. No. 813-1164D Date 4-82 Eng. [illegible]

In spite of the difficulties, it is probable that the batholith chosen for shaft sinking as a potential repository site will, for purposes of expediency and ease of construction, feature a relatively small depth of overburden. Thus, for the purposes of this study, it has been assumed that, except for the normal provisions for collaring and preshafting, the overburden conditions do not present a major influence on shaft design and construction.

The following data pertains to granitic rock which is assumed to occupy the larger portion of the shaft depth.

5.2.2.2 Stratigraphy

Plutons are massive bodies of intrusive rocks which solidify beneath the earth's surface. Generally, plutons are too small or discontinuous to be considered as a repository for high level nuclear waste. The largest of plutons, i.e., batholiths, are generally granitic in composition. Most batholiths are emplaced in mountain belts and are a result of numerous plutonic events. The locations of the major intrusive bodies are shown in Figure 5-5. The intruded country rock surrounding this molten mass may be metamorphosed and deformed. If the plutonic event is forceful, fracturing and faulting occur within the existing pluton as it is cooled and solidified.

The thickness of the alteration zone within the country rock depends on the temperature and depth of emplacement of the mass. If emplacement occurs within four miles of the ground surface, the lower temperatures will generally result in a smaller alteration zone than emplacement at a greater depth (Golder Associates, 1979).

When the intruded country rocks are composed of sediments (shales, slates and limestones) their metamorphosed equivalents will consist largely of gneiss close to the granite, grading to schist farther away. Roof pendants of metamorphic rock may be found within the granitic body.

The western batholiths are found in zones of rugged topography that are active orogenic belts. The batholiths in portions of the Appalachian and shield areas are in gentle terrain masked by alluvium or glacial drift (Golder Associates, 1978).

Models of exposed plutons illustrating the various topographic expressions are presented in Figure 5-6.

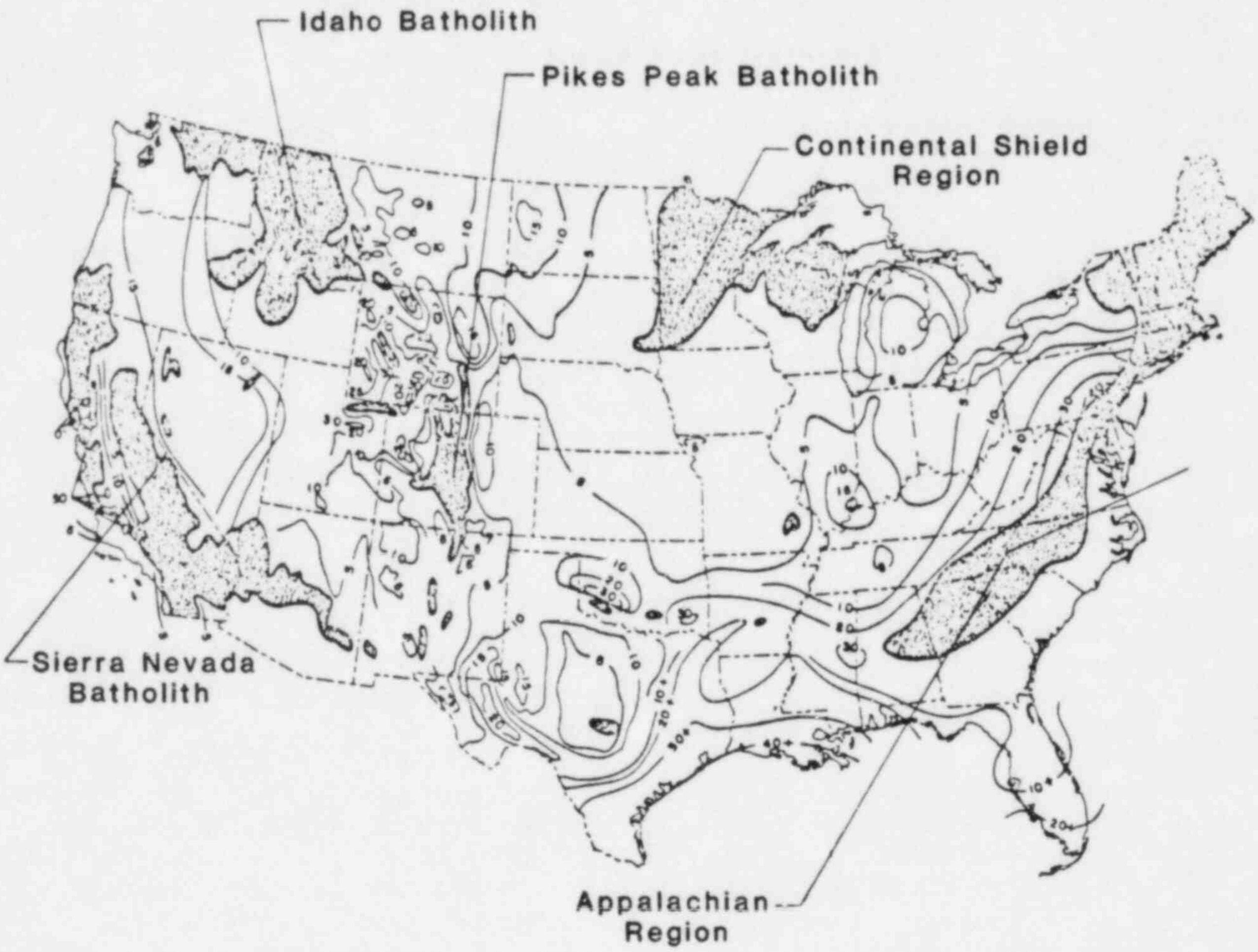
5.2.2.3 Structural Properties

The structural features of importance include fractures, faults, joints, and flow-related structures.

As an igneous magma is intruded, contact with the surrounding country rock can result in a differential shear across the body. This shear can

PRINCIPAL AREAS OF CRYSTALLINE BASEMENT ROCKS AND THICKNESS OF SEDIMENTARY ROCKS IN THE UNITED STATES

Figure 5-5



0 500
Scale, miles

EXPLANATION

Crystalline basement rocks.

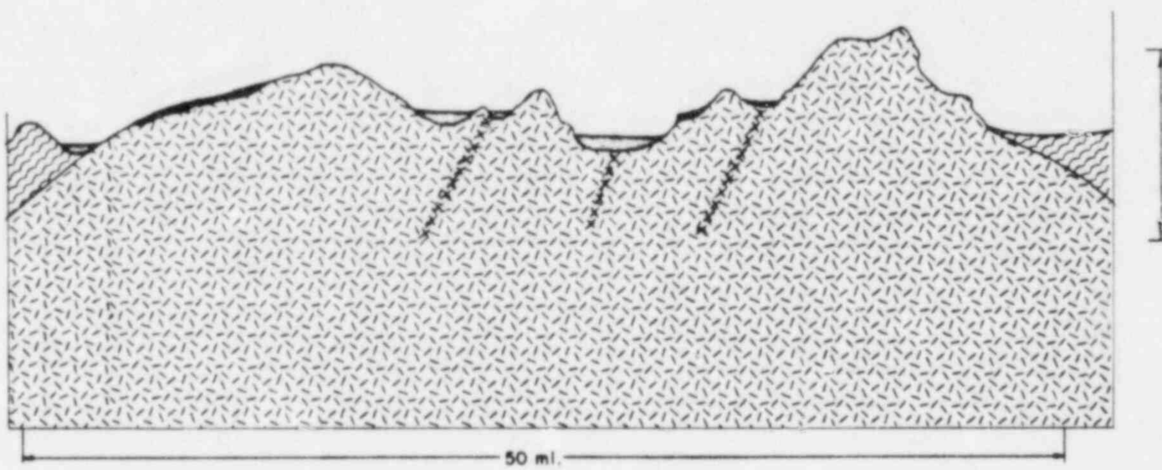
—10— Isopach line in thousands of feet, interval variable, + indicates basement rock not reached; contours omitted where data is lacking.

After EKREN et.al. 1974

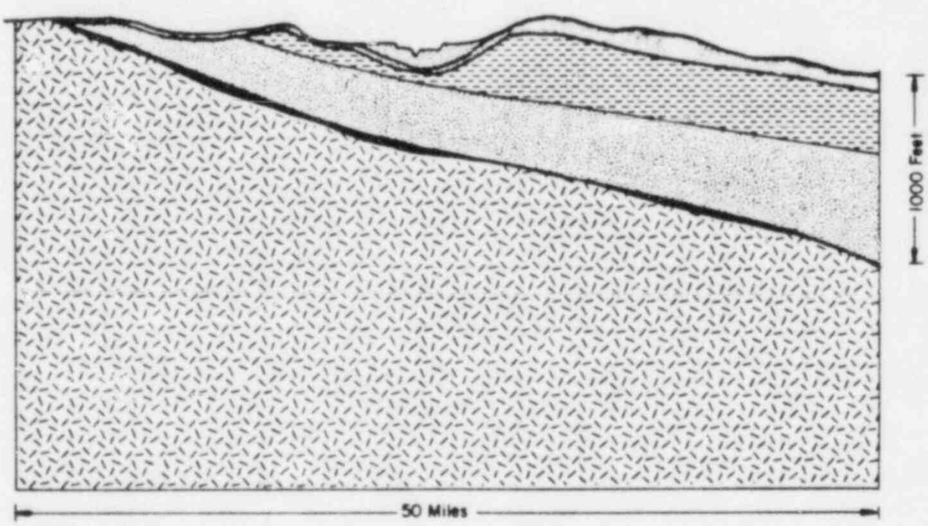
Rev. Dwg. No. Date 6-82 Eng.

GENERIC CRYSTALLINE BASEMENT ROCK MODELS Figure 5-6

GENERIC CRYSTALLINE BATHOLITH MODEL



GENERIC SHIELD-PIEDMONT CRYSTALLINE MODEL



LEGEND

- Glacial and/or Alluvial Sediments
- Crystalline Batholith (Granitic)
- Metamorphic Boundries
- Fracture Zones
- Siltstone - Shale
- Sandstone
- Saprolite (clay)

After Golder, 1977

Rev. Dwg. No. 813-1168D 020 Date 3-82 6-82 Eng. LG align platy or tabular minerals parallel to the flow direction to produce a linear or planar flow structure. This flow banded structure may grade imperceptably into a massive phase of the intrusion, and can be a determining factor in the orientation of joints, fractures, and faults (Golder Associates, 1979). The depth and mode of emplacement of the intrusion can lead to different flow pattern and joint orientations (Cloos, 1921).

The transition of a magma to a solid state involves the crystallization of the major mineral phases leading to an increase in viscosity. This transition, in the outer layer of the pluton, can result in a series of joints and faults around the intrusion (Figure 5-7).

Three mutually orthogonal joint or fissure sets with a fourth interposed at an oblique angle are common. These discontinuities may be filled with quartz, pegmatite, aplite, or clayey gouge material or may remain open. Interaction between the igneous intrusion and regional tectonic stresses can produce orientations significantly different from those indicated in these simple models. Super-position of regional tectonic stresses at a later date may introduce new discontinuities, but more frequently tends to emphasize pre-existing zones of structural weakness.

Secondary fractures and joints at shallow depth and/or along the fringes of a pluton, are relatively numerous. However, these decrease with depth and toward the interior of the intrusive (Golder Associates, 1978).

Within the Precambrian Shield rocks of western Minnesota, several granitic plutons are exposed but most are buried under glacial debris. The presence of monumental stone quarries is indicative of the wide joint spacings often found in granite.

The granitic plutons of the Appalachian Region are very complex. Flow banding usually dips at high angles but in places it rolls over the crests of domes or arches or dips at low angles over wide areas. The structure involves much flowage of the rocks and thickening and thinning of the units with little breaking or faulting. Nearly all the faults of the crystalline area formed after the rocks had been deformed and solidified.

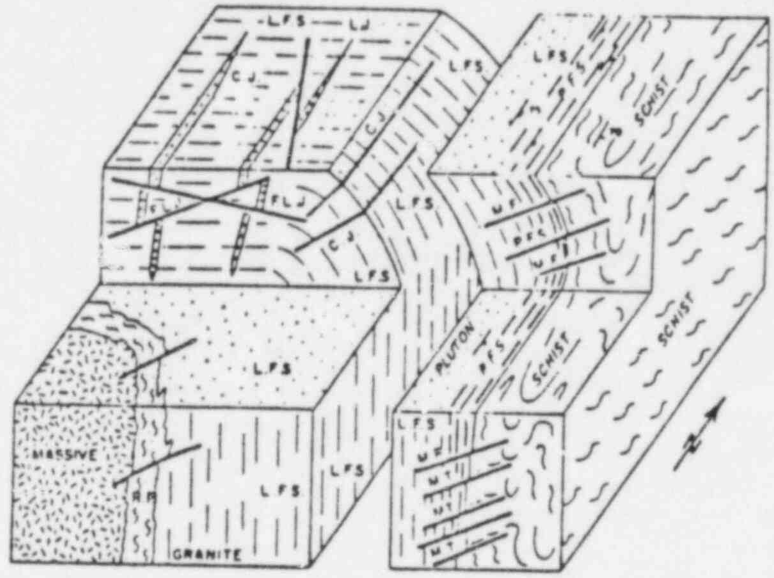
The Sierra Nevada batholith is a complex structure consisting of numerous granitic plutons. The major recently active faults are along the north, south and eastern borders. Jointing occurs in at least one area on 50-ft centers for the master joints. Minor joints are much more variable. (Golder Associates, 1978).

The granite of the Pikes Peak batholith shows a crude foliation due to a planar alignment of the mica plates and feldspar laths. Principal fault movement occurred along planes of foliation with the movement either upward or downward along those planes (Warner and Hornback, 1971). Joints also appear to be parallel or normal to the flow banding and were related in orientation to the minor faults. Joints become irregular in complexly folded metamorphic rocks. (Golder Associates, 1977a).

GRANITE FLOW FABRIC AND STRUCTURE

Figure 5-7

FLOW FABRIC AND STRUCTURAL RELATIONS OF AN INTRUSION

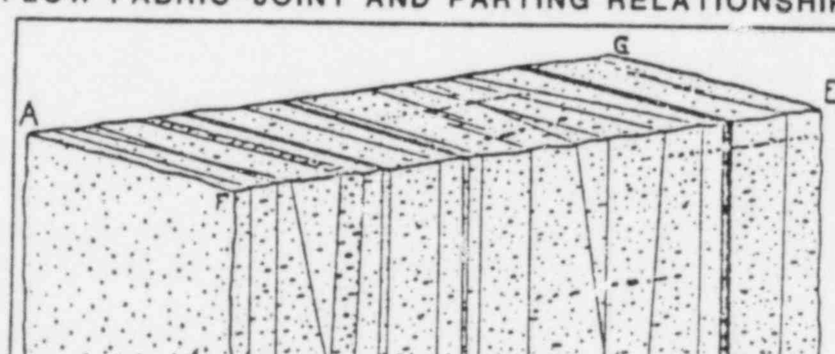


P.F.S. Platy Flow Structure
L.F.S. Linear Flow Structure
C.J. Cross Joints
R.P. Roof Pendant

M.F. Marginal Fissure
M.T. Marginal Thrusts
F.L.J. Flat-Lying Joints
L.J. Longinal Joints

After Badgley, 1964

FLOW FABRIC-JOINT AND PARTING RELATIONSHIPS



Cross Joints(J) ,dikes(RSTU),horizontal flow lines trend from left to right. The joints and dikes are perpendicular to flow lines. The parting planes(ABCF, CFED, AFGE)may, or may not, coincide with exfoliation planes.

After Balk, 1937

Rev. Dwg. No. B13-1164D 021 Date 3 82 6-82 Eng. [illegible]

Systematic documentation of joint orientation in a number of plutons indicates that joint position, spacing, continuity and orientation are highly variable so that average joint conditions cannot be meaningfully defined (Atomic Energy of Canada, Ltd., 1980).

5.2.2.4 Mechanical Properties

The mechanical properties of significance to shaft construction and operation are the in situ stress state and the strength properties of the rock mass and rock discontinuities.

The in situ pre-excavation stresses influence the stability of the shaft opening and the temporary and permanent support measures in a major way. Table 5-6 summarizes the documented measurements of in situ stresses in various granitic bodies within the United States. It is considered that as noted from measurements in other rock types, one principal horizontal stress may be approximately equal to the vertical stress and the other principal horizontal may be as much as twice the vertical stress.

The magnitude of the rock mass strength relative to the stresses induced around the periphery of the shaft by excavation control the stability during construction. Intact strength of granite in the unconfined state is generally in the range 15,000 to 30,000 psi. Thus, the rock mass strength is controlled by the strength of the discontinuities. Fractures in granite are most frequently hard and rough. It is concluded that except for isolated zones of alteration, weathering, faulting and shearing, no stability problems during construction are expected. Similarly, the high confined strengths of granite permit minimal lining requirements for permanent support.

5.2.2.5 Geohydrology

The lack of hydrologic data in intrusive igneous rocks is due mainly to their unimportance as water-producing formations, particularly at depths in excess of several hundred feet. The absence of weathering and the tendency for fractures to be less common with depth, result in very low water yields (Golder Associates, 1979).

Intergranular porosity in solid fresh intrusive rock is generally less than 3 percent and most often less than 1 percent (Krynine and Judd, 1957). Pores are poorly interconnected resulting in low to zero permeabilities (Golder Associates, 1978).

Towse (1979) has adapted the following ranges of intact porosity and hydraulic conductivity values for granite from Ekren et al (1974):

Porosity	0.5 to 3 percent
Intact Rock Permeability	6.0 x 10-11 to 1.0 x 10-4 (cm/sec)
In situ Hydraulic Conductivity	3.5 x 10-8 to >7.0 x 10-2 (cm/sec)

RESULTS OF IN SITU STRESS MEASUREMENTS IN GRANITE

Table 5-6

LOCATION	REF	DEPTH(ft)	VERTICAL STRESS (PSI)	AVERAGE HORIZONTAL STRESS / VERTICAL STRESS
Red Mountain, Colorado	1.	2050	2625	0.56
Henderson Mine, Colorado	1.	2592	3509	1.23
Montello, Wisconsin	2.	446	--	3.29

REF.

1. Hooker, et al, 1972
2. Jaeger and Cook, 1976

Rev. Dwg No. 813-11642 Date 4-82 Eng.

The flow through a granitic rock mass is controlled by secondary permeability. The only exception to this is when the mass is deeply weathered (Golder Associates, 1979). The secondary permeability of any granitic aquifer is controlled by joints, fractures, faults, hydrothermal alteration, and weathering. Jointing is usually the most important factor.

At shallow depths and/or at the fringes of the intrusion, permeable fractures commonly produce low to moderate rock mass permeability. With increasing depth, the fractures tend to close or are nonexistent. At depths exceeding a few thousand feet, extensive areas of crystalline rock may exist which contain very few permeable fractures (Golder Associates, 1978).

Weathering and alteration can substantially increase porosities and permeabilities. Depths of weathering from 5 to 50 ft are normal but may extend to 300 ft or more in areas of intense weathering (Davis and DeWeist, 1966).

The permeability of overlying meta-sediments frequently associated with batholiths are typically in the range 10^{-5} to 10^{-10} cm/sec. These results were obtained by Davis and DeWeist (1966) from laboratory tests on samples taken from depths up to 2000 ft. The implications of these geohydrological conditions to the sinking of the upper shaft fall within the realm of normal provisions for pregrouting of saturated and moderately permeable strata.

Yardley and Goldrich (1975) report that most groundwater flow occurs in the upper few hundred feet of a batholith. The open fractures may yield substantial quantities of water; however, permeability is generally low. Most water wells in granite are located in the upper few hundred feet. Very little groundwater flow occurs below a depth of 3000 ft (Golder Associates, 1978).

5.2.2.6 Summary

Generally, granite intrusives are buried beneath hundreds of feet of sediments or they have been uplifted and eroded to the extent they are now surface features.

Batholiths overlain by the original country rock have many common features. The sedimentary formations in close proximity to the intrusion have been altered and are generally more competent. Their metamorphosed equivalents consist of gneiss close to the granite, grading to schist further from the intrusive.

The outer shell of the intrusive will consist of numerous sets of joints and shears as a result of the transition from a magma to a solid. These features may also extend into the surrounding country rock. These discontinuities may later be filled with quartz, pegmatite, aplite or clayey gouge material, the result of which would be an effective seal of the fractured zone, or they may remain open and provide numerous avenues for fluid flow. The thickness of the alteration zone within the country rock depends on the depth of emplacement of the intrusion.

The state of stress within granitic rocks is highly variable and site dependent. Generally, the horizontal principal stresses are one to two times the vertical stress.

The secondary fractures and joints, while being numerous near the contact with country rock, decrease with depth toward the interior of the intrusion.

Granitic rocks are generally quite impermeable. This is mainly due to absence of weathering and the wide spacing of fractures. Normally, primary permeability in a granite does not exist, and groundwater flow is a result of secondary permeability.

The results of numerous laboratory tests on granite also indicate it is a favorable medium from the point of view of excavation of a shaft. These results are summarized in Table 5-7.

5.2.3 <u>Tuff</u>

5.2.3.1 Introduction

The tuffs at Yucca Mountain in the Nevada Test Site (NTS) are being investigated by the Department of Energy as a possible repository site (Figure 5-8).

A review of the available geological/geohydrological data for the tuff at Yucca Mountain has shown it is not adequate to fully define these characteristics. The acquisition of data is still at the site selection stage.

5.2.3.2 Stratigraphy

Tuffs are pyroclastic deposits produced when the gas content of a magma is explosively lost; they may be deposited either directly from explosive volcanic eruptions or as reworked and redeposited sediments. This mode of origin results in a high degree of variation between different tuff deposits. In the Basin and Range Province, the accumulation of tuff locally exceeds 9840 ft in thickness, and individual units may be tens of miles in lateral extent. Thus it is important to restrict the area discussed in this report to Yucca Mountain at the Nevada Test Site. The majority of relevent data on tuff has in fact been obtained from investigations at Yucca Mountain.

Tuff deposits that cool as a single entity are commonly referred to as a cooling unit (Figure 5-9). Such a deposit typically has a core of welded material. The welded zone is characterized by a lack of bedding,

PHYSICAL PROPERTIES OF GRANITE

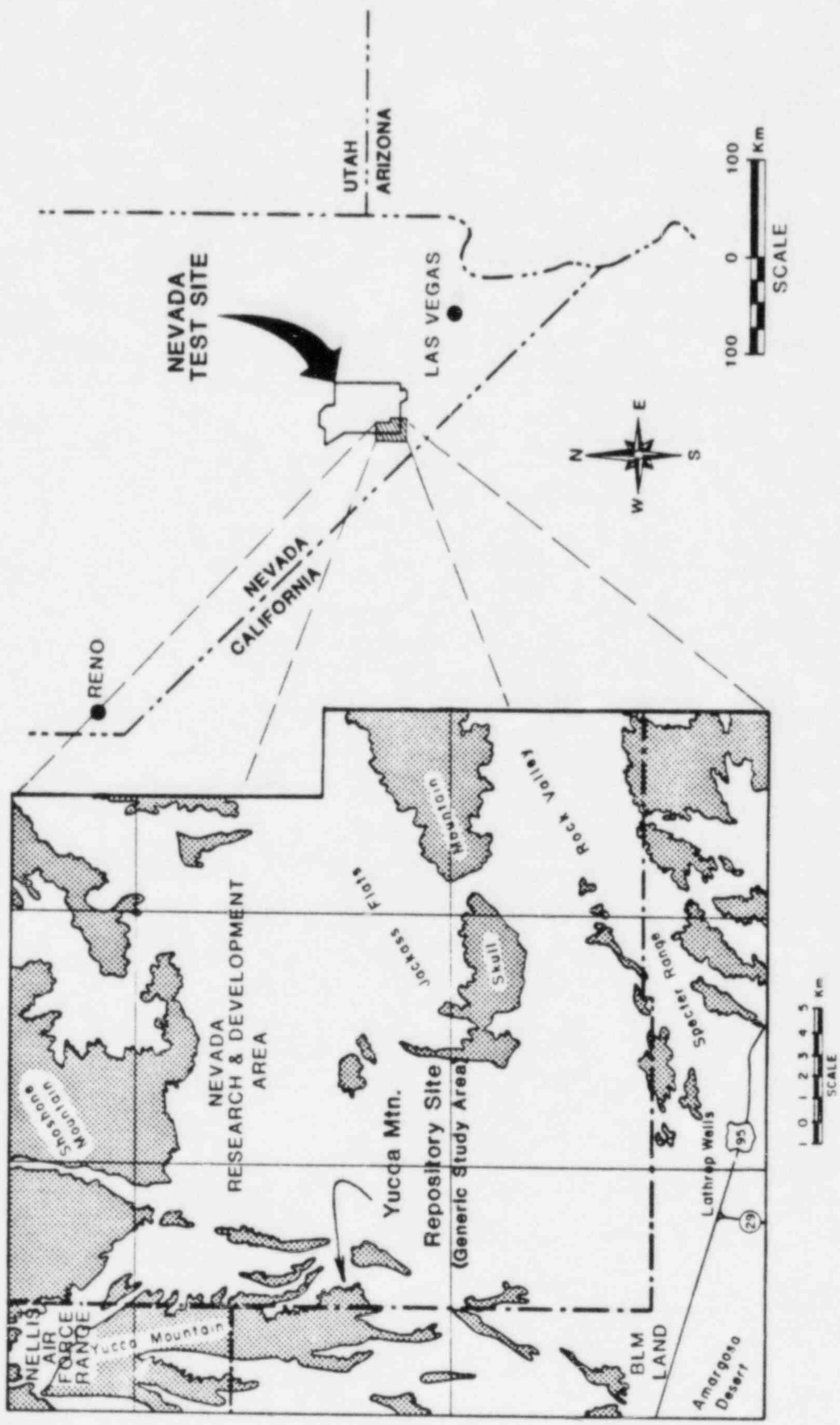
Table 5-7

Porosity 0.5% to 3.0%
Intact Hydraulic Conductivity 1.0×10^{-4} cm/sec to 6.0×10^{-11} cm/sec
In Situ Hydraulic Conductivity 3.5×10^{-8} cm/sec to 7×10^{-2} cm/sec (depending on amount of fracturing
Uniaxial Compressive Strength 29,000 - 37,700 psi

Rev. Dwg. No. 813-1164D 245 Date 4-82 Eng. L.G.

LOCATION OF GENERIC STUDY AREA
WITHIN THE NEVADA TEST SITE

Figure 5-8

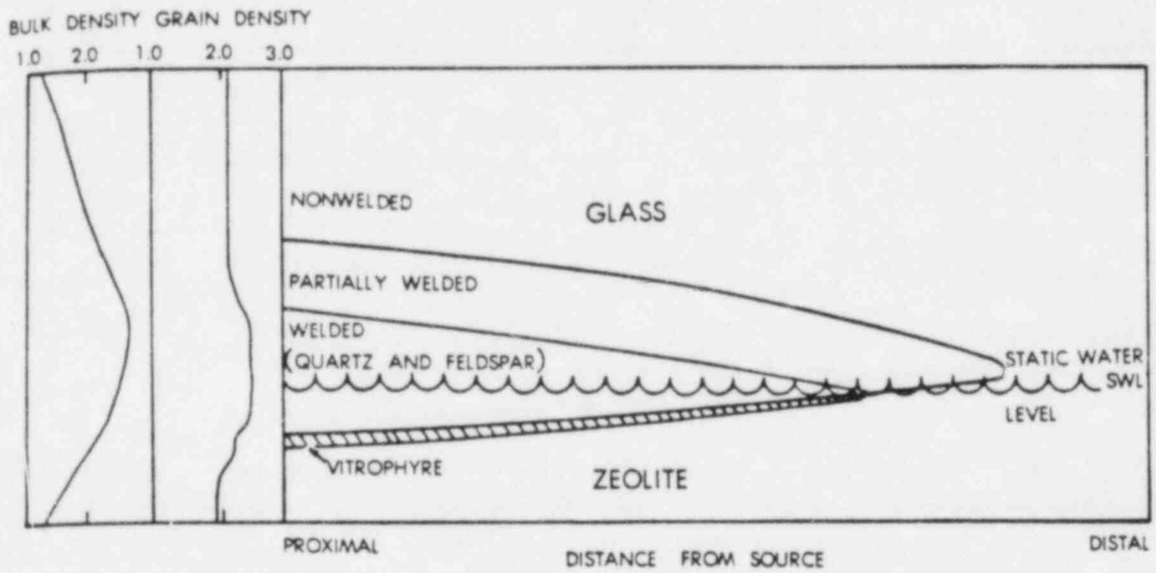


After DOE, Aug. 1981

Project No. Reviewed Date

SCHEMATIC CROSS SECTION THROUGH AN ASH FLOW TUFF COOLING UNIT

Figure 5-9



After Sykes and Smyth, 1980

columnar jointing, and spherulitic structures. At the base of the welded zone, there is typically a layer of densely welded material that has not devitrified, but, instead, remains a dense glass called a vitrophyre. The degree of welding decreases outward from the core so that the welded zone is surrounded by zones of decreasing density, competence, and strength (Figure 5-10). An unsorted, nonwelded, horizon of loosely aggregated pumice and ash similar to the air fall unit is commonly present at the base of the ash flow deposit. The transition between the soft unwelded upper portion and the hard-jointed, welded zone is commonly gradational, but over a very narrow interval.

