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A mining research contract report
MAY 1986

GROUTING EFFICIENCY IN VERTICAL SHAFTS

FINAL REPORT

Contract J0156004
Engineers International, Inc.

BUREAU OF MINES

UNITED STATES DEPARTMENT OF THE INTERIOR

U.S. DEPARTMENT OF THE INTERIOR ★ BUREAU OF MINES ★
SAFETY EFFICIENCY
MINERAL INDUSTRIES

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This report presents a review of the state-of-the-art in shaft grouting technology, and an assessment of how current knowledge and practices are supportive of the performance requirements for a high-level nuclear waste repository shaft. Case histories of various shafts recently sunk by conventional drill-and-blast techniques as well as blind-drilling have been studied and the results discussed. The existence of voids, their formation, detection, monitoring, and control during grouting are devoted special attention. The long-term performance of grouts was examined. The need for further research and investigations is documented.

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FOREWARD

This report was prepared by Engineers International, Inc., under U. S. Bureau of Mines Contract Number J0156004. The contract was administered under the technical direction of the Pittsburgh Research Center, with Mr. Charles D. Taylor acting as Technical Project Officer. Ms. Janice D. Johnson, Washington, DC, was the Contracting Officer and Mr. Earle Amey, Washington, DC, was the Program Manager.

Engineers International would like to express its appreciation to Mr. Charles Taylor, Technical Project Officer, for his cooperation and assistance, and for fostering an open, productive atmosphere for pursuing the work. EI would also like to thank the many individuals and organizations within the drilling, oil, and shaft sinking communities who offered particular and extraordinary assistance: to Mr. Wallace Hammer and Mr. P. Ken Ortego, Fenix and Scisson, Nevada Test Site, for their time and cooperation; to Mr. Karl Hahn and Mr. Pat Rowe, Reynolds Electric and Engineering Company, Nevada Test Site, for arranging the NTS site visit for the Project Engineer and for continuing to be of assistance afterwards; to Ms. Lillian Wakeley, Waterways Experiment Station, Vicksburg, Mississippi, for her ongoing updates on grout research; to Mr. Dwight Smith, Halliburton Company, Duncan, Oklahoma, for his extraordinary effort in providing information on casing cementing technology, and particularly to Messrs. John Gallerani and Steve Balone of Bechtel Engineering, Waste Isolation Pilot Plant site, Carlsbad, New Mexico, for arranging the WIPP site visit by the Project Engineer and for making available a considerable quantity of information, both before, during, and after the site visit. In addition, EI would like to express its appreciation of Mr. Fred Edwards, Dynatec Mining, Ltd., Ontario; Mr. Jim Friant, The Robbins Company, Seattle, Washington; Mr. W. Earl Cunningham, Las Vegas, Nevada; Mr. Hassell E. Hunter, Houston, Texas; Mr. Harold Hendrick, Drilco Industrial, Las Vegas, Nevada; Mr. Paul Richardson, Orange, California; Messrs. David Whitley and Robert Pliska of Santa Fe Shaft Drilling Company, Santa Ana, California; Mr. William Hawes of Sohio Oil Shale, Salt Lake City, Utah; Messrs. Warren Harrison, Dan McFadden, and Don Morrison of Frontier-Kemper Constructors, Evansville, Indiana; Mr. Billy McLaurin, Dresser Industries (Magcobar), Denver, Colorado; Mr. William Nash, McCarthy Brothers Construction, St. Louis, Missouri; Mr. Jim Cobbs of Cobbs Engineering, Tulsa, Oklahoma; Mr. Jake Webster, Basalt Waste Isolation Project, Hanford, Washington; Messrs. John Stormont and Chuck Gulick, Sandia Laboratories, Albuquerque, New Mexico; Mr. Eric Nelson, Dowell-Schlumberger, Tulsa, Oklahoma; "Dutchy" Boettcher of Grouting Advisors, Inc., Grand Junction, Colorado, and Mr. Wayne S. Adaska, Portland Cement Association, Skokie, Illinois, all of whom gave freely of their considerable expertise in grouting, cementing, and shaft sinking.

For EI, the project was managed by Dr. Madan M. Singh. As Project Engineer, Mr. Robert A. Cummings provided the technical direction and is the major author of the report. Mr. S. K. Mukherjee assisted in the literature identification. Mr. Gregory Zeihen was in charge of collecting and maintaining the more than 6,000 pages of literature references and drawings necessary to support the project. Mr. Peter J. Huck contributed most of the discussions on void detection, grout placement monitoring, and seepage detection technology, and Mr. Grant Bumâ contributed to the discussion of water movement mechanics in voids.

The efforts of EI's Subcontractor, The Cementation Company of America, Inc., were coordinated through Mr. Stephen H. E. Phillips. Cementation provided detailed case history data on shaft sinking projects in the U.S., Canada, and Great Britain, and also contributed significantly to the discussions of shaft sinking and lining technology.

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1.0 INTRODUCTION

The U. S. Department of Energy (DOE) is responsible for the planning, design, and construction of a geologic nuclear waste disposal facility, such that the facility could begin to accept waste by the mid-1990s. A major milestone was recently reached when the DOE issued the draft Environmental Assessments (EAs) for public comment. The EAs, among other things, nominate 5 specific sites and recommend 3 of these, for site characterization. As required by 10CFR60, the law governing the siting and design of a repository, site characterization activities will include subsurface testing in the anticipated horizon or host rock to confirm its suitability for emplacement of waste and to provide data for performing assessments. Proposed host rocks include tuff, basalt, and bedded salt. Since subsurface testing will be required, an exploratory shaft (ES) will have to be sunk, to gain access. The ES will be incorporated into the repository geological setting, once a site is selected, as one of the repository shafts. Not only is the disposition of the ES important in overall site suitability, but the ES construction will be a source of information for the constructability of the repository shafts.

1.1 REPOSITORY SHAFT PERFORMANCE REQUIREMENTS

The following discussion provides the background necessary to understand the reason for the study of void occurrence and control in the shaft grouting that is reported upon herein. Although this discussion focuses on the ES, chiefly because this is the imminent concern, the principles will apply to repository access shafts as well.

The site characterization activities, as stipulated in 10CFR60.10(D)(1) shall be conducted so as to "limit adverse effects on the long-term performance of the geologic repository to the extent practical." This has several implications. Any penetration of the host rock can constitute a preferential pathway for radionuclide release, and hence all pentration designs should be conceived in such a way as to minimize the hazard. This applies to deep boreholes, whose numbers are to be limited, and to the ES and other openings created during the site characterization phase. A similar degree of confidence will be required for the repository access openings (to be constructed later). Three sites will be characterized, but only one will become the first repository; chances are that the other two will never be developed as repositories. Thus, only one ES of the three built will have to function in such a way as to not provide a preferential pathway, but since it will not be known which of the three this will be at the time the ES is constructed, each ES will have to be designed and built in accordance with the intent of 10CFR60. This implies that the design and construction will need to be executed with great care and attention to the potential isolation requirement of the facility which is also set forth in 10CFR60. A deliberate, careful procedure is therefore mandated by this requirement.

In apparent conflict with this requirement is the Nuclear Waste Policy Act (NWPA), which, among other things, sets a schedule for repository development seeking to alleviate, at the earliest practicable date, the problem of growing inventories of high-level-waste (HLW) in surface temporary storage. Added to this is a requirement to control costs, stemming in large part from the assertion that many of the funds to be spent on site characterization are at considerable risk, since the President will only select one site to be the eventual repository.

The DOE has indicated that it strongly favors the blind boring method to construct the ES at some sites. (Present concepts for construction of the later repository shafts are for these to be constructed by drill-and-blast methods.) The DOE has supported its position by pointing out its expectation of substantial cost and schedule savings with blind boring, and has indicated its belief that the blind boring method is suitable for preserving the isolation capability of the host rock.

The U. S. Nuclear Regulatory Commission (NRC), the agency with responsibility for licensing the repository facility, must have reasonable assurance that the isolation requirement of the host rock, and eventual repository, will be met. Accordingly, the NRC has reviewed the shaft construction concepts presented to it by the DOE in some detail, and has expressed concern that the isolation capability of the site not be compromised by site characterization activities, including shaft construction.

With respect to the ES construction and particularly to blind boring, the NRC has observed that the blind boring method necessarily does not permit direct observation and control of construction, lining, and sealing operations to the extent that the drill-and-blast method does. Even shafts constructed by drill-and-blast methods and concurrently lined, pose sealing concerns in both the pre-closure and post-closure periods.

The NRC has asked DOE how the shaft liner-to-rock grouting operation will be conducted, controlled, verified, and what remedial actions will be taken, to ensure that the end product will meet the statutory requirements for waste isolation. The question is most critical in reference to the ES, but is germane to construction of the later repository shafts as well. The DOE has responded, in essence, that it believes state-of-the-art procedures will meet the regulatory requirements, and has, in correspondence with the NRC, indicated what measures are planned towards this end.

1.2 PURPOSE AND SCOPE OF THIS REPORT

In order to aid an enlightened and objective review of the waste repository shaft construction plans, the NRC, through the U. S. Bureau of Mines, requested a review of state-of-the-art shaft grouting technology, and an assessment of how the state-of-the-art is supportive of the performance requirements for shaft lining grout quality. The objectives were threefold:

1) Review case histories of projects in the mining, defense, and oil industries where grouting in vertical shafts was done between the shaft casing and the rock face, with special emphasis on the size and frequency of voids that occurred in the grout, on the manner of water transport through void systems, and on how the grout placement was monitored and controlled.

2) Report on the capability of state-of-the-art grout monitoring systems to detect the size and frequency of voids occurring in the grout.

3) Summarize the state-of-the-art of grouting in vertical shafts, presenting a scenario of the lowest level of voids that can be expected, the ability to monitor the occurrence of these voids, and the effect of these voids on the movement of water from one waterbearing zone to another.

In considering the information needed, Engineers International, Inc. (EI), realized that there was potential for voids occurring in the grout placed behind the linings of conventionally-sunk (drill-and-blast) shaft linings as well, and that these should be included, to provide a proper perspective. It was also realized that the concept of "void" that is relevant to the problem at hand surpasses that of only open, mud-filled cavities and extends to any influence that could result in the shaft having an unanticipated deleterious effect on the isolation capability of the site. In essence, this meant that the voids considered should include the natural porosity of the grout, the presence of cracking (secondary permeability), and effect of a micro-annulus, as well as the occurrence of larger cavities. The voids of concern, although owing their occurrence to the method of construction, might not be physically manifested until long after the shaft was completed, for example, a gradual detachment of the grout from the shaft casing due to long-term deformation and temperature changes. Thus it is also relevant to consider actions of the lining and rock mass that could contribute to void formation though time.

So, the philosophy adopted in the study was somewhat larger than strictly defined by the stated objectives. It regards, the shaft lining, grout, and affected rock as a system, and emphasizes the mechanism that could operate within each component of this system, either to compromise the grout behind the shaft casing in blind drilled shafts, or, in the case of directly placed shafts, to compromise the analogous backwall grout behind a concrete or composite lining.

The objectives as stated in the contract are clear in that penetration grouting and pre-grouting were not be the focus of the research, and EI has generally held to that restriction. However, these types of grouting are salient remedial or preventive actions that could mitigate the types of voids that are being considered in detail, and are contributors to the integrity of the overall shaft/lining system, the optimization of which would tend to reduce the effects of all voids. Thus, all forms of grouting are considered relevant to some extent.

It was decided to limit the types of shaft sinking methods for which void occurrence was to be considered in detail, to those methods either contemplated by the DOE or appearing to hold the greatest potential for application in a repository.

1.3 APPROACH

The information considered was entirely from the published literature or from personal communications. It was found that the types of information available on shaft sinking are varied and that the amount information on methods is voluminous; however, there is little devoted to the watertightness of the finished product in most cases, and usually nothing was found on the potential occurrence of voids or how these affected leakage. There were some notable exceptions, but for void occurrence information, EI was forced to rely in large part on the observations and recollections of involved personnel and experts in the field.

In order to arrive at a deduction as to the occurrence of voids in grout and the contribution those may have made to water movement in any particular case, it was necessary to delve into the construction records in considerable detail -- well beyond the level of detail provided in most readily-available publications. Because of the intensive effort involved in performing these kinds of in-depth assessments, EI chose to focus on several case histories for which the level of detailed information needed was available, and which held particular relevance to the requirements of repository shafts.

1.4 PARTICIPATION

The work reported upon herein was performed by Engineers International, Inc. under U. S. Bureau of Mines (USBM) Contract No. J0156004. Mr. Charles D. Taylor was the Technical Project Officer representing the USBM, Ms. Janice D. Johnson was the Contracting Officer, and Mr. Earl Amey was the Program Manager.

Engineers International's effort was managed by Dr. Madan M. Singh. The technical direction was provided by Mr. Robert A. Cummings, Project Engineer, who was assisted by Messrs. S. K. Mukherjee, G. Buma, P. J. Huck, and G. Zeihen. The Cementation Company of America, Inc. as a subcontractor, through Mr. Stephen H. E. Phillips, provided several of the case histories and contributed to the discussions of backwall grouting and shaft lining technology.

The U. S. Nuclear Regulatory Commission (NRC) contact for most of the project was Mr. David H. Tiktinsky but towards the end was Dr. Banad Jagannath, both of the Division of Waste Management, Engineering Branch, in Silver Spring, Maryland.

1.5 REPORT CONTENT

The report is organized into 6 sections, including the introduction. A list of the many references used for the preparation of this report is presented at the end.

Section 2 addresses grouting technology. It is intended to develop concepts of how grout mixes and placement could affect the likelihood of voids and how the existing grouting technology might support the elimination of voids. It is not an exhaustive treatment of this very broad subject, but focuses on those methods that are of the most significance for annular grouting of shaft linings and casings.

Section 3 gives background on shaft sinking and lining technology. There are separate discussions on excavation methods and on lining installation, since the selection of an excavation method no longer necessarily preselects a lining type for modern shafts. Again, only those methods are considered that have the most utility for repository shafts, although the principles involved could be extended to other types of linings and excavation methods as well.

Section 4 describes documented or presumed occurrences of voids in lined, vertical shafts. There is no statistical analysis presented, since it would be of little or no value to do so given the extent and reliability of the available information on void occurrences. The greatest emphasis is on shafts with watertightness requirements approaching those of repository shafts, and the section deals exclusively with shafts where leakage has occurred. Some experience from metal mine and other types of shafts is presented for completeness, although the relevance to repositories, in terms of void likelihood, may be weak. Section 4 concludes with a subsection (4.5) that synthesizes shaft lining experience with respect to modes of void occurrence in primary grouting materials, in primary lining materials, and in the rock mass.

Section 5 describes the steps that could be taken to minimize void influence and occurrence likelihood. The content of Section 5 was developed from the case histories, particularly those whose performances indicated some success at achieving a major reduction in annular flow or those that exhibited some particularly noteworthy technique. Most of the suggested measures were developed from general grouting and shaft sinking experience. Other subsections describe the state-of-the-art techniques for detecting voids, monitoring grout placement, and detecting seepage. The discussion also summarizes what limited knowledge is presently available about the mechanics of water movement through voids systems, and concludes with the benefit to the grout system that could be derived from sealing.

Section 6 provides an evaluation of the best case of void occurrence that would be expected of a realistically-constructed repository shaft. The evaluation is subjective, because there are no existing shafts that are exactly like repository shafts to compare against. Therefore the evaluation was made by assuming that the construction and design of the repository shafts would consider and incorporate all reasonable presently-available precautions against the occurrence of voids, and that the remaining probability of void occurrence is due to inherent limitations of grouting of shaft sinking technology, or to uncertainties in rock mass response to shaft construction.

Throughout the report, examples form actual grouting experience and shaft construction are used to illustrate the discussion.

2.0 REVIEW OF GROUT AND CONCRETE TECHNOLOGY

Numerous concepts of grouting and shaft liner cementing are evaluated in this report that require an understanding of grout formulation and chemistry, and the behavior of grouts and concretes in natural environments. This section is included as an aid to the reader in interpreting the conclusions that follow in later sections. It is not intended to be an exhaustive treatment of the chemistry and formulation of grouts.

In as much as the subject is the formulation, chemistry, and application of grouts, it is appropriate to make a few, somewhat philosophical comments at the outset.