Because the surface of the deposit is loose and poorly consolidated, it is readily reworked by surface processes. Such processes give rise to sorted, bedded deposits termed bedded tuffs. A wide gradation exists between true tuffs and sedimentary deposits with a tuffaceous content.

The Yucca Mountain region is underlain by a tuff sequence which may locally exceed 10,000 ft in thickness. Four members within the tuff sequence have been selected as potential repository horizons by the Department of Energy (DOE, August 1981). The Bullfrog and Tram Members (subunits of the Crater Flat Tuff) (Figure 5-11) are considered the leading candidate repository horizons. Three other potential repository horizons are also shown.

The reader is directed to Golder Associates (1982a) for a more complete discussion of the genesis of tuff.

Based on very limited subsurface data, the tuff units at Yucca Mountain with one notable exception, the Upper Tram, appear to be fairly uniform in thickness and continuity. Summary descriptions of the stratigraphic characteristics of the Yucca Mountain site are given in Figure 5-11.

The strata in Yucca Mountain appear to be gently dipping. Reported dips range from 5° and 7° (DOE, August 1981). The Bullfrog and Tram Members appear to thin to the north. It is considered that available data are insufficient to accurately evaluate stratigraphic continuity at Yucca Mountain.

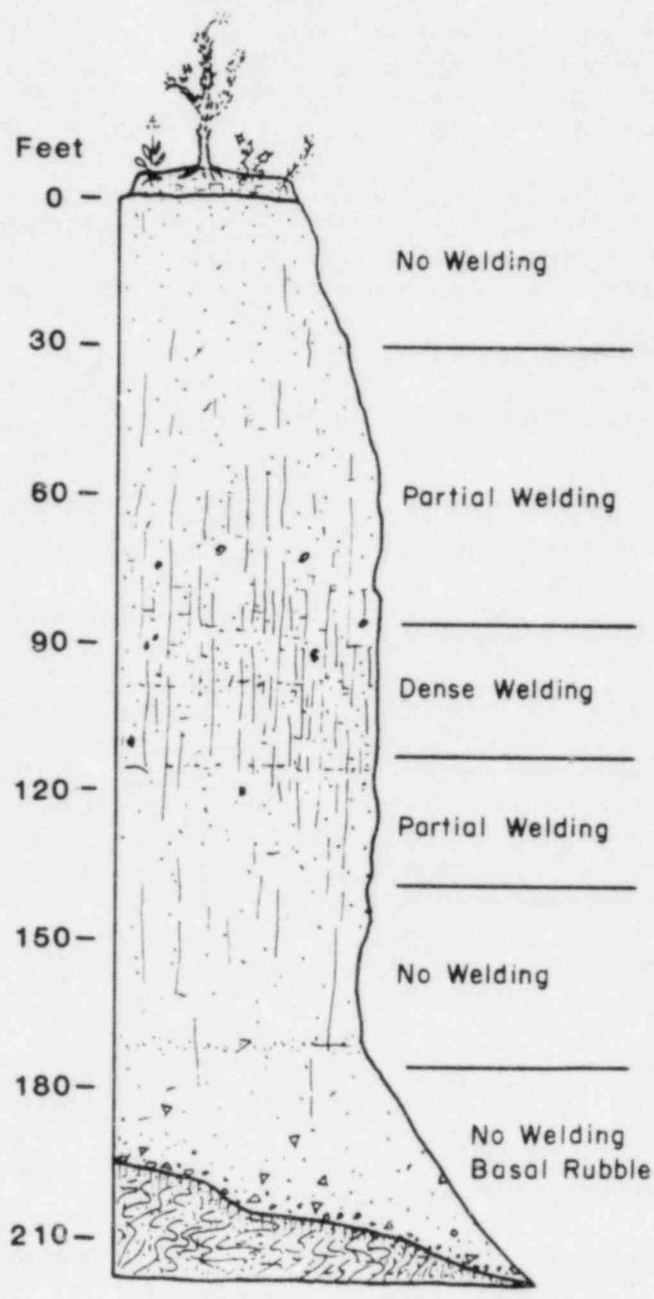
5.2.3.3 Structural Properties

Significant structural features of the tuff which should be considered in the design and construction of a repository are:

- Bedding
- Discontinuities
- Folding
- Faulting/shearing.

SCHEMATIC STRUCTURAL SECTION THROUGH A SINGLE ASH-FLOW TUFF BED

Figure 5-10



After Winograd, 1971

Project No .... Reviewed ...... Date ......

STRATIGRAPHIC SEQUENCE IN GENERIC STUDY AREA

Figure 5-11

FORMATION

ALLUVIUM (Qa) Gravel, sand, silt, containing fragments of densely welded Tiva Canyon Member, and partially welded Yucca Mtn. Member.

PAINTBRUSH TUFF

TIVA CANYON MEMBER (Tpc): A nonwelded to partially welded, vitric tuff; pumice.

YUCCA MTN. MEMBER (Tpy): A nonwelded vitric ashflow tuff; pumice.

PAH CANYON MEMBER (Tpp): A nonwelded, vitric ashflow tuff.

TOPOPAH SPRINGS MEMBER (Tpt): Densely welded, devitrified ashflow tuff (upper 78.6 meters are glassy). Lithophysae between 133.6 - 217.6 meters and 248.5 - 365.8 meters in hole USW-G1.

TUFFACEOUS BEDS OF CALICO HILLS (Tct): Nonwelded, zeolitized ashflow tuff with occasional layers of bedded and reworked tuff. Zeolitization locally ranges from 10 to 80 percent.

CRATER FLAT TUFF

PROW PASS MEMBER (Tcfp): Primarily a partially to moderately welded, devitrified tuff; locally exhibits vapor phase crystalization and argillic pumice. A bedded and reworked zeolitized horizon is located between 656.4 and 662.0 meters in hole USW-G1.

BULLFROG MEMBER (Tcfb): Partially to moderately welded, devitrified tuff with local vapor phase crystallization, slightly argillic between 673.5 and 707 meters in hole USW-G1. Zeolitized pumice horizon between 793.5 and 805 meters in hole USW-G1.

TRAM MEMBER (Tcft): Partially to moderately welded, devitrified tuff (locally nonwelded); argillic and zeolitized between 981.6 and 1074.2 meters in hole USW-G1. Upper contains concentrations of lithic fragments and is partially to moderately welded. Lower is nonwelded to partially welded, zeolitized argillic.

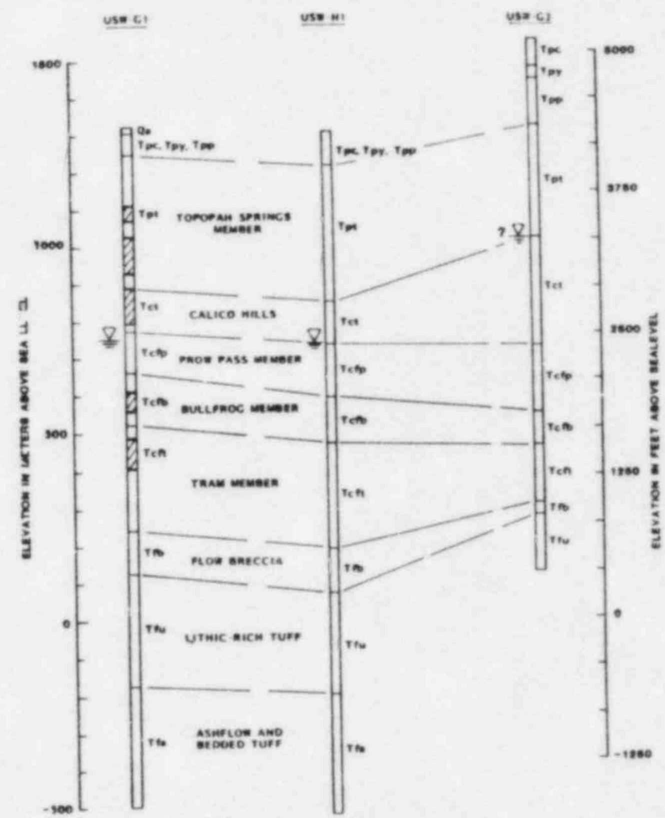
FLOW BRECCIA (Tfb): Interstratified breccia, rhyodacite lava, bedded/reworked tuff, and ashfall tuff; primarily devitrified; moderately to well indurated; lower 7.9 meters (25.8 feet) of unit is argillic; basal 1 meter (3.4 feet) of unit is zeolitized.

LITHIC RICH TUFF (Tfu): A thick, 319 meter (1046 feet) thick ashflow tuff over a bedded/reworked tuff. The ashflow tuff is partially welded and well indurated with 5 to 15 percent rhyolitic and intermediate lithic fragments, lithics decrease in size and abundance in lower 12.2 meters (40 feet) of interval; argillic and zeolitic. The bedded/reworked tuff is moderately indurated and devitrified, individual beds range from 4 cm to 1.5 meters (0.13 to 5 feet). Lowermost bed, .3 meter (1.2 feet) thick, contains 30-40 percent lithic fragments.

ASHFLOW AND BEDDED TUFF (Tfa): A thick sequence, 323 meters (1060 feet), of ashflow tuffs, airfall tuffs, bedded/reworked tuff, and tuffaceous sandstone. Induration varies from partially to well indurated. The degree of welding in the ashflow and airfall tuffs ranges from nonwelded to densely welded. Nearly all horizons within the unit are zeolitic and/or argillic. Individual beds within the bedded/reworked horizons range from 2 cm to 2.4 meters (0.01 to 8 feet).

THICKNESS IN G1 (m)

12
8
34
363
115
113
142
147 (upper)
133 (lower)
119
274
320 +



TARGET REPOSITORY HORIZONS AS SELECTED BY DEPARTMENT OF ENERGY 8/81.

After DOE, August, 1981

Bedding

The thickness of an ash-flow tuff depends on the volume of material erupted and the topographic configuration over which it is deposited. Ash-flow tuffs tend have even upper surfaces with very low original dip angles. By contrast, the base of an ash-flow tuff may be quite irregular, especially if it was deposited upon uneven topography. Successive depositional surfaces within a particular flow become progressively more level as topographic irregularities are filled. This pattern differs markedly from the overall blanketing of the topography by ash-fall tuffs units.

A principal characteristic of ash-flow tuffs is the common occurrence of nonsorted and nonbedded materials. This characteristic is in direct contrast to ash-fall tuffs in which pronounced bedding is commonly present (Ross and Smith, 1960.)

The NNWSI Peer Review (DOE, August 1981) suggested that the tuffs at Yucca Mountain being of ash-flow origin, are laterally continuous. Further conformation of this feature is required.

Bedding should be treated as a type of discontinuity. Due to the probable irregular and nonuniform deposition of tuff at the bottom of a flow, it is considered that bedding should be treated as a characteristic of minor significance only.

Discontinuities

The degree and frequency of joints and fractures for the Yucca Mountain tuffs are not well defined.

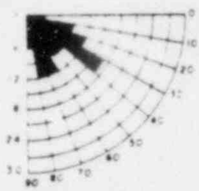
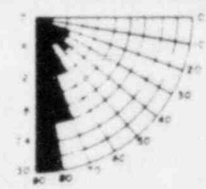
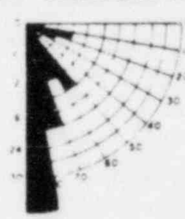
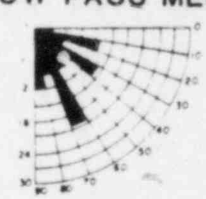
Columnar jointing is a common feature of many welded tuffs. They normally do not occur in noncrystalline nonwelded parts of the ash-flow units. Joint spacing may vary from 2 in. to 3 ft. The more closely spaced joints are usually found in the zones of most intense welding. Unlike columnar joints in lava flows, which characteristically form 5- or 6-sided polygonal columns, columnar joints in welded tuffs form rectangular to square columns.

The most common type of jointing is vertically oriented. Some welded tuffs have developed fan jointing, while others have distorted vertical joints that give rise to bent or warped columns.

A study by Spengler et al, (1979) of joint occurrence and distribution in drill hole core indicates that, in general, the densely welded ash flows are highly fractured, whereas bedded tuffs and nonwelded to moderately welded ash flows are less fractured.

Figure 5-12 shows the distribution of joint inclinations (percent of joints in each 10° increment) for the five major stratigraphic units. Inclinations are expressed in degrees of dip as measured from the horizontal. As displayed, joints within the Tiva Canyon member indicate random orientation ranging from 0° to 90°.

INCLINATIONS OF JOINTS WITHIN STRATIGRAPHIC UNITS Figure 5-12

INTERVAL FEET	INTERVAL METERS	PERCENT OF JOINTS PER 10° INCREMENT	NUMBER OF JOINTS MEASURED	PERCENT OF TOTAL JOINTS	AVERAGE NUMBER OF JOINTS PER 10-FOOT INTERVAL
54.0-270.0	16.5-82.3	TIVA CANYON MEMBER 	159	17.3	7.[illegible]
270.0-1,363.9	82.3-415.7	TOPOPAH SPRING MEMBER 	494	53.7	4.5
1,363.9-1,835.7	415.7-559.5	TUFFACEOUS BEDS OF CALICO HILLS 	46	5.0	.98
1,835.7-2,333.2	559.5-711.1	PROW PASS MEMBER 	181	19.7	3.6
2,333.2-2,500.6	711.1-762.2	BULLFROG MEMBER 	40	4.4	2.4

PERCENT OF FRACTURES

After Spengler et al, 1979

Project No 813-1162-C Reviewed L.B. Date 12-81 813-1164D 026 4-82

The attitude and spacing of the discontinuities in the tuffs of the Yucca Mountain sequence are considered to be critical characteristics because of their impact on stability and the hydrologic properties of the rocks.

Folding

There is no record in the literature of folding of the Yucca Mountain beds. Some fault blocks are locally tilted. Folding is not regarded as being significant in the design and construction of a shaft.

Faulting/Shearing

Five fault zones were recognized in one drill hole core by Spengler et al (1979). Evidence for faulting in this hole was based on brecciated core, abrupt changes in the dip of pumice layers, zones of granulation, and striations and slickensides on fracture surfaces. Due to the absence of any thin, well-defined marker beds, the magnitude of displacements within fault zones could not be established.

Two faults were encountered in a second drill hole, both within the Tram Member. The first, located at a depth of 3522 ft, is 1 in. thick. The second fault is situated at the base of the member at 3558 ft. The fault corresponds with a 0.8 ft thick layer of "swelling" green clay.

Although it has been shown that the potential repository sites currently being studied at NTS have avoided the worst zones of faulting, the intersection of several faults by the proposed shafts is likely.

The fact that two of the few holes drilled to date have intersected faults indicates that they may be more widespread than initially supposed. The width, composition, texture, and the continuity of individual shears within a fault zone may have a direct influence upon groundwater flow. Unfortunately, little is known on fault occurrence and characteristics.

5.2.3.4 Geochemical

Pyroclastic rocks, particularly fine-grained varieties, are readily altered, both chemically and physically. This is because of their high porosity, the large surface area of constituent particles, and the inherently unstable nature of the glassy fragments.

Devitrification of glass is the initial alteration phase and usually occurs fairly rapidly.

One of the more common products of devitrification is the expansive clay mineral montmorillonite (which is of the smectite group). Montmorillonite is an expanding-lattice clay mineral which exhibits swelling on wetting and shrinking upon drying due to the introduction or removal of interlayer water.

Clay minerals at NTS are reported as sodium saturated montmorillonite-beidellites (DOE, August 1981). The presence of sodium saturated montmorillonite is extremely important in that this particular form of montmorillonite has a significantly higher swelling potential than the other common variety of montmorillonite, which is composed of adsorbed calcium cations; the swelling potential is approximately three times greater for sodium than for calcium montmorillonites. The potential for a volume decrease upon drying is also correspondingly greater. Associated with a volume decrease upon drying is the development of dessication cracks and the widening of joint apertures.

Zeolite and clay alteration zones have been recognized in the stratigraphic section penetrated by drill holes (DOE, August 1981). These are presented in Figure 5-13. Zone 1, the upper tuff stratigraphy in the drill hole down to 1296 ft, contains the altertion assemblage Na-K montmorillonite clays. Zone II which extends from 1296 to 3133 ft below the surface contains minor amounts of clays, which again are Na-K montmorillonites. The top section of Zone III Na-K dioctahedral montmorillonite similar to that in Zones I and II. These montmorillonites are interstratified with less than 15 percent illite, and preliminary data indicate no clear trend of increasing interstratifications with depth (DOE, August 1981). Below 5080 ft, authigenic albite and K-feldspar become the dominant secondary minerals in both the nonwelded and welded units.

5.2.3.5 Geohydrology

Hydraulic considerations for repository access shaft design and construction are concerned primarily with groundwater inflow during excavation and control of seepage during operation of the facility.

Hydraulic Conductivity

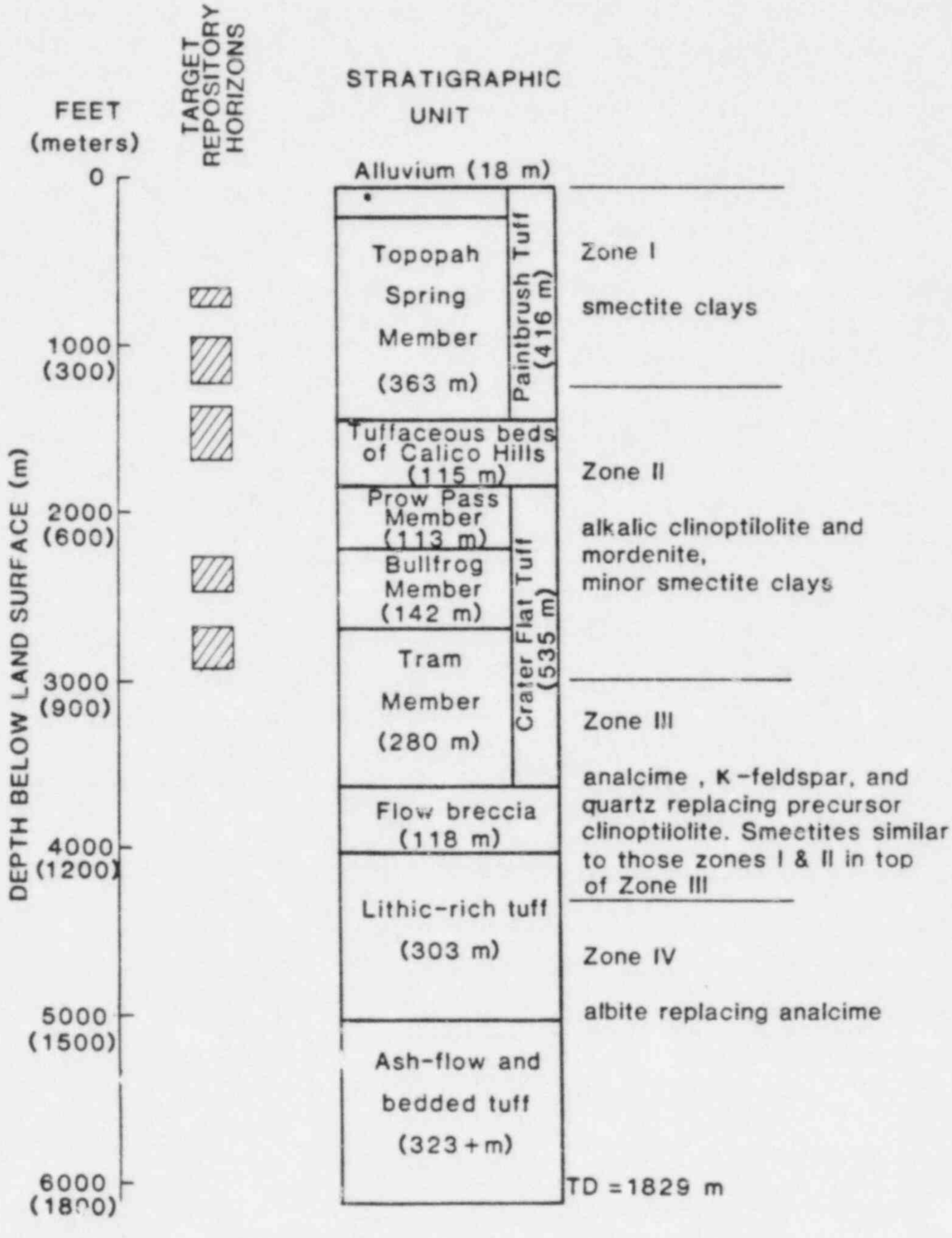
In situ hydraulic conductivity in the nonwelded zones is mainly controlled by the matrix characteristics. An irregular rubble zone is common along the base of some flows, but since the rock fragments are completely surrounded by matrix material, i.e., not interconnected, these zones do not result in appreciable permeability.

Within the partially and densely welded zones, hydraulic conductivity is controlled exclusively by joints and fractures. Observations of tuff outcrops and core samples indicate that the cooling joints tend to form perpendicular to bedding. Jointing results in welded zones with anisotropic hydraulic conductivities. Since the jointing has a preferred vertical orientation, the vertical hydraulic conductivity (Kv) tends to be greater than the horizontal hydraulic conductivity (Kh).

Permeability characteristics of welded tuff core samples at the NTS have been described by Winograd and Thordarson (1975). Laboratory analysis indicates matrix hydraulic conductivities that vary inversely with the degree of welding, ranging from 10^{-4} cm/sec in nonwelded

ZEOLITE AND CLAY ALTERATION ZONES IN TYPICAL YUCCA MOUNTAIN TUFF SEQUENCE

Figure 5-13



After DOE, August 1981

Rev. Dwg. No. B3-1682 027 Date 4-82 Eng. LB zones to 10^{-10} cm/sec in densely welded zones. In unfractured nonwelded tuff, the matrix hydraulic conductivity of core samples is probably similar to the in situ hydraulic conductivity, but such a relationship is not valid in the welded zones where hydraulic conductivity is controlled by fracturing.

Observations of underground workings (tunnels and test chambers) in saturated zeolitic tuff of the Indian Trails Formation were made by Thordarson (1965). Although this tuff unit is not saturated below Yucca Mountain, the descriptions of fracturing and groundwater inflows provide useful comparative qualitative information on the in situ hydraulic conductivity. Most joints have near-vertical attitudes and are generally closed. Open joints, however, have widely variable apertures and can be nearly closed at one location and open as much as 2 in. just a few feet away. Only a small percentage of the joints are water-bearing. About 50 to 60 percent of tunnel inflows resulted from faults or breccia zones and 40 to 50 percent of this was attributed to fractures. The initial discharge of water from most fractures was less than 20 gpm but the discharge from one fault zone was about 200 gpm. The discharge from all fractures decreased rapidly with time and within a few days, was a small fraction of the initial flow rate. Water-bearing joints tended to be poorly connected and tunneling often intersected saturated joints 100 meters away from joints which had been dewatered several days earlier. Joint densities reached a maximum of one per meter of tunnel, but many sections of tunnel up to 10 meters long were unjointed.

Groundwater inflow in a test chamber located 1000 ft below the regional groundwater table at Pahute Mesa was estimated at less than 4 gpm. In a deeper chamber located 600 m below the water table, inflow rates of the order of 1 gpm were observed. (Winograd and Thordarson, 1975).

Hydraulic Gradient

Little data exists with regard to vertical hydraulic gradients in the tuff sequence below Yucca Mountain. The data from deep boreholes suggests a decrease in hydraulic head with depth which would indicate downward vertical flow (DOE, August 1981). In northern Yucca Flat, to the northeast of Yucca Mountain, the piezometric head in the tuff aquitard is as much as 130 ft higher than that of the underlying carbonate aquifer (Winograd and Thordarson, 1975).

Porosity

Effective porosities have not been measured in welded and nonwelded tuff. Total porosities vary inversely with the degree of welding, ranging from 50 percent in nonwelded zones to 5 percent in the central densely welded zones (Winograd and Thordarson, 1975). In nonwelded tuff, groundwater flow is primarily through the rock matrix and therefore, effective porosity may be similar to total porosity. In densely welded tuff, where flow is controlled by fractures, effective porosity is likely to be much less than the total porosity.

Specific Storage

Specific storage is of significance only with respect to the transient groundwater pressure response, such as during depressurization (shaft excavation) and repressurization (post-excavation stabilization). In nonwelded zeolitic tuff to partially welded tuff, specific storage may range from 10^{-5} to 10^{-7} cm^{-1}. In jointed densely welded tuff, values of 10^{-7} to 10^{-8} cm^{-1} may be considered realistic (Walton, 1970).

5.2.3.6 Mechanical Properties

During shaft excavation, limited areas of exposed tuff may overlap fractures or even become unstable. This increased fracturing will affect the ground control measures both during excavation and subsequently when sealing is carried out to reduce the potential for radionuclide migration.

One of the most compehensive studies to date on the strength properties of Yucca Mountain tuffs is that carried out by Sandia National Laboratories and reported in Olsson and Jones (1980). The study included confined and unconfined tests on intact and jointed samples of tuff from all the major flows obtained from Boring UE25a-1 and G-Tunnel. Deformability and creep were also studied in these tests. The properties of samples obtained from Boring UE25a-1 are shown on Table 5-8.

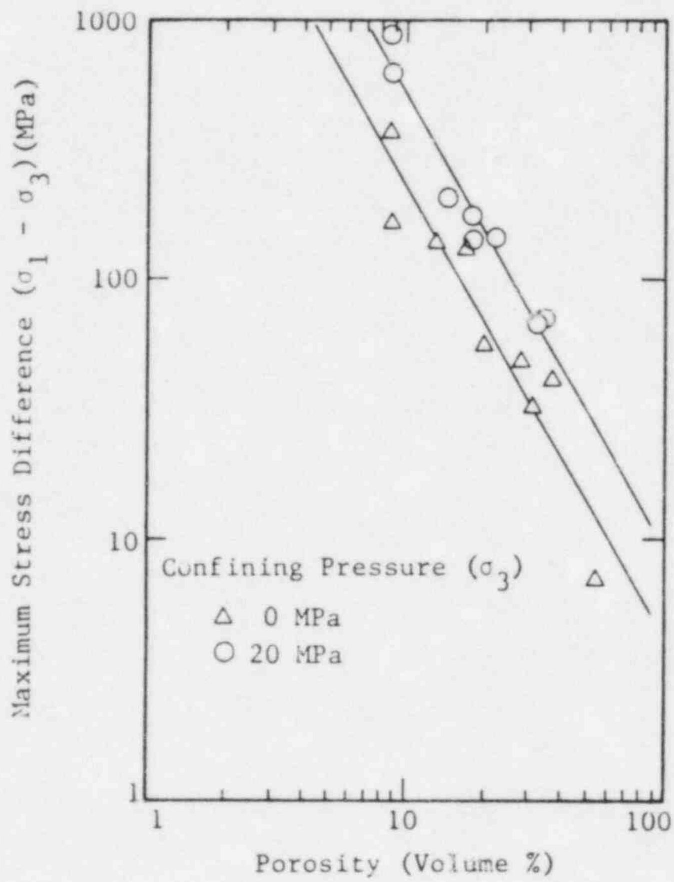
The strength of tuff is a function of the formation, strain rate, degree of saturation, porosity (degree of welding) and alteration. A typical strength-porosity relationship is shown in Figure 5-14. The influence of alteration on both strength and modulus warrants special mention. As well as leading to a noticeable deterioration in the mechanical properties of the Yucca Mountain tuffs, alteration results in the formation of materials with significant swell potential. Because of the high variability of strength with welding and porosity of the flow beds, it is expected that in certain horizons at the greater depths, occasional stress-related stability problems may be experienced.

For the densely welded tuffs, which generally exhibit brittle behavior, the tensile strength may be an important mechanical property, particularly when an unfavorable combination of in situ stresses and excavation induced stresses result in significant tensile stresses around the excavation. Unfortunately, no such information is available for Yucca Mountain.

The only significant and available information on modulus data for Yucca Mountain tuffs are included in Table 5-8. The wide scatter of results suggests that, despite the small specimen size, discontinuities may have a significant influence. Varying degrees of welding, as shown by the calculated porosity values, may also partly explain the scatter (Figure 5-15). Significant modulus anisotropy is indicated.

COMPRESSIVE STRENGTH OF TUFF AS A FUNCTION OF POROSITY AND CONFINING PRESSURE

Figure 5-14



After Olsson and Jones, 1980

Project No. 83-1162C Reviewed __ Date 12-81

83-1162 028 4-82

MECHANICAL PROPERTIES OF TUFF AT YUCCA MOUNTAIN (from Hole UE25a-1)

Table 5-8

Unit	Specimen Number	Confining Pressure (MPa)	Temperature (°C)	Max. Stress Difference (MPa)	Young's Modulus (GPa)	Poisson's Ratio	Calculated Porosity (%)
Tiva Canyon	87.6	0	RT	364	57.5	0.31	8.8
Tiva Canyon	87.6	10	RT	396	43.9	0.30	8.8
Tiva Canyon	87.6	20	RT	875	58.3	0.22	8.8
Tiva Canyon	185	20.7	200	105	----	----	26.7
Tiva Canyon	212.7	0	RT	7.03	0.41	0.28	54.0
Topopah Springs	723	0	RT	138	40.4	0.22	12.9
Topopah Springs	739	20.7	200	133	23.9	0.15	11.3
Topopah Springs	1250	0	RT	166	61.8	0.30	8.8*
Topopah Springs	1250	10	RT	412	73.0	0.23	8.8*
Topopah Springs	1250	20	RT	618	59.9	0.21	8.8*
Calico Hills	1490	0	RT	47.7	12.3	0.14	28.1
Calico Hills	1605	20.0	RT	26.1	7.99	0.22	29.5
Calico Hills	1634	20.7	RT	67.5	8.50	0.27	32.2*
Calico Hills	1662	20.0	RT	70.3	9.57	0.25	34.9
Calico Hills	1692	0	RT	40.8	14.0	0.20	36.6
Prow Pass	1948	100.0	RT	299	22.0	0.20	19.1
Prow Pass	1968	20.0	RT	176	27.0	0.20	18.0
Prow Pass	1985	20.7	RT	207	31.0	0.25	14.5
Prow Pass	2014	0	RT	130	47.9	0.30	16.7
Prow Pass	2039	0	RT	32.2	7.84	0.18	31 *
Bullfrog	2401	50.0	RT	174	18.7	0.19	21.9
Bullfrog	2421	20.0	RT	145	19.2	0.23	22 *
Bullfrog	2452	0	RT	54	6.37	0.05	20.3
Bullfrog	2491	20.7	RT	140	22.1	0.28	17.7

RT = room temperature

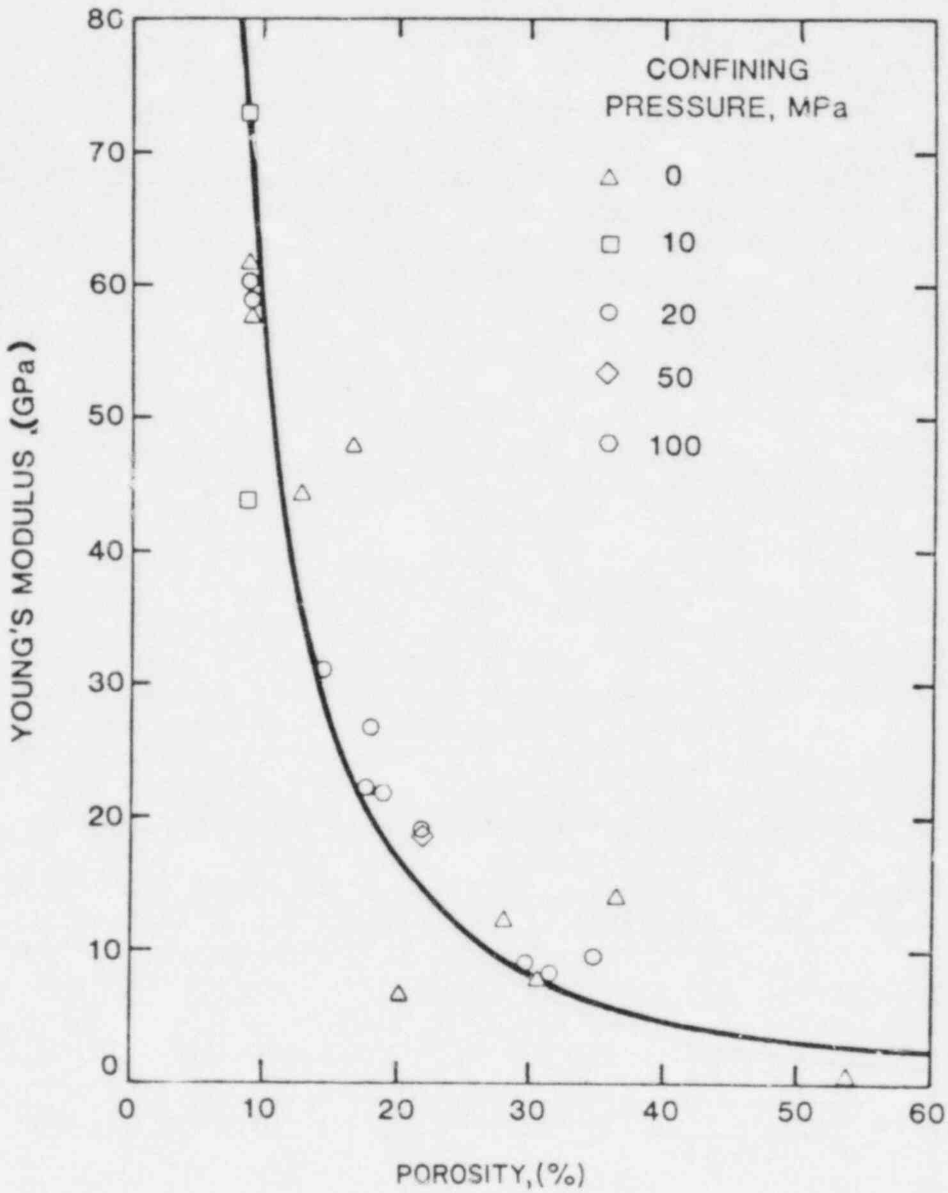
* = estimated

after Olsson & Jones, 1980

Rev. Dwg. No. 8L3-116YD Date 9-82 Eng. LG

YOUNG'S MODULUS OF TUFF AS A FUNCTION OF POROSITY FOR VARIOUS CONFINING PRESSURES

Figure 5-15



After Olsson & Jones, 1980

Project No. Reviewed Date

Creep Deformations

The deformational response of tuff appears to be time-dependent. This creep behavior may have on important influence on the physical integrity of shaft excavations, especially post-construction, and should be allowed for in-shaft design.

In Situ Stresses

The behavior of the shaft will be affected both by the original in situ stresses and the stresses induced during excavation. A knowledge of the in situ stresses is indispensable in assessing both the short-term and long-term deformational response of the shaft/rock structure particularly in the weaker tuff formations. Measurements by Haimson et al (1974) at NTS yielded horizontal stress values just slightly less than vertical. Further testing is required before definitive conclusions can be made regarding the nature of in situ stresses at Yucca Mountain.

5.2.4 Composite Geological Profile - Hard Rock

5.2.4.1 Introduction

In an effort to develop a composite profile for basalt-granite-tuff, it has been necessary to formulate a typical geological profile which is in fact not obviously similar to either of those for basalt, granite or tuff. This results from the fact that although all three media can be considered as hard rock, the diversity of characteristics (stratigraphic, structural, geohydrological and mechanical) is considerable. Clearly, it will not be possible to assign specific rock types to the elements of the profile, only characteristic properties.

In the formulation of a composite profile, the data for the three media have been incorporated with a deliberate bias in the following order of decreasing importance: basalt, tuff and granite. This is considered appropriate because of two factors:

- The extensiveness of data decreases in this order
- The proposed sites for tuff and granite are less specific than for basalt.

The typical stratigraphic profile has been developed in two stages. In the first stage, a general section is synthesized which represents the large-scale features, particularly the flow beds. This aspect is of significance to the adaptability aspect of the shaft construction. Characteristic details of the elements of this profile are determined in the second stage. The profile is drawn for an assumed depth of 4000 ft.

5.2.4.2 Summary Descriptions of Geological Conditions

The basalt is typically overlain by about 800 ft of overburden in the form of clays, sands and some moderately consolidated sediments. Below this are a sequence of basalt flows of similar properties but interspaced with brecciated flow tops and interbed sediments of significantly inferior quality.

The tuff consists of numerous ash-flow tuff units with minor amounts of bedded and reworked tuffaceous sediments. Overburden depths are typically less than 150 ft. The ash-flow tuff has similar properties throughout. The main body is moderately competent and the weakness features are generally more pervasive than those which exist in the basalt.

The profile for the granite consists of about 300 ft of glacial till/outwash, 150 ft of a contact metamorphism zone with exfoliation jointing and the remainder of essentially sound granite, increasing in competency with depth.

5.2.4.3 Composite Profile

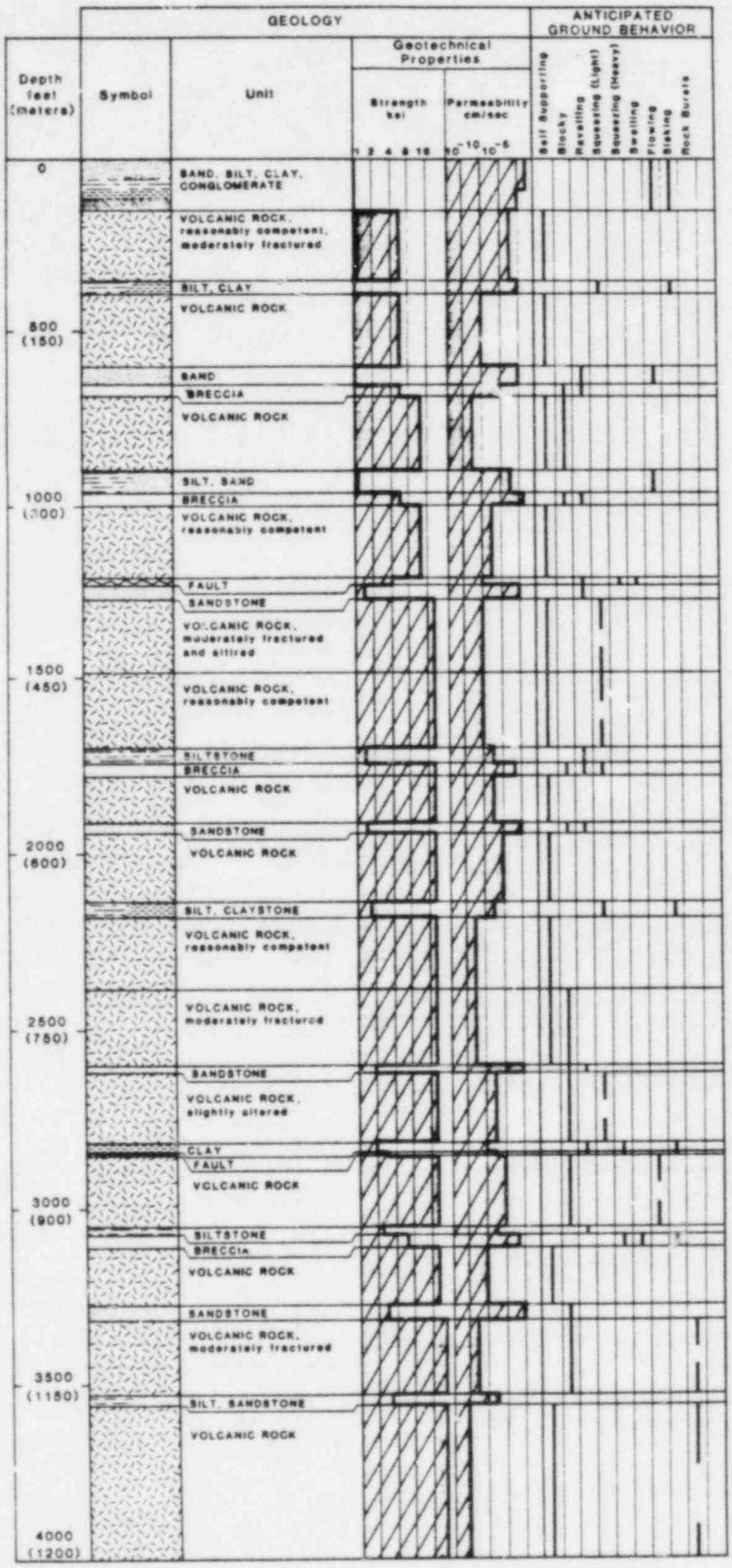
The concept of using composite profiles for the present study is given in Section 5.1. Figure 5-16 shows the composite profile developed for hard rock. The top 150 ft consist of alluvium, sands, clays, silts and some conglomerates. These materials are for the better part, weakly or moderately consolidated. The water table is 30 ft below the surface. The stratigraphic sequence below this overburden consists of flows each about 250 ft thick which is typical of tuff and basalt. These flows are interspersed with strata of lower quality rock typical of breccia flow tops and interbeds in basalts and reworked tuff. These relatively thin intermittent strata are relatively permeable and of low strength although consolidated. More frequently than not, they are saturated with pressures up to hydrostatic and their behavior under unlined and uncontrolled shaft sinking conditions are such as to control the stability of the shaft and the associated grouting, freezing, sealing, lining and sinking requirements. Ravelling, squeezing and occasionally swelling are anticipated.

The thicker main flows are generally competent and self-supporting although blocky and ravelling ground may be experienced in the upper portions of the flow.

A gradual variation of strength with depth is indicated. Without preventative or remedial measures, potential inflows to an unlined shaft for this geological sequence are roughly estimated to be between 5,000 and 50,000 gpm. It is not expected that unlined shafts will be stable or result in acceptable flows into the repository horizon without extensive grouting and freezing measures. In addition, design of a lining to withstand a hydrostatic head appears prudent for a repository application. As envisioned by estimates cited and reported in Golder

COMPOSITE SECTION - HARD ROCK

Figure 5-16



Rev. Dwg. No. Date Apr 82 Eng.

Associates (1982a), the potential problem of water inflow to the shaft and its implications to construction and lining is extremely difficult to assess. Nevertheless, these conditions markedly limit the available technically feasible and economically viable construction methods which can meaningfully be prepared for evaluation.

5.3 SALT (BEDDED-DOMAL) COMPOSITE MEDIUM

5.3.1 Bedded Salt Deposits - Palo Duro Basin

5.3.1.1 Geologic Setting

The Palo Duro Basin is a large asymmetrical basin between the Matador arch on the south and the Wichita-Amarillo uplift on the north (Figures 5-17 and 5-18). Its east-west-trending axis is about 5 miles north of the Matador arch. The basin has a length of about 175 miles and a width of about 60 miles.

Initial development of the Palo Duro Basin began in Pennsylvanian time and the principal period of deposition was during the Permian. A total of 11,000 ft of sedimentary rocks overlie the basement complex of igneous and metamorphic rocks: pre-Pennsylvanian rocks are about 1000 ft thick, Pennsylvanian rocks are about 1000 ft thick, Permian rocks are about 7000 ft thick, Triassic strata are about 1500 ft thick, and the Tertiary sediments are several hundred ft thick.

All Permian units are thicker in the southern part of the basin, and they are thinner northward toward the Amarillo uplift. Permian rocks dip gently to the south and southwest over most of the basin, and the dip inclination is typically 20 to 40 ft per mile.

The western part of the basin is located in the High Plains physiographic province. The eastern part of the basin is located in the Central Lowlands province. The boundary between the two provinces is defined by a prominent escarpment formed by the caliche caprock on top of the Ogallala formation of Tertiary age. Most salt dissolution occurs east of the boundary, so repositories would probably be sited to the west (Dutton, 1979).

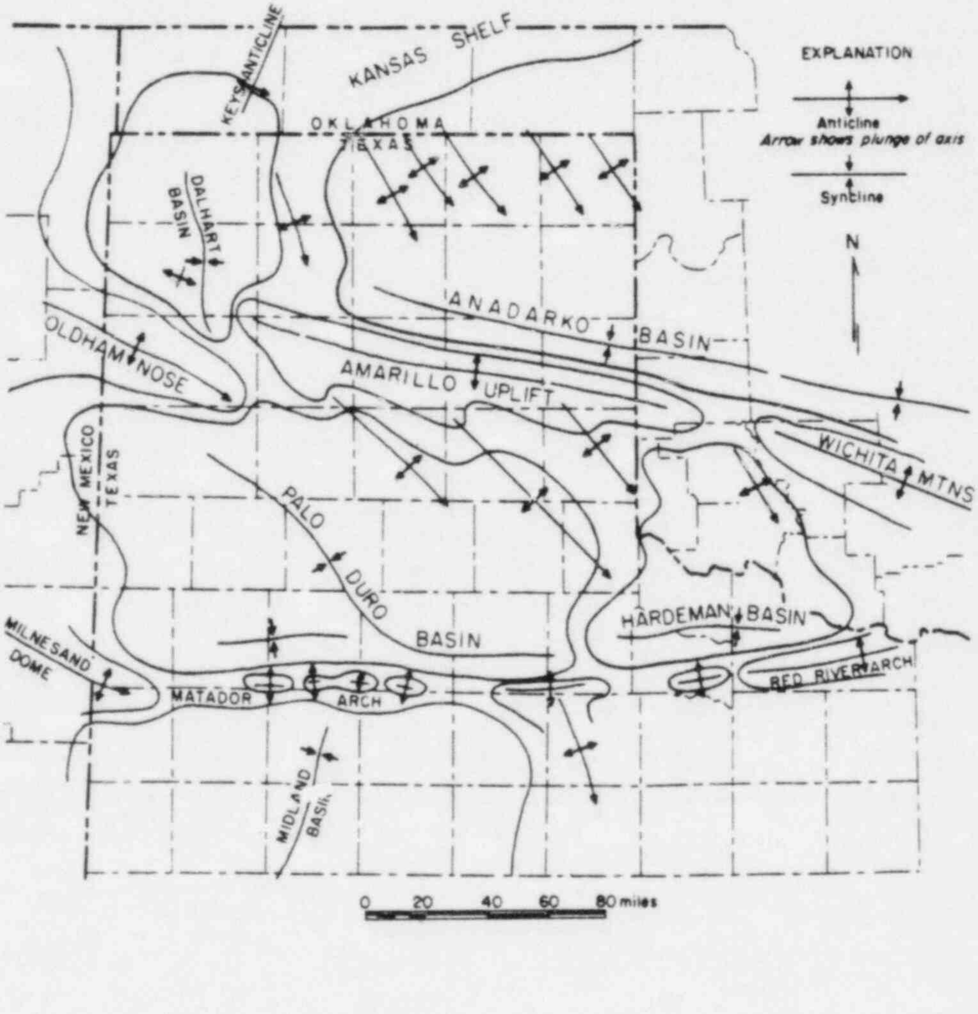
5.3.1.2 Stratigraphy

The stratigraphy of the Palo Duro Basin is shown on Figure 5-19 to 5-21. The main units present in the basin consist of the Ogallala formation, the Dockum group, and the upper Permian evaporite sequence.

The Ogallala formation is up to 200 ft thick and consists of sandstone with occasional thin beds and lenses of siltstone and claystone. Most of the sandstone is weakly cemented, but there are some thin, well cemented beds. The formation is an important aquifer in the Texas Panhandle.

STRUCTURAL ELEMENTS AND LOCATION MAP OF THE TEXAS PANHANDLE

Figure 5-17

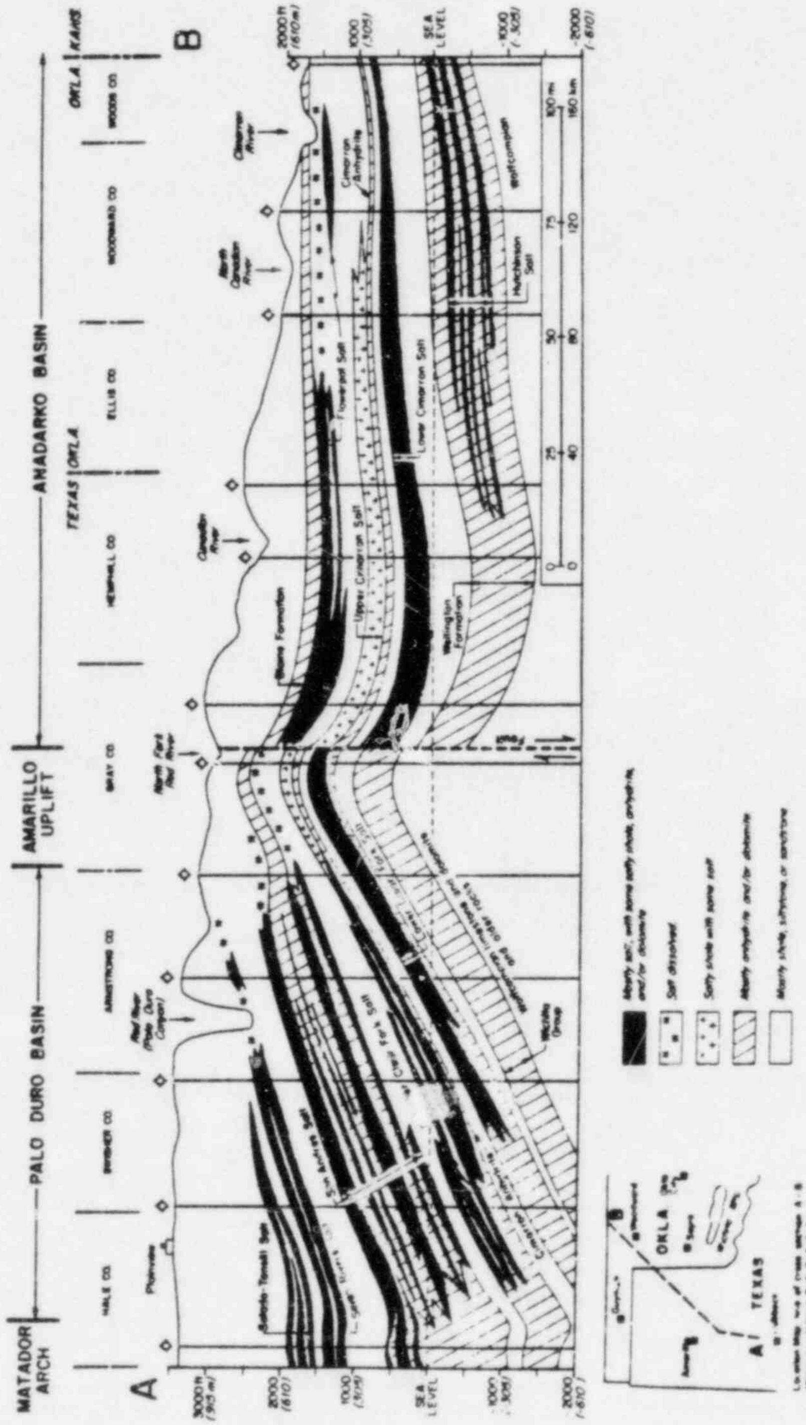


After Nichclson, 1960

Rev. 1164D A03 4-82
Dwg. No. RBL3-1168D Date 3-9-82 Eng. WW

GENERALIZED STRUCTURAL CROSS SECTION SHOWING PERMIAN SALTS AND ASSOCIATED STRATA IN THE TEXAS PANHANDLE AND WESTERN OKLAHOMA

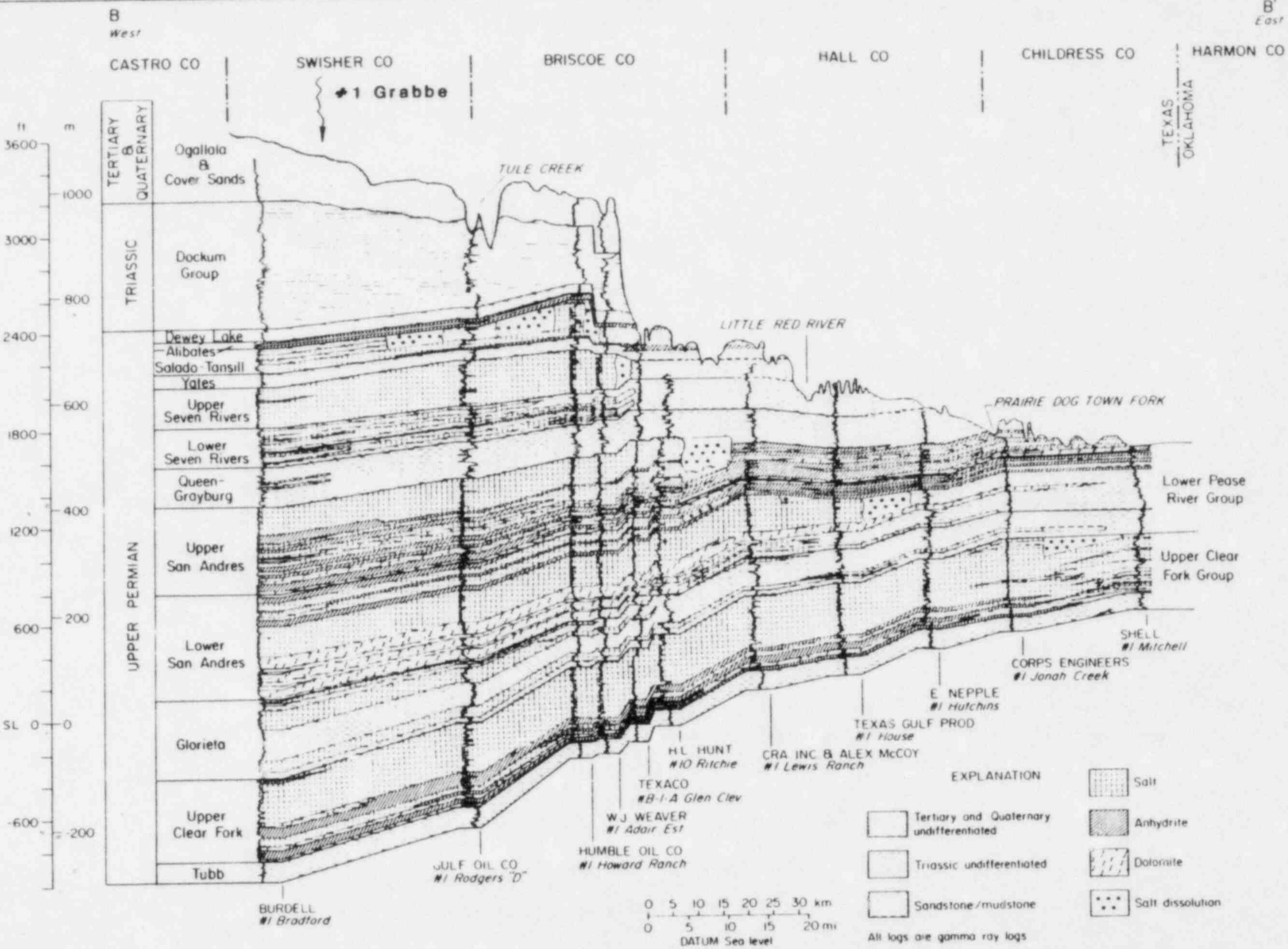
Figure 5-18



After Johnson, 1976

Rev. 83-1164D 032 4-82 LO
Dwg. No. AB13-1164B Date 3-11-82 Eng. M.K.