The context of this report is the efficiency of state-of-the-art grouting, in vertical shafts that might be used in a repository. The repository application makes requirements of the grouts and lining materials, in terms of longevity, stability under unfavorable temperature and hydrochemical conditions, and required performance level, that are not reflected in the historical development grouts. Grouts evolved, generally, in response to constructibility and maintenance needs. In a sense, grouting originated as a kind of contingency item that came into play when a clear need existed to repair some problem that threatened the integrity of a structure. The first record of shaft grouting was to repair brickwork in a shaft in 1864 in Germany. Thus, the performance requirements were such that any improvement at all was at least some improvement, and would tend to forestall the threat of failure. The degree of improvement accepted was a function of available technology and, to a major extent, cost. The prevention and ground improvement role of grouting was a relatively recent development that occurred in response to more ambitious projects, less ideal sites, and enhanced analytical abilities and methods that were better able to quantify the effect of site conditions and to predict with some reliability the benefit of grouting.

For effectiveness in planning for a repository, grouts should be reviewed secondarily in the repair/contingency mode and primarily in the prevention/ground optimization mode. Repositories will require performance of the grouts that come as close as possible to those of the "ideal" grout--high penetrability, strength, chemical stability, pumpability, and reliability, and low permeability, shrinkage, thermal response, and toxicity (see Section 2.1.7). In order to bestow sufficient confidence in the prevention/optimization mode, the grout will have to perform satisfactorily in modes and bench-scale testing, followed by in-situ testing and overtests.

The following discussion identified two groups of grouts, cementitious and chemical. The former, being multi-phase, are suspensions; the latter, being single-phase until a reaction occurs, are solutions. A third category, emulsions, are two-phase dispersed materials that rely on expulsion of water for consolidation.

Because of this, emulsions, such as bitumen/water, are not normally used for sealing, and are not discussed further. With the advent of additives and particulates, the distinction between the categories may become blurred.

The Office of Nuclear Waste Isolation (1981; ONWI255) (p. 121) identifies 5 basic functions of grout:

- increase of strength
- transfer, reduction, or addition of stress
- reduction of deformability
- reduction in permeability
- protection against chemical or physical attack.

The following discussion identifies how the various grouts can fulfill one or more of these requirements.

2.1 CEMENTITIOUS MATERIALS

Cementitious grouts are generally thought of as Portland cement of one or more types, although this conviction is somewhat restrictive. There are other cements, including slag cement, gypsum cement, and resin gypsum cement. Blast-furnace slag can make a cement when reacted with sodium hydroxide either by itself or in combination with Portland cement. Resin gypsum cements set up quickly and are slightly expansive, but are also expensive. The great majority of cementitious materials used in shaft grouting are Portland cements, and these are the subject of this section. Smith (1976) provides an excellent discussion on cementing, with reference to oil well cements.

2.1.1 Types of Basic Cements

Cement grouts are formed basically from ordinary Portland cement and water, and as such, are considered suspensions. Other cements or additives may be used to impart certain specialty properties, but the most commonly-desired ones, such as early setting times, sulfate resistance, and low heat of hydration for temperature control, are available pre-blended, and are identified by Type or Class.

The American Society for Testing and Materials (ASTM), the American Concrete Institute (ACI), the American Petroleum Institute (API), and the American Association of State Highway Officials (AASHO), all have specifications for Portland cements.

Portland cement is formed from limestone, silica, alumina, and iron, along with small quantities of gypsum and magnesium. Through the calcining process, four basic components are formed: tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tricalcium aluminoferrite (C_4AF). (The acronyms in parentheses are not chemical formulae, but are commonly used shorthand designations for the components). Each has special properties that it imparts to the cement mix. Upon contact with water, these components hydrate to form dense calcium aluminosilicate crystals, along with the liberation of heat.

Variation in the proportions of these constituents and the addition of additives, pozzolans, and the like can alter the properties of the mix and the hardened compound. The American Society for Testing and Materials recognizes 5 combinations or types of cement. ASTM Types I and II cements are essentially ordinary Portland cements, with Type II having slightly less C_3S and C_3A and C_4AF, than Type I. It has somewhat higher sulfate resistance that Type I. Type III cement emphasizes the C_3S component and is a high-early-strength mix. Type IV includes a greater proportion of C_2S and C_4AF and is a low heat cement. Type V cement has a reduced proportion of C_3A, which is susceptible to chemical attack by sulfate. Sulfate reacts with lime and C_3A to form calcium sulfoaluminate, which is expansive. Type V cement should not be used with calcium chloride as an accelerator. It contains slightly more calcium sulfate, to increase sulfate resistance, and is commonly used in salt mine shaft concrete.

Portland cements can be mixed with ground blast furnace slag, for better chemical resistance. Other important cements are higher alumina (bauxite and limestone) which set slowly but harden quickly with great heat liberation, and supersulfate cement (ground blast furnace slag, calcium sulfate, and ordinary portland cement), which has a very high resistance to seawater and sulfate attack.

This report also mentions the API Classes of cements (Table 1). The API cements are extensively used in oil-well cementing work and are extensively referenced in blind hole drilling. The API recognizes 8 basic mixes of cements, in recognition of the special and more variable setting environments involved in oil-well cementing. The standards include moisture content, soundness, fineness of grind, thickening time, compressive strength, and free water content (Suman and Ellis, 1977). API Class A is roughly the equivalent of Type I and is rated to 6,000 ft, where special properties are not required. Depth ratings are functions of thickening time and strength/pressure relationships). Class B is equivalent to Type II and is a moderately sulfate-resistant cement also rated to 6,000 ft; a special high-sulfate-resistant Class B is also available. API Class C is roughly equivalent to Type III, and is rated to 6,000 ft for conditions where high early strength is required. It is available in three classes of sulfate resistance. Types IV and V do not have any direct API analogs. API Class D is rated from 6,000 to 10,000 ft for moderately high temperatures and pressures, in two classes of sulfate resistance. Class E is a high-temperature, high-pressure blend rated from 10,000 to 14,000 ft and is also available in two classes of sulfate resistance. Class F is an extremely high-temperature, high-pressure class rated at from 10,000 to 16,000 ft and is only available in the high sulfate resistant type. Classes G and H are basic cements

Table 1: API cement classes

Class	Depth range,* ft.	Available sulfate resistance	Characteristics, availability
A	0 - 6,000	Ordinary	Common (construction), widely avail.
B	0 - 6,000	Moderate	Special (construction), avail. California, Canada
C	0 - 6,000	Ord., mod., high	High early strength, fine grind, widely avail.
D	6,000 - 10,000	Mod., high	Coarse grind, retarded, not avail. North America
E	10,000 - 14,000	Mod., high	Same as D
F	10,000 - 16,000	Mod., high	Same as D
G	0 - 8,000	Mod., high	Basic cement, no chemical retarder, avail. West. U. S.
H	0 - 8,000	Moderate	Basic cement, coarse grind no chemical retarder, Gulf Coast & Mid-Continent
J	12,000 - 16,000	High	Resists strength retrogression, min. temp. 230°F

* As manufactured. Based on normal size cement job in well with geothermal gradient of 1.5°F per 100 feet.

designed to be custom-blended with additives for particular conditions. They are rated for 0 to 8,000 ft, and can be blended with accelerators and retarders for a wide range of temperature and pressure conditions. Class G cannot be manufactured with any additives except calcium sulfate or water, or both, and comes in two classes of sulfate resistance (moderate and high). Class H cannot also contain any additives except calcium sulfate or water, or both, and comes only in high sulfate resistance. In the moderate sulfate resistance classes, Classes G and H have the same chemical formulations; Class G slurries are normally a little lighter.

Other variations in properties of cements can be produced by varying the grind. Microfine cement (Shimoda and Ohmoni, 1982) has a significantly smaller particle size, greatly improving its penetration capability (Section 2.1.7).

Further information on Portland cement formulations can be found in the extensive literature on the subject, for example, Littlejohn (1982).

2.1.2 Specialty Cements

Smith (1976) describes several different types of specialty cements: pozzolanic, resin or plastic cements, diesel oil cements, expanding cements, latex cements, and permafrost cements.

Pozzolans are siliceous materials that by themselves are not cementitious, but that develop cementitious qualities in the presence of lime and water. Examples are diatomite, crushed pumice, heat-treated clays, and fly ash. Fly ash works by combining with the calcium hydroxide that is liberated during the hydration of Portland cement. This can result in a denser, hence stronger, cement. Pozzolan-lime cements (Smith, 1976) are combinations of pozzolans, fly ash or silica, lime, and a little calcium chloride, and hydrate to form calcium silicate. They are high-temperature compatible, lightweight, and easily retarded.

Resin or plastic cements are specialty materials used for positive plugging or squeezing (remedial grouting of an oil well). Under squeezing pressures, the resin component separates to enhance the seal.

Gypsum cements are based on calcium sulfate. "Gyp cements" set rapidly with a high early strength, and can expand to about 0.3 percent by volume. Gypsum is soluble and not particularly strong; cements using gypsum usually are based on a Portland-type cement, such as Class A, G, or H. Gyp cements are occasionally specified at the Nevada Test Site as shaft casing cements (Ortego, 1985).

Diesel oil cements are normally Class A, B, G, or H mixed with diesel oil, kerosene, and a surface active agent. Diesel oil cements will not set unless in communication with water, and so are used mostly for remedial work, for example on fluid loss zones.

Expansive cements are of great interest for casing cementing, prevention of a microannulus, crack filling, and positive sealing. Portland cements expand through the formation of ettringite from C_3A and sulfates. Smith (1976) describes 3 types of commercial expanding cements: Type K, which expands from 0.05 - 0.2 percent expansive calcium sulfoaluminate; Type S, which has a high C_3A content and 10 - 15 percent gypsum, capable of expanding .05 - 0.2 percent; and Type M, which contains small amounts of refractory cement to lend expansive capability. There are many other ways to achieve expandability, such as adding gypsum to Class A, using pozzolans, and adding salt at more than 5 percent to Class A, G, or H. Salt cements can have side effects, such as fluid loss. Rowe (1985) describes a specialty mix that expanded more than 4 percent. Often, though, the expansion takes place at the expense of density and strength. Because of the timing of the expansion with respect to set time and hydration, and due to thixotropy, the expandability is not well-suited to filling highly irregular voids.

One commercial expansive cement, Chem Comp, has been widely used as the standard breakout zone cement for large-diameter shaft casings, for example, at Amchitka and Crownpoint. ESC (expandable salt concrete) tested by Sandia National Laboratory (SNL) for use at the Waste Isolation Pilot Plant (WIPP) site, is based on Chem Comp (Gulick, 1986). In general, expanding cements do not respond well to retarders, and many require special handling and storage procedures.

Griffin et al. (1979) describe a commercial product known as Self-Stress that is claimed to expand before it attains its full strength of from 2,500 to 5,000 psi. Self-Stress is claimed to penetrate mud films by crystal growth and to be sulfate resistant.

Dowell Schlumberger advertises expansive cements that can attain and maintain 0.35 percent linear expansion after 25 days and continue to expand at a decreasing rate thereafter. Data provided on this particular product were only for surface ambient conditions. Also, the permeabilities of this product were reported to be 0.004 md as opposed to 0.0001 md for neat Class A. The product is still less permeable than prehydrated gel cements, however. Twenty-four-hour strength is claimed to be 2,000 psi.

A special expansive cement mix was specified for the first cementing stage near the shaft key area at the WIPP exploratory shaft, which was blind-drilled (see Section 4.4). There have been many other expansive cements developed and used.

Calcium aluminate cements were described earlier. They are bauxite and lime blends that provide high early strength along with resistance to chemical attack.

Latex cements actually are blends of powdered latex and Class A, G, or H, and are used to improve bonding strength and filtration control in wells.

Permafrost cement is designed for use in oil-well applications in permafrost regions. It is typically a gypsum or refractory cement blend that has high early strength and low heat of hydration characteristics, so that it will achieve the necessary strength before it freezes, without doing undue damage to the permafrost.

2.1.3 Cement Additives

There is a wide assortment of additives available to obtain the desired properties for any given cementing application. When the polymers and proprietary additives are considered, the assortment can be bewildering. For any practical application, it is best to thoroughly examine the situation and to investigate all the potential interactions of the additives before deciding on any particular mix.

Smith (1976) provides a useful grouping of additives, as follows:

- acceleratoꝛs
- lightweight additives
- heavyweight additives
- retarders
- lost circulation control agents
- filtration control agents
- friction reducers
- specialty materials.

Through the use of additives, it is possible to: vary the slurry density between 10 and 25 lb/gal, vary the final strength from 200 to 20,000 psi, adjust the set time from a few seconds to 36 hours, lower the rate of filtration, vary the flow properties, provide corrosion resistance, control fluid loss by using bridging agents, improve resiliance, reduce permeability, provide expansion, and control the heat of hydration through the use of inert and heat-absorbing fillers. Morozov et al. (1970) report that surface acting substances such as soap-naptha, abietate sodium, latex, bituminous and polyvinyl acetate emulsions, and polyacrylnitrile, can densify and reduce the permeability of cements with simultaneous improvement in water retention and workability of the slurry.

<u>Accelerators</u> include calcium chloride (Figure 1), sodium chloride (salt), gypsum, sodium silicate, and reduced-water cements. Calcium chloride is an especially effective accelerator, and is the

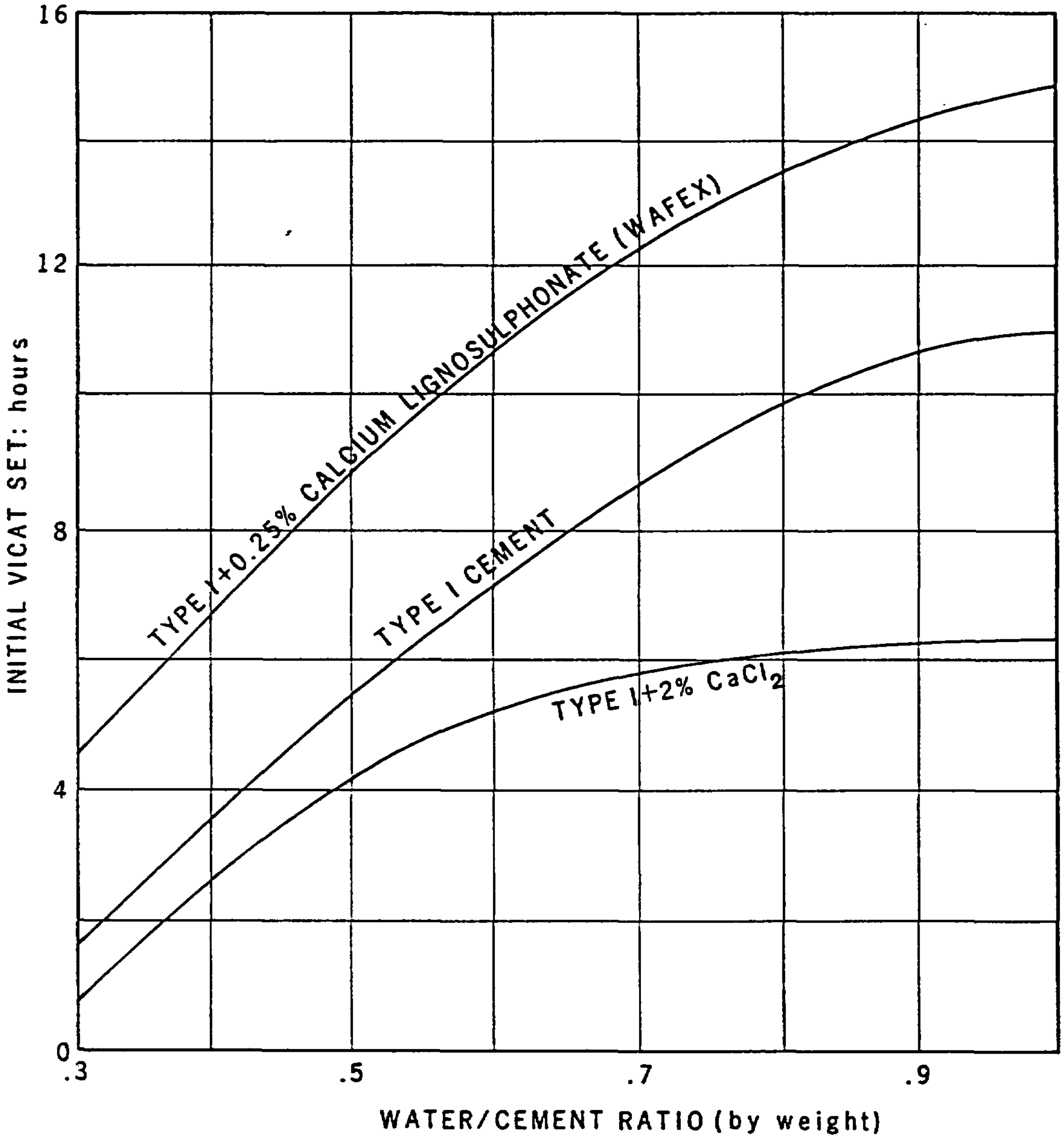


Figure 1. Effect of Accelerators and Retarders on Vicat Set (after Littlejohn, 1982)

most commonly used. It can adversely affect sulfate resistance, however, and must be thoroughly mixed in. Calcium chloride can also accelerate steel corrosion and enhance drying shrinkage. Salt is used but is less effective, and in large quantities is actually a retarder. Seawater can serve the same function. Gypsum can reduce setting times due to the tendency of the gypsum to set early. In amounts of 4-10 percent, gypsum will impart a flash set that can be useful in lost circulation zones -- 30 to 50 percent of gypsum content will set in 12-20 minutes regardless of the conditions. Gypsum in moderate amounts will also enhance thixotropy. Sodium silicate is most commonly used when certain specialty retarders are in the mix, normally carboxy methyl hydroxyethyl cellulose (CMHEC). Acceleration through densification entails the use of friction reducers and reduced water content.

Light weight additives (Table 2) reduce the column weight of the slurry, increase yield, lower the cost of the slurry, and may lower the filter loss. These include bentonite, prehydrated bentonite, modified cements (bentonite and calcium lignosulfonate dispersant), high-gel salt cements (bentonite, 3 - 7 percent inorganic salt, and a dispersant), diatomaceous earth, perlite, and gilsonite. Of these, bentonite (sodium montmorillonite) is by far the most common.

Bentonite can be dry-mixed or premixed with water in a high-shear mixer to produce a prehydrated gel. This homogenizes the bentonite and enhances water wetting -- 1 percent prehydrated bentonite is equivalent to about 3.6 percent if dry-mixed. Prehydrated gel at 2 percent by volume, a very commonly used ratio, will extend the yield by about 8 - 10 percent. The cementation of the Amchitka shafts in Alaska (Hunter, 1986) used considerable prehyrdrated gel. The procedure also allows the increased use of water (which can enhance bleed), extends set times, reduces density and weakens the hardened product. Clay contents of 2 - 5 percent by weight of water generally produce tolerable reduction in strength, but the strength lost (and hydraulic bonding lost) at higher contents can be significant. The amount of strength reduction also depends on water content (grout thickness) (Deere, 1982). With moderate addition of water, clay can actually reduce bleed, by enhancing the suspension of the cement particles (Littlejohn, 1982).

Bentonite cements tend to exhibit slow strength development with poorly-defined setting times (Littlejohn, 1982). This action is similar to that of a plasticizer. Because of their lower densities and greater initial water contents, bentonite cements can be more susceptible to chemical attack.

Kaolin and illite clays seem to affect hardening and durability less (Morozov et al., 1970) because of their relative inertness; they in essence act as fillers. Bentonite, however, has a high cation exchange capacity and can chemically interfere with hardening (Morozov et al., 1970).

Table 2: Common light weight cement additives (from Suman and Ellis, 1977)

	Extender, % by wt. cement					Density, ppg, for comp. strength:	
Type cement	Gel	Salt	Sodium meta-sil.	Water	Diacel D	Above 500 psi	Below 500 psi
Gel							
Class H	4	-	-	-	-	14.1	-
	8	-	-	-	-	13.1	-
	12	-	-	-	-	-	12.6
	16	3	-	-	-	-	12.5
Class C	4	-	-	-	-	13.1	-
	8	-	-	-	-	12.5	-
	12	-	-	-	-	-	12.0
Prehydrated gel	1.5*	-	-	-	-	14.2	-
	2.0*	-	-	-	-	13.7	-
	2.5*	-	-	-	-	12.8	-
	3.0*	-	-	-	-	-	12.3
	3.5*	-	-	-	-	-	12.1
	4.0*	-	-	-	-	-	11.8
	4.5*	-	-	-	-	-	10.7
Pozzolan and fly ash 50/50	2	-	-	-	-	14.1	-
	6	-	-	-	-	-	13.3
	10	-	-	-	-	-	12.8
	18	-	-	-	-	-	12.4
Silicate	-	-	1.0	-	-	14.2	-
	-	-	2.0	-	-	12.5	-
	-	-	3.0	-	-	-	11.4
Calcined shale-cement**	-	-	-	65	-	13.7	-
	-	-	-	85	-	12.8	-
	-	-	-	95	-	-	12.4
	-	-	-	115	-	-	12.0
Pozzolan and bentonite†							
Class H	6†	-	-	74	-	13.6	-
	6†	-	-	83	-	-	13.1
	6†	-	-	104	-	-	12.4
Class C	6†	-	-	104	-	-	12.0
Diacel D	-	-	-	-	10	13.2	-
	-	-	-	-	20	-	12.4
	-	-	-	-	30	-	11.7
	-	-	-	-	40	-	11.0

* Percent by weight water
** Trinity Lite-Wate data. Similar cement available from Texas Industries.
† 65/35 cement and Pozmix A % gel based on combined weight.

Heavy weight additives are used to densify slurries to offset hydrostatic pressures or to lend greater mass to concrete.

The most widely-used common weighting additive is hematite. Others (Table 3) are ilmenite, barite, sand salt, and low-water mixes (through the use of plasticizers). Note that the use of fine barite requires additional water.

Retarders are used to improve pumpability and grout travel by retarding the set, and to offset the effect of self-induced or external high temperatures (Figure 2). Some cements contain retarders as-manufactured, to meet the depth or pressure requirements. The compatibility of any contemplated retarder with the base cement and the other additives should be carefully checked.

The most common retarders are lignosulfonate (derived from wood processing), carboxymethyl hydroxyethyl cellulose (CMHEC), saturated salt, borax, sugar, and tartaric acid. Setting rates under hot conditions or when pumping over long distances can be reduced by as much as 100 percent using retarders (Littlejohn, 1982).

Lost circulation additives for cement are much the same as those used in drilling muds for the control of circulation losses, and their use is for the same purpose. Typical materials include ground walnut shells, cellophane flakes, perlite, gilsonite, coal, and nylon fibers. Accelerated or flash-setting cements will also help, as will reducing the slurry density.

Filtration control is important in the cementing of large holes and oil wells. Filtrate loss refers to the migration of the slurry liquid into the formation prior to the initial set, leaving behind a mix without sufficient water for proper hydration. Such cements are prone to weakness, higher porosity, and shrinkage. There are various organic polymers that perform a filtrate-control function, as does CMHEC, latex, and bentonite when used in conjunction with a dispersant. Latex can also improve bonding strength. The effects of bentonite have been covered previously.

Friction reducers (dispersants, plasticizers) have the effect of improving the flow properties for a given water/cement ratio (Table 4). The use of a friction reducer will allow sufficient pumpability with reduced risk of bleed and filtrate loss, by lowering the gel strength and yield point of the slurry. Common friction reducers are polymers, alkyl-aryl-sulfonate, polyphosphate, lignosulfonate, organic acid, fluid-loss agents, and salt. Polymers are lubricants and set time controllers, and do not retard the mix or promote bleed. Most are not compatible with salt. Salt controls the set time of slurries, particularly those containing bentonite, diatomaceous earth, or pozzolans. Salt can also affect slurry weight, set time, and can be a cause of foaming. (Foaming can be overcome by dry blending or with admixtures).

Table 3: Heavyweight cement additives (from Suman and Ellis, 1977)

Material	Specific gravity	Grind (mesh)	Max density, ppg	Extra water needed	Effect on comp. strength	Effect on pumping time
Ottawa sand	2.63	20 - 100	18	None	None	None
Barite	4.25	325	19	20%	Reduce	Reduce
Coarse Barite	4.00	16 - 80	20	None	None	None
Hematite	5.02	40 - 200	20	2%	None	None
Limenite	4.45	30 - 200	20	None	None	None
Dispersant	-	-	17.5	None	Increase	Increase
Salt	-	-	18	-	Reduce	Varies

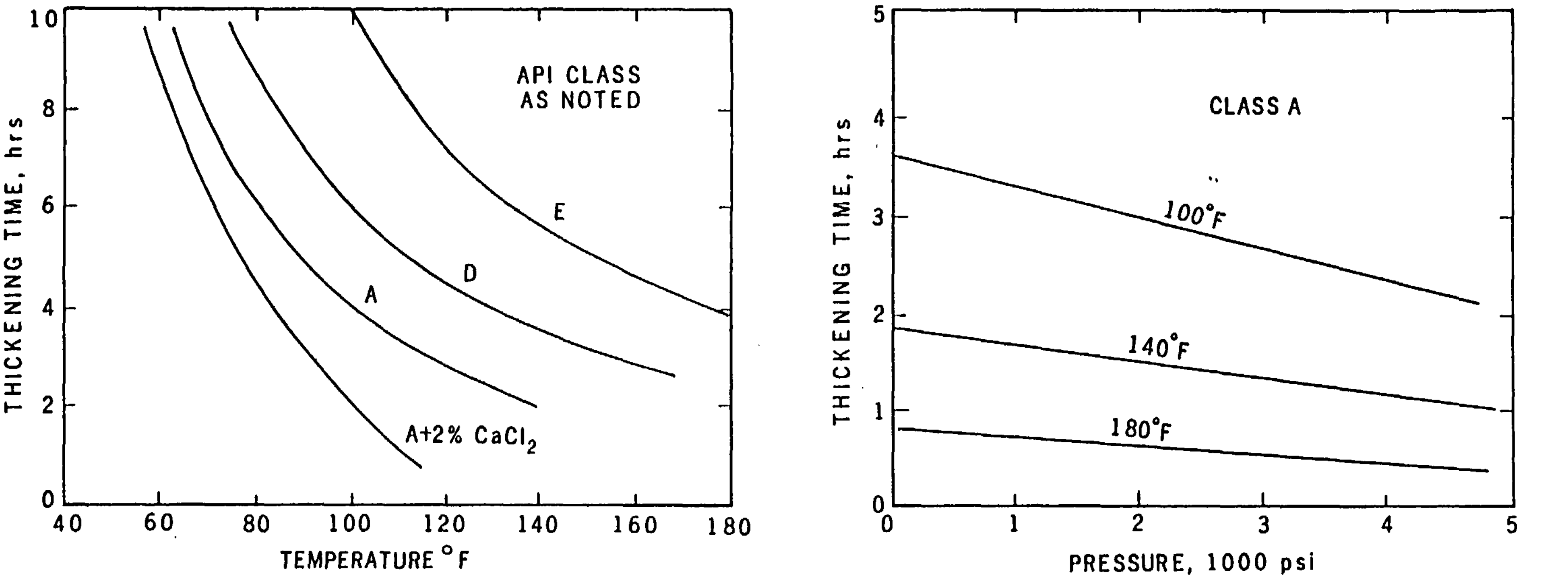


Figure 2. Effect of Temperature and Pressure on Cement Thickening Time (after Suman and Ellis, 1977)

Concrete workability can be improved with air entrainment, but at the expense of density.

Special additives include mud decontaminants (for removing the effect of co-mingling of drilling mud and cement), fibers for adding resiliency, silica flour for strength loss prevention (a problem at elevated temperatures), dyes and tracers for identifying grout pathways, and gypsum.

2.1.4 Mixing Agents

Cements and concretes can be mixed using water, oil or other petroleum products, or brine.

The use of brine can have a beneficial effect in controlling shale swelling and of course is prescribed for applications in salt, potash, gypsum, trona, or other water-soluble formations. Often when the use of brine-based cements is required, a saturated brine is specified. It is difficult to achieve a truly saturated brine in practice -- brines saturated under ambient conditions at the mixing station may not still be saturated under downhole pressure and temperature conditions. Also, the necessary chemistry of the brine should be determined beforehand and the interaction of the brine with the other cement additives assessed.

The use of diesel oil, kerosene, and other petroleum products, was discussed in Section 2.1.2. Commonly these are used along with a surfactant to enhance the hydration of the slurry when the watercourse is intercepted.

Water is by far the most common mixing agent used in the formulation of grouts and concrete. The amount of water used in the mix has a great effect on pumpability, penetratability, strength, permeability, and other properties. These properties are taken up separately in Section 2.1.7.

2.1.5 Aggregate

The essential difference between cement and concrete is that concrete contains an inert coarse filler -- aggregate. Typical aggregates are slag, sand, washed pea gravel, lightweight aggregates such as expanded perlite, crushed stone, and coarse gravel. Aggregate performs the multiple functions of absorbing heat of hydration, interrupting fracture plans that would, if continuous, reduce failure strength, and extending the mix.

Desirable qualities of aggregate are low harshness (angularity), durability or soundness, appropriate size distribution (grading), abrasion resistance, appropriate specific gravity, and inertness to alkali-aggregate reactivity.

Manufactured aggregates such as crushed slag or crushed stone (particularly basalt) can be very harsh, meaning that extended handling mixing, and pumping will be accompanied by accelerated wear on the equipment.

Soundness is most commonly measured by the magnesium and sodium sulfate soundness tests and are indirect measurements of frost resistance. Better results are obtained with direct freeze-thaw tests. Durability tests, which are combined immersion and abrasion tests, also point out nondurable components.

Alkali-aggregate reactivity (AAR) may occur if the aggregate contains amorphous or finely-divided silica, which can react with the high-alkali portion of the cement, producing an increase in volume at the point of the reaction. This disintegrates the concrete. AAR can be readily detected by a sodium hydroxide quick-chemical test. Oxidation of sulfides and reaction of alkali with argillaceous dolomites are other damaging reactions that have been observed. In general, aggregate containing sorption-sensitive carbonates, chert, chalcedony, siliceous siltstone, and sulfides should be avoided.

Large (8-ft diameter) plugs used for sealing shaft casing leaks at the Nevada Test Site were mixes containing 200 percent (by weight of cement) sand for heat control (Rowe, 1985).

Grouts may incorporate fine graded sand, pea gravel, or even coarse gravel as a filler, and to provide a heat sink. Morozov et al. (1970) found that calcite, quartz, and kyanite promote crystallization of cement, feldspars and hornblende were only slightly beneficial, and micas significantly reduced grout durability.

2.1.6 Mix Properties

The foregoing discussion has described the components of most cementitious grouts and concretes, and it has been pointed out that practically any desired mix or endpoint property can be theoretically attained. This subsection discusses the important properties of mixes in somewhat more detail.

The most important consideration in grouting for seepage control is to deliver a stable, low-permeability material of suitable strength to the maximum number of water conduits possible. The ability to do this depends on the viscosity and grain size of the grout and the manner in which it is injected.

Flow properties result from dynamic interparticle forces of attraction and repulsion, and in dense grouts, dilatency of moving particle matrix to expand as the particles interact (Littlejohn, 1982). It has in the past been the practice to enhance pumpability by thinning the mix with water; however, this can result in high water/cement ratios that are prone to bleeding. It also does little to solve problems related to grain size in the slurry. Continuous agitation can delay the buildup of shear resistance by overcoming thixotropic tendencies. Nonetheless, when thixotropic grout enters the ground, the agitation ceases, and penetrability is reduced.

Cements will exhibit a flash set that precedes the main set, occurring only minutes after mixing. This is due to the behavior of the gypsum component. It is important to either have the grout in place by this time, or to continue agitation until the agglomeration caused by this set is overcome, in order to reduce pumping problems and the possibility of bridging.

Thixotropy is a property that, simply put, means the shear resistance in motion is far less than the shear resistance at rest. Thixotropic grouts can be poured into a paper cup and, if allowed to settle without beginning the hardening process, the cup can be inverted and the cement will remain in place. If the same mix is then stirred, it pours easily. Chem Comp is well-known for its thixotropic properties. Gypsum will impart thixotropic behavior to cement.

Thixotropy can be a help or a hindrance. Near the limit of the grout penetration, the velocity of the grout drops and at that point thixotropic grout exhibits a drastically reduced ability to penetrate any further without very large increases in pressure. On the other hand, thixotropic grouts have high gel strengths that can be essential in open voids or under moving water conditions. Bulkheads at the Nevada Test Site for containment of detonations in horizontal tunnels require gas-tight sealing to 1,000 psi. Despite its expansiveness, Chem Comp was found unsuitable because of its thixotropic properties and inability to penetrate fine cracks (Tibbs, 1985).