Rev. 8/3-1164D 033 4-82
Dwg. No. A8/3-1164B Date 3-9-82 Eng W.W.



Note the off-stepping development of the salt dissolution zones in stratigraphically older units. Dip reversals are apparently a result of collapse over area where salt has been dissolved.

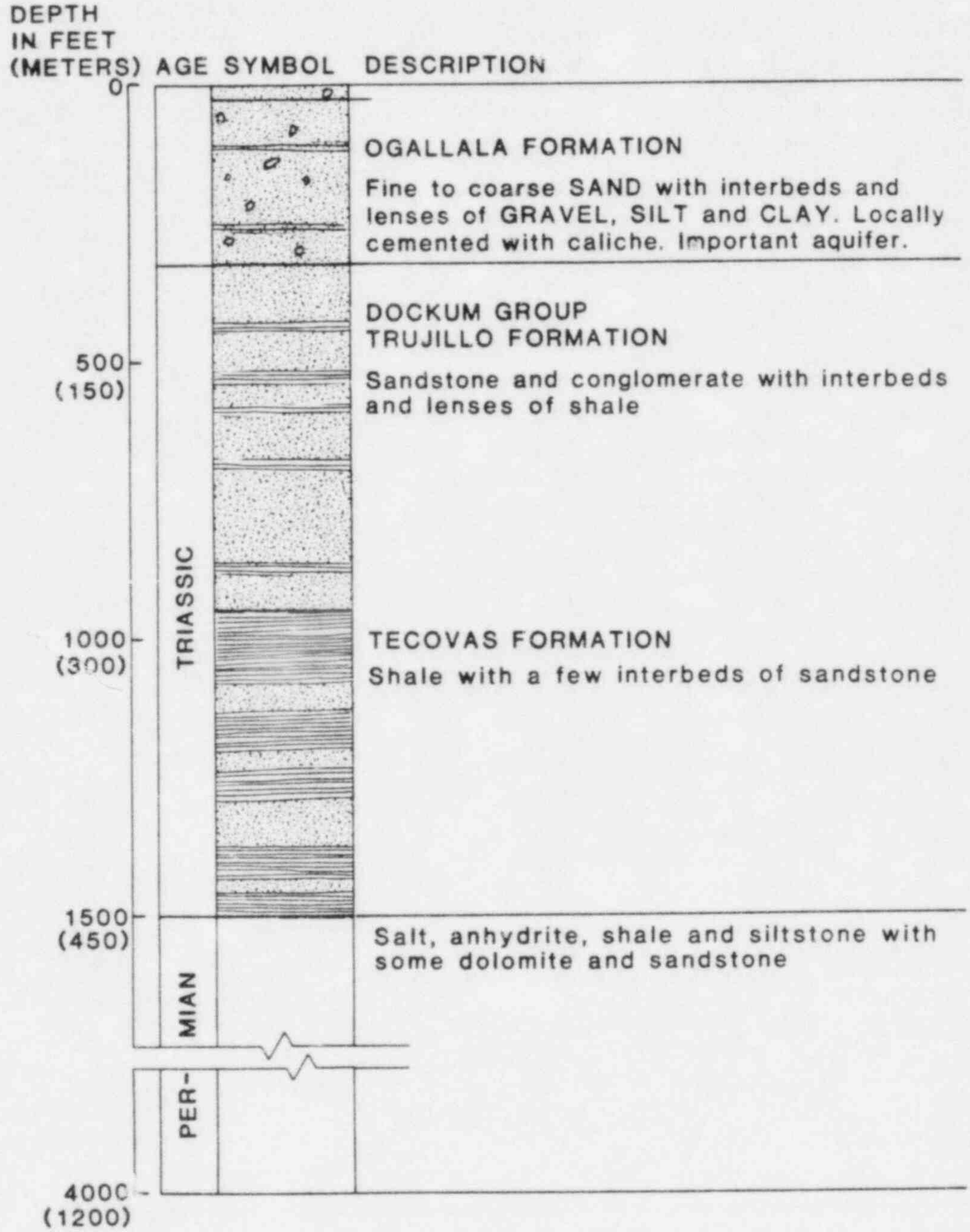
After Gustavson et al, 1981

STRUCTURAL AND STRATIGRAPHIC CROSS SECTION OF THE EASTERN MARGIN OF THE PALO DURO BASIN

Figure 5-19

STRATIGRAPHIC COLUMN OF POST-PERMIAN UNITS, PALO DURO BASIN

Figure 5-20

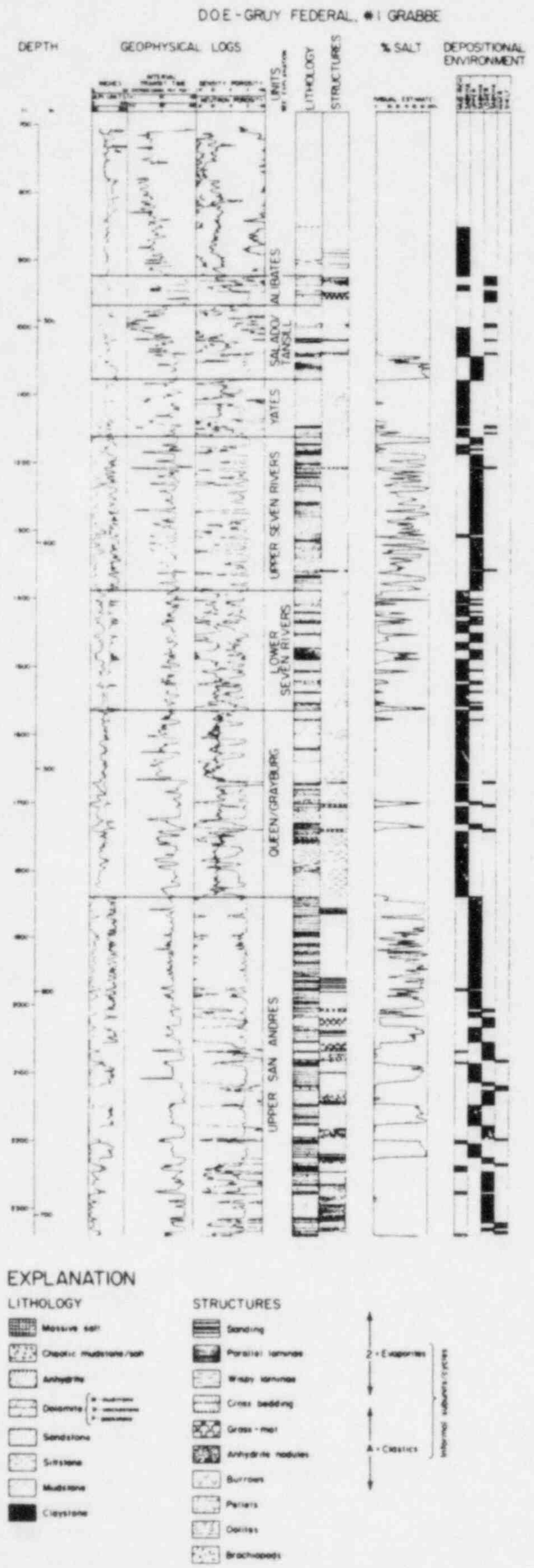


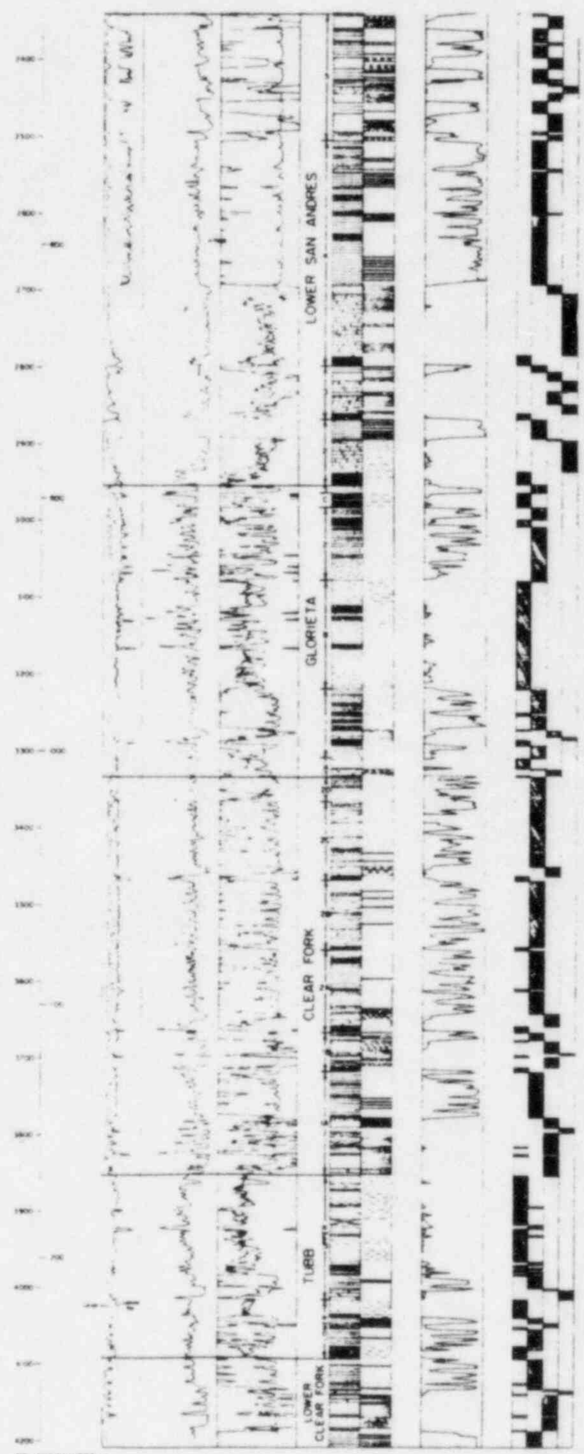
After Johnson, 1976

Rev. Dwg. No. Date 4-19-82 Eng.

CORED TEST WELL IN BEDDED SALT, SWISHER CO., TEXAS

Figure 5-21





See figure 5-19 for location

After Gustavson et al, 1981

Rev. 1164D 035 4-82 L.B.
Dwg. No. A1813-11648- Date Mar 82 Eng. W.W.

The Dockum group is up to 1500 ft thick and consists of the basal Tecovas formation and the Trujillo formation (Bachman and Johnson, 1973). The Tecovas formation consists of shale with a few beds of uncemented sandstone. The shale is weak and deteriorates in water. The Trujillo formation consists of sandstone. The degree of cementation of the sandstone is not reported in the references reviewed.

The upper Permian evaporites consist mainly of salt, with interbeds of anhydrite, claystone, dolomite, and sandstone. The thickness of the individual beds ranges from a few feet to about 200 ft. No information is available on the geotechnical characteristics of any of the units, except for the salt.

5.3.1.3 Geohydrology

Figure 5-22 is a schematic of the groundwater flow pattern in the Palo Duro Basin. The figure shows the main geologic units and the approximate permeability of each of the units.

The Ogallala formation is the most important aquifer in the region and is the main source of water for irrigation (Wyatt et al, 1977). Wells drilled into the Ogallala formation yield up to several hundred gallons per minute. The permeability of the Ogallala formation is on the order of 10^{-2} cm/sec. The vertical permeability of the formation is markedly lower than the horizontal permeability because of clay and silt layers. Perched water tables could cause dewatering problems during shaft sinking.

The Dockum group is used as an aquifer in areas where the Ogallala formation has been removed by erosion. The Dockum group has a permeability on the order of 10^{-4} cm/sec which is lower than that of the Ogallala formation because of the higher content of clay and silt and because of the better cementation of the sandstones.

The Permian and other Paleozoic units are not used for water supply because the water has a high dissolved solids content. The salt, anhydrite, siltstone, and mudstone have very low permeability on the order of 10^{-10} cm/sec. Within the impermeable units, there are interbeds and lenses of higher permeability material, including sandstone and dolomite which may have average permeability values of the order of 10^{-6}cm/sec. The permeability of the sandstone and dolomite may be several orders of magnitude higher in areas where they are more jointed. These units may be tens of feet thick and could result in large groundwater inflows and difficulty in sealing during shaft construction.

5.3.1.4 Strata Properties

The physical properties and competency of the various strata through which the shaft would be sunk are not known. While it is probable that

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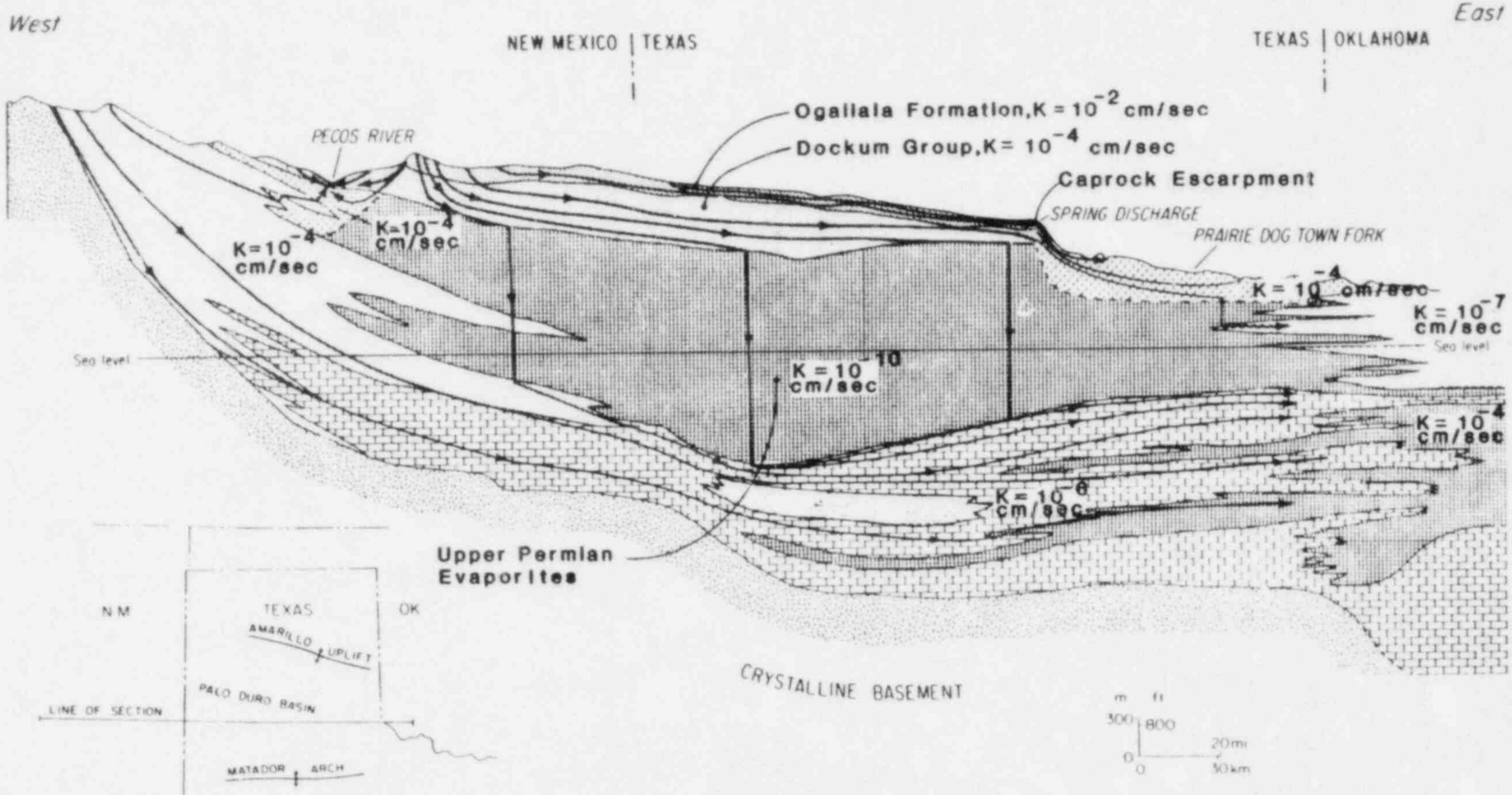


GEOLOGICAL CROSS-SECTION

After Bassett et al, 1981

CONCEPTUALIZED FLOW LINES THROUGH THE MAJOR HYDRAULIC UNITS, BASED ON COMPUTED POTENTIOMETRIC SURFACES AND AVERAGE PERMEABILITIES FOR PALO DURO BASIN

Figure 5-22

the more competent beds such as limestones, dolomites, anhydrite and salt would not present any problems to shaft sinking, the design of the support and sinking method would be controlled at the deeper sections by the weaker materials. Phenomena such as slaking, swelling of clays, squeezing, and flowing ground would be expected to some extent.

5.3.1.5 Stress State

No published results of stress measurements or interpretations for the basin are available. The most likely state of stress in the upper Permain zone which contains substantial volumes of anhydrite and salt is hydrostatic. Except for squeezing noted above, no stress related problems would be expected during shaft sinking.

5.3.2 Domal Salt

5.3.2.1 Geologic Setting

Salt domes form in areas where thick evaporite sequences have been progressively buried by sedimentary deposits; characteristically, this situation develops in subsiding environments. As temperature and pressure increase, the salt becomes unstable and the salt beds may become mobilized and intrude into the overlying sediments. Such intrusive salt bodies are termed diapirs or stocks; the term "salt dome" encompasses the salt itself, the external sheath of deformed material, and the caprock.

The intrusive evaporite sequence generally consists of halite (rock salt) with minor amounts of anhydrite and gypsum, occasionally separated by thin beds of clay or silt.

Because of their diapiric formation, salt domes have great vertical continuity. The beds are folded and sheared about vertical axes which contrast markedly with the horizontal continuity of the surrounding rocks. The domes may be continuous with the "mother bed" which is present at some considerable depth or, alternatively, they may be well separated from it. The salt becomes progressively more diapiric as the sedimentation increases and results in uplifts and basins during deposition, hence, it may exert an influence on the pattern and nature of the sedimentation itself.

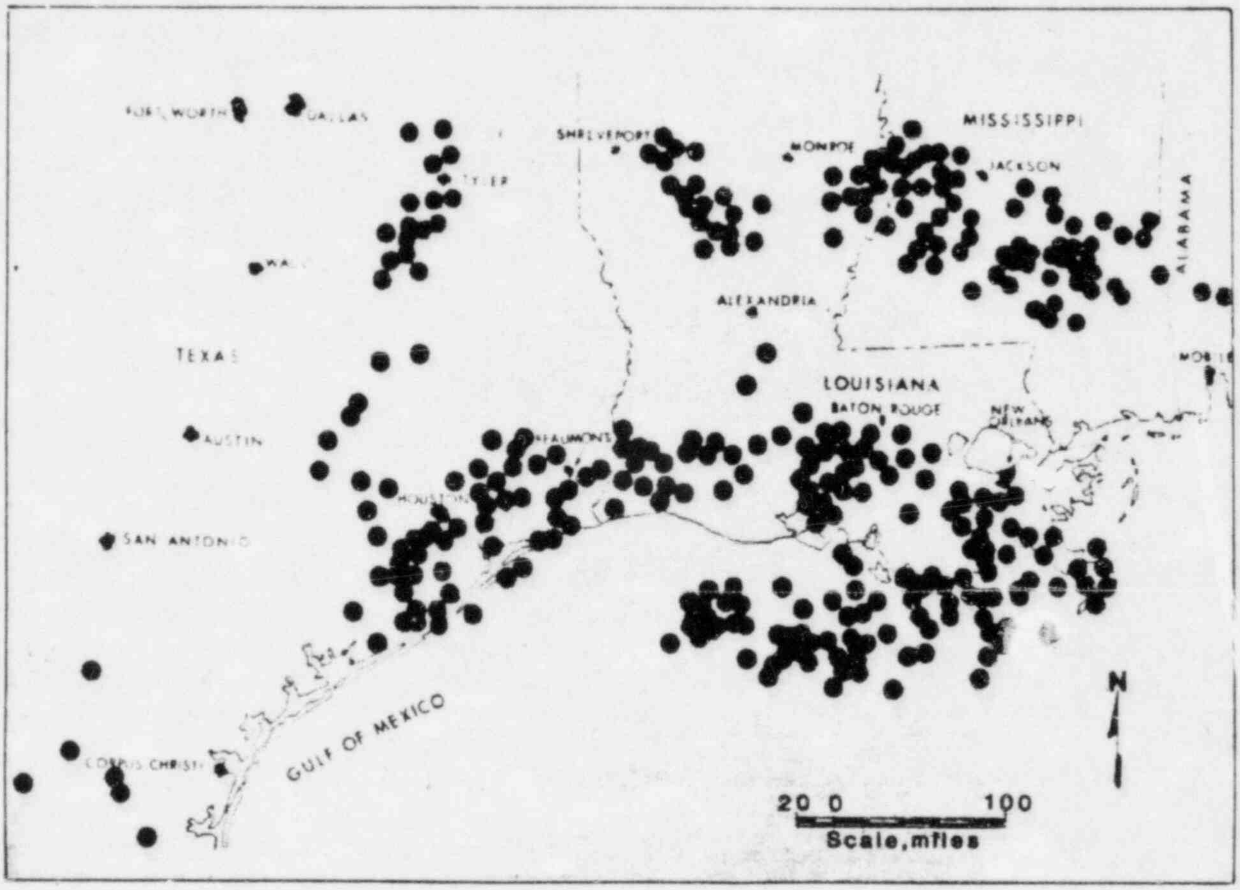
Figure 5-23 shows the locations of salt domes in the Gulf Coast region. Many of these domes have been considered as potential repository sites.

5.3.2.2 Stratigraphy

The stratigraphy of domal salts and the surrounding strata is strongly related to the history of dome intrusion. Domal salt stratigraphy is highly site dependent because of the influence of the dome on sedimentation patterns.

LOCATION OF GULF COAST SALT DOMES

Figure 5-23



After Hawkins and Jirik, 1966

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Table 5-9 shows the stratigraphy of part of the Gulf Coast area of the United States for units younger than Paleocene. The salt has originated in the Louann Formation of lower Jurassic age. Diapiric structures have been intruded from that stratigraphic level to various horizons within the geological sequence.

The stratigraphy is of major significance to underground design and construction because it determines the suitable repository level and the strata that will be penetrated by access shafts.

5.3.2.3 Structure

The structural characteristics for the salt and caprock/overburden are discussed below.

1) Salt

- Bedding

  The presence of bedding is evidenced in salt domes by color banding which varies in various shades of black to white; the darker bands are usually rich in anhydrite. The beds range in thickness from one inch to several feet. Bedding attitudes are usually steep; the coast domes of Louisiana have near vertical bedding, yet those in the north of that state have dips as shallow as 50° (Golder Associates, 1977b). Bedding has a high degree of persistence and can often be correlated between several levels in a salt mine.

- Discontinuities

  Remote from excavations, the salt is not normally jointed or fractured because of its plasticity, low strength, and ease of recrystallization, which cause it to heal any joints or fractures that might form. In one documented case of natural jointing in a salt mine, joints spaced 0.5 to 1 ft apart were found in an area of unusually hard salt (Golder Associates, 1978).

  Most discontinuities observed in salt excavations have resulted from stress-relief on unloading or from blasting. In both cases, the planes are believed to be of limited persistence. Golder Associates (1977b) also describes contact fractures between individual mineral grains and cavities or natural pockets which may contain brine or gas. The latter are often found in "anomalous zones" located near internal boundary zones or at the exterior of the dome.

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Table 5-9

STRATIGRAPHY AND WATER BEARING CHARACTERISTICS OF GEOLOGIC UNITS NEAR RICHTON AND CYPRESS CREEK DOMES

SYSTEM	SERIES	GEOLOGIC UNIT		DOMINANT LITHOLOGY	WATER BEARING CHARACTERISTICS
QUATERNARY	PLIO PLIESTOCENE	CITRONELLE FORMATION		UNCONSOLIDATED SAND AND GRAVEL. FINE-TO-COARSE GRAINED QUARTZOSE SAND WITH TRACES OF CHERT AND SOME CLAY.	A FEW WELLS CAN PRODUCE UP TO 200 GPM BUT MOST WELLS YIELD 5-25 GPM. PERMEABILITY (K) = 10^{-1} to 10^{-4} cm/s
TERTIARY	MIOCENE	HATTIESBURG AND CATAHOULA FORMATIONS, UNDIFFERENTIATED		INTERBEDDED SAND AND CLAY. SAND IS FINE GRAINED AT THE TOP AND GRADES TO COARSE GRAINED GRANULAR QUARTZ. PYRITE IS COMMON THROUGHOUT.	A HIGHLY TRANSMISSIVE WATER-BEARING FORMATION. AN IMPORTANT AQUIFER. PUMPING RATES VARY FROM 125 TO 850 GPM. K = 10^{-1} cm/s
TERTIARY	OLIGOCENE	VICKSBURG GROUP AND JACKSON GROUP		CLAY, LIMESTONE, AND MARL, MINOR INTERBEDS OF FINE SAND. LIMESTONES VARY FROM CHALKY TO SANDY AND SILTY.	DOES NOT TRANSMIT WATER READILY BECAUSE OF CLAY K = 10^{-2} to 10^{-4} cm/s
TERTIARY	EOCENE	CLAIBORNE GROUP	COCKFIELD FORMATION	CONSISTS OF BEDS OF FINE-TO-MEDIUM-GRAINED SAND. SANDY CANDONACEOUS CLAY, AND THIN BEDS OF LIGNITE. BEDS OF INTERBEDDED CLAY AND LIMESTONE OCCUR LOCALLY	HIGH TRANSMISSIVITIES AND HYDRAULIC CONDUCTIVITIES NORTH OF STUDY AREA. K = 10^{-5} cm/s
TERTIARY	EOCENE	CLAIBORNE GROUP	COOK MOUNTAIN FORMATION	LIMESTONE, VARIABLE CRYSTALLINE TO FOSSILIFEROUS, CONTAINS SOME SAND AND SILT.	GENERALLY NOT CONSIDERED AN AQUIFER, BUT CAN TRANSMIT WATER LOCALLY. K = 10^{-6} to 10^{-3} cm/s
TERTIARY	EOCENE	CLAIBORNE GROUP	KOSCIUSKO (SPARTA) FORMATION	LIMESTONE, CLAY AND SAND INTERBEDDED UPDIP OF STUDY AREA FORMATION IS COMPOSED PREDOMINANTLY OF ROUNDED QUARTZ GRAINS.	GOOD FLUID TRANSMITTING PROPERTIES UPDIP OF STUDY AREA WHERE SPARTA IS EXTENSIVELY UTILIZED FOR PUBLIC AND INDUSTRIAL WATER SUPPLIES. K = 10^{-6} to 10^{-2} cm/s
TERTIARY	EOCENE	CLAIBORNE GROUP	ZILPHA, WINONA, AND TALLAHATTA FORMATIONS	ALTERNATING BEDS OF CLAY AND LIMESTONE WITH SOME BEDS OF SILTSTONE AND SHALE.	DOES NOT TRANSMIT WATER READILY.
TERTIARY	PALEOCENE	WILCOX GROUP		INTERBEDDED SAND, CLAY AND SILTSTONE WITH MINOR SHALE, LIMESTONE AND LIGNITE. SAND IS VERY FINE-TO-FINE GRAINED.	CAPABLE OF TRANSMITTING WATER IN SOME AREAS. K = 10^{-4} to 10^{-2} cm/s
TERTIARY	PALEOCENE	MIDWAY GROUP		PREDOMINANTLY CLAY, WITH INTERBEDDED SAND, SHALE AND LIGNITE. LIGNITE, PYRITE, AND GLAUCONITE ARE COMMON THROUGHOUT.	DOES NOT TRANSMIT WATER READILY.