This report mentions the water/cement ratio (w/c) in discussing case histories, whenever this information was available. Customary U. S. practice is to express the w/c ratio in terms of volume with respect to loose cement. Thus, in this report a 3:1 w/c grout contains 3 parts by volume of water to 1 part by volume of cement. A standard 1-cubic-foot bag of dry cement weighs about 94 lb. Since some weight-based ratios are used in the literature it is necessary to know which is being reported: a 6:1 w/c ratio by volume is equivalent to a 4:1 ratio by weight.

Because of settlement, mixtures containing a large proportion of coarse aggregate or sand should use plasticizers rather than increased water content to promote pumpability and workability (Auld, 1983b) (see Table 4). Concrete, in particular, suffers strength losses if placed at high w/c ratios, and in jump-formed shaft linings, the settlement that is almost inevitable can promote leakage. Auld (1983b) recommends the absolute minimum water to achieve about

Table 4: Effect of plasticizer on shaft concrete slump (from Auld, 1983b)

Site	North Selby, Shaft Lining	North Selby, Bulk Filling
Total cementitious content	500 kg/m^3 (30% OPC*, 70% Cemsave)	400 kg/m^3 (250 kg/m^3 OPC, 150 kg/m^3 pfa)
Sand	Blaxton Zone 3 595 kg/m^3	Elvaston Zone 2 770 kg/m^3
Sand % of total aggregate	34%	42%
Coarse aggregate	Blaxton gravel 1150 kg/m^3	Elvaston gravel 1050 kg/m^3
Water	180 li/m^3	180 li/m^3
Water:cement ratio	0.36	0.45
Slump without plasticizer	60 mm (2.36 m)	50 mm (2 m)
Plasticizer	Flocrete N	Flocrete N
Slump with plasticizer	160 mm (6.3 m)	160 mm (6.3 m)

* Ordinary Portland Cement

an inch of slump for each 1000 ft of fall, for vertically-placed concrete. The shaft concrete in the Miami, AZ No. 11 and 12 shafts was placed at a much higher slump (8 in.) and was not vibrated for this reason.

Slump of concrete is measured with a slump cone and is essentially a measure of the shear resistance of viscosity of the concrete. The column of mix is initially 12 in. high in the inverted cone; when the cone is removed, the difference in the height of the "slumped" concrete and the original height is the reported slump. Some very fluid concretes have been used in underground construction. Slumps of concrete used for drift support at the Lakeshore Mine in Arizona were reportedly as high as 11 in. because of the long pumping distance (the aggregate was 3/4 in.). Normal tunnel concrete is placed at a slump of 4 - 6 in. If it is necessary to place shaft concrete at a high slump, segregation can be controlled by thorough remixing at the bottom, by keeping transport distances low, and using vibration to aid in pre-settling and spreading the mix.

For grouting with cements, the classic w/c ratio is 2:1 (Houlsby, 1982b), thinner for fine cracks or thicker for open voids. It is now possible with modern additives, to achieve acceptable penetrability and workability at lower w/c ratios.

Penetrability of grout in larger cracks is related to the viscosity of the mix and to the injection pressure. In smaller cracks, the penetration can be affected by the grain size of the cement. A widely accepted rule of thumb is that a crack cannot be penetrated by grout whose maximum particle size is more than about 1/3 the width of the crack. Particles larger than this will bridge (Annett, 1969). The U.S. Army Corps of Engineers Waterways Experiment Station (1956) found that penetration of artificial apertures as small as 0.25 mm was possible using high w/c grouts. The use of microfine cements will enhance penetrability -- microfine cement has an average grain size on the order of 4 microns, and a maximum grain size of 10 microns. The generally-accepted limit of hydraulic conductivity that can be acceptably penetrated with a standard cement grout is about 0.01 cm/sec, although much lower values are reported for the use of microfine cement (Tibbs, 1985).

Recent developments in grout mixes have gone a long way towards improving the penetrability and workability of non-bleed (actually limited-bleed) grouts. Cement grouts developed by the Waterways Experiment Station (WES) for plugging and sealing tests for repositories are based on Class H cement and, with the aid of plasticizers, retarders, and dispersants, and the addition of silica, retain good workability and pumpability for 3 - 4 hours at w/c of about 0.3. Although these are sanded grouts, it would seem that for crack filling where heat control is not as great a consideration, the sand could be omitted from the mix and a fine grind used to improve crack penetration.

The preceding discussion has made mention of bleed, and the concern regarding bleed occurs repeatedly in this report. Intuitively, bleed refers to the segregation of water from the mix and can occur at any time following mixing. The water that is expelled, can travel through the gelled grout mass, creating channels. In the absence of dispersants or stabilizers, such as bentonite, bleed is generally enhanced at high w/c ratios (Figure 3) where settlement is facilitated by low slurry viscosity. Increases in w/c ratio are marked by accompanying gradual increases in bleed rate and amount of bleed, thus, the threshold of bleed-proneness is difficult to pinpoint. Generally without additives w/c ratios less than about 0.5 are resistant to bleed and w/c ratios greater than about 1.0 are prone to bleed. The working w/c limit with respect to bleed is, of course, a function of the grout application and the mix, which can develop a set before bleed occurs. Microfine cement (MC-500) is easily wetted because of its fine grain size, so water contents can be higher, gel times (especially with sodium silicate added) can be less (1-3 minutes), and separation slower and less, that ordinary Portland cement (Shimoda and Ohmori, 1982). Bleed action in grout is discussed in Section 2.3 and its implication in shaft grouting and cementing is covered in Section 4.5.2. Bentonite can, in moderate amounts, reduce bleed by aiding the suspension of the cement particles. Other additives are available to reduce bleed -- methocel was used to prevent bleed at the Riccall shaft at Selby (Fotheringham and Black, 1983).

2.1.7 Cement Grout and Concrete Performance

The performance of grouts and concretes in void mitigation can be addressed in terms of strength and density, permeability, uniformity, and longevity. This section discusses how these properties operate in producing or discouraging void formation.

2.1.7.1 Strength and Density. Solid density and strength are positively correlated for concretes and cementitious grouts. Bentonite tends to weaken cementitious materials by reducing the number of cement gel bond linkages and by reducing the density.

The w/c ratio is a major determinant of strength. This is because the hydration of the cement particles produces products that take about 2.3 times up more space than the solid particles did originally (Powers et al., 1955); the excess space is interparticle volume formerly occupied by water. Excess water discourages dense interconnected crystal growth and can even be trapped in the pores of the set product. Strength loss is approximately proportional to water content. Grouts containing little air typically exhibit porosities of 6.5 percent at w/c = 0.55, and 3.5 percent at w/c = 0.4 (Littlejohn, 1982). Too little water for thorough hydration can also be weakening. High-shear mixing can ensure thorough particle wetting at low water contents.

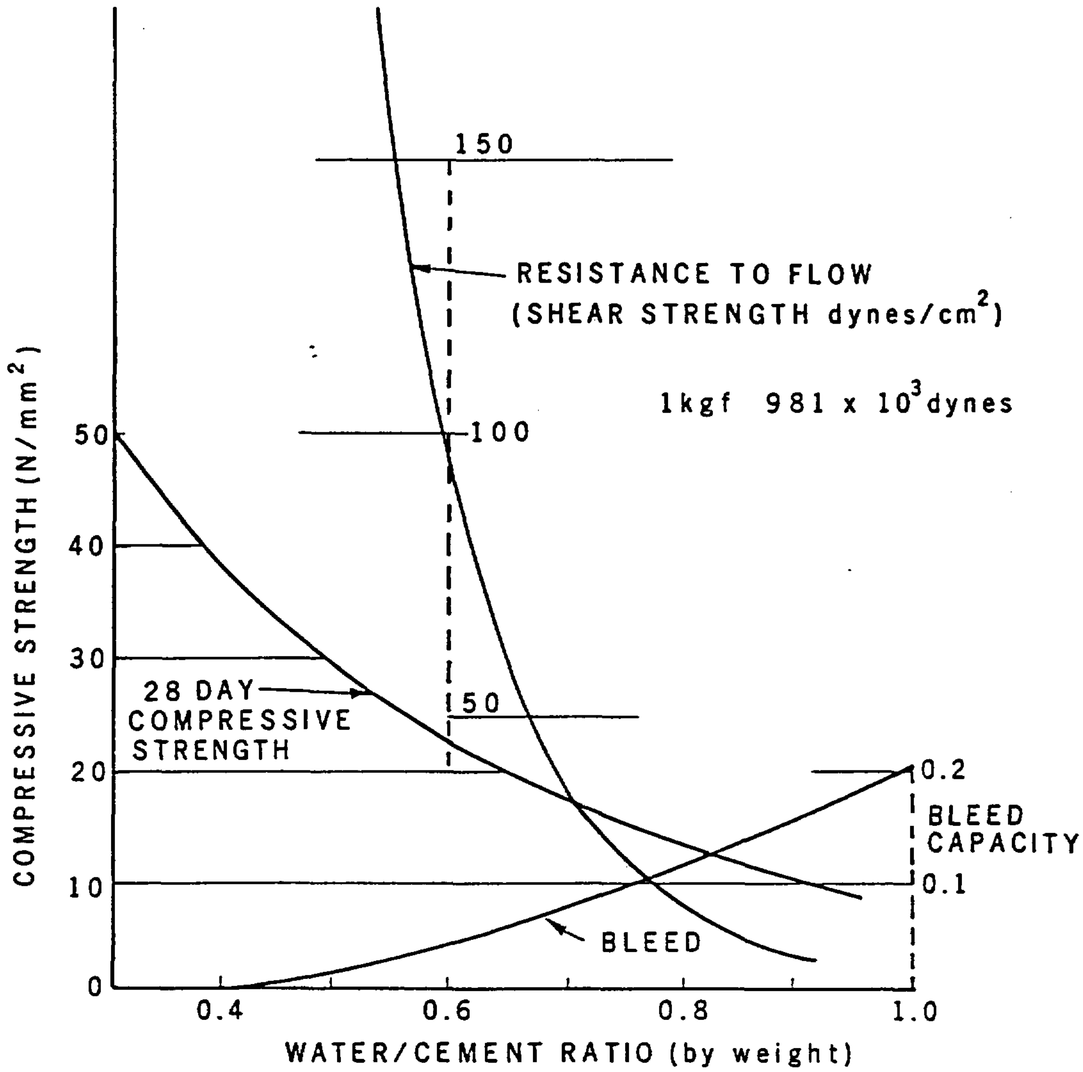


Figure 3. Effect of Water Content on Grout Properties (after Littlejohn, 1982) (1 N/mm^2 = 145 psi)

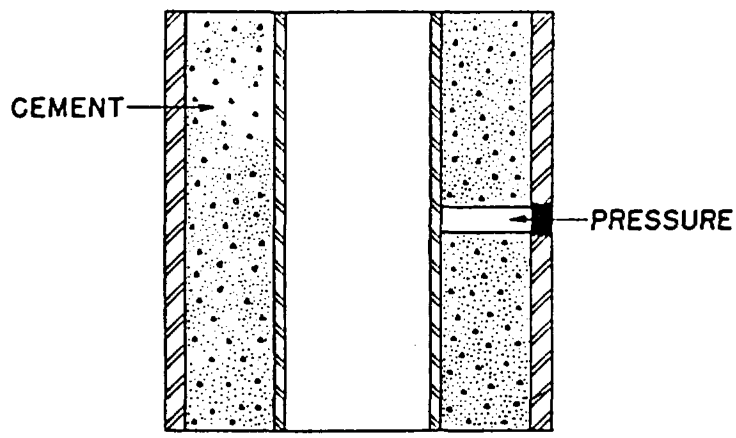
Another determinant of strength is time. Generally, cementitious mixtures increase in strength with age. Concrete strength, for example, is usually referenced to its value in 7 days or 28 days. However, concrete will continue to gain strength at a decreasing rate for months to years, unless dried out or attacked chemically or mechanically.

Strength is normally measured in unconfined compression. Previous sections have pointed out that a wide range of strengths is possible, and that additives such as silica flour can greatly increase strength. Commonly, shaft concrete is in the 3,000 - 6,000 psi range if water or earth pressures are anticipated. For example, Newman (1978) reports the strength of shaft concrete at the Harmony Gold Project in South Africa as being 4,500 psi, using Type I and calcium chloride accelerator. Strengths of 6,500 psi were attained on some British coal mine projects by lowering the w/c ratio and using admixtures to sustain the workability (Section 3.4.2). Microfine cement (MC-500) mixed at a typical w/c ratio of 4.5:1 to 6:1 can attain 9,000 psi (Shimoda and Ohmori, 1982).

An important consideration in preventing the formation of a micro-annulus is hydraulic bonding. Hydraulic bonding is a measure of the resistance of the cement casing or cement-rock interface to separation by pressurized water or gas (Figure 4). It depends on the condition of the surfaces involved (Table 5) and on the cement strengths, and responds in about the same way to additives such as bentonite (Figure 4). Boughton and Dellinger (1965) report hydraulic bonding of about 375 psi for salt-saturated expanding cement, and about 275 psi for salt-saturated Class A cement, in the holes drilled for Project Dribble at Tatum Dome.

Cement and concrete strengths are affected by the admixtures used and by the environment they are placed into. Filtrate loss and freezing can have serious deleterious effects, although these can be considered as mix changes occurring prior to the set. High heat can produce a too-rapid initial set and a lower ultimate strength than when properly cured. Some formation chemistries are important, also. Research on the use of microfine cement in salt at Sandia Laboratories seems to indicate reductions in strength and durability when placed against salt, that are not present in nonsalt applications (Gulick, 1986). Cements that have low density and high porosities are more susceptible to sulfate attack and temperature changes, particularly freezing and thawing. Littlejohn (1982) recommends a maximum w/c ratio of 0.4 in frost-prone areas to reduce porosity.

2.1.7.2 <u>Cement Grout and Concrete Permeability</u>. Permeability is another critical parameter for grouts and concretes. Some very low values have been reported, but these are usually associated with ideal laboratory conditions. The higher the cement content, the lower the permeability. Silica flour, in addition to increasing the strength and density, can also reduce cement permeability, to as



HYDRAULIC BOND TEST

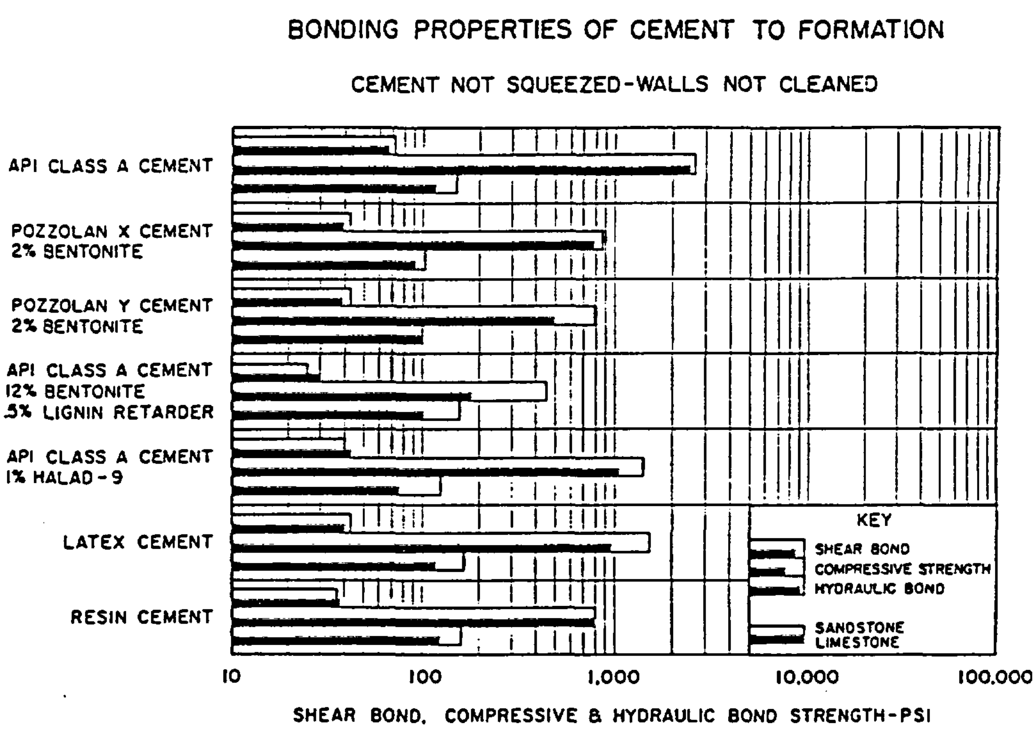


Figure 4. Effect of Additives on Cement-Formation Hydraulic Bonding (lab tests) (from Evans and Carter, 1962)

Table 5: Effect of casing condition on hydraulic bonding

Surface finish	Type mud wetting	Hydraulic bond (psi)	
		Water	Gas
New mill-varnished	None	200-250	
Varnish removed (chemical	None	300-400	
Varnish removed (sand-blast)	None	500-700	150
Varnish removed (sand-blast)	Fresh water	100	50
Varnish removed (sand-blast)	Invert oil emulsion	100	50
Varnish removed (sand-blast)	Oil base	100	50
Resin-sand coat (new, sand blast)	None	1,100-1,200	450
Resin-sand coat (new, sand blast)	Fresh water	100	55
Resin-sand coat (new, sand blast)	Invert oil emulsion	100	45
Resin-sand coat (new, sand blast)	Oil base	100	45

Cement: API Class A
Water Content: 5.2 gal/sk
Curing temperature: 80°F
Curing time: 24 hours
Casing size: 2 in. inside 4 in.

Examination of these specimens revealed no discrete cracks and it was thought that shrinkage had disrupted the hydration product structure. Daemen et al. (1985) also note higher permeabilities in dry-cured specimens. Abel et al. (1979) report 0.07 percent shrinkage for controlled curing of a 7-sack mix. This can be more than tripled with air drying.

Thermal cracking is due to temperature gradients and is usually associated with massive concrete where heat can become trapped in the interior. Cement hydration temperatures in cementing casings in blind-drilled shafts can reach 212°F (100°C), but boiling does not occur because of the high pressure. Pours greater than 3 ft thick (some shaft concrete is over 4 ft thick) are particularly susceptible. Rowe (1986) used a 200 percent (by weight) variable sand content as a heat sink in a 6-ft-diameter by 8-ft-high shaft plug. Internal temperatures reached 195°F at 26 hours, but no visible stress cracking occurred. A hot environment (air or ground temperature) can accentuate the problem.

Cracking due to sulfate attack, oxidation of sulfides, and aggregate reactivity, have been discussed earlier.

Other sources of cracking are corrosion of reinforcing steel, poor mix handling practice, overloading of immature concrete, inadequate reinforcement, and failure to consider tensile strain in design.

Shaft concrete and grout may also be compromised by lack of uniformity (segregation). One form of segregation in concrete is honeycombing (an open, gravelly structure). In grouts, bentonite has been suspected of separation from the cement, in fine fissures. Separation also occurs in fine fractures due to bridging or thixotropy. The irregularity of the stage top surfaces in the Crownpoint drilled shaft was due to thixotropy of Chem Comp cement. Deere (1982) reports that 6:1 w/c mix grouts can undergo up to 60 percent sedimentation in 2 hours.

Expansion can compensate for shrinkage effects. Much has already been said about achieving expansiveness in cementitious materials. The expansion takes place during the hardening process and normally should be designed to be less than 1 percent. Depending on how a high expandability is achieved, serious strength deficiencies and permeability increases can result. Rowe (1986) described the problem resulting from a Type-A based cement that expanded over 4 percent. Normally a few tenths of a percent is sufficient. Boughton and Dellinger (1985) describe a salt-saturated cement that achieved 0.68 percent expansion in 9 days and a neat cement that expanded 1.04 percent at 120°F and 1,500 psi confinement in 9 days. Often interparticle bonding in salt-saturated cements are not as strong as with ettringite-forming expansive cements.

Buck et al. (1985), have developed sanded salt grouts for repository sealing exhibiting about 0.4 percent expansion over the long-term (1 - 2 years). Strengths are 5,000 - 6,000 psi, and permeabilities are about 2 microdarcies for the unsanded version and unmeasurable for the sanded grout. Good bonding to salt core, exhibiting very little preferential flow, was achieved with care; however, prolonged exposure to high pressures resulted in an increase in permeability, probably due to a shift to partial saturation with the increase in pressure. Bleed with this mix (Table 6) is less than 1 percent. Another grout developed at the Waterways Experiment Station (Buck, 1985) is formulated from Type H high-sulfate-resistant cement, Class C fly ash, plaster, water reducers, de-air additives, sand, silica flour, and reaches 15,000 psi compressive strength at 1 year. Initial permeability and expansion are good; however, slight permeability increases at a contact with anhydrite, were noted.

Expansion of Chem Comp, as that of other expansive cements, depends on the curing environment. In addition to requiring sufficient water for initial hydration, most cements show full expandability only when cured under 100 percent humidity or in contact with water. Upon the application of heat, expanding cements could be expected to shrink somewhat, depending on the way the water is bound up in the cement matrix. In this regard, it seems that the so called "true" expansive cements, which form well-bonded aluminosilicates, would be more stable. The expansiveness of some cements is affected by confining pressure, while that of others is not. Rowe (1986) describes cement mixes that did not show reduced expansion under confinement, but that did show lower effective linear coefficients of expansion at elevated temperatures. This suggested that the component of expandability due to crystal formation may have been adversely affected by heat.

2.1.7.3 Longevity. One major concern for grouts and concretes used in repository shafts is the expected life. Grout used only in operational system components may not need to have 1,000-year performance, but even the operational lifetimes alone (on the order of 90-100 years, including the retrievability period) are well in excess of the period of time that the special grouts discussed herein have been in use. The life of conventional industrial grouts, even those not subjected to high heat levels or thermomechanical ground deformation, is between 20 and 50 years. These grouts may not display the ideal characteristics that are demanded of repository grouts -- expandability, heat control, low water-cement ratios, chemical compatibility, and low permeability. The release of bonded water or interstitial water that can occur with some cements can, along with chemical attack, be a major limitation on grout life. New mixes at WES will be tested for indications of long-term instability by very close monitoring, overtests (chemical and thermal), and identification of deterioration mechanisms by study of ancient grouts and concretes (see Section 6.3).

Table 6 : Formulation of limited-bleed, expansive grout (BCT-1-F) (from Buck et al., 1985)

Constituents	Amount Required for 1-cu-ft Batch, lb
RC-881 Class H Cement*	39.32
AD-592(2) Fly Ash*	13.21
Cal-Seal (Plaster)*	4.63
Melgran 0**	0.86
Salt (NaCl)†	7.69
De-Air No. 1	0.1422
20-40 Sand	20.25
D-30 Sand	20.25
Silica Flour	8.11
Water	20.69
Ratio of Water to Cementitious Materials (0.36)	
Flow was 21 sec at 5 min, 20 sec at 60 min, and 19 sec at 120 min	
Bleeding was 0.58 percent	
Actual Unit Weight, 136.43 lb/ft^3	

* Considered to be cementitious materials.

** Melamine powder high-range water reducer (superplasticizer); 1.50 percent used by weight of cementitious materials.

† 37.2 percent by weight of water (BWOW).

Chemical grouts, discussed in the next section, can exhibit marked instabilities in the long term.

2.2 CHEMICAL GROUTS

There are many shaft performance problems for which chemical grouts were the only successful solution. Chemical grouts have the advantages of having readily controllable gel times and in some cases, very low particulate contents and viscosities, that enable close agreements between takes for grout and water. Thus, some difficult sealing problems associated with fine cracks or fine-grained soils have been solved with chemical grouts. Unfortunately many chemical grouts have serious drawbacks for repository application. These include a tendency for soils and unconsolidated materials grouted with some chemical grouts to creep, toxicity, potential gel instability in the presence of high thermal conditions, extreme shrinkage under wet-dry (and possibly saline) cycling, low strength, and for some chemical grouts, poor penetrability due to high viscosity even with low particulate contents.

The most common chemical grouts can be classified as sodium silicate formulations, acrylamides, lignosulfonates, phenoplasts, and aminoplasts (Karol, 1982). All are true solutions. The lignosulfonates and sodium silicates are the precipitating type, involving a reaction with an acid to produce a salt and gel. The acrylamides, phenoplasts, epoxies, and resins are polymer-forming grouts.

Most chemical grouts are solids when fully congealed, but are generally mixed with water in the field to enhance the penetration and reduce the cost. The water contents in the injection mix are typically 80 - 90 percent. When water is added, the reaction product is a gel, in which the polymers are linked in a relatively open structure, entrapping water molecules. This water (88 - 92 percent by weight (Karol, 1982) is not chemically bonded to the gel and is likely to leave the gel structure in response to mechanical (pressure), chemical, or thermal changes, such as freeze-thaw or wet-dry cycles, or reaction with soil and groundwater to form soluble reaction products (Karol, 1982). Most grouts will re-swell upon rehydration (within limits), but once fractured, they will not re-heal.

Some chemical grouts have viscosities as low as 2 centipoise (cp), which approaches that of water (1 cp), and these are usable in ground with permeability as low as 0.0001 cm/sec. The chrome lignins and phenoplasts have viscosities on the order of 5 cp, and are suitable in ground permeabilities of 0.001 cm/sec. Viscosities of chemical grouts can range up to 7000 cp and more. All grouts are difficult to place in soils with more than 20 percent silt and clay (Karol, 1982).

Lab viscosities can be misleading. Depending on the ground, pump times in fine-grained soils or fine cracks can be long, resulting in a discrepancy between the lab viscositity and the usable viscosity. A 2 cp (lab) grout may have a usable viscosity of 10 cp.

Chemical grouts, with the possible exception of epoxies, normally do not greatly increase the strength of the grouted medium. A coarse soil grouted with a chemical grout typically has a shear strength of about 70 psi. Chemical grout strengths normally do not compare with cement grout strengths. Chemically-grouted soils are known to be susceptible to creep in unconfined compression, and creep endurances can be as little as 40 percent of the instantaneous strength. Graf et al. (1982) reported a decrease in grouted soil strength with time and water immersion. Because of their open structure, chemical grout gels can be pervious to some ions, including dissolved salts if the gel was originally fresh-water based.

Grout reactions are seldom 100 percent complete, as actually applied in the field. This affects pregrouting planning when considering toxicity, degree of reliance on achieving the laboratory properties, factoring in temperature and chemical changes, and in committing to long-term uses.

For many temporary shaft sealing applications, however, the chemical grouts have been notably successful. For example, the upper 100 ft of the International Minerals Corporation (IMC) No. 5 shaft in New Mexico was injected with 60,000 gallons of a 20 percent Geoseal mixture and inflow was reduced from the projected level of 1,500 gpm to 35 gpm, enabling sinking to proceed.

The often-remarkable penetrability of chemical grouts is due to low viscosity, low particulate content, and controllable gel time that approximates the pumping time. This means that the grout gels over a very short period as compared with the pumping time, with little increase in viscosity prior to the onset of gelling. This feature is pronounced in the acrylic polymers, such as AM-9, and less so for the silicate grouts. At long pumping times, dilution near the grout front can be a serious problem.

2.2.1 Sodium Silicate-Based Grouts

Sodium silicate grouts are the most widely used of the chemical grouts because of their performance, relatively high strength, and low cost. Unfortunately, they are also of relatively high viscosity, and susceptible to synerisis (water explosion and shrinkage) and dissolution. Dissolution is caused by unreacted soda, which reverses the reaction that formed the polysilicic acid. Low-viscosity silicates do exist, but have lower strength, permanence, and gel time control. Silicate grouts also tend to coat fissures and are used as pregrouts in some applications to reduce pressure losses for later cement grout injection.

Sodium silicate is syrupy and has a pH of near 11. When reduced by acidification or saponification, a gel of silicon dioxide and hydroxides is formed. The grout is non-toxic and very low in corrosiveness.

Sodium silicate grouts in sand specimens stored under water for extended periods have been observed to deteriorate and experience a partial to total loss of strength (Karol, 1982). Grouting can reduce soil permeability by up to 6 orders of magnitude, but the effect is commonly not permanent, due to synerisis (Baker, 1982; Davidson and Perez, 1982). Creep tests have shown failure after as little as 3 days at less than half the instantaneous strength (Baker, 1982; Borden, Krizek, and Baker, 1982). Creep is less important in confined conditions.

Hoshiya et al. (1982), describe a new grout based on a non-alkaline silica soil that exhibits higher strength and penetrability than water-glass grouts with organic reactants.

2.2.2 Chrome-Lignin Grouts

Chrome-lignin grouts are derived from lignosulfonates, which are by-products of wood processing. Because the reagents are industrial by-products, there are 6 grades, depending on viscosity, resin and sludge content. Grouts have lignosulfonates and a hexavalent chromium component. Solution viscosities vary from 3 - 8 cp. They produce a tough, rubbery gel, imparting about the same strengths in soils as the acrylamides. Creep endurance limits are about 25 - 50 percent of the instantaneous values. As with most gels, soils grouted with chrome-lignins are freeze-thaw and wet-dry susceptible, and are therefore considered temporary stabilizers in most applications. The dichromate salts used in the grouts are toxic.

2.2.3 Acrylamide Grouts

Discovered in 1951, the first commercially-produced acrylamide grout (AM-9) was hailed as being very nearly the "ideal" grout: very low viscosity, excellent gel time control, maintenance of low viscosity throughout injection, and sufficient strength (though modest) for most applications. Unfortunately, the acrylamides are extremely toxic, and are no longer manufactured in the U. S. Acrylamide grouts are a mixture of acrylamide and a methyl derivative. The polymer formation is accomplished by a catalyst that controls gelling time according to concentration. The rate of polymerization is slower in air than in water and is affected by the presence of salt and alkalinity.

Acrylamide gels are nearly 90 percent water and will shrink in strong brines unless made with a saturated solution (Annett, 1969).

Acrylamide grouted sands have creep endurance limits of about 25 percent of the unconfined compressive strength, and are susceptible to wet-dry cycling.

Although acrylamides are no longer available from domestic manufacturers, they can be obtained from Japan, although at a high cost. After the acrylamides were removed from the market, there was a major effort to develop replacements. Berry (1982) describes a related grout, Injectite-80, based on low toxicity, biodegradable organic reagents and inorganic salts, that delivers comparable performance to acrylamides. The viscosity of Injectite-80 is somewhat higher than the acrylamides (10 - 50 cp as opposed to 1 - 2 cp) but it offers some offsetting features. It is less susceptible to wet-dry cycling and is less likely to become diluted when injected into waterbearing ground, and the reduced tendency for dilution also offsets the higher viscosity. It is chemically related to nylon, and is chemically stable. Gel times tend to be longer but less grout is normally needed for a given application. Another "substitute" grout, acrylate, is discussed in Section 2.2.6.

2.2.4 Phenoplasts

Phenoplasts are formed by the reaction of phenol and analdehyde. An acid environment is required at ambient temperatures. They have relatively high strength (comparable to high-concentration silicates), low viscosity (1 - 3 cp), polymerize essentially instantly, and maintain constant viscosity throughout injection.

The most common phenoplast grout is resorcinol reacted with formaldehyde. Set time is controlled by the concentration of the components. Phenoplasts will completely disintegrate if subjected to wet-dry cycles. Although only the catalyst is likely to be mobile in groundwater, all three components of phenoplast grouts are toxic. Common commercial phenoplasts are Geoseal, Terranier, and Rocagil.

2.2.5 Aminoplasts

Aminoplasts are made with urea and formaldehyde. They will set up only in environments of pH less than 7. Their solution viscosities are similar to those of the acrylamides and phenoplasts, but prepolymers are commonly added to control the product, which can increase the viscosities into the 10 - 20 cp range. As chemical grouts go, they are somewhat more resistive to creep, although they do exhibit the tendency to break down under wet-dry or freeze-thaw cycling. If the final gel contains unmixed formaldehyde, there can be a toxicity concern. Some commercial products are Herculox, Diarock, and Cyanoloc 62.

2.2.6 Other Chemical Grouts

There is a wide range of chemical grouts besides the ones listed above. These are somewhat less common but are suitable for specialized applications. Variants exist of the common ones above, with additives to offset some of the undesirable attributes.