After Law Engineering Testing Company, 1980

- Folding

As the salt moves upward, the beds of halite become increasingly deformed and isoclinal folds develop by plastic flow and ductile faulting. These folds have near vertical axes and range in size from a few inches to thousands of feet in wave length. Parasitic folds may develop on the limbs of larger folds and, in some domes, more than one generation of folds is apparent. In the strata surrounding the salt domes, dips are generally away from the dome towards the rim synclines.

Folding of the salt is not generally of great significance for design of underground structures other than in its relationship to the process of intrusion and internal shearing.

- Faulting/Shearing

Faulting occurs both within the domal salt and in the surrounding rocks. In the salt, ductile faulting is believed to have occurred where there has been continuous permanent strain without loss of cohesion normal to the fault at the last time of motion (Odom and Hatcher, 1980). Above and adjacent to the stock, brittle faults are formed by the tensional forces created by the uplift during domal growth. Faults may also form in the rim synclines due to subsidence. The whole exterior of the dome may be considered as a ductile fault or shear because of the differential movement between salt and surrounding rocks. Ductile faults may also form within the salt where spines move at different rates and where the beds have become highly attenuated in the fold limbs.

The indication of faulting or shearing within the salt prior to excavation is often tenuous and substantiated only by indirect evidence. Open zones created by faulting tend to leak water during the short term and become self-sealing, in the long term.

Faults have important effects on strength and hydraulic conductivity and are considered to be of critical significance for design. However, with appropriate site investigation, shafts can normally be sited to avoid faults.

- Inclusions

Gas, brine, and nonsalt sediments can all form inclusions within a salt stock. These may be primary if they were deposited along with the salt, or secondary if they were incorporated into the salt during domal growth. All three types of inclusions are most commonly found near the external sheath and within the internal boundary zones.

Gas pockets of carbon dioxide and hydrogen sulphide may be encountered within the salt. Pressure pockets can also be found deeper than 1000 ft below ground, usually at spine boundaries or near the exterior of the dome. When pressure pockets are encountered, salt may be broken out of the excavation in excess of that planned by the blasting.

Brines may be found anywhere within the dome, but are most common near ductile fault zones where water was probably trapped within the salt as it pushed through the sediments.

The sediments encased within a dome are usually pod or lenticular shaped. They too are most common near internal boundary zones and the external sheath, but they may be present throughout the dome if they were deposited within the salt.

Although inclusions have provided mining problems in some parts of the world, they have not been shown to be of major significance in the Gulf Coast domes.

- Solution Features

Rocks present in the Gulf Coast domes are all of high solubility. Solution may readily occur under unfavorable hydrologic conditions. When it does occur, it could be a progressively deteriorating situation leading to collapse of the mine. Extreme caution is exercised in mine excavations in salt domes to ensure that known zones of high hydraulic conductivity are avoided.

The location of solution zones or solution-enlarged discontinuities is difficult to predict except at the stock margins. Predictive techniques including drilling ahead of the face and geophysical surveying.

Because of the potentially disastrous consequences, solubility of the salt is a critical characteristic of shaft design and construction in domal salt.

2) Caprock/Overburden

All Gulf Coast salt domes are overlain by caprock. The caprock generally has the same configuration as the salt stock and has formed during the intrusive process. It may overhang the salt on all sides and is normally thicker near the center.

Caprock generally consists of anhydrite, gypsum, calcium carbonate, sand, and traces of clays. Of these, anhydrite and gypsum make up the largest percentage of the material.

A typical caprock would consist of an upper calcitic portion interbedded with gypsum. The calcites are subject to dissolution which result in vuggy or fractured zones sometimes filled with water and/or sand and clay. Some limestones may be found towards the perimeter. The lower portion consists mainly of anhydrite containing zones of gypsum and salt stringers. The gypsum is encountered primarily in the zone of probable hydration underlying the water bearing calcite, where anhydrite has been altered.

The nature of the contact between the caprock and the salt stock is likely to vary from place to place. Conditions range from sharp and tight contacts to possible dissolution cavities.

All of the characteristics found to be of critical significance to the domal salt would be equally important in the caprock.

The nature of overburden is very much site dependent. Generally, it consists of sands, clays and limestone. Typical groundwater conditions are noted in Table 5-9. The sands and limestones form aquifers and the clays form aquicludes. At the most likely location of repository access shafts, in the central part of the dome, dominant structural features in the caprock and overburden are expected to be near-horizontal.

5.3.2.4 Geohydrology

1) Domal Salt

Laboratory permeability data indicate that natural groundwater flow in domal salt would be through fractures. Kupfer (1974a and 1974b) postulates that a zone of sheared salt containing 10 to 50 percent shale exists along the external boundaries of most salt domes. The geometry and hydraulic conductivity of these shear zones is largely unknown, since excavations avoid the outer regions of the salt stock. Many domes contain internal shear zones which separate spines that have moved differentially. Since the attitudes of internal shear zones are predominantly vertical, they are not readily intercepted by vertical exploration boreholes. Therefore, in situ tests have not been routinely performed in these zones of presumably higher hydraulic conductivity.

Naturally occurring fractures are rare in domal salt due to plastic flowage over geologic time. Stress relief fractures around excavations could intercept an existing shear zone and provide a pathway for groundwater flow.

Case studies of salt mines have demonstrated the low in situ hydraulic conductivity of domal salt. Groundwater seepage rates tend to be low and the mines remain dry without large-scale dewatering operations. The movement of groundwater is primarily through fissures and shear zones or inclusions of impure salt. Three types of fluid leaks have been observed in mines in Louisiana salt domes (Golder Associates, 1978) and a description of these leaks provides some insight on the in situ hydraulic conductivity of domal salt. They include:

- Short-Duration Leaks. These leaks are relatively common in Louisiana mines. Typically, brine containing hydrocarbons begins dripping from the ceiling after new workings are opened. Volumes may be significant (i.e., exceed evaporation rate) for several days to a month, after which flow diminishes and generally ceases within six months. The larger and more persistent drips are commonly near shear zones. These are probably flows from inclusions.

- Increasing Volume Leaks. These leaks tap water sources from outside the salt dome. Because there is a large source of fresh water in the surrounding sediments, these leaks can rapidly increase in size due to salt dissolution and cause flooding of the mine. Because of this potential flooding hazard, miners carefully seal shafts penetrating the dome and minimize excavation in the outer regions of the dome.

- Sporadic - Continuing Leaks. These leaks, found in only two Louisiana mines, occur near boundary shear zones. They begin slowly, gradually increasing to a peak discharge rate and then stabilize at a much lower rate. They are normally grouted at this time. However, new leaks commonly develop within several months, and the process is repeated.

  The leaks are thought to tap an external water source which is hydraulically connected to the mine by a semipermeable boundary shear zone. If the groundwater is not saturated with sodium chloride, dissolution could locally increase hydraulic conductivity in the shear zone and cause flooding although this has not actually occurred to date.

2) Caprock

It is generally believed that groundwater flow in upper caprock is primarily through joints, fractures and solution cavities. Indurated, unfractured caprock has a low hydraulic conductivity, probably less than 10^{-6} cm/sec. The upper parts of the caprock are often brecciated or sheared and may contain solution channels so that in situ hydraulic conductivity will be locally higher where these secondary features exist. Hydraulic conductivities on the order of 10^{-3} to 10^{-5} cm/sec were measured (Law Engineering Testing Company, 1980). Furthermore, some caprocks are unconsolidated at the salt-caprock boundary and drillers have experienced circulation losses in what they describe as "loose anhydrite sand." The presence of such zones seems likely to indicate active migration of fluids along the boundary of the salt stock.

Straddle packer tests, conducted at the salt-caprock interface of the same domes (Law Engineering Testing Company, 1980), indicated hydraulic conductivities of 7.0 x 10-6 and 1.8 x 10-6 cm/sec, respectively. These values suggest a relatively tight contact. In some caprocks, drillers have reported abnormally high hydraulic heads at the salt-caprock interface which suggests that caprock can be a relatively impermeable natural barrier to groundwater flow.

3) Overburden

Sedimentary deposits of the interior Gulf Coastal basin are characterized by thick accumulations of interbedded sands, silts, and clays. The sediments tend to be poorly lithified near the land surface and become more indurated with depth. Sediment type is the primary factor controlling permeability with consolidation and cementation playing a secondary role. Although the sedimentary formations tend to be characterized by a dominant rock type, horizontal and vertical changes in lithology result in a range of hydraulic conductivity that can exceed two or three orders of magnitude. Since the sediments are subhorizontally stratified and mainly unfractured, the vertical hydraulic conductivity tends to be less than that in the horizontal direction. The sedimentary units around the Richton and Cyprus Creek domes are summarized in Table 5-9.

The most reliable measure of hydraulic conductivity is from in situ borehole tests. Such tests are an effective measure of horizontal hydraulic conductivity, but are relatively insensitive to vertical hydraulic conductivity. The results of the borehole packer tests conducted in the Mississippi and Louisiana study areas are summarized in Table 5-9. The values presented are horizontal hydraulic conductivities of the more permeable strata and may be considered upper bounds on vertical hydraulic conductivity.

5.3.3 Composite Geological Profile - Salt

The preparation of a composite geological profile and set of conditions which are typical of both bedded and domal salt first requires a comparison of their main difference noted above. It is expected that in the overburden, the same types of ground conditions would be encountered, although the depths would differ. The synthesis of a composite profile for the lower shaft section in the salt formation is more difficult, because of the basic difference in stratigraphy.

5.3.3.1 Comparison of Bedded and Domal Salt

1) Salt

The main differences between bedded and domal salt are the extent of the salt deposits and the character of the interbedded material.

Bedded salt in the Palo Duro Basin is normally in beds that may be a few feet to a few hundred feet thick and may be continuous over an area of several hundred square miles. Salt domes have an areal extent of a few square miles to a few tens of square miles, but extend to depths of 20,000 ft or more.

All of the major bedded salt deposits contain thin interbeds and stringers of anhydrite, dolomite, clay and occassionally sand. In addition, there are thick beds of claystone, siltstone, dolomite, and some sandstone within the upper Permian evaporite sequence. All of these units will be encountered in shafts dug to repository horizon.

Domal salt contains thin bands of anhydrite and occasional inclusions that may contain dolomite, sand, clay, gas, or liquid. However, there are very few thick beds or large inclusions of dolomite, sand, or clay.

2) Caprock

Domal salt is almost always overlain by caprock consisting of calcite, gypsum, and anhydrite. The caprock is often brecciated, vuggy, and very permeable near the upper contact and tight near the contact with the salt. The caprock may be a few tens of feet to several hundred feet thick. Bedded salt is not overlain by caprock.

3) Overburden

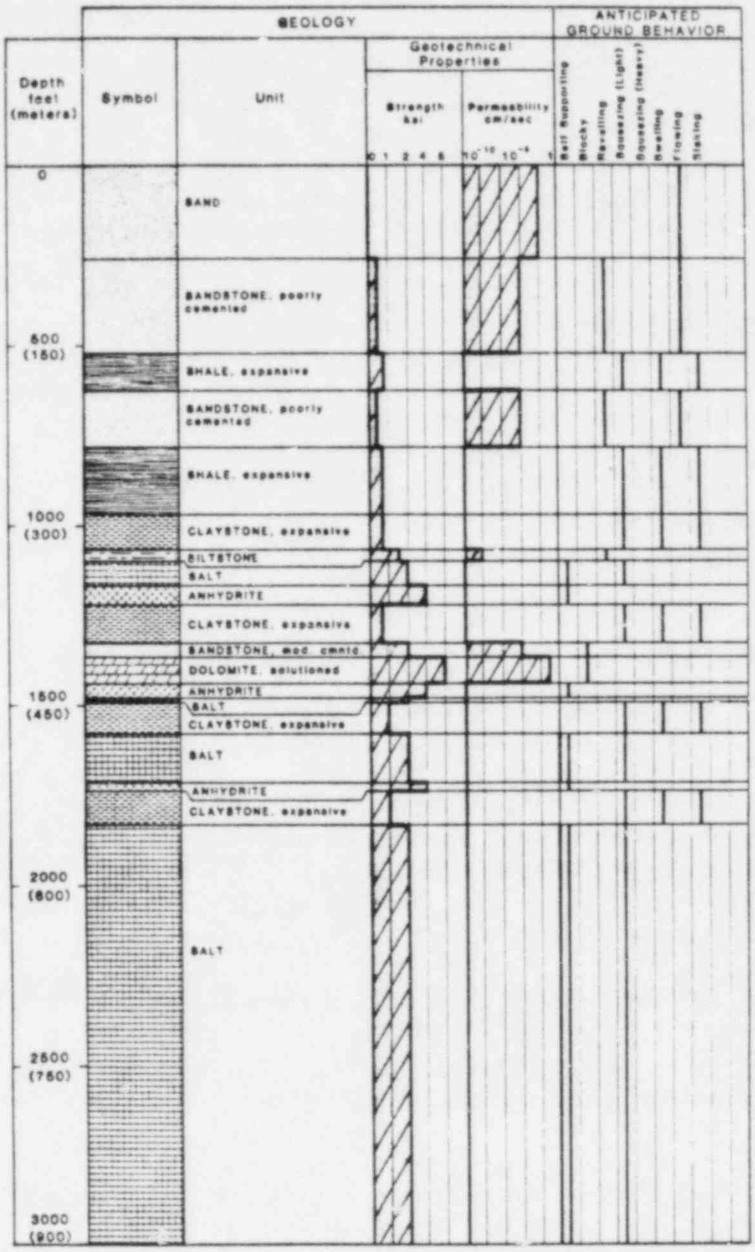
Both bedded and domal salt are overlain by sedimentary formations. There is no available information on the mechanical characteristics of the overburden in either case. However, it appears that the overburden in the Palo Duro Basin is more competent and better lithified than the overburden above the salt domes. In both cases, the overburden includes permeable sands and low permeability clays and silts. The clays are compacted, but do not appear to be cemented, so that on exposure they will most likely deteriorate with time. The clay interbeds will increase the difficulty of dewatering the sands and silts.

5.3.3.2 Composite Profile

The concept for the use of composite profiles for the present study is given in Section 5.1 The composite geological profile shown in Figure 5-24 is considered to best represent the geology of the bedded and domal salt sites from the point of view of shaft design and construction requirements. The profile has been drawn for a repository depth of 3000 ft which is considered to be a reasonable depth for both applications.

COMPOSITE SECTION - SALT

Figure 5-24



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The salt domes currently being considered as potential repository sites have 500 to 800 ft of caprock and overburden (Law Engineering Testing Company, 1980; DOE, 1981). For bedded salt sites, the corresponding overburden depth are of the order of 100 to 1500 ft.

Because of the greater probability of locating a first repository shaft in bedded rather than domal salt, emphasis has been given to this medium in the formulation of typical stratigraphic characteristics. Compared to domal salt, the overburden is more lithified. Geohydrological conditions can be defined as alternating layers of permeable sands, sandstones, and impermeable clays or shales. Typical aquifer permeabilities would be in the range 10^{-2} to 10^{-4} cms/sec. The inferred inflow rates suggest that for all aquifers below the surface, extensive positive control of groundwater would be necessary prior to shaft sinking to the respective horizons.

Except for the possible impact of squeezing clays and slaking/swelling shales on the design and timing of the lining process, no other constraints arising from temporary and permanent stability of the strata are expected. The key to the excavation and support of the shafts would be in the control of the groundwater.

The stratigraphic section for the middle portion of the shaft has purposefully been selected as being more typical of that for bedded salt. This sequence compresses in order of predominance, salt, anhydrite, shale, dolomite and sandstone. Sandstone and shale conditions are likely to mirror those encountered in the overburden. For these strata, ground preparation and control procedures similar to those predicted for the overburden would be required, although drier conditions are anticipated. The possible existence of solution features in the anhydrite and dolomite should be considered in the shaft design.

The salt can be considered competent, impermeable and requires no temporary support. However, it is subject to considerable creep deformations around an excavation and requires careful management of inflow water, e.g., proper sealing of fresh-water aquifers at the higher elevations.

The uncertainty regarding the development of long-term pressures on the lining as a result of creep requires that for repository shaft applications, a conservative lining design be adopted (Muir and Cochrane, 1966). The alternating sequence of the strata with differing excavation/support/foundation requirements will have an unfavorable effect on the progress of shaft sinking.

6.0 DETAILED SHAFT DESIGN

6.1 INTRODUCTION

This chapter defines the various shaft sinking and lining designs considered to be most appropriate for each of the two composite media formulated in Chapter 5. The designs were finalized in conjunction with The Cementation Company of America.

In an attempt to include all possible comparisons of interest, the rotary sinking method has been subdivided into two groups, those methods which have bottom access and those methods which do not. This is illustrated in Table 6-1. This subdivision will allow the following direct comparisons for at least one composite medium:

1) Comparison of conventional and rotary methods where there is no bottom access, as for example in a first shaft

2) Comparison of ream-and-slash and rotary methods where bottom access is available

3) Comparison of the two most suitable methods of rotary sinking in which there is and is not bottom access, respectively.

This third comparison is particularly interesting since depending on the ground it is not necessarily advantageous to utilize a bottom access for a production shaft even if one is available.

Within the constraints of each method, the optimum set of shaft sinking conditions, i.e., shaft design, have been determined based on experience, thus alleviating the need for subsidiary comparisons of details such as the best method of lining, grouting etc. This is applied in a simple manner to conventional shaft sinking and ream-and-slash methods to determine for example, optimum drill-blast-muck-support cycle details. This application of this approach to the rotary methods requires firstly a choice of the best method, i.e., drill or bore, full face or ream, and then a selection of the machine and construction details.

For the rotary method with bottom access, raise drilling has been chosen as the most promising. For the rotary method without bottom access, large-diameter drilling in mud using top-drive equipment has been adopted. These two choices are virtually dictated by the current technological limitations.

It must be emphasized that while seven separate shaft designs have been formulated, some in fact might not be economically sensible or practical and might never be proposed in reality. However, for the present purposes of comparisons, these "marginal" designs have been retained. The essential features of each method are given in Table 6-1, while details are provided in Sections 6.2 and 6.3 of this chapter.

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SUMMARY OF DETAILED SHAFT DESIGNS

Table 6-1

	BASIC METHOD			
	CONVENTIONAL	REAM-AND-SLASH	ROTARY	
BLIND				
BOTTOM ACCESS				
HARD ROCK (0-4000 ft)	DRILL-BLAST-MUCK CYCLE WITH INTERMITTENT PROGRESSIVE LINING (Grout during sinking)	RAISE DRILL AND SLASH WITH PROGRESSIVE LINING DURING SLASHING (freeze to full depth)	MUD DRILL FULL DIAMETER USING TOP DRIVE EQUIPMENT (float steel liner)	NOT CONSIDERED FEASIBLE
SALT (0-3000 ft)	DRILL-BLAST-MUCK CYCLE WITH INTERMITTENT LINING (Grout during sinking)	RAISE DRILL AND SLASH WITH PROGRESSIVE LINING (Freeze to top of salt)	BRINE MUD DRILL FULL DIAMETER USING TOP DRIVE EQUIPMENT (Float steel liner)	STAGE REAM FROM BOTTOM-CONCRETE LINE AFTER (Freeze to top of salt)

Many assumptions have had to be made in order to extend the engineering detail of the shaft facilities sufficiently to be able to develop the required reasonable estimates of construction costs and schedules. The major assumptions that were made are noted. Many are concerned with the geology through which the shafts are to be sunk. Thus, as the geology intimately influences the construction techniques and water inflows into the shaft, it has a very significant effect on cost and time schedules.

The method of shaft construction in many cases is dictated by the water control methods, if any, that are required. The alternatives which are available and have been considered for the geological conditions of this study are freezing and grouting. Due to the depths of operation and quantity of water involved (in both the salt and hard rock media), pumping methods are not considered appropriate.

The assumptions regarding freezing and grouting have a major impact on the cost and schedule estimates. Freezing offers a technique which produces positive results and overall time schedules which can be relied upon with a high degree of certainty. As well as preventing water inflow, it also strengthens the ground and allows excavation in generally poor ground with a minimum of extra support. However, there are a number of factors which should be considered which affect both the technical and economic viability of the process.

Firstly, it is generally considered that a groundwater flow rate of 3 to 5 ft/day is the maximum that can be tolerated for a successful freeze operation. Secondly, groundwater composition and temperature have a considerable effect on the economics of the freezing operation. The presence of salts in the groundwater can lower the freeze point considerably such that a more extensive freezing operation will be required. Similarly, the presence of high groundwater temperatures would have the same effect. Based on the conditions presented in Chapter 5, it has been assumed that no anomalies exist and that a conventional brine coolant can be used and that the freezing schedule will be unaffected.

Grouting can impart both strength and impermiability to the ground, but unlike freezing cannot be conducted successfully in all types of ground. It can be carried out in either of two ways or a combination of both. The shaft can be pregrouted through a series of holes drilled and grouted around the shaft perimeter from the surface prior to the shaft being sunk or through holes drilled from within the shaft as it is deepened and reaches each water-bearing zone. Pregrouting from the surface has the advantage that particularly where a large part of the shaft is potentially water bearing, its use can result in significant time savings to the overall sinking schedule, but it requires the use of accurate drilling techniques. In some types of ground where the hole spacing must be close and particularly where the holes must be deep, the difficulty of drilling and the problems of having to carry out grouting at long range make the system technically and economically unsuitable. In-shaft grouting is used in the present designs, where appropriate, for this reason.

Lini~ ~nd sealing requirements have been given careful consideration. When the ground is allowed to thaw after freezing and even after grouting, some residual water is able to enter the shaft excavation and penetrate the lining through any unsealed construction joints that may have been formed as a consequence of the lining technique used. This can be minimized by using correctly designed construction joints, backsheeting and relief pipes. A water stop incorporated into the joints can help in this situation, but backsheeting is essential in that it forms a barrier between any water on the wall and the concrete, thus ensuring a good finished product. In addition, it creates a circumferential channel between the concrete and the rock into which a high shear cement grout can subsequently be injected to prevent water migration. This is called backwall injection.

French drains, or gravel-filled channels, formed behind the lining are also an important part of a properly designed backwall injection program. These drains catch and control the water by leading it into pipes and directly to pumps until the backwall program can fill up and seal the drain prior to forcing the grout up behind the lining to effect a final lining seal. Any final residual water make into the shaft will be caught in a water ring and handled conventionally by pumping to the surface.

These factors have been carefully considered in the choice of water control methods used for each of the shaft sinking techniques considered in these shaft designs and estimates.

In the shafts for salt, it is essential that the final water make into the shaft be essentially zero by the time the shaft has reached the salt contact zone so that the dissolution of the salt by the unsaturated water does not cause undermining of load-bearing structures. Thus water control techniques must be applied to ensure that there is no final water inflow. In addition, a seal is constructed behind the shaft lining to prevent the migration of water down behind the lining and into the zone containing soluble salts. The purpose of the seal is to incorporate a means by which the rock and concrete lining can be post-stressed at a pressure in excess of the estimated pore-water pressures at that level. While the quantities of water anticipated in this region are small, such flows over a period of time if allowed to pass into the soluble salt zone, could develop a wash-out which could conceivably put the shaft lining at risk.

Finally, it is noted that all the detailed designs are drawn from the category of shaft sinking methods described as currently technologically feasible. A subset of this category is whether the method has been proven or not. Both proven and nonproven methods have been included. A method is described as technologically feasible if:

- Equipment is currently available to apply the method at the scale proposed without regard to nontechnical factors

- The method has already been applied at some smaller scale and an extension to larger or deeper shafts or more onerous conditions does not present new technological barriers
- The method has been applied under very similar conditions to that proposed.

Only in the third case, can the method be considered proven. If the method incorporates a new concept which has not actually been tried, such as lining using concrete segments or slipforming in a mudfilled drilled shaft, it is considered beyond the available technology and is excluded from consideration here.

6.2 HARD ROCK

The shaft designs for the three shaft sinking methods proposed are given in Figures 6-1 to 6-3. The geological conditions are also shown for ease of comparison. In these shafts it is not essential to keep the water make as low as for the salt shafts because a small water flow into the hard rock shaft does not threaten the integrity of the shaft and the repository. However, water inflow will be kept to a practical minimum in order to keep down pumping costs over the life of the repository and to reduce the impact of shaft construction on the geohydrology of the area.

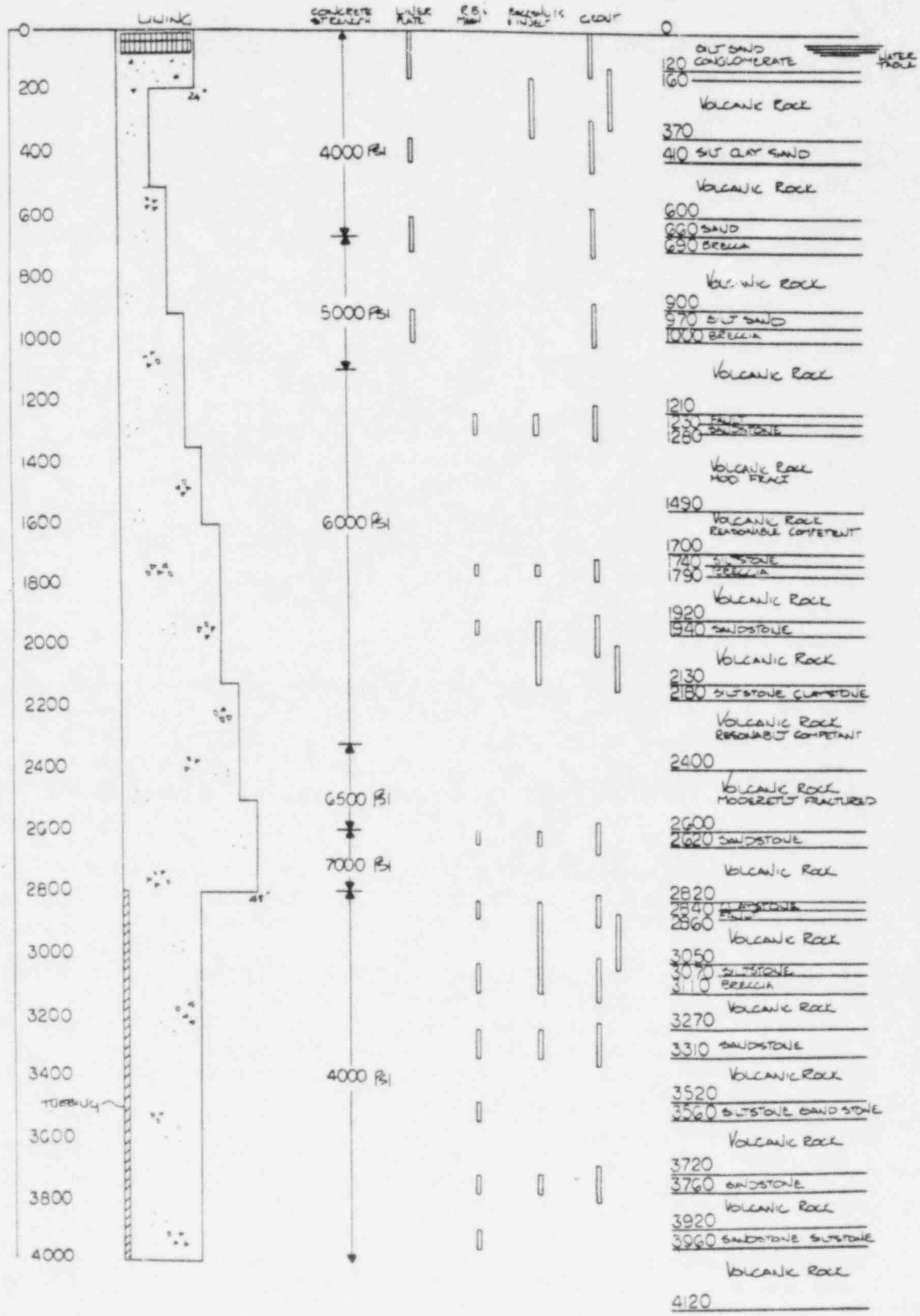
6.2.1 Drill-and-Blast

The method proposed involves full face drill-and-blast excavation with staged lining construction in 20 ft lifts immediately behind the shaft bottom. The controlling factor on the design of the lining and ground improvement measures is the discrete zones of interbed sediments and breccias, the majority of which are regarded as highly permeable and saturated. These water-bearing rocks are amenable to grouting.