For example, Berry (1986) reports that experiments are underway under the sponsorship of the Department of Energy's Strategic Petroleum Reserve to investigate the response of grouts equilibrated under ambient conditions to changes in the salt dome chemical environment. These experiments are varying the polymer content and the mix water chemistry, adding dessication protectants such as ethylene glycol, and adding fillers such as celite. There has been no formal reporting of the results to date, as the experiments are still in progress and results are preliminary. However, it appears that the products tested may be more stable when the mix contains higher polymer contents. Ethylene gycol is helpful in reducing the effects of wet-dry cycling, but not in all cases. Some acrylates tested seem to be more susceptible to changes in chemistry of the formation water and some polyacrylamides appear to be more stable.

Clarke (1982) describes a popular acrylate grout, AC-400. It is a mixture of acrylate polymers that is polymerized with an oxidation/reduction catalyst system and is cross-limited with a small amount of methylene bisacrylamide. The solids content is about 40 percent and the viscosity is about 2 cp. It swells slightly in contact with water. AC-400 has the advantage of being non-toxic.

There is a number of grouting materials that set up or produce foams upon contact with water; of these, the polyurethanes are probably the best (Karol, 1982), because of their superior mechanical properties. Polyurethanes are two-component reactions controlled with a catalyst. Although high-viscosity (22 to 300 cp for TACSS) some low-viscosity polyurethanes have been introduced recently. Grieves (1975) reports the use of polyurethane foam for sealing marl which was moisture-sensitive and to be placed behind concrete. Polyerethanes are capable of penetrating fissures in the 0.01 - 0.03 mm size range.

In the USSR, abundant vegetable wastes have been used to produce a furfurol grout that exhibits good performance in irregular cracks (Adamovich and Baushev, 1970). Furfurol products displace water adhering to mineral grains, have a low viscosity (2 - 13 cp at full strength), are resistant to chemical degredation, and are inexpensive. Depending on the chemical environment, the furfurol can be hardened with several compatible media. They are not strong (2-month strength is only about 57 psi). They also swell upon contact with water.

Various polymer grouts will accept fillers (Demin and Popov, 1974). Fillers can increase strength (Figure 5), lower cost, and reduce shrinkage. Filler materials include fly ash and clay. Penetrability of 0.06 mm was achieved with high-shear mixing.

2.3 GROUT PLACEMENT MECHANICS

Prior to grouting, a water test is commonly run to determine takes, potential flow paths, and communication. The take of a water is usually less than that of the grout that follows. In fractured ground, water testing also serves the purpose of washing out the hole and fractures. At the same time, adverse chemical effects (such as acid groundwater if an acid-setting grout is to be used and a flash set is to be avoided) can be mitigated with a chemical wash, or sodium silicate might be used to prepare the ground for the acceptance of cement grout. Fresh-water washes prior to injection of silicate into formations bearing high Ca and Mg in groundwater will reduce bridging due to early precipitation of calcium and magnesium silicates.

The water-take is measured in lugeons. One lugeon is defined as a water take of 1 liter of water per meter of hole per minute at 10 bars pressure. A lugeon corresponds roughly to a permeability of 1.3 x 10^{-5} cm/sec hydraulic conductivity. Lugeon values in excess of 100 are quantitatively almost meaningless (Houlsby, 1982a). Soil with hydraulic conductivity of 10^{-4} to 10^{-5} is considered marginally groutable and anything of less permeability is considered ungroutable (Baker, 1982).

Modern grouting programs have graduated from the piston-type pump to the Moyno pump. Piston pumps can exhibit rapid piston wear, valve clogging, difficulty in cleaning, frequent blockages, and the potential for transient pressure losses during the piston cycle. The Moyno pump uses a helically-grooved shaft that rotates within a rubber sleeve. The grout is thus displaced forward at all times for a uniform flow, with high pressures possible. There is less dependence on valves and the replacement of the rubber sleeve is straightforward.

Circulating grout line systems should be used to make the best of pump efficiency. At the end of a stage, when flow volume decreases but pressures are the highest, a return line will allow the maintenance of flows commensurate with the pressures involved. The use of return lines will help forestall blockage before the true refusal point is reached. This and the use of smaller-diameter lines aids in reducing pressure losses with thixotropic grouts due to the higher velocities maintained throughout the stage.

Grouting is normally done in stages, which can be descending or ascending. Generally the choice of which is dependent on ground stability. If holes cave badly or there are structures (including

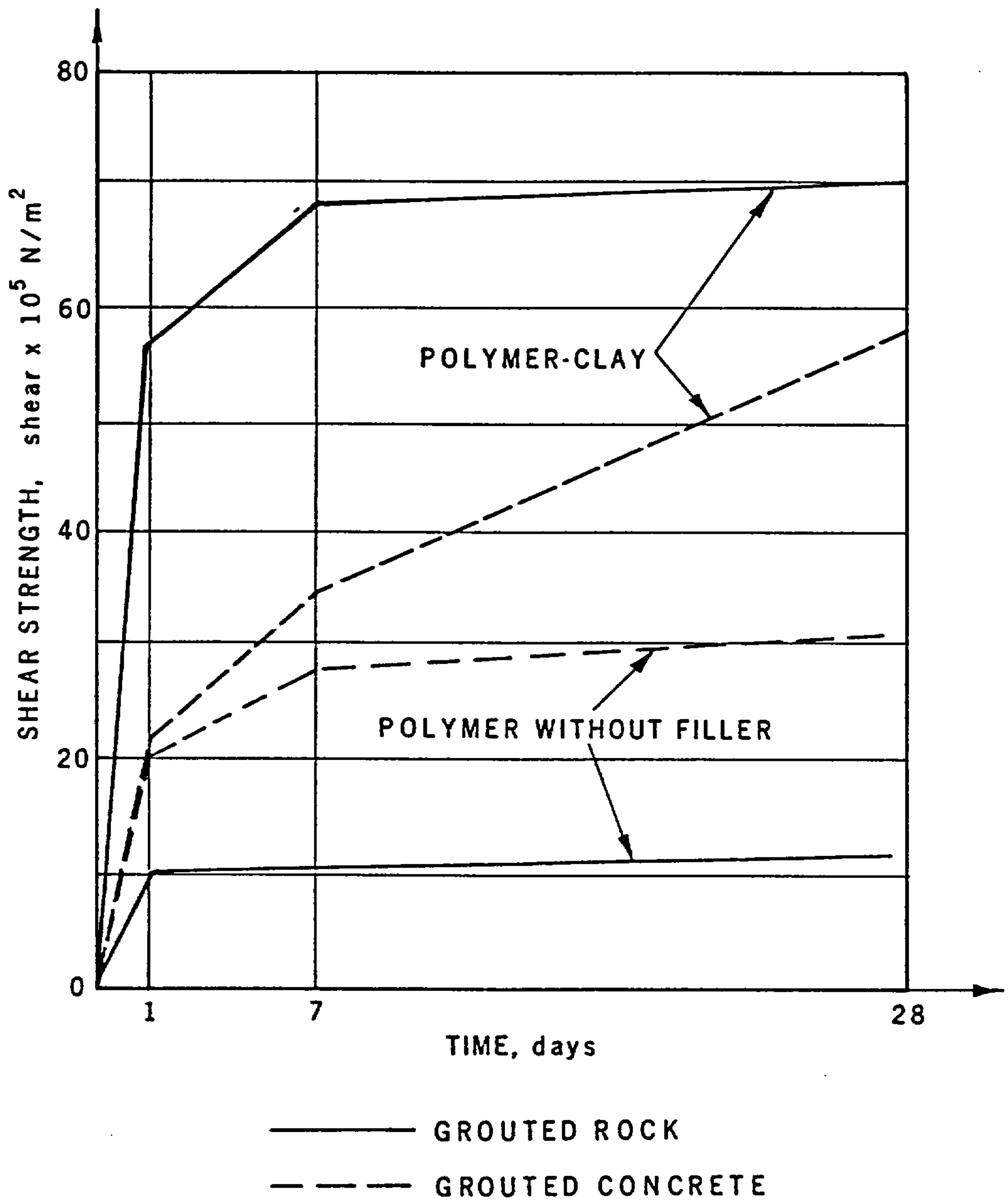


Figure 5. Strength Enhancement of Polymer Grouts with Fillers (after Demin and Popov, 1974)

little as 0.001 md (cured at 350° F) (Suman and Ellis, 1977). Powers et al. (1955) tested the hydraulic conductivity of cement pastes and extrapolated ultimate values to 0.6 x 10^{-10} cm/sec, observing a steady decrease with age. High w/c ratios also produced increases in permeability (Figure 6). Work at the Waterways Experiment Station has resulted in mixes proposed for repository sealing that have set permeabilities of 10^{-8} darcies (Wakeley, 1985).

Examination of the data suggest that the effects of shrinkage, thermal cracking, bleed, impurities, and interface effects that are difficult to control in the field, may not be operating in these tests. Daemen et al. (1985) in testing the permeability of various plugs (Type A Portland with expansive agents and dispersants at w/c of about 1) observed substantially higher permeabilities at the plug-borehole interface than through the plug itself. The plug hydraulic conductivities were on the order of 10^{-6} cm/sec and the interface conductivities varied. Both increased on heating (Table 7). Concrete cured at high temperatures (200 - 300 F°) may exhibit increased permeabilities. Retarded concrete is especially susceptible to this effect. Wakeley and Roy (1985), in work related particularly to salt repository sealing, found that cement grouts can be difficult to bond to anhydrite, due to the formation of calcium hydroxide. Wakeley (1985) demonstrated this in tests comparing grout plug effectiveness in siltstone and anhydrite. Powers et al. (1985) observed high permeability coefficients in specimens without visible channels and attributed this to bleed channels that were not completely filled with hydration products.

Although it is interesting that very low ideal grout permeabilities can be attained, it is more relevant to consider the permeability of the as-placed grout or concrete, the nature of the interface, and the groutability of the rock mass, when addressing water migration in shaft systems.

The American Concrete Institute (ACI, 1985) provides a comprehensive list of the sources of cracking in cementitious materials (concrete).

<u>Plastic shrinkage cracking</u> can occur due to moisture loss rates at the concrete surface in excess of bleed water replacement rates.

<u>Settlement cracking</u> occurs following placement and vibration, adjacent to local restraint, such as (in shafts) stiffener rings, grout ports, pour boundaries, or reinforcement.

<u>Drying shrinkage</u>, occurs in both concrete and neat cement grout and is caused by moisture loss following hydration. Excess bentonite can dehydrate with the same effect. Up to 1 percent strain can occur, and if it is non-uniform, substantial cracking can result. Powers et al. (1955) noted a 70-fold increase in permeability of specimens cured at 79 percent humidity rather than at 100 percent.

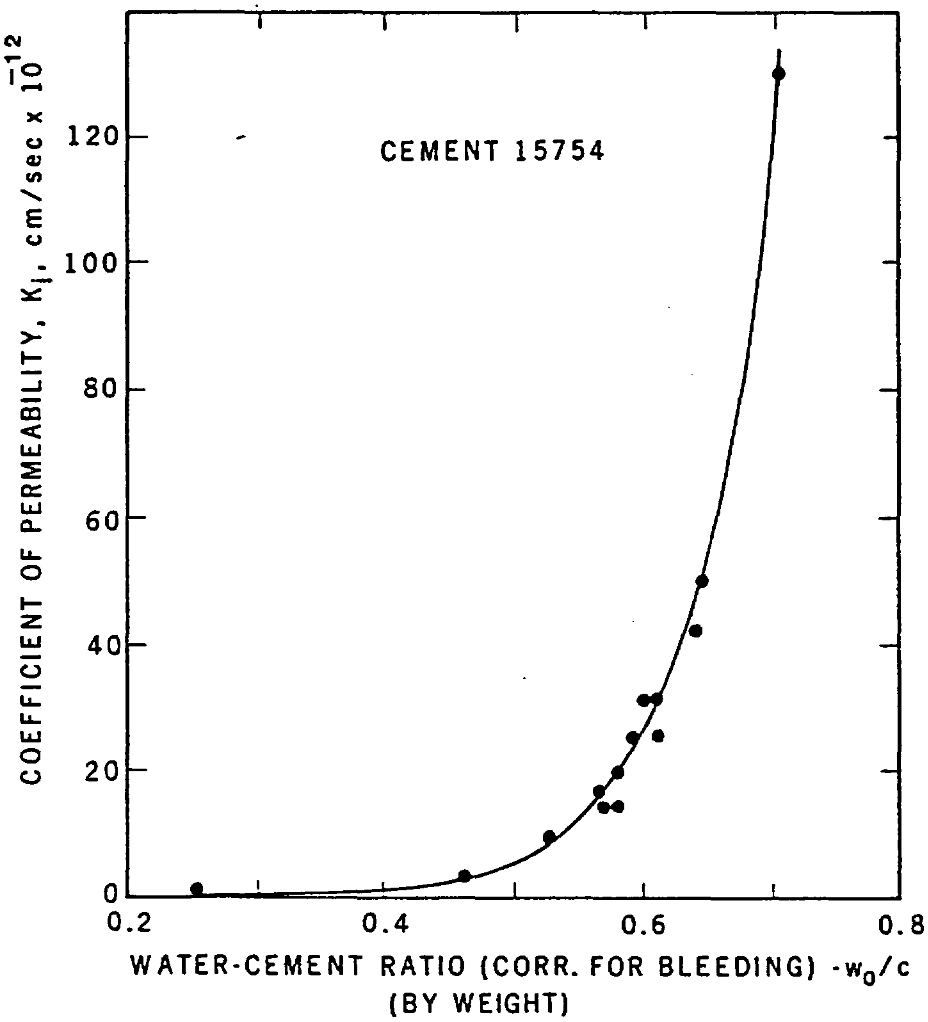


Figure 6. Effect of W/C Ratio on Permeability of Cement Paste (after Powers, et. al., 1955)

Table 7: Hydraulic conductivity of artificial borehole plugs at ambient and elevated temperatures (from Daemen et al., 1985).

Temperature of Cement °C (°F)	Injecting Pressure MPa (psi)	Hydraulic Conductivity cm/min	Instrinsic Permeability cm^2 (Darcy)
		SAMPLE SC-1	
22 ± 2 (71 ± 3.8)	0.52 (75)	3.889×10^{-6}	$34. \times 10^{-15}$ (4.29×10^{-6})
22 ± 2 (71 ± 3.8)	0.55 (80)	3.118×10^{-6}	508.8×10^{-15} (51.55×10^{-6})
22 ± 2 (71 ± 3.8)	0.57 (82)	2.322×10^{-6}	378.8×10^{-15} (38.38×10^{-6})
40 - 55 (104 - 131)	0.55 (80)	4.47×10^{-6}	420.9×10^{-15} (42.64×10^{-6})
		SAMPLE SC-2	
22 ± 2 (71 ± 3.8)	0.41 - 0.55 (60 - 80)	12.07×10^{-9}	1969×10^{-18} (199.5×10^{-9})
22 ± 2 (71 ± 3.8)	0.66 - 0.69 (95 - 100)	8.001×10^{-9}	1305×10^{-18} (132.3×10^{-9})
40 - 43 (104 - 109)	1.15 (167)	1.453×10^{-9}	157.4×10^{-18} (15.94×10^{-9})
40 - 43 (104 - 109)	3.35 (486)	1.634×10^{-9}	176.9×10^{-18} (17.93×10^{-9})
49 - 50 (120 - 122)	2.0 (290)	5.077×10^{-9}	477.7×10^{-18} (48.40×10^{-9})
49 - 55 (120 - 131)	0.75 (109)	3.75×10^{-9}	342.8×10^{-18} (34.73×10^{-9})
57 - 58.5 (135 - 137)	2.0 (290)	6.095×10^{-9}	507.5×10^{-18} (51.42×10^{-9})
70.9 - 71.9 (160 - 161)	2.0 (290)	11.47×10^{-9}	805.2×10^{-18} (81.58×10^{-9})
83 - 84.5 (181 - 184)	2.0 (290)	34.41×10^{-9}	2042×10^{-18} (206.9×10^{-9})

the shaft bottom) nearby that must be protected from heave, descending stages are used. Grouting in ascending stages requires the use of packers down the hole, which is difficult to effectively do in weak ground. It does, however, remove the necessity for repeated drill setups on the hole.

Descending-stage grouting, in essence, involves grouting in a surface pipe with a nipple and a shut-in valve. The length of the pipe depends on the need for near-surface grouting and the likelihood of grout communication to the surface. Grout is then injected to refusal or some other shutoff criterion, such as total volume, and the hole is shut in to allow the grout to set. In some cases, the stage is regrouted with progressively thicker mixes, the theory being that the thinner, early injections traveled the furthest from the hole and may not have sealed larger features. Normally, the sequence is completed in each hole, and then in the manner designed for all holes in the pattern, prior to deepening the holes. After the designed depth of grouting is reached, it may be desired to drill and grout a secondary or even a tertiary pattern, each of which normally splits the spacing of the preceding holes.