Thus, to minimize disturbance to the groundwater regime and to limit inflows to 100 gpm, a hydrostatic lining will be required. A graduated thickness concrete lining with a thickness of 4 ft and a strength of 7000 psi will be used down to 2800 ft. From 2800 ft to 3900 ft, a composite lining of concrete and cast iron tubbing will be necessary. In addition, intermittent grouting of water-bearing strata will be carried out at 15 locations during sinking to seal off flows and to improve the strength and resistance to deformation of fractured zones. Back-wall injection between the lining and the impermeable membrane will be required at these same locations.

To prevent the migration of water axially along the disturbed rock annulus, both from the repository and between aquifers, a shaft seal at 3900 ft and numerous aquifer seals are considered prudent design features. Controlled perimeter blasting will be used to minimize the disturbance to the rock as described for example in Hardy and Heley (1979). The formulation of aquifer seals involves the grouting of this

SHAFT DESIGN FOR HARD ROCK- DRILL-AND-BLAST Figure 6-1



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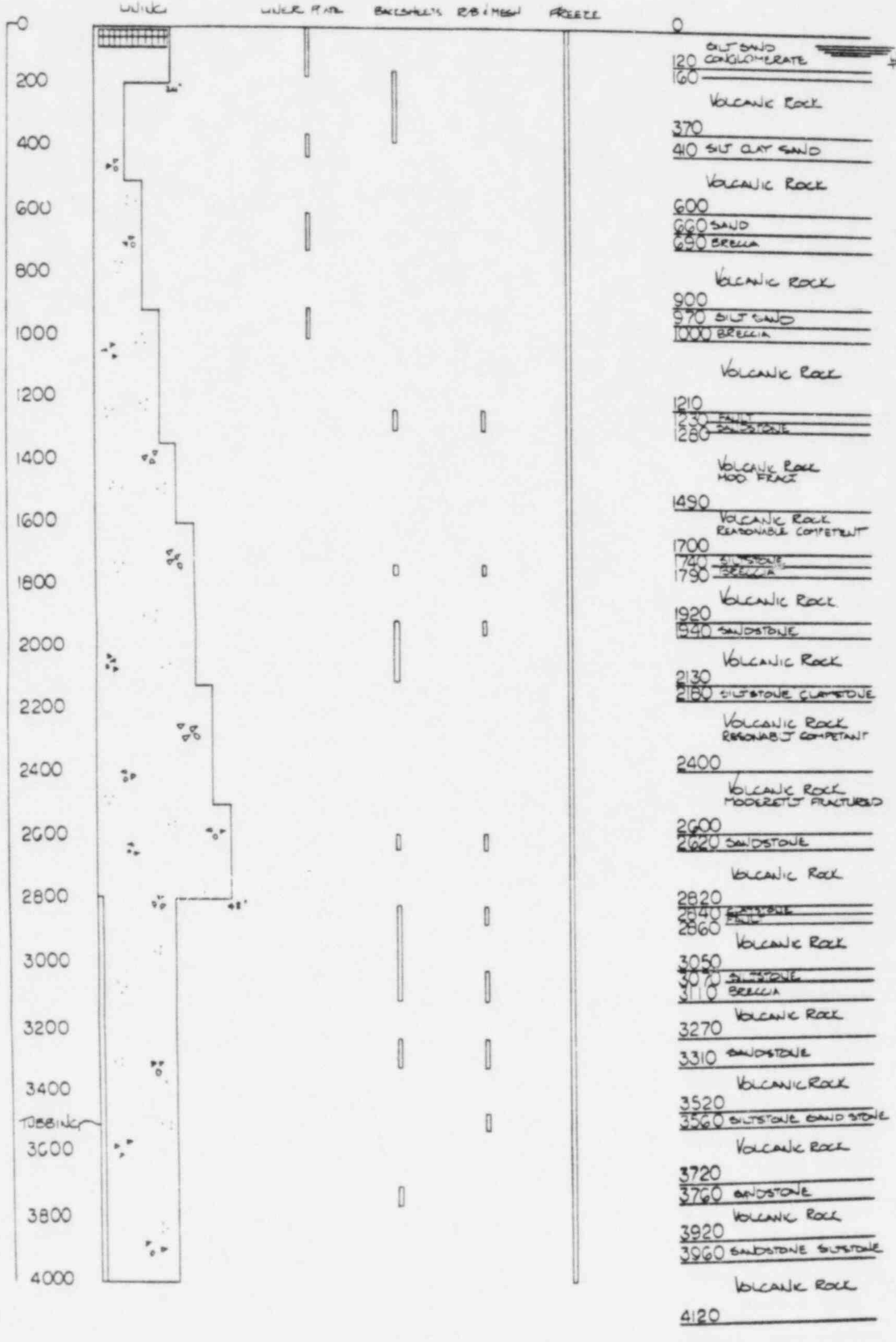
SHAFT DESIGN FOR HARD ROCK- REAM-AND-SLASH Figure 6-2



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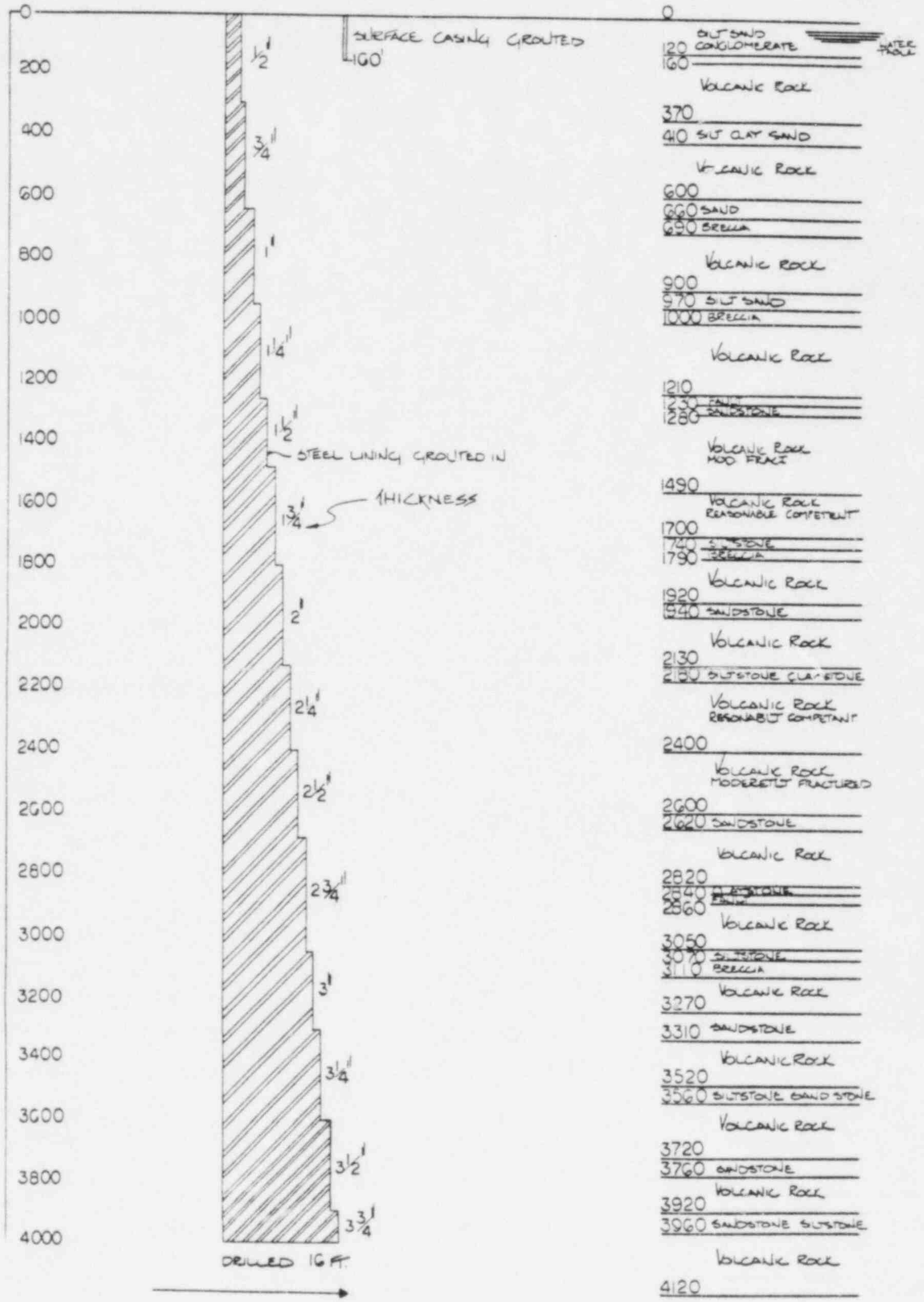
SHAFT DESIGN FOR HARD ROCK-BLIND ROTARY (DRILLING)

Figure 6-3



disturbed zone through holes in the lining with durable permanent cement or chemical mixtures. The grouting has the dual purpose of aiding construction and providing a barrier to radionuclide migration.

In the upper 1000 ft of the shaft some stability will also be required in the sand and breccia formations. This will also be achieved by the grouting. However, over these zones, while sinking, additional temporary support will be provided by liner plate. At the lower elevations where temporary support is required in the sandstones, breccias, and swelling and slaking siltstone and claystone, rock bolting and mesh will be adequate. Backsheeting and subsequent backwall injection will be required in those zones producing a residual water make into the shaft.

Temporary pump stations will have to be constructed during sinking to handle the water make. Water rings will be constructed at various intervals down the shaft to collect the residual water.

For the typical section in hard rock, the use of conventional sinking is considered technically feasible with no major problems with groundwater or stability control.

6.2.2 Ream-and-Slash

Because of the large quantity of water available to enter the shaft, it is imperative that the water be immobilized prior to the raise drilling of the pilot hole. Thus, this method would probably not be adopted for the particular ground conditions presented because of the exorbitant cost of freezing to the full depth. Apart from this excessive cost, this scale of freezing application is unprecedented and the feasibility is suspect.

Pregrouting or freezing is considered necessary to prevent the large flows of water, estimated at 50,000 gpm on average into the shaft during the raising of the 6 ft bore. This estimate is entirely speculative. Lining construction and the use of shaft and aquifer seals and back wall injection would be as for the conventional technique described above.

6.2.3 Blind Rotary

The most promising and cost effective method of blind rotary sinking is considered to be large-diameter drilling using top drive equipment, e.g., equipment similar to the Hughes Tool Co. rig. This method has been adopted here.

Geological conditions are considered appropriate to this method. The hole is drilled full of mud in one pass to full depth. Special muds are not considered necessary though weighted muds may be required to cope with unusually high aquifer pressures. In this way, good control of inflow or outflow with minimum disturbance is expected without the need for grouting.

With the hole still full of mud, the steel casing is floated into place by the progressive welding of ring sections at the collar during sinking of the vessel. The lining is then grouted in place. Lining thicknesses at 3900 ft will be of the order of 4 in. A shaft seal will be used at the 3900 ft level but because of the excellent rock surface produced, aquifer seals will not be required.

Steel linings have been installed in this way in blind bored shafts but not to 4000 ft. Thus, while the technology is available, this application is not considered proven. Alternative lining schemes using precast concrete segments or slip-forming under mud have been proposed (Skonberg, 1980) but have not been developed to date. For the present application, it is unlikely that a hydrostatic concrete lining could be constructed in this way.

6.2.4 Rotary - Bottom Access

The most promising method of shaft construction by bottom access using rotary methods is considered to be that of large-diameter raising.

This method suffers the same major disadvantage as with the ream-and-slash viz, the problem of controlling groundwater in the open shaft during raising. In this case, however, because the final raise diameter is much larger, the problems are compounded. The integrity of the exposed strata under water pressure is also brought into question. Further consideration will not be given to this shaft design as it is considered inappropriate and not technically feasible for the geological conditions.

6.3 SALT

6.3.1 Drill-and-Blast

For the geological section in salt, conventional sinking is considered a prime candidate method. To properly cope with the unconsolidated variable sediments, freezing down to 830 ft would be required. The shaft would then be collared and sunk and lined concurrently. Grouting to seal off water flows between 850 ft and 1850 ft, if required, would be carried out from within the shaft, as this is more economic than freezing to the greater depth.

The plain, mainly unreinforced concrete hydrostatic lining would terminate at 1850 ft. Backwall injection would be required down to 850 ft and between 1350 and 1450 ft. Aquifer seals would be located at 500, 600, 850, 1350 and 1450 ft. Below 1850 ft, the shaft would be unlined. This would allow the construction of optimum backfill seals during decommissioning. Where the lining crosses intermediate anhydrite or salt beds above 1850 ft, a special lining backing will be used to allow for creep effects.

Two shaft seals will be installed. One at 1450 ft, immediately below the lowest water zone and a second at 1830 ft, in the top of the salt. The upper seal is located in the most competent rock available above any rock which contains insoluble salts and so prevents the ingress of any water from the main water-bearing zones into the area containing soluble salts. The lower seal is a secondary measure to guarantee that no water can flow behind the lining to prevent leaching of the salt.

The collar will be excavated down to the water table at 30 ft using liner plate and ring beams for support. The zones containing water will be backsheeted so that a program of backwall injection can subsequently be carried out to seal the lining and to isolate the aquifers and so cause a minimum of disturbance to the groundwater regime.

Any residual water will be caught in the water ring installed at 1440 ft. Rock bolts and mesh will be required as temporary support for sinking in those zones of expansive, slaking shales and claystones. Some rockbolts and mesh may also be required for permanent support in the unlined salt section. Thus, the lining design and water control methods ensure that a secure shaft is sunk down to the salt zone, sealing off sufficient groundwater "enroute" to prevent uncontrolled leaching of the salt in the shaft bottom and the repository area.

The method is considered technically feasible. The design is shown in Figure 6-4.

6.3.2 Ream-and-Slash

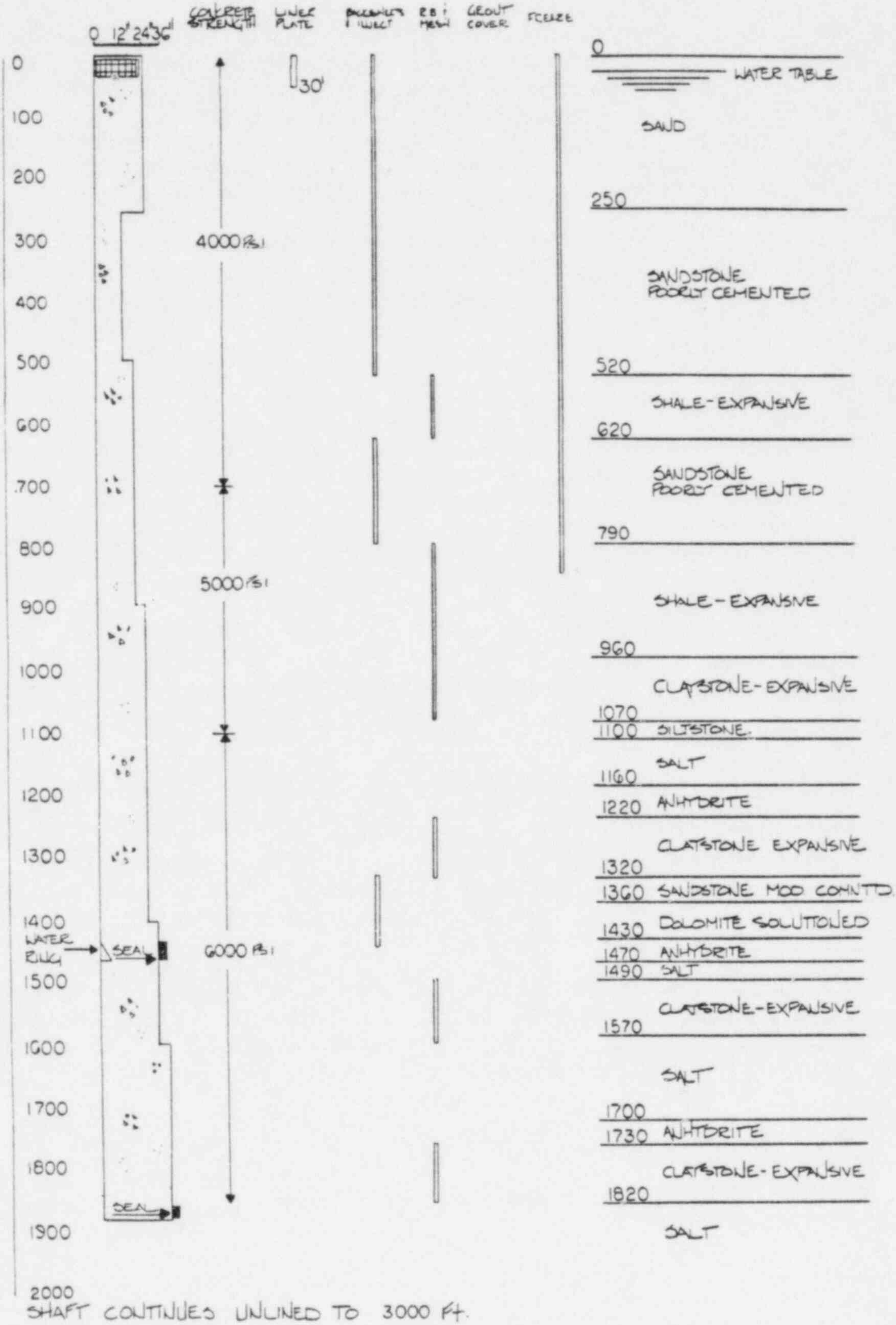
This method involves the drilling of a pilot hole to the bottom access, back-reaming to about 6 ft diameter and then slashing down again. Complete freezing to 1430 ft would be required to prevent erosion of the salt formation and destabilization of the raise bore by inflows at the higher levels. The use of back-wall injection, shaft seals and aquifer seals would be as for the conventional sinking method described in 6.3.1.

Compared to the conventional method, the question here is, "is the extra freezing requirement more than compensated for by the facility and advantages of mucking through the raise?" The method is considered technically feasible and proven. The design is shown in Figure 6-5.

6.3.3 Blind Rotary

As for the hard rock, large-diameter drilling using top drive equipment is proposed here.

SHAFT DESIGN FOR SALT- DRILL-AND-BLAST Figure 6-4



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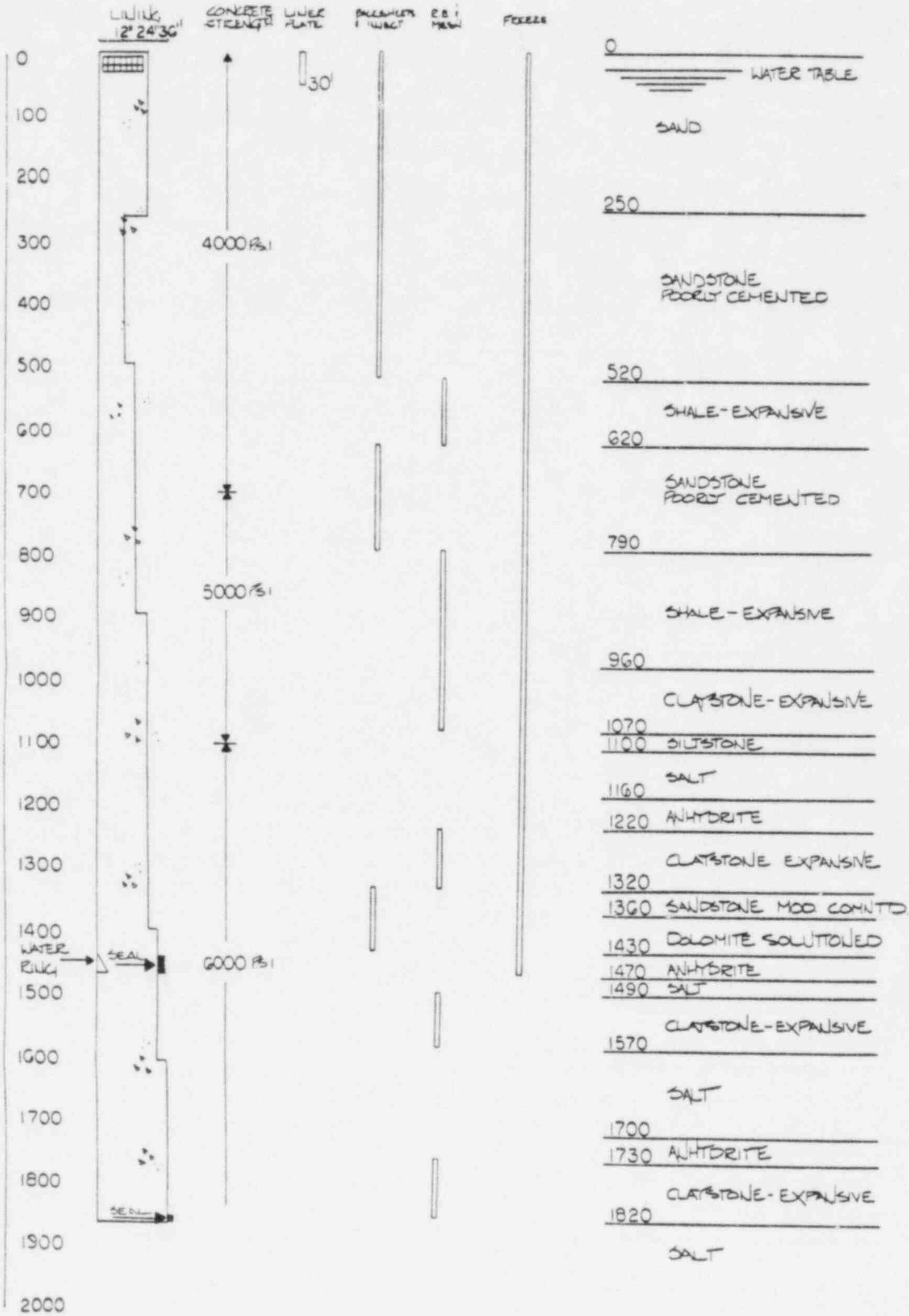
SHAFT DESIGN FOR SALT- REAM-AND-SLASH Figure 6-5



RAISEBORE AND SLASHING CONTINUES TO 3000 FT

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The drilling will be carried out with a brine-based mud to stabilize the hole and to control the water. A surface casing, 18 ft diameter will be installed and grouted to 250 ft through the unconsolidated sand. The shaft will be drilled to a diameter of 16 ft to the full depth so that a 14 ft diameter casing can be installed. The final steel casing will be floated into place by pumping mud into the blanked-off casing and supporting the residual load on the drilling rig. Additional sections will be added and welded into place as the lining is sunk.

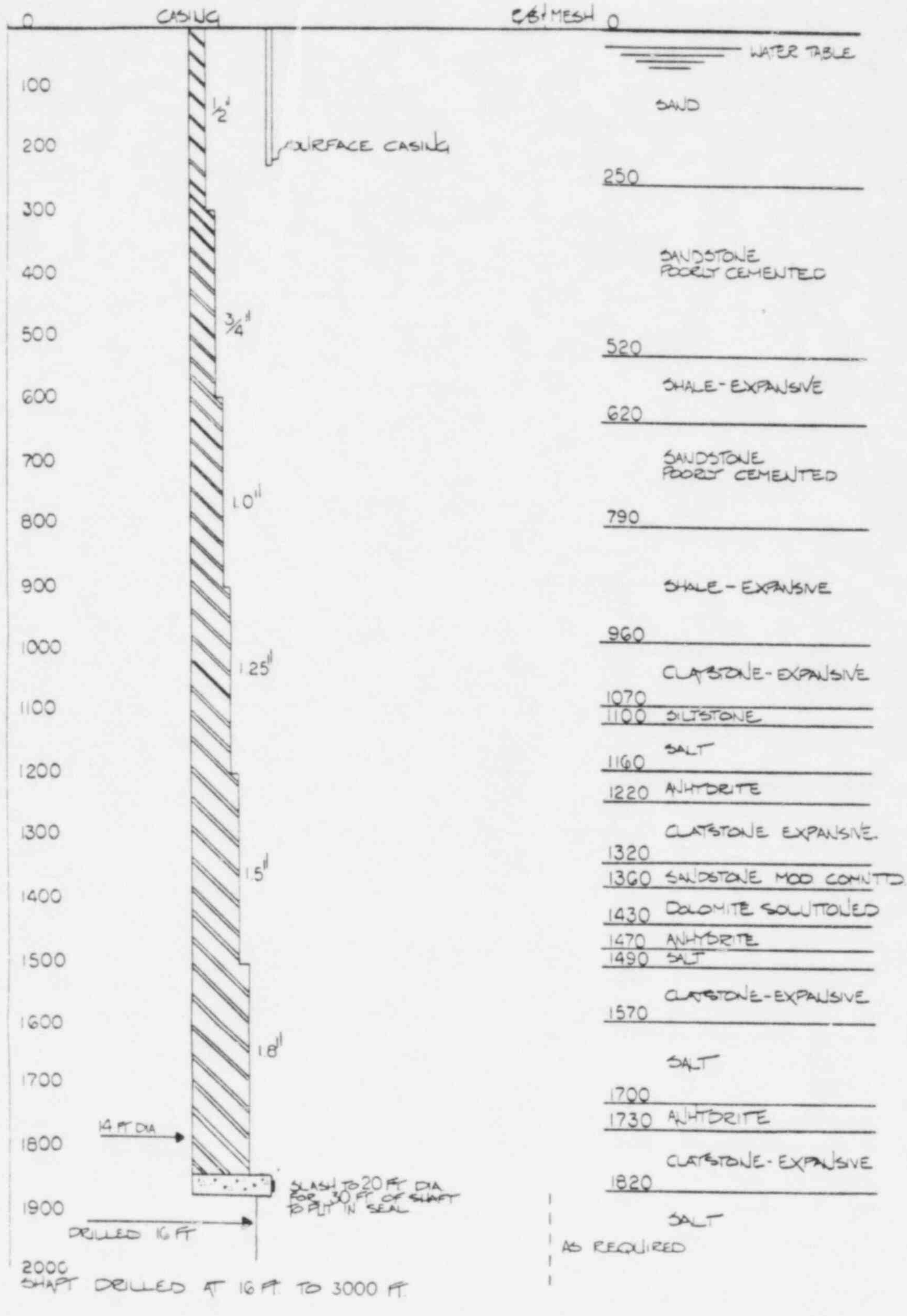
When the lining is in place down to the top of the salt (at 1850 ft), it is grouted into place. The cement grout is pumped into the annulus above a bottom seal between the steel casing and the rock and the mud is displaced by cement grout. This will be conducted in at least 2 stages. In the final stage the grout will be pumped until the return fluid is essentially pure cement grout, so that it is certain that virtually all the mud has been displaced and the grout has good contact with the steel and the rock. This should provide a good seal between the casing and the rock, but to ensure that a final seal is made at the top of the salt, a seal will be constructed in a special concrete-lined section below the steel casing, but hydraulically integral with it. The design is shown in Figure 6-6.

6.3.4 Rotary-Bottom Access

The same arguments are valid for this method as for the ream-and-slash method described in Section 6.3.2. Bottom access is necessary and so the whole section must be frozen to 1850 ft before the pilot hole can be drilled. The hole will be reamed to 17 ft so that a relatively economical hydrostatic lining can be installed. This will be installed from the top of the shaft and progressing downwards at the completion of the raise-drilling. This lining would be plain concrete down to 1300 ft except for the reinforced collar section. Below 1300 ft it would be a composite lining of an outer shell of steel welded in situ and grouted into place against the rock with an inside layer of 14 in. of concrete. High strength concrete will be used in the lower parts of this shaft. This is not normal practice as great care in handling and placing the concrete is required in order to maintain consistently good concrete in an environment not always conducive to such measures. However, it is feasible.