Hole spacings, inclinations, stage depths and procedures are entirely site-specific and, although designed and planned beforehand, are commonly changed as the grout acceptance character of the grout is learned.

As cracks near filling, the thickening and thixotropy or the grout results in a fall-off of pressure at the grout front, although the pump pressure may not change. Wide cracks may carry fluid grout past the thickening grout in the finer ones, and can burst through the thickened front, causing a drop in pressure and an abrupt increase in take. When the grout has flowed as far as it can in the regional cracks, it starts to fill bleedwater channels. Refusal occurs after the bleedwater channel filling has run its course. This is not to say that bleedwater channels do not remain after the conclusion of grouting. Often, flat-dipping cracks do not offer a path for the release of bleedwater and the channels can remain after grouting is completed. Bleedwater may be trapped beyond the grout terminus in cracks, although field experience indicates that most of it escapes. Trapped bleedwater may indicate the need for regrouting.

"Displacement grouting" is done with the intent of forcing cracks open to accept more grout which is typically thin, and thereby to enhance penetration. The method initially dilates the rock structure and relies on relaxation to expel bleedwater. "Compaction grouting" is a technique for densifying and dewatering soils by injecting thick grout under high pressure, which squeezes the medium. It is usually used as a settlement pallative or dewatering aid. "Penetration grouting" uses moderate-thickness grouts and does not rely on hydraulic dilation of cracks and subsequent relaxation to enhance rock mass densification, as does displacement grouting. Pressures are therefore always less than lithostatic and are usually just in excess of hydrostatic.

The specifics of shaft wall grouting and pregrouting are discussed in Section 3.5.

3.0 EXPERIENCE IN SHAFT LINING AND GROUTING

This section is a review of shaft lining and grouting practices as they have developed from actual projects. The discussions are supported where appropriate with representative case histories.

The scope of this section is limited. It is not an in-depth treatment of the subject of shaft sinking or liner design. Rather, these elements are briefly described and illustrated to facilitate understanding of how voids that could require grouting can occur. The shaft sinking and lining methods described herein are typical but not necessarily complete. They probably do, however, encompass the range of shaft sinking practices that would be appropriate for a repository.

This discussion treats sinking (excavation and temporary support) and lining separately, although they may be considered simultaneous operations for some shaft construction methods. This distinction is to enable a separate focus to be placed on lining methods and void prevention technology later. Such a distinction is possible because most of the lining methods are compatible with a of excavation methods, ground conditions permitting. Prefabricated linings have been floated into place in drill-and-blast shafts. Blind-drilled shafts have been slipformed. From the broad point of view of the occurrence of voids in shaft linings, there are relatively few areas where the excavation method is of much consequence.

3.1 SITE EVALUATION

Effective lining and void prevention begins at the site exploration stage. It is important to determine the hydrogeologic and engineering geology of the strata to be intercepted by the shafts, to provide a sound basis for selecting the excavation method and for designing the lining. It generally is far more difficult to stop shaft lining seepage after it has initiated than to prevent its occurrence in the first place. As mentioned earlier, repository shaft construction will emphasize seepage prevention.

Hydrogeologic evaluation for shaft sinking is keyed to predict where inflows may occur and what their magnitudes might be, to support the design of preventive programs, and estimate hydrostatic lining procedures and flow paths. The program should also provide complete background to support any remedial work or maintenance grouting that could be required in the future. It should include ground water sampling and gas pressure determinations, and hydrochemical assessments as well.

Hydrogeologically, a shaft is a large well; the lining represents the casing. In this case the well is to be kept dry, with the casing resisting some or all of the hydrostatic head. The subsurface field testing is therefore about the same as for any water well. Any appropriate program will involve pump tests, slug tests, or flowmeter inflow tests. Double-packer drill-stem tests can help pinpoint zones prone to inflow. Generally, the interval packed off varies from 50 - 100 ft, but can be outside this range depending on the expectation of aquifer characteristics. Drill-stem tests were run for 24 hours on two occasions prior to cementing the casing at Tatum Dome, and showed no inflow.

At the Selbý coalfield, pre-grouting hydrogeological tests relied upon a pressure/depletion recovery test (PRT), which is essentially a measurement of aquifer pressure recovery using gages and valved relief pipes following a free-draining period (Black et al., 1982; Tunnicliffe and Keeble, 1982). This method gave results comparable to those of drill-stem tests at the Riccall shafts (Fotheringham and Black, 1983).

Geophysical logging, in preparation for shaft grouting and sinking projects, includes density, resistivity, neutron porosity, caliper, temperature, three-dimensional velocity, and in some cases television. Television can, for example, verify that drill cuttings do not clog fractures and porosity in the grouthole walls. Some logs may be run at intervals to determine aquifer properties such as inflow.

Care should be taken when interpreting the subsurface data for planning shaft grouting and sinking. Often, the greatest permeability is associated with steep dipping fractures or cavities that are under-represented in vertically-drilled boreholes. Corex (1978) reports British pre-sinking investigations at Riccall where the measured permeabilities in the vertical direction (0.01 md to 15.4 md) were much lower than in the horizontal direction (0.01 md to 300 md, commonly in the 10-25 md range). Even a carefully-conducted series of pump tests in vertical boreholes would have incorrectly estimated the true water inflow to the shaft in this case.

Section 2.3 mentioned water acceptance tests (Lugeon tests) that are normally run prior to grouting in a borehole. Such tests are similar to slug tests, and the latter are useful indicators of grout acceptance for pregrouting projects from the surface or the shaft bottom. The typical Lugeon test can underestimate the permeability since turbulence can be introduced that does not operate in the same way during grout injection.

Hydrochemical investigations must address grout setting conditions (chemical, hydrological, temperature, and physical). On-site water may be used in grout or shaft concrete. Dissolved species can cause penetration to be limited by inducing an early set, can adversely affect the longevity of the grout, influences the selection of both grout and concrete (the salt mines in the Cleveland area are developed beneath the Oriskany sandstone, which carries strongly acidic water), and can retard the set or prevent it entirely.

Earlier sections have described how several types of grouts and concrete perform poorly in hostile environments. Ground water samples are accordingly retrieved and analyzed for chemical (ionic) content, pH, specific gravity, resistivity, and suspended solids. The water analysis will also aid freeze wall planning.

Engineering geological investigations are aimed at the determination of the active earth pressures on the shaft lining, the construction behavior of the ground and the related stability and safety concerns, occurrence and characteristics of fractures and porosity, presence of swelling materials, drilling resistance and muck handling problems, and where applicable, freeze-thaw behavior. Field logging of the drill core or cuttings includes lithology (texture, weathering, rock type, grain size, fabric, irregularities), core condition (recovery, fracturing, RQD, evidence of slaking or erosion) and rock structure (attitude of beds, spacing and orientation of discontinuities, fracture condition, fillings). The geologist should also monitor the drilling operation itself and record advance rates, fluid loss zones, bit type, hole diameter, mud type, depth and length of core run, water levels before and after core runs, and delays.

Samples taken and returned to the laboratory are generally tested for strength, porosity, specific gravity, permeability, grain size distribution (soils and poorly consolidated rock), clay mineralogy, chemical content, and possibly Atterburg limits or slake durability on shales, silts, or clays. Special tests are done in some cases, such as the creep characteristics of salt, grouted specimens, or frozen specimens. Great care should be used in extrapolating data from laboratory tests of frozen ground to the in-place condition.

Specialized tests that are done in some cases include hydrofracturing to determine stress distribution and dye or tracer tests to determine sources, rates, and directions of inflows. The rate of ground water movement is important in assessing the setting environment of grouts, particularly the dilute chemical grouts, and in assessing the amenability of shaft freezing. Historical studies of shaft excavation and drilling in the area or similar areas are also important in planning drilled shafts.

Near-surface investigations are done for placement of the shaft collar, headframes, freeze plant if required, and other facilities. Generally, the design of the shaft lining in the overburden sections is different from below, and may consist of corrugated metal pipe, liner plate, or concrete. The near surface investigation is typical of shallow construction projects. Geophysical investigations may be used to determine the depths to bedrock in the area, and to identify bedrock lows where water could accumulate. Drilling may be done with a hollow-stem auger. Split-spoon samples are taken and tested in a soils laboratory. Penetration rates (blow counts), field description of the soil, weathering interface, water levels, and so on, are noted.

3.2 SITE PREPARATION

Site preparation involves the initial ground improvement to enable execution of the chosen shaft sinking and lining methods. The degree of ground preparation required depends on the initial severity of the conditions the sensitivity of the excavation method to the subsurface conditions, and the performance expected of the finished product. The two objectives are generally: reduce deleterious ground movements, and control or eliminate water inflows to the excavation. Water occurrence may be responsible for adverse ground movements, and vice versa. Related concerns, such as design for rock loading, resistance of settlement or creep, control of aquifer pollution, and prevention of dissolution, are encompassed by these two objectives.

To fulfill these, there are essentially three options: pregrouting for consolidation and water blockage, ground stabilization and inflow prevention by freezing, and reduction of hydrostatic head and water inflow by depressurizing or dewatering wells. Freezing is generally the preferred method in fine-grained, waterbearing rock or soil. Grouting or depressurizing/dewatering wells are preferred in coarser grained or fractured rock and soil. Successful freezing will require suitable water temperature, chemistry, and rate of movement. The benefits of pregrouting are lasting, although not necessarily permanent. Freezing and depressurizing/dewatering are essentially used for constructability, and are not long-lasting. However, there is some ground that simply does not respond well to grouting. In the potash districts in Saskatchewan, for example, freezing was employed on almost all the presently-used shafts through the Blairmore aquifer, after some thoroughly unsuccessful attempts to grout. Deeper rock aquifers in the area have in most cases responded favorably to grouting.

Shaft pregrouting can be done prior to sinking, from the surface, or from the shaft bottom during sinking. The two methods can and have been combined on some projects. Grouting from the surface can involve some very deep holes and long stages, which presents grout penetrability, setting, and drillhole control problems. It is expensive, requiring long drilling footages and presumes that the behavior of problem aquifers can be adequately characterized and controlled from subsurface boreholes, normally drilled vertically. However, pregrouting from the surface takes the ground improvement activity out of the shaft sinking cycle, which offsets the high initial cost. In-shaft grouting involves a cessation of sinking, but the greater proximity to the horizon to be grouted offers better understanding of its characteristics, better control of the grout injection and travel, shorter drillholes, and the potential for a superior product by enabling some flexibility in grouthole orientation for optimal coverage of fracturing.

The following discusses these methods in somewhat more detail and gives examples of their use.

3.2.1 Pregrouting

3.2.1.1 Surface-Based Pregrouting. Shaft pregrouting from the surface is normally justified only in the expectation that sufficient problems will be encountered during sinking, and that the outcome will be effective enough, to warrant the commitment of up-front time and funds to undertake the program. Surface-based pregrouting has been used to improve expected lost-circulation zones for drilled shafts, as well as to facilitate water control for drill-and-blast shafts with directly-placed linings.

Surface-based pregrouting is analagous to any other downhole foundation grouting program, with the major exception being the great depths and large stage intervals that may be used. The grout holes are drilled as deep as required, and are surveyed and directionally controlled. In South Africa, stages have been as long as 1,000 ft (Dietz, 1982). Newman (1958) describes the Harmony Gold Mine shaft pregrouting project in South Africa that involved 3 holes to 2,000 ft.

The grout selection factors and the procedures are site specific. The most common practice in mines for surface-based pregrouting is to use a cement-based grout mix with set times and for the chemical, temperature, and pressure conditions at the horizon to be grouted. Consideration is also given to conditions in the grout pipe itself; the grout slurry weight may need to be controlled to reduce bottom-hole pressures that could adversely affect grout set or hydrofracture weaker horizons.

Pregrouting from the surface is not always effective, and it can be difficult to assure the effectiveness before starting to sink the shaft. The grout pattern can miss steeply-dipping fractures or singular fissures that can govern the inflow behavior to the much larger shaft. Nel (1981) describes an instance where, despite a pregrouting program, a later 118-ft (36-m) grout cover hole hit a water pocket under pressure (600 psi) that flooded the Elandsrand (South Africa) shaft.

The IMC No. 5 shaft in New Mexico's Carlsbad potash district was extensively pregrouted (Cementation Corporation of America, unpublished case history). The ground contained vuggy, brecciated dolomite aquifers in the interval 0 - 100 ft. The potential inflow was 5,000 gpm, as derived from pump tests. Grouting was done 5 ft outside the neat line through 6 in. standpipes, with (initially) Type

V cement, at initial w/c ratios of 6:1, decreasing to 1:1. There were 24 holes -- 6 primary, 6 secondary, and 12 tertiary. Stages were 45, 75, 90 and 100 ft. After injecting 280,000 lb of cement, the inflow had been reduced only to 1,200 gpm (projected), so the ground was re-grouted with chemical grout (Geoseal). Fresh-water preflushes were used to prevent the saline ground water from retarding the setting times. An additional 24 holes were drilled inside the cement ring so as to split the cement grout hole spacings, and, in all, 60,000 gal of Geoseal mix were injected at a completion pressure of 1 psi/ft. This reduced the inflow as measured from pump tests, to 35 gpm, and sinking was begun. Deepening of the shaft required the injection of almost 1.9 million more pounds of cement and fly ash into the 100 - 290 ft (to top of salt) interval. Upon excavation, the residual water make from the whole shaft was found to be about 75 gpm.

3.2.1.2 <u>In-Shaft Pregrouting</u>. In-shaft pregrouting is much more common in the United States. The adverse impact of the delay in sinking can be minimized, if the characteristics of the expected inflow horizon are well-known during shaft planning. Often, however, the pregrouting occurs in response to an unexpectedly severe condition that surfaces during the sinking. It is common practice to carry at least one probe hole well in advance of the shaft bottom as a precaution, and some projects have used as many as 4 holes, one in each quadrant. The suitability of multiple boreholes is illustrated by the Wills-Weaver No. 3 shaft in the Carlsbad potash district, where one episode of pregrouting of the Culebra dolomite involved 25 first-stage holes, 15 of which were dry. Of the remainder, one produced 132 gpm.

The expectation of in-shaft, or cover, grouting, is not to establish complete sealing but only to afford the necessary degree of inflow control to enable the planned sinking and lining operations to be carried out. A typical cover grouting plan is shown in Figure 7.