Shaft seals will be constructed at 1430 ft and 1830 ft depth. Backsheeting and backwall injection will be carried out at the water-bearing zones in the plain concrete-lined section. A water ring at 1440 ft will handle the residual water inflow into the shaft. The design is shown in Figure 6-7.

SHAFT DESIGN FOR SALT- BLIND ROTARY (DRILLING) Figure 6-6



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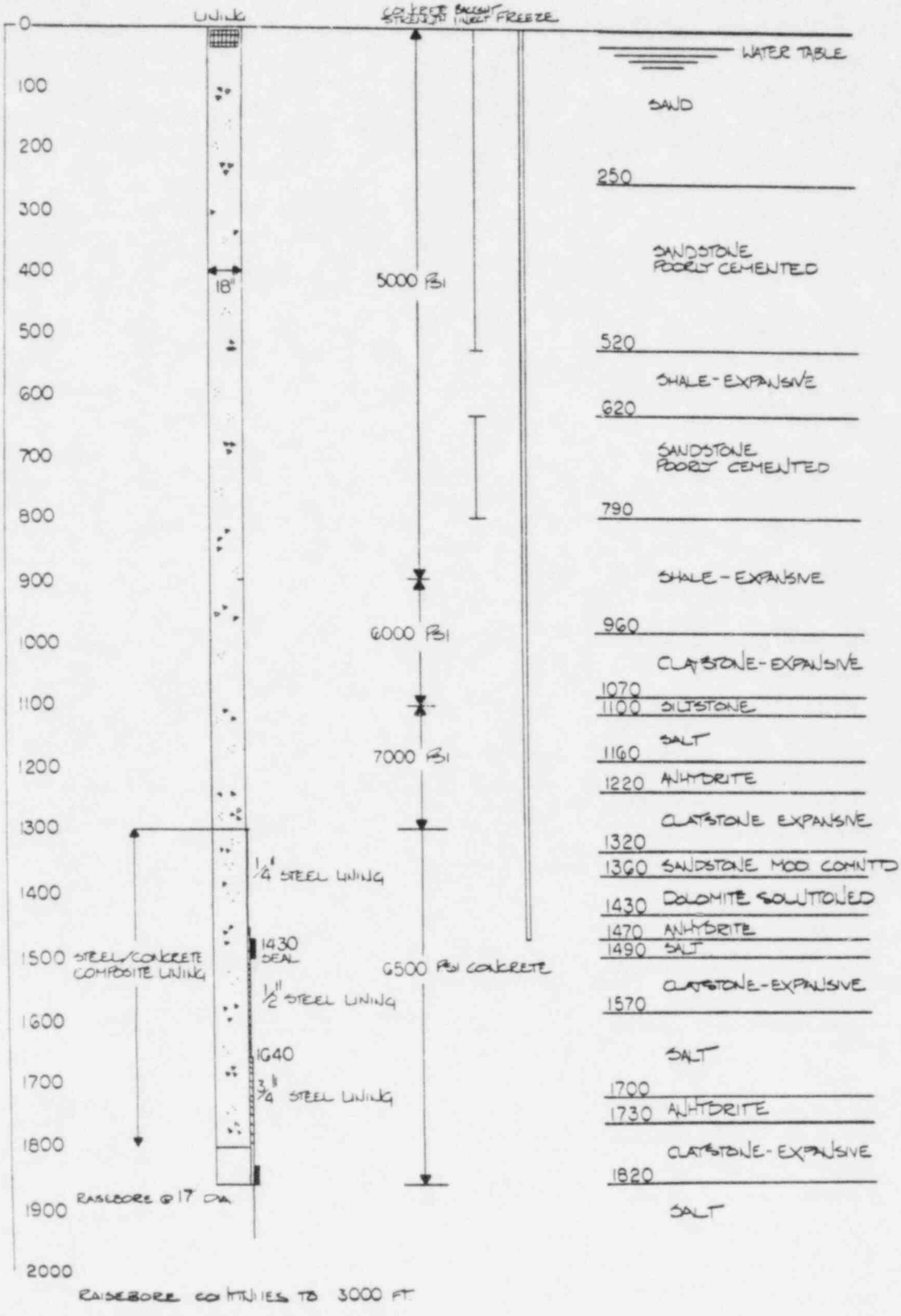
SHAFT DESIGN FOR SALT- BACK-REAMING

Figure 6-7



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7. EVALUATION OF SHAFT SINKING METHODS FOR REPOSITORY APPLICATIONS

7.1 INTRODUCTION

The seven shaft designs developed in Chapter 6 are summarized here:

	Hard Rock	Salt
• Drill-and Blast	o	o
• Ream-and-Slash*	o	o
• Blind Rotary (Drilling)	o	o
• Back-Reaming*		o

*Requires bottom access.

The practical significance of these designs is evaluated in terms of cost, time of construction, and short- and long-term repository performance.

Blind rotary boring was eliminated from further consideration in Chapter 6 because it was not considered to be currently technically feasible, especially for depths of 3000 ft or more.

For each of the composite media, the cost and schedule estimates and the review of technical factors are provided. These estimates were prepared independently by The Cementation Company of America. The review of technical factors also has a slightly broader approach and considers the attributes of the methods within the context of a range of geological and geometrical conditions. This review is given in Section 7.5 which also collectively evaluates both cost/schedule and technical factors for each method with particular regard to the suitability, advantages and disadvantages for repository application.

7.2 ESTIMATING ASSUMPTIONS

The evaluations are restricted to the shaft sinking and lining operation itself, i.e., the effect of schedule on capitalized cost to the repository is not considered. Other assumptions, some repeated here for the sake of completeness are:

Geometry

- The shaft is vertical, circular and hydrostatically lined to 14 ft inside diameter with concrete, reinforced concrete or an equivalent lining where appropriate.

- The shaft depths are 4000 ft and 3000 ft in hard rock and salt, respectively

- Shaft stations are not provided.

Construction Factors

- The sinking is carried out with minimum disturbance to the rock and groundwater regime
- At the completion of sinking, the water inflow into the shaft will be a practical minimum, but in all events will be less than 100 gpm
- Controlled blasting will be used where applicable
- Grouting and sealing will be designed for long-term requirements
- Only currently available technology will be assumed
- It is assumed that several shafts will be constructed at the site using the same method.

Siting

- Equipment will be transported 1000 miles
- Power, water etc., are already available on site.

Working Conditions

- Adequate skilled labor is available locally and payment will be made at Bacon-Davis rates
- No extraordinary working conditions such as snow, elevation, underground temperatures exist
- All year round working is possible.

Productivity Factors

- Some allowance is made for lost time due to interruptions for geological/geotechnical studies (drill-and-blast and ream-and-slash only)
- Small usual bidder contingency is provided for breakdowns, delays and unusual ground conditions. No allowance has been made in the estimate of schedules for the additional interference generally factored into government projects.
- Sinking schedule and utilization factors are structured for critical path construction requirements.

Costing Basis

- Costs are based on July 1982 rates
- General and overhead expenses of site management are not included
- No serious delays are experienced in equipment delivery
- Because of schedule constraints, temporary hoist and surface facilities are used
- The provision and installation of shaft furnishings or permanent conveyances or hoisting systems are not included
- Major equipment is written off over four shafts
- Surface facilities are shared where possible
- Grouting and freezing equipment are estimated on a shared rental basis
- Only temporary pumping requirements are considered with 100 percent back-up facilities for safety.

7.3 SCHEDULE AND COST ESTIMATES

7.3.1 Hard Rock

Figure 7-1 shows the time schedules for the construction of the seven shafts, including the three in hard rock. The drill-and-blast method has the longest schedule of 32.5 months, a result primarily of the slow sinking rate, of the order of 150 ft/month.

In comparison, the total time for the reaming and slashing operation is 31.5 months, giving an average sinking rate of 225 ft/month. Without the requirement for freezing, a 20 to 30 percent savings in schedule could be effected using the ream-and-slash.

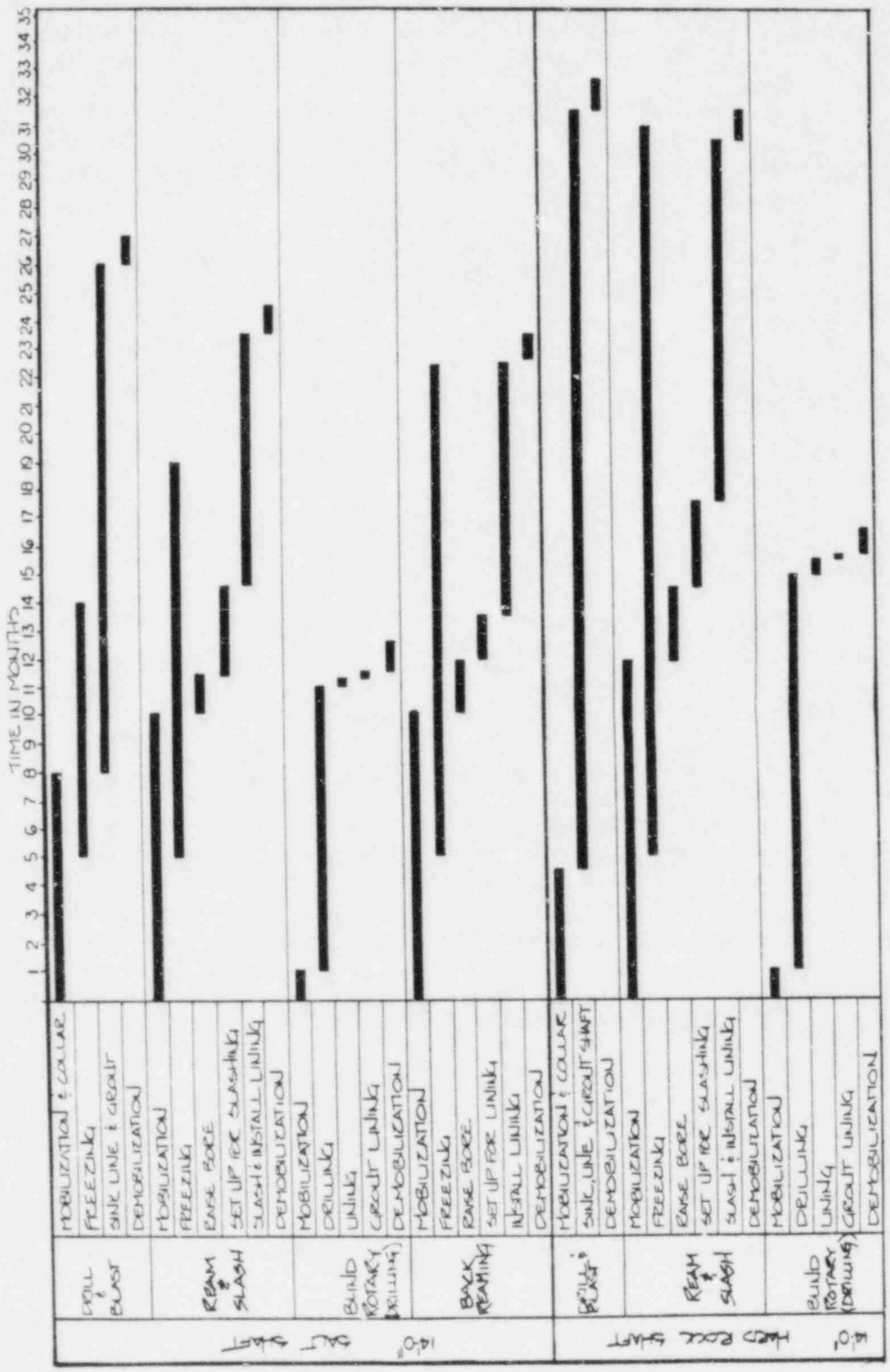
The short mobilization time and the rapid penetration rate of 290 ft/month allows the blind-drilled shaft to be completed in about 16 months. This is considerably less than for the other two methods, even though conservative estimates of the advance rate have been assumed.

A summary of the schedules and costs for hard rock are given in Table 7-1. Costs are presented in more detail in Table 7-2 and Figure 7-2.

These costs include the materials and equipment, plant rental, consumables, labor and levies, power and contractors profit that are concerned with the shaft construction. However, they do not include any

SHAFT CONSTRUCTION SCHEDULES

Figure 7-1



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SUMMARY CONSTRUCTION DURATIONS AND COSTS FOR SHAFTS IN HARD ROCK

Table 7-1

Construction Method	Construction Duration (months)	Cost (million dollars)
Drill-and-Blast	32.5	34.0
Ream-and-Slash	31.5	37.0
Blind Rotary (Drilling)	16.0	47.0

See Section 7.2 for cost assumptions

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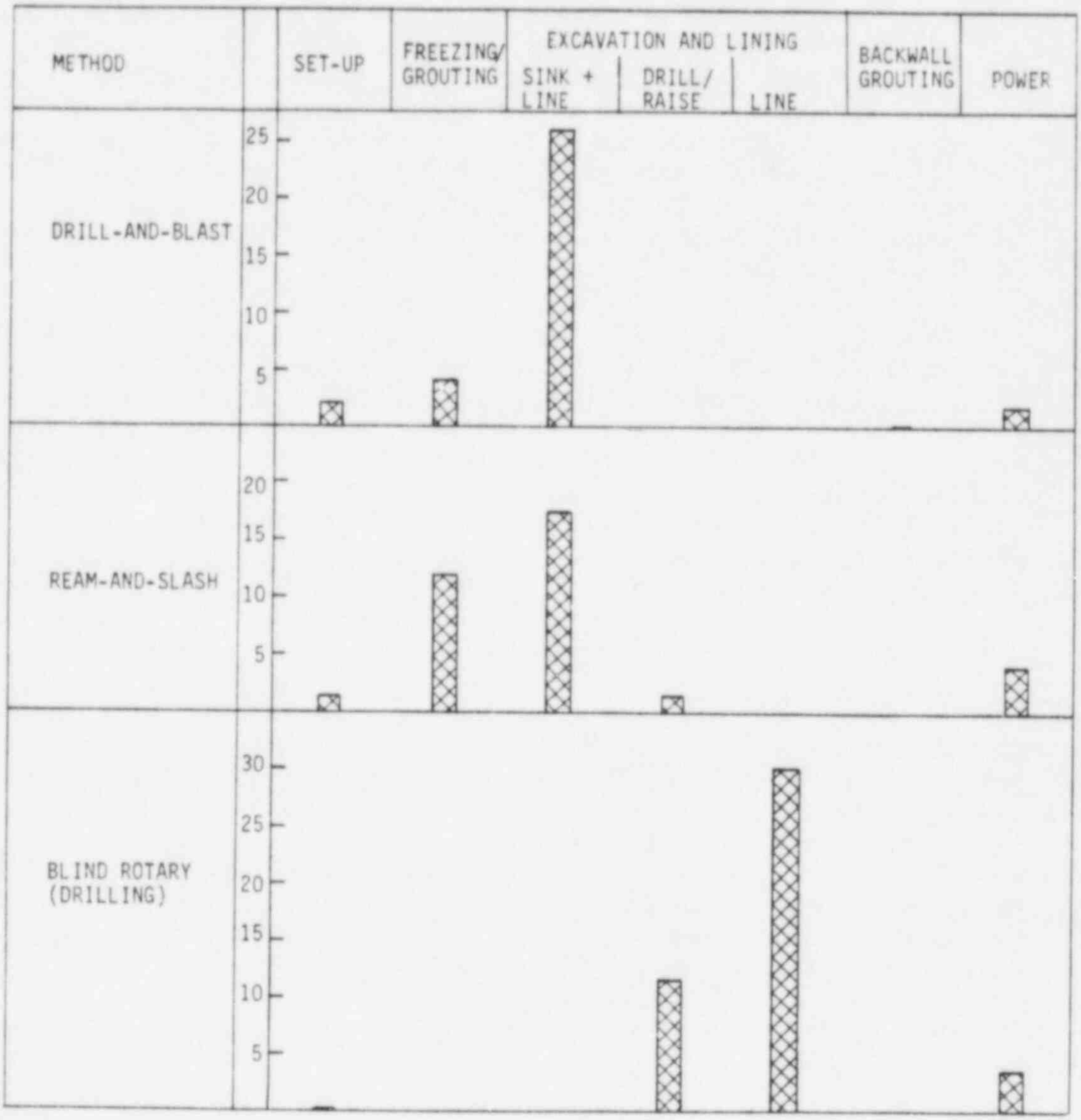
COST BREAKDOWN FOR SHAFTS IN HARD ROCK

Table 7-2

		Off Site Mobilization	Mobilization Driller	Set Up And Sink Collar To 100'	Sink Shaft And Line	Drill Shaft	Raise Bore	Install Lining	Grout Behind Casing	Backwall Grouting	Freezing	Pressure Grouting	Power	Demobilization	TOTAL
14' 0" HARD ROCK SHAFT	Drill-and-Blast	80,000		1,579,388	26,154,122					182,000		3,772,700	1,798,000	144,576	33,710,786
	Ream-and-Slash	50,000	100,000	1,173,496	17,650,000		1,520,000				12,000,000		4,000,000	136,800	36,630,296
	Blind Rotary		110,000			11,500,000		30,000,000	1,200,000				3,600,000	115,000	46,525,000

COST BREAKDOWN FOR SHAFTS IN HARD ROCK

Figure 7-2



NOTES

1) Costs are in millions of dollars.
2) Set-up includes mobilization, demobilization and collaring to 100 feet.
3) Freezing/Grouting covers preparations prior to or during sinking to control ground/groundwater.
4) A breakdown of excavation and lining costs for the drill-and-blast method is not appropriate here.
5) Backwall grouting includes grouting behind casing.
6) Power includes refrigeration, compressed air, rotary drives and mud pumping.

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estimate for the cost of disposing of the muck through the repository for the ream-and-slash and back-reaming methods.

For the drill-and-blast design, the major cost item is the sinking and concurrent lining. Even for the relatively unfavorable geological conditions, the cost of pressure grouting is only about 10 percent of the total cost. In contrast, the cost of groundwater control (freezing) for the ream-and-slash method is about 300 percent higher, or 30 percent of the total respective cost. It can be seen clearly from Figure 7-2 that given good geological conditions where freezing would not be considered necessary to stabilize the raise bore, the cost of using ream-and-slash could be as little as two-thirds that of using the drill-and-blast method.

For the blind rotary drilling design, the major cost is for the construction of the lining, amounting to about 65 percent of the total cost. As noted elsewhere, it is expected that much cheaper lining alternatives which will also be suitable for repository shafts, will be developed in the near future. Thus, it is possible that the total cost of blind rotary drilling will be comparable to that of drill-and-blast and ream-and-slash while retaining the important advantage of the much shorter construction duration.

7.3.2 Salt

From Figure 7-1, it can be seen that the sinking methods have the same relative order of duration for hard rock and salt. The drill-and-blast method is still the most time-consuming, requiring 27 months at an average rate of 170 ft/month. The ream-and-slash method at 24.5 months is slightly more advantageous in terms of construction time for salt than for rock. This is because of the higher average sinking rate (330 ft/month) and the shorter freezing time. Overall construction times for the shafts in salt and hard rock are comparable considering the two different depths involved.

Average blind drilling advance rates of 300 ft/month lead to a total construction time of 12.5 months.

The back-reaming method requires the same preparation times and results in the same advance rates as for the ream-and-slash. Thus, the total construction time of 23.5 months is approximately equal to the 24.5 months for the ream-and-slash.

A summary of the schedules and costs for salt are given in Table 7-3. Costs are presented in more detail in Figure 7-3 and Table 7-4.

The comments on the relative costs of the component items for the shaft designs in hard rock also apply here in general terms. Because of the greater importance of groundwater control for the shafts in salt, the cost of freezing relative to the total construction cost is higher. Again, the cost of only the ream-and-slash operation is particularly

SUMMARY CONSTRUCTION DURATIONS AND COSTS FOR SHAFTS IN SALT

Table 7-3

Construction Method	Construction Duration (months)	Cost (million dollars)
Drill-and-Blast	27.0	13.0
Ream-and-Slash	24.5	9.5
Blind Rotary (Drilling)	12.5	25.0
Back-Reaming	23.5	11.0

See Section 7.2 for cost assumptions

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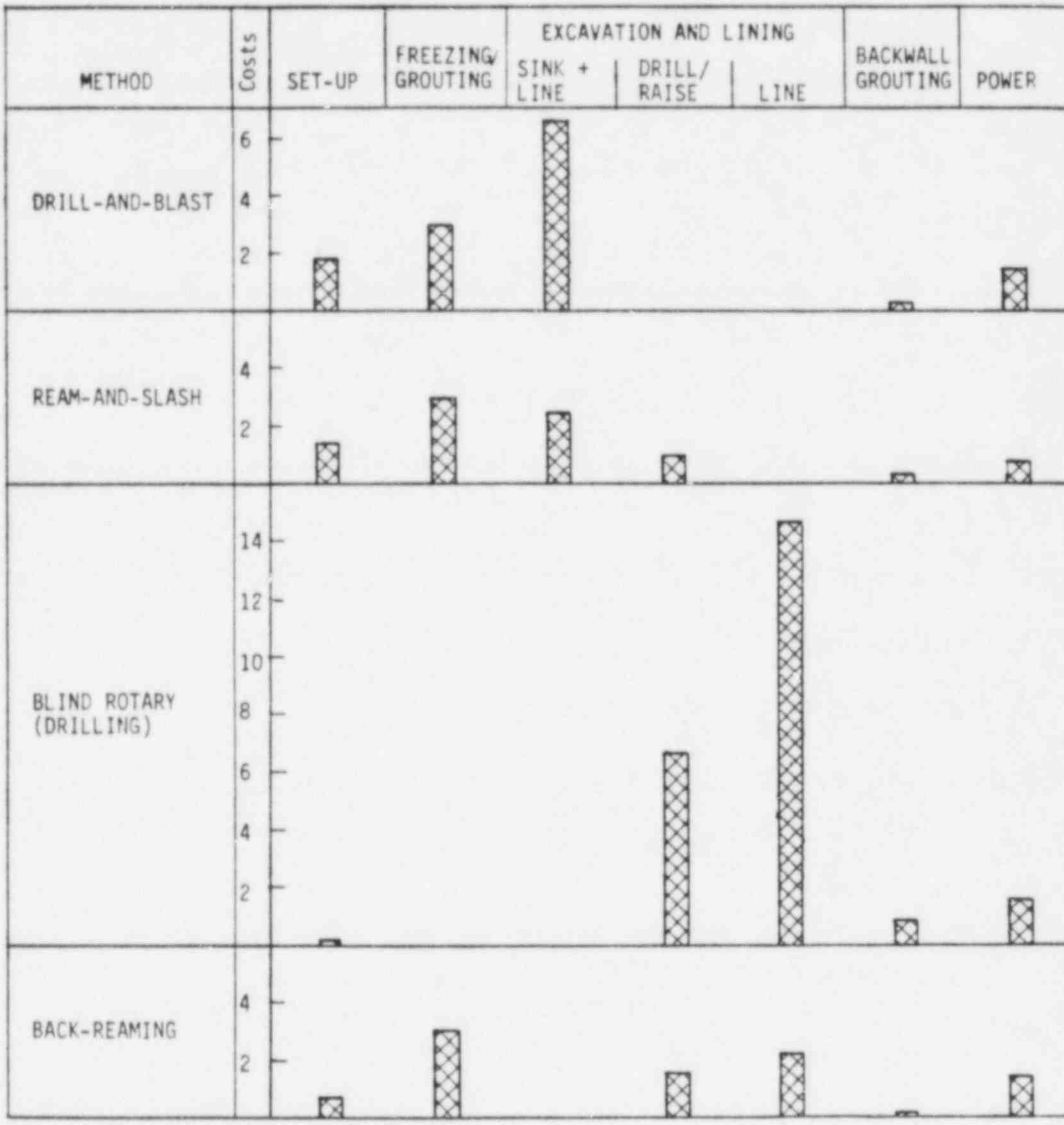
COST BREAKDOWN FOR SHAFTS IN SALT　　　Figure 7-3



NOTES
1) Costs are in millions of dollars
2) Set-up includes mobilization, demobilization and collaring to 100 feet
3) Freezing/Grouting covers preparations prior to or during sinking to control ground/groundwater
4) A breakdown of excavation and lining costs for the drill-and-blast method is not appropriate here
5) Backwall grouting includes grouting behind casing
6) Power includes refrigeration, compressed air, rotary drives and mud pumping

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COST BREAKDOWN FOR SHAFTS IN SALT

Table 7-4

		Off Site Mobilization	Mobilization Driller	Set Up And Sink Collar To 100'	Sink Shaft And Line	Drill Shaft	Raise Bore	Install Lining	Grout Behind Casing	Backwall Grouting	Freezing	Pressure Grouting	Power	Demobilization	TOTAL
14' 0" SALT SHAFT	Drill-and-Blast	80,000		1,579,348	6,520,785					250,000	2,400,000	500,000	1,400,000	144,576	12,874,709
	Ream-and-Slash	50,000	100,000	1,173,446	2,509,254		1,020,000			290,000	3,100,000		764,640	136,800	9,144,190
	Blind Rotary		100,000			6,594,000		14,601,825	878,457				2,500,000	115,000	24,789,282
	Back-Reaming	50,000	100,000	480,000	1,600,000		1,560,000	2,253,554		250,000	3,100,000		1,135,000	136,800	10,665,354

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advantageous compared to the drill-and-blast excavation. Thus, overall, the ream-and-slash method is cheaper. If more favorable geological conditions existed, the benefits of using ream-and-slash would be more pronounced.

Again, as with the shaft in hard rock, the major cost item for blind drilling is the lining operation. While this particular cost item may be substantially reduced with future technological developments, it is believed that blind drilling will not be competitive with the other methods, purely in direct cost terms unless particularly poor geological conditions are encountered.

The cost of the back-reaming method benefits considerably from a well-advanced technology, although its application to the poorer range of geological conditions is more uncertain than for the other three methods. It can be seen that if the major cost item of freezing is unnecessary, a particularly economic method of shaft sinking would be available.

7.4 INFLUENCE OF SHAFT DIAMETER

Access to the repository will most probably require the construction of a number of shafts of different diameters. So as to place the evaluations of costs, construction durations and technical factors in proper perspective, a limited quantitative determination of the influence of shaft diameter on costs and schedules was carried out.

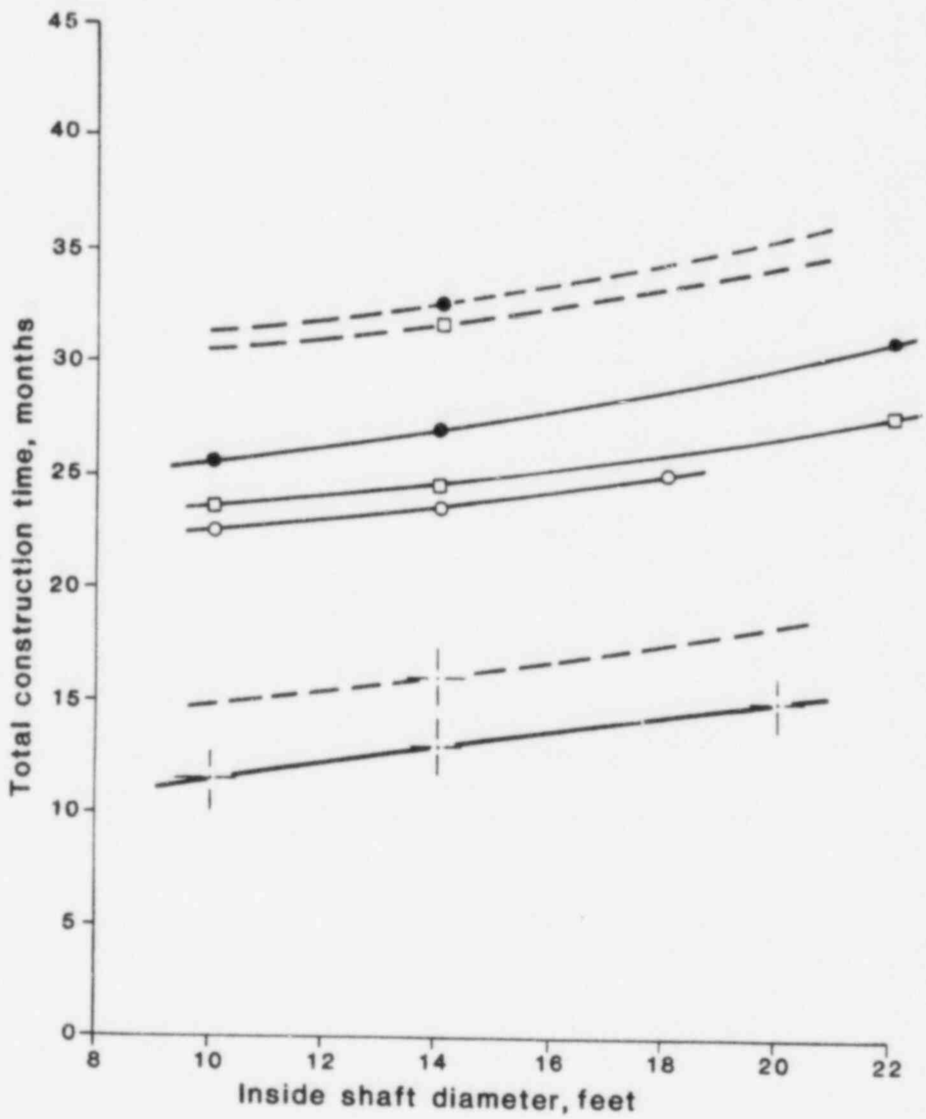
This determination was performed explicitly for salt for which four detailed shaft designs are available. The sensitivity of cost and schedule to diameter were assessed by first establishing cost and duration as a function of diameter for each of the component activities. Determinations were made for inside lined shaft diameters as shown in Figures 7-4 and 7-5. No significant departures in trends of cost and schedule with diameter could be ascertained between the three shafts designs in hard rock and the corresponding three designs in salt. The corresponding relationships for hard rock are therefore presented on the same figures with the necessary adjustment shown for absolute values of cost and construction time.

It can be seen from Figure 7-4, that the construction duration is relatively insensitive to the shaft diameter. For example, a 100 percent increase in diameter from 10 ft to 20 ft results in only a 10 to 25 percent increase in construction time. It would appear therefore that construction time is not a factor in the selection of optimum shaft diameters for the repository.

The variations of construction costs with shaft diameter follow a slightly more diverse trend. For the drill-and-blast and ream-and-slash methods, there are no inherent technological constraints on the shaft diameter that can be constructed. Thus, the cost trends for these two methods are very similar to the corresponding schedule trends. For the

INFLUENCE OF SHAFT DIAMETER ON CONSTRUCTION DURATIONS

Figure 7-4



LEGEND

———	Salt
– – – –	Hard Rock
●	Drill-and-Blast
□	Ream-and-Slash
○	Back-Reaming
–¦–	Blind Rotary (drilling)

NOTES

1) See text for method of estimating
2) Back-Reaming applies only to shafts in salt

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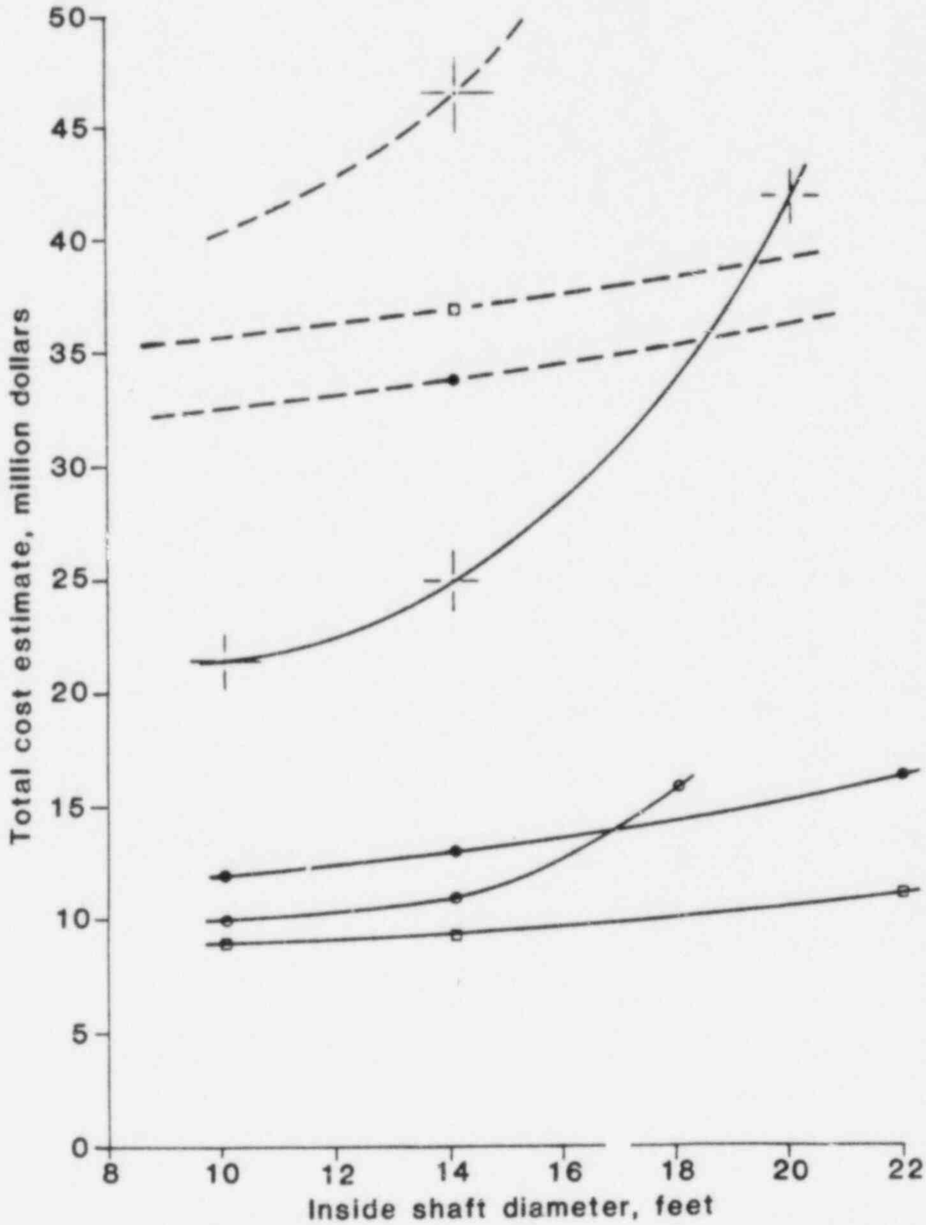
INFLUENCE OF SHAFT DIAMETER ON COSTS

Figure 7-5



LEGEND

- ——— Salt
- – – – Hard Rock
- ● Drill-and-Blast
- □ Ream-and-Slash
- ○ Back-Reaming
- —¦— Blind Rotary (drilling)

NOTES

1) See text for method of estimating
2) Back-Reaming applies only to shafts in salt

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rotary methods however, the use of the larger diameters borders on the feasibility of construction, and the attendant costs tend to escalate rapidly for the larger diameters. It also follows that for rotary sinking at large diameters, costs are somewhat less certain.

7.5 REVIEW OF TECHNICAL FACTORS AND CRITICAL EVALUATION FOR REPOSITORY SHAFTS

The summary evaluations of the advantages and disadvantages of the various shaft sinking methods in generic terms have already been presented in Chapter 4. By focusing on the conditions for each of the two composite media, and the design constraints for repository shafts, seven detailed designs were developed in Chapter 6 using four of these methods. This section reviews the technical factors for these seven specific shaft designs.

Tables 7-5 and 7-6 have been presented in an effort to rank the various significant factors for a critical evaluation of repository shaft sinking techniques.

This process is inevitably subjective. Value judgments may also change, as technology develops further, as uncertainties in geological or construction conditions are clarified or as shaft performance criteria are modified. Certainly, the ranking of these factors will be different for the first and subsequent shafts at a site. Nevertheless, this ranking gives some indication of the relative importance of each factor to shafts in general and to the relative difference of significance between different shaft sinking methods.

Cost and construction duration are included as factors in this ranking. This introduces a complication in that it is unreasonable to compare costs between methods which may differ widely in technical specifications and performance characteristics, e.g., suitability, performance or reliability of construction. This approach provides a professional evaluation, although considerably subjective, from neither the point of view of the NRC charter for public safety nor the programmatic point of view of the applicant, DOE.

No factors leading to explicit exclusions can be identified provided the following assumptions are held valid.

- Cost and construction duration are not primary exclusionary factors
- Freezing can be carried out to the depths and accuracy required
- Shaft diameter and depth are less than 16 ft and 4000 ft, respectively for which raise drilling and large-diameter drilling are considered feasible.

OVERALL COMPARISON OF SHAFT SINKING METHODS - HARD ROCK

Table 7-5

Factor	Range of Weighted Scale	Blind: Drill-and-Blast	Blind: Rotary (Drilling)	Bottom Access: Ream-and-Slash
1. Sealability/ Damage	0-10	6	8	6
2. Feasibility/ Predictability	0-10	10	8	5
3. Construction Duration	0-8	4	7	3
4. Inspection/ Testing	0-6	6	3	6
5. Safety	0-5	1	5	1
6. Alignment	0-5	5	2	4
7. Cost	0-5	3	2	3
Overall Rating		35	35	28

Note: A higher rating indicates a more favorable condition/outcome

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OVERALL COMPARISON OF SHAFT SINKING METHODS - SALT

Table 7-6

Factor	Range of Weighted Scale	Blind: Drill-and-Blast	Blind: Rotary (Drilling)	Bottom Access: Ream-and Slash	Bottom Access: Back Reaming
1. Sealability/ Damage	0-10	6	8	6	10
2. Feasibility/ Predictability	0-10	10	9	6	4
3. Construction Duration	0-8	3	6	4	4
4. Inspection/ Testing	0-6	6	3	6	3
5. Safety	0-5	2	5	2	5
6. Alignment	0-5	5	2	4	2
7. Cost	0-5	4	1	4	4
Overall Rating		36	34	32	32

Note: A higher rating indicates a more favorable condition/outcome

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The most significant remaining factors, in order of importance are judged to be as follows (factor is deemed significant if the outcome is an important shaft consideration, the outcome is sensitive to the factor per se and if the factor exhibits notable variations between sinking methods):

1) Effect on sealability of the shaft and groundwater environment for long-term repository performance

2) Predictability of construction outcome for short and long term

3) Duration of shaft construction to commissioning

4) Potential for direct inspection, testing and design verification

5) Safety in construction and operation

6) Shaft alignment and associated operational compromises

7) Cost of construction and maintenance.

These factors are weighted as shown in Tables 7-5 and 7-6 and ranked according to a value judgment on the expected outcome of that factor for each method for the composite media of hard rock and salt. A higher number indicates a more favorable outcome.

Comments on the weighting of the scale ranges are:

- Disturbance to the ground and its potential effect on long-term sealability and groundwater is ranked high because it has a major effect on the performance of the shaft. However, the current perspective on this factor is that with controlled blasting and the most sophisticated grouting techniques, effective sealing can be carried out for a drill-and-blast shaft. Thus, this type of rock disturbance is considered to be a major rather than an exclusionary factor.

- The predictability of construction including sinking, lining and sealing to a satisfactory standard and potential time and cost overruns is a major factor. It depends on the technical capabilities of the method, its versatility and the geology encountered. Raise drilling and freezing to great depths are obvious considerations here.

- The duration of construction is ranked reasonably high because it reflects not only on the capitalized cost of the repository but also on the urgent need for an available repository for permanent waste storage.

- The facility of inspecting the shaft walls is not ranked high since it is considered that for a production shaft, geological conditions in the area should be fairly well established by the exploratory shaft. Where mud is used, any testing required to verify the design and construction can be carried out after lining. Freezing and lining limit the scope but do not exclude the facility of geological and geotechnical studies.

From Tables 7-5 and 7-6, it can be seen that for the ranking shown, no one particular method of sinking exhibits a clear overall superiority. Of the blind sinking methods, large-diameter drilling offers better safety and schedule and least damage but at a greater direct cost. With indirect costs factored in, it would appear to be the more favorable method.

The facility of bottom access does not appear to offer any advantage overall in comparison to blind sinking methods. This is, in large part, a result of the depths of the shaft (which affect costs, duration and feasibility directly) and the adverse geological conditions. With better geological conditions, bottom access sinking would result in a relatively more favorable outcome, as has been deomonstrated on many projects to date.

Some understanding of the most suitable areas of application of each method can be deduced from these tables as follows.

The controlling constraints on the use of back-reaming is the availability of bottom access. Other constraints include good ground conditions with minimal inflow, minimal stability problems, geological conditions which allow accurate drill alignment and nonexcessive shaft depths and diameters. If these requirements are fulfilled, back-reaming offers a very rapid and cheap form of excavation, especially in salt and in developments where schedule contraints have a major impact of economic feasibility.

The blind rotary boring method has been eliminated because of the need for further technological development to validate the feasibility. The uncertainty in coping with all but the best groundwater conditions is also likely to moderate near-future development for this type of medium. The high initial capital cost of the machine and machine development is unlikely to be compensated for by rapid excavation rates until much deeper shafts can be sunk.

Blind rotary drilling from the surface with mud is currently suited to shafts up to moderate diameters, (16 ft) and depths (4000 ft) in very poor rock which would otherwise present serious problems with the use of drill-and-blast. However, it is finding increasing application in soft rock formations of mediocre rather than bad quality. Recent technological advances over the last 10 years have extended the shaft diameter range in which the method is cost competitive with drill-and-blast from 10 ft to about 18 ft (Peck and Deere, 1969). The major limitation of excessive cost of lining is likely to be removed in the near future with the adoption of new lining methods.

Compared to drill-and-blast, the ream-and-slash method offers significant cost and schedule advantages if geological conditions are good and the shaft not too deep. Depth affects the alignment accuracy and the feasibility of raise-drilling. The necessity to pregrout or prefreeze to great depth in adverse geological conditions is generally not compensated for by the faster advance rate afforded by raise mucking. Shaft depth and geological conditions which do not favor sinking by the ream-and-slash method are also those not conducive to the use of the back-reaming method.

The main attributes of the drill-and-blast method are the lack of practical limits to diameter and depth and the security and predictability of construction for a wide range of geological conditions. The high unit costs and the relatively long construction durations are less significant disadvantages for sinking deep shafts in poor ground. However, the indications are that the technological advances for mechanical excavation are gradually shrinking the range of conditions for which the drill-and-blast is the preferred method.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

As stated in Chapter 1, the objective of this report is to present a comparative evaluation of the various available shaft sinking techniques within the context of the particular short- and long-term engineering performance requirements of repository access structures. The particular attributes of primary significance in this evaluation are:

Technical

- Influence on geohydrological regime, damage to shaft walls and effect on sealability
- Facility for in situ testing and design verification
- The predictability and feasibility of construction.

Nontechnical

- Safety of construction and operation
- Cost of construction
- Duration of construction.

The main set of control conditions, derived from contemporary repository shaft design considerations, which form the basis of the evaluation are:

- Vertical shaft configuration in the size range 10 to 22 ft in diameter and 3000 to 4000 ft in depth

- Hydrostatic lining to restrict groundwater inflows to 100 gpm or less

- Shaft construction and design to specifically acknowledge the critical schedule constraints and long-term repository sealing (against radionuclide migration) criteria

- Restricted to technology feasible, but not necessarily proven, in 1982

- Detailed evaluation of the respective sinking methods for two "composite" media representative of overburden and host rock conditions for basalt, granite and tuff (designated "hard rock") and for bedded and domal salts (designated "salt").

The primary comparative evaluation has been conducted for 14 ft internal diameter shafts developed in two composite media using four different methods of sinking/lining. The comparisons draw a major distinction between shafts sunk blind and those which utilize bottom access.

Based on the system of ranking introduced to grade the significant attributes of each ranking method and the resulting design, it is concluded that for application to repository access, no one particular method of sinking exhibits a clear overall superiority. It is stressed however, that this is a subjective ranking. Nevertheless, some significant differences in technical and nontechnical characteristics between the various sinking methods have been identified; namely:

- For the stipulated shaft requirements, drill-and-blast, ream-and-slash, and rotary drilling using top drive equipment are considered to be technically feasible. Back-reaming is regarded as marginally feasible while blind boring using in-hole equipment is discounted as currently not viable.

- Of the blind sinking methods, rotary drilling offers distinct advantages in terms of safety, minimum construction duration and least damage to the rock and groundwater regime. It is considerably more expensive than all other methods. However, this disadvantage is often more than compensated for by the considerable savings in capitalized costs for the repository project which accrue from the attendant short construction time. However, these machines are not currently available and considerable lead time for procurement exists. Blind rotary drilling is particularly favorable for the construction of shafts in the size range less than 18 ft diameter and located in unfavorable geological conditions.

- The ream-and-slash and drill-and-blast methods of construction both impart considerable damage to the shaft walls, are unfavorable from the safety aspect and generally involve long construction times. Their main advantage is low direct cost. Because of the torque limitations on rotary sinking methods and the subsequent impact on advance rates, both ream-and-slash and drill-and-blast methods are generally more advantageous for the deeper larger shafts in good geological conditions. In fact, the ream-and-slash method is very competitive in terms of both cost and schedule if good ground conditions exist.

- Back-reaming methods of shaft construction are only marginally feasible at the upper range of shaft geometries being considered here. Although raise-drilling technology from which it derives is well developed, there are definite inherent mechanical limits to the scale of operations. When geological conditions are good and for the smaller range of shaft diameters, back-reaming combines the safety, schedule and minimum disturbance advantages of mechanical construction methods with the cost advantage of drill-and-blast sinking.

The primary evaluation has concluded that the facility of bottom access does not appear to offer any significant overall advantage in comparison with blind sinking methods. This is, in large part, a consequence of the considerable depths of the shafts chosen and what may be considered to be "adverse" geological conditions requiring expensive time consuming freezing preparations. For better geological conditions and smaller, shallower shafts, bottom access sinking methods would appear as much more favorable alternatives.

The primary comparative evaluation of shaft sinking method deals with a single shaft diameter and only two types of geological media. It has been possible however, by careful study of the controlling factors and limitations of each sinking method, to ascertain for different shaft diameters and geological conditions, the practical limitations and preferred areas of application of each method as concluded above. It is also concluded that for the application of the four feasible methods of shaft construction to the repository determined conditions cited earlier, schedule and cost variations within each and every method are a relatively insensitive function of shaft diameter. In short, shaft diameter per se does not affect the choice of sinking method within the range of conditions considered.

Finally, the shaft designs presented in this evaluation illustrate the scope of a shaft sinking operation and the level of design considered appropriate for vertical repository access facilities. These designs represent the practical interpretations of the currently envisaged design conditions and criteria for the construction of shafts for high level nuclear waste deep geologic repositories.

Recent developments point the way for a continued trend towards the greater use of mechanized shaft sinking. This will be particularly evident in the construction of deeper shafts where water problems are minimal. The economic feasibility of proposed ventures depends more than ever on the time to complete shaft access. Thus, the technology of shaft construction can be expected to play an even greater role than at the present in the planning of underground structures.

Because of the limited scope for reducing manpower and the use of increasingly more stringent safety laws for shaft construction, it appears inevitable that cost trends for drill-and-blast sinking will increase disproportionately in comparison to those for mechanized sinking methods. Furthermore, in comparison to mechanized sinking methods, there is limited scope for technological advances. Most probably, the major near-term technological advances will be in the development of top-drive blind rotary drilling. As diameters became larger, torque transmission (and hence sinking rate) may become a limiting factor. The main scope for improvement of shaft drilling technology lies with:

- Development of improved cheaper lining systems
- Improvements in bit cleaning technology
- Improved penetration rates through more efficient circulation systems.
- Development of part-face shaft sinking machines.

In the longer term, it is expected that in-the-hole shaft boring machines, which do not have overriding torque and thrust limitations, will develop to the necessary practical standards for routine application.

Part-face machines also warrant serious consideration as major future contributors to rapid mechanized shaft sinking. Such a machine has been designed by the Harrison-Western Corporation (1982). This machine, similar in concept to the part-face tunneler or road header type machine, allows access to the working face at any time to cope with groundwater inflow, grouting, support or lining problems as required. The bottom of the shaft is excavated by a rotating cutterwheel which is half the diameter of the excavated shaft.

8.2 RECOMMENDATIONS

The following recommendations are directed towards resolving some outstanding questions in the evaluation of a preferred sinking method for a specific application and in improving the application of shaft designs for repositories.

It is cautioned at the outset, that the evaluations and conclusions presented herein should be restricted to the particular set of assumptions and conditions noted. In the selection of the preferred shaft sinking method for a specific site, it is recommended that the shaft designs be reviewed and the comparisons re-evaluated. This re-evaluation should address the actual geological conditions, the then-current technology, and the updated repository design constraints. Conclusions different from those noted here are distinctly possible and probable.

Some of the most promising methods of shaft sinking are on the forefront of technology, i.e., rotary sinking. Thus, because of the associated uncertainty in construction and performance outcome, it is suggested that the first shaft (the exploratory shaft) in a proposed site be a smaller shaft sunk using such a method. This will allow a more rational choice as to the best method of excavation, a demonstration of the sinking requirements and provide in the planning of a future shaft to improve the reliability of construction. In this way, the choice of the sinking method for the production shafts will not be compromised by the uncertainty of construction outcome or the ability to verify the shaft design.

Consideration should be given to the possibility of using more than one method for the sinking of the several shafts at one site. This has a number of advantages:

- It improves the possibility of earlier initial access to the repository level
- It reduces the overall construction uncertainty
- Simultaneous sinking of a number of shafts and organization of equipment needs are much easier
- Total construction time may be reduced

- Adaptation of each method to the respective shaft diameter is possible.

It is suggested that preventative/conservative investigation, design and construction approaches be adopted. It is important to conduct presinking site evaluations such that potentially troublesome areas can be recognized in advance. After-the-fact remedies to stability problems and disturbance have an irreversible impact on the "quality" of the final facility.

Recommended areas for further study include:

- Assess the immediate future potential for using lower cost liners in blind drilling applications

- A detailed evaluation of how shaft diameter affects the feasibility of drilling to great depths

- The effect of long-term regional thermal deformations around the repository on shaft lining integrity and sealing

- The effect of drilling mud of sealability on the shaft

- The effect of freezing on long term disturbance to the shaft walls and on the reliability of geological/geotechnical investigations.

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GLOSSARY OF MINING AND GEOTECHNICAL TERMS

Some confusion and inconsistency has arisen in the construction industry regarding the use of terms for describing mechanized shaft sinking techniques. This is primarily a consequence of a newly and rapidly developing technology. The following definitions and nomenclature have been adopted in this report.

Rotary sinking refers to all full face mechanized methods of shaft excavation. In one sense, it may be drilling or boring and in another, it may be blind or with bottom access.

Boring refers to the method of excavating using a completely self-contained machine at the excavation heading in the sense of a mole.

Drilling refers to the configuration where the drive unit is remote from the cutter at the face and torque and thrust are transmitted through a drill stem.

Reaming is the process of stepwise enlarging the opening to successively larger diameters. It may be a boring or drilling process. Thus, the widely used technique of raise boring is really raise drilling or raise reaming.

Blind drilling or boring refers to the excavation process where excavation proceeds full face into virgin rock. It includes the conventional drill and blast method. All other methods where a pilot hole is used for mucking, ventilation or alignment are not considered blind. With very few exceptions, nonblind methods rely on bottom access for some part of the sinking operation.

Other definitions and terms follow in alphabetical order.

Aquifer Seal. Seal created in the rock by grouting injection to isolate aquifers and prevent groundwater flow longitudinally along the shaft outside the lining in the fractured or destressed rock.

Backsheeting. Corrugated or plastic sheeting placed against the shaft wall to prevent water leakage from running into the mass concrete during pouring. Also used to facilitate backwall injection.

Backwall Injection. Cement injection of the thin annular space between the poured concrete lining and the excavated rock.

Bulkhead. A tight partition of wood, steel or concrete used as a barrier against fire, gas, or water or wall or partition erected to resist ground or water pressures.

Bunton. A steel or timber element extending across the shaft at intervals of several feet. They serve to carry cage guides and to compartmentalize the shaft for hoisting and ladderways.

Crosshead. A runner or framework that runs on guides, placed a few feet above the sinking bucket in order to prevent it from swinging too violently.

Draw Works. In rotary drilling, that part of the equipment functioning as a hoist to raise or lower drill pipe and in some cases to transmit torque to the rotary table.

Dump Scrolls. Device, usually located as part of headframe, for maneuvering or emptying skips, often automatically.

French Drains. A covered hydraulic conduit containing a layer of loose or fitted stone or other pervious material.

Galloway Stage. Multidecked platform suspended near bottom of shaft during sinking. It can be raised or lowered as required during drilling, blasting, mucking and concreting.

Hydrostatic Lining. Concrete steel or cast-iron lining capable of resisting external water pressures corresponding to the theoretical head of water at the particular section in the shaft.

Jumbo. A drill carriage on which several drifter type drills are mounted.

Liner Plate. Bars or plate placed between other supports to reinforce sets against collapse from heavy strata pressure.

Pregrouting. Grouting of ground using either cement or chemical injection ahead of the excavation face and prior to disturbance of stress or groundwater. It may be performed from the surface or successively from the shaft bottom.

Ring Beam. Beam of concrete cast at the bottom of a pour to act as the support for a lift of concrete lining to be cast above the beam and below the previous lift.

Rock Bursts. The explosive release of accumulated strain energy in the rock in the bottom of a shaft, generally occurring in brittle rock below about 2000 feet depth.

Rock Disturbance. The disturbance of the original stress state and the creation of artificial fractures in the rock immediately surrounding the excavation.

Rock Support. The placement of reinforcement (bolts) shotcrete, steel or wood sets, concrete lining or other materials offering resistance to or improving the strength of the rock.

Safety Dog. Mechanical locking device usually at the crosshead which engages the guide in the event of hoisting rope failure.

Shaft Collar. The oversize concrete structure at the junction of the shaft and the surface constructed prior to sinking.

Shaft Seal. Seal created in the disturbed rock zone by grouting to pressures greater than the anticipated external groundwater pressures so as to ensure an impermeable lining/rock seal to axial flow.

Sheave Deck. The upper deck of a shaft sinking stage containing the pulleys connecting the stage and hoist rope.

Shotcrete. Shotcrete is concrete or mortar conveyed through a hose and pneumatically projected at high velocity onto a surface, the force of the impact compacting the materials.

Skip. A guided steel hopper usually rectangular used in vertical and inclined shafts for hoisting men, materials or ore.

Slickline. Pipe used for conveying concrete from the surface to the stage.

Slipforming. Construction of a prismatically shaped concrete structure using a slowly moving form to contain the wet concrete.

Spiling. Forpoling over timber or steel supports to support in advance of excavation, weak loose rock.

Spears. Tapered rigid plates used with rope guide hoists for guiding the cage into position at the top and bottom of the shaft.

Squeezing Ground. Ground experiencing excessive deformation and yielding as a result of in situ stresses greater than the resistance strength of the fractured ground.

Stoper. A light percussive drill incorporating a pneumatic cylinder to provide support and thrust.

Swelling Ground. Ground experiencing excessive deformation and dilation into the opening as a result of swelling of the clay fraction associated with the absorption of water.

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16. ABSTRACT *(200 words or less)* The purpose of the complete project is to provide the NRC with technical assistance to enable the focused, adequate review by NRC of specific aspects related to design and construction of an in situ test facility and final geologic repository, as presented by the DOE in Site Characterization Reports and License Applications (LA).

This report provides a comparative evaluation of various shaft sinking techniques for production shafts for a repository. The primary comparative evaluation has been conducted for 14 ft internal diameter shafts developed in two composite media using five different methods of sinking/lining. The technical, cost and schedule comparisons distinguish between shafts sunk blind and those which utilize bottom access. Based on the system ranking, it is concluded that no one particular method of sinking exhibits a clear overall superiority.

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