The procedure is to stop the shaft excavation at a suitable distance (30 - 50 ft is common) above the aquifer to be grouted. If water inflow is already occurring or an insufficient thickness of rock exists to accommodate the grouting pressures, it may be necessary to install a concrete pad and grout it into place, to seal leaks. The pad may also be necessary to adequately set the standpipes. If the lining is in place, it may be carried down and keyed to the pad, and sealed with grout. The grout holes are angled outward slightly and "spun", that is, drilled at angles that in the horizontal plane are not perpendicular to the shaft excavation surface. This enables coverage of a greater percentage of fracture system orientations all around the shaft perimeter. The outward-looking orientation is necessary because there is no way other than excavating a shaft grouting station (which has often been done) to inject beyond the shaft perimeter with vertical holes, from the shaft

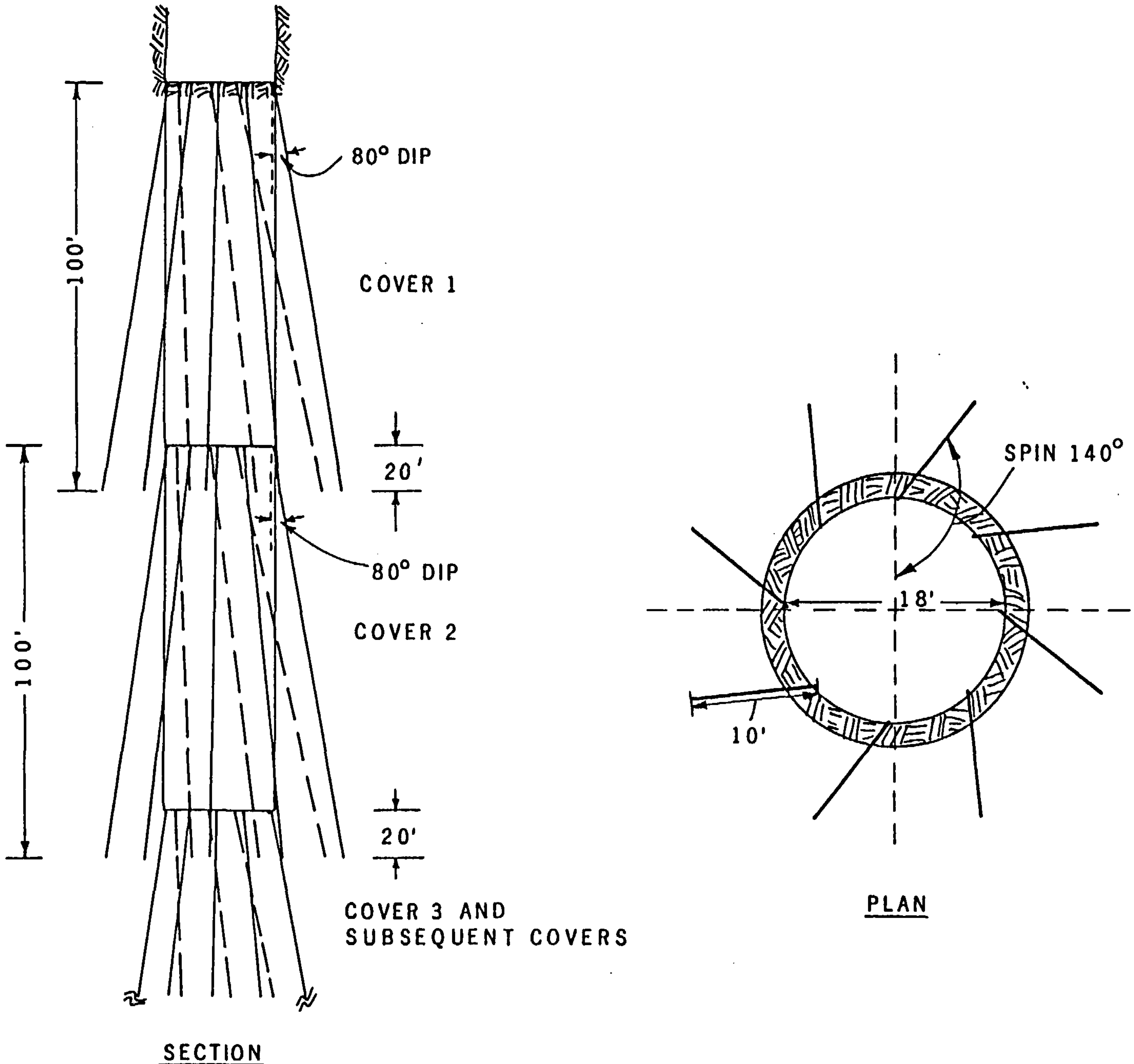


Figure 7. Typical Pre-Grouting Treatment from Shaft Bottom. Note, the overlap of the covers and the angle, and the coverage due to an 8-hole pattern (after York, 1964)

bottom. Thus, the holes diverge with depth and it is necessary to overlap the covers as the shaft proceeds downwards to keep to the optimal spacing and minimize the quantity of drilling done.

The layout of the covers is entirely site-specific and depends on the ground characteristics, the penetrability of the grout, the thickness and pressure of the aquifer, the expected grout hole deviation, and so on. Spin angles are normally 10 - 25 degrees from perpendicularity with the shaft walls and are limited by the necessary clearance for the drilling equipment. Holes are typically 100 ft long (York, 1964) but again will vary according to conditions. Scott (1963) describes a grouting program in a Canadian potash mine that uses 240-ft long holes. The vertical angles vary from 80 to 65 degrees from the horizontal, but could be less if a very thick grout wall must be formed.

Grout holes are typically 3 1/2 in. in diameter. Standoffs are seamless pipe grouted into place with quick-set cement and pressure-tested before deepening to the horizon to be grouted. The pressure-testing also evaluates the concrete-rock seal. Grouting pressures may be as high as twice hydrostatic but are limited by uplift of the grout pad and the pressure limitations on the lining, if one is present. The grouting is normally done through descending stages, and all holes are completed in a given stage before deepening the holes. Stages can be 10 ft to 100 ft but are commonly in the 20 - 40 ft range, depending on penetrability. Scott (1963) using sodium silicate pre-injection achieved over 500 ft of penetration in vertical fracture.

Greenslade et al. (1981) describe a planned in-shaft pregrouting program at Nose Rock, New Mexico, that used a cement grout at w/c of 12:1 varying down to 1:1. A grouted annulus was established, with a thickness of near 20 ft. The procedure was successful in sealing about 75 - 90 percent of the inflows.

Nash (1984) describes several instances of in-shaft pregrouting. In one case, the shaft was only 50 ft deep when inflows were encountered up to 100 gpm. A nearby shaft developed 10 gpm from the same horizon. The aquifers had not been anticipated due to the placement of surface casing in the exploratory boreholes to 60 ft depth, which was the starting depth of the geophysical logging. Since the shaft was for a trona mine, it was critical to arrest this water, and grout covers were placed in a manner similar to the one described above. One difference was that a gravel pad was placed beneath the concrete, and was keyed outwards into the shaft perimeters. Both concrete and gravel pad were 4 ft thick and the gravel was grouted after the concrete had set. Final water control was achieved after injecting a polyphenolic resin grout into the surrounding strata.

Taylor (1932) reports that 132 grout holes were used to treat a 1 ft-thick sand layer above the caprock at the Grand Saline salt mine. The grouting was done from the shaft bottom. When the layer was mined through, it was found to have been completely replaced by grout, but still leaked.

The Wills-Weaver shaft mentioned previously was pregrouted for its construction to control 50 - 118 gpm inflows from the Culebra dolomite aquifer. There were 4 phases of pregrouting but complete sealing from pregrouting was not achieved, despite the use of thin silicate chemical grout on the last stage (ONWI255). The liner was eventually backwall grouted 2 ft into the rock, which temporarily arrested the leak. An inspection years later (1979) revealed that the inflows had increased to 2 - 3 pgm. This illustrates the expectation that pregrouting, even with great care, is not likely to completely eliminate inflows during construction.

Nash (1984) reports reducing a flow of 275 gpm to 60 gpm after an in-shaft program that injected 81,900 cu ft of chemical grout and 1,259 cu ft of cement grout into the bottom of a coal mine shaft under construction. A companion shaft with a potential inflow of 375 gpm (owing to a 65 percent larger cross-sectional area) was reduced to 65 gpm using a polyphenolic resin and no basal gravel pack.

At Selby, 30 grout covers were used to control water issuance from the Bunter Sandstone, using both chemical and cement grouts. The Basal Sand was also grouted, even though its water make was only 30 - 40 gpm because of the chance for erosion.

3.2.2 Ground Freezing

Ground freezing was developed in 1883 to sink shallow shafts through waterbearing ground in Germany. The first U.S. record of the method is in 1888 at the Iron Mountain, Michigan, mine. The method is well-established and the technology is well-developed. It is generally preferred over drilling for large-diameter shafts in very soft, very hard, or highly variable ground. Nevertheless, the freeze-thaw behavior and some other aspects of the method pose concerns for repository shaft grouting and lining sealing.

Basically, freezing for shaft excavation involves drilling a ring of freeze holes to the total depth required, in a very controlled manner, some distance outside the excavation neat line. The holes are normally 6 - 8 in. in diameter, as for example in the Kellingly shaft described by Firth and Gill (1963), and are cased. The ground is frozen by circulating chilled brine (lithium or calcium chloride are the most common) through freeze pipes set within the drillholes. Site preparation can take up to 4 months and waiting time to complete freezing can require similar or longer periods, depending on conditions. A pilot hole is drilled down the center of the shaft to allow relief of the water forced out of the formation by the freezing action. Additional observation and temperature monitoring holes are provided to ensure that the freeze wall is complete and stays that way throughout the excavation process, and until the lining is in place and grouted.

Excavation takes place within the frozen cylinder. Excavation of frozen ground is almost always more difficult than in unfrozen ground. Excavation can be by drill-blast, impact breakers, or by hand. Care must be taken not to rupture the freeze pipes or the freeze wall through ground movement, drilling, or by blasting vibrations. Hegemann and Jessburger (1985) describe a freeze pipe rupture that occurred in the Bunter sandstone at the Voerde shaft; sinking continued as the sand ruptured, but did not fail. Dry-percussive drilling can pose major problems in some rock due to cuttings removal problems. Wet-percussive is prone to freeze-ups. At Selby, it was necessary to design and fabricate special steels and bits to drill out the rounds in some strata, and hole closure was rapid and a consistent problem (Tunnicliffe and Keeble, 1981). Certain explosives do not function well in frozen ground.

The refrigeration plants for freezing projects are large and require a large output to withdraw the heat to establish the initial freeze. For perspective, the Voerde shaft freezing project, which at the time of its completion in 1980 was frozen twice as deep as the deepest prior freezing project in Germany, required a refrigeration plant with a capacity equivalent to 10,000 householed refrigerators.

Freezing can be successfully done in ground with saturation as low as 10 percent (Schuster, 1984). Unsaturated soils can slough. Depending on water salinity, flow velocities higher than about 5 ft per day can be very difficult to freeze, although the use of lithium chloride brines and modern, powerful refrigeration plants mean that this is not a strict limit.

Ground freezing has been used extensively in the Saskatchewan potash district, to over 2,000 ft depth (Ostrowski, 1967). One early instance, IMC's Yarbo No. 1 shaft, was frozen from the surface through glacial till, and the Blairmore was frozen from an underground freezing station. The underground station was necessary because 29,950 gal of AM-9 and 12,000 bags of cement expended in a pregrouting program were ineffective at controlling the Blairmore.

The layouts of freeze shafts varies according to the ground water characteristics and the loads (hydrostatic and earth pressures) to be resisted by the freeze wall. The Voerde project required 38 pipes on a 5-ft (1.55-m) spacing (Hegemann and Jessberger, 1985), in addition to 4 temperature monitoring holes, 2 observation holes, and the center pilot hole. The Haltern 1 and 2 shafts (Stoss and Braun, 1983) used a freeze pipe spacing of 4-ft (1.2 m), with allowable hole deviation described by the maximum hole spacing of 5.25 - 6.25 ft (1.6-1.9 m) (depending on depth). The freeze holes are of course concentric to the excavation.

Freeze wall design for many years used the Dopke formula but in recent years, more refined and less conservative methods have been developed. Klein (1980), Hegemann (1981), Wild and Forest (1981), Stoss and Braun (1983), and others, describe the strength assessment and design of freeze walls and freezing plants, the details of which are beyond the scope of this report.

The use of laboratory freezing behavior in freeze wall design must the viewed with extreme caution, since the processes involved are quite different than those in-situ. Hegemann and Jessberger (1985) describe one method for controlling, interpretting, and using these data.

Because hole spacing is important to the establishment of a sufficient icewall thickness, the control of freeze hole deviation is critical, and every shaft freezing project carries a specification for permissible hole deviation. At Voerde, the deviation tolerance was 0.25 percent of the depth, and the alignment was checked every 50 ft with a single-shot survey, during drilling. The specifications at Haltern, in addition to the diametral tolerance mentioned above, included a radial tolerance to be within the 1.6 ft (0.5 m) annulus between circles 46 and 49 ft (14 and 15 m) in diameter as referenced to the shaft centerline. The Prosper No. 10 shaft deviations were allowed to be within 0.5 percent of depth and the surveys confirmed that the furthest out was 0.43 percent (Hausler, 1972). Wild and Forest (1981) report that the use of careful, controlled drilling (using stabilizers and directional drilling control) resulted in an average error of 1.5 ft (0.45 m) at 984 ft (300 m) depth for the freeze holes at Selby.

The major concerns for voids in shaft linings connected with freezing arise from the ground behavior during freeze-thaw cycling and the effect of the low wall temperature on the lining materials. The heave of the ground associated with freezing (Figure 8) evidences ice expansion and soil movement that produces a migration of water towards the relief hole at the shaft centerline and can overconsolidate clayey and silty members of the rock column (Chamberlain, 1981). The effect is less pronounced in sands, especially when undersaturated (Altounyan et al., 1983). The freeze wall temperatures at depth are usually lower than they are near the surface and it may be necessary to insulate the shaft walls to prevent local thawing. After lining, the upper parts of the shaft walls may not refreeze.

Shaft wall temperatures are typically 32°F to 14°F (0° to -10°C and decrease with distance (Figure 9) towards the freeze pipes where the temperature may be 0°F (-18°C) with calcium chloride brine

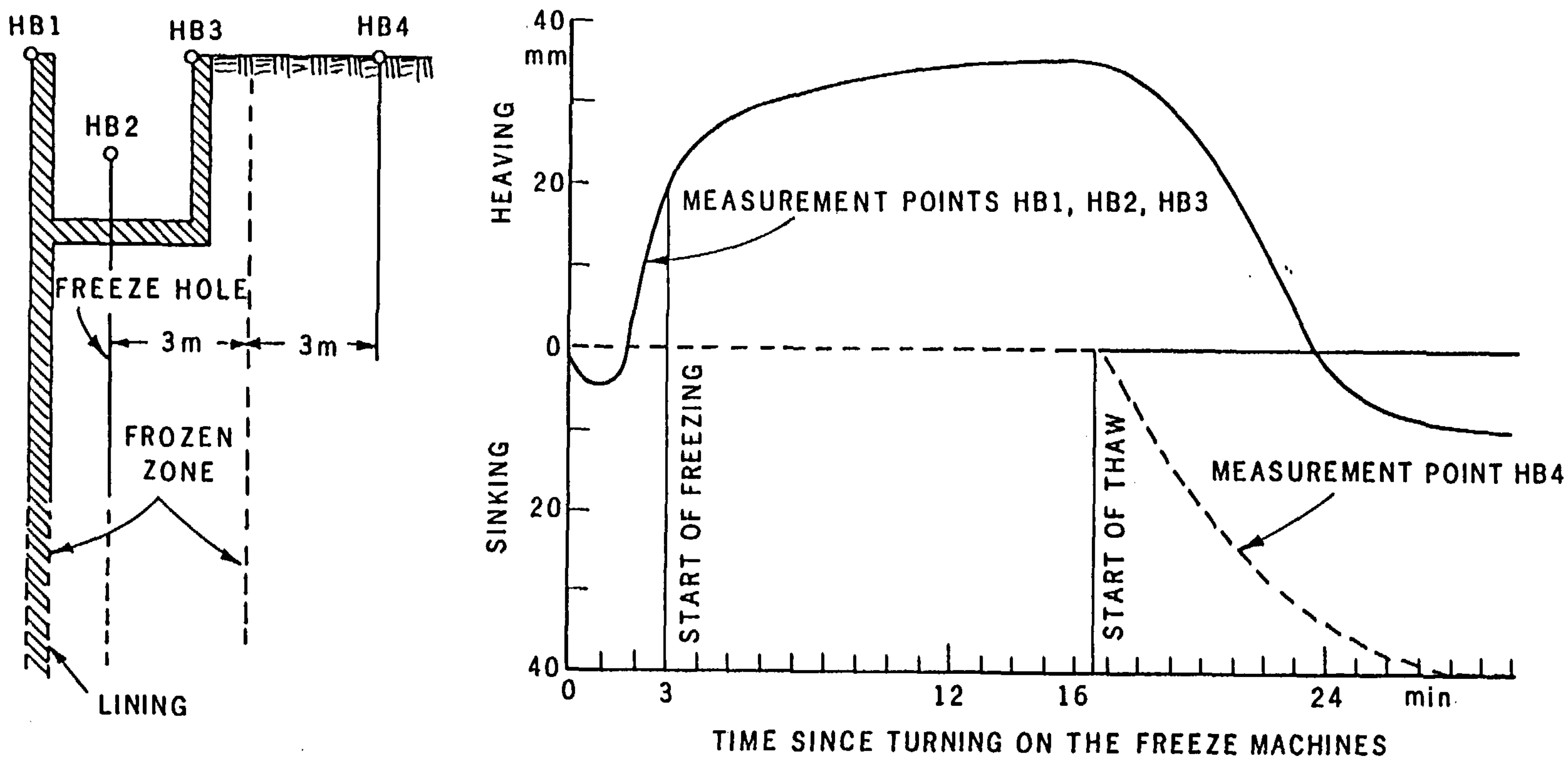


Figure 8. Surface Effect of Shaft Freezing at Wulfen 1 and 2 (after Kampschulte, Lehmann, and Link, 1964)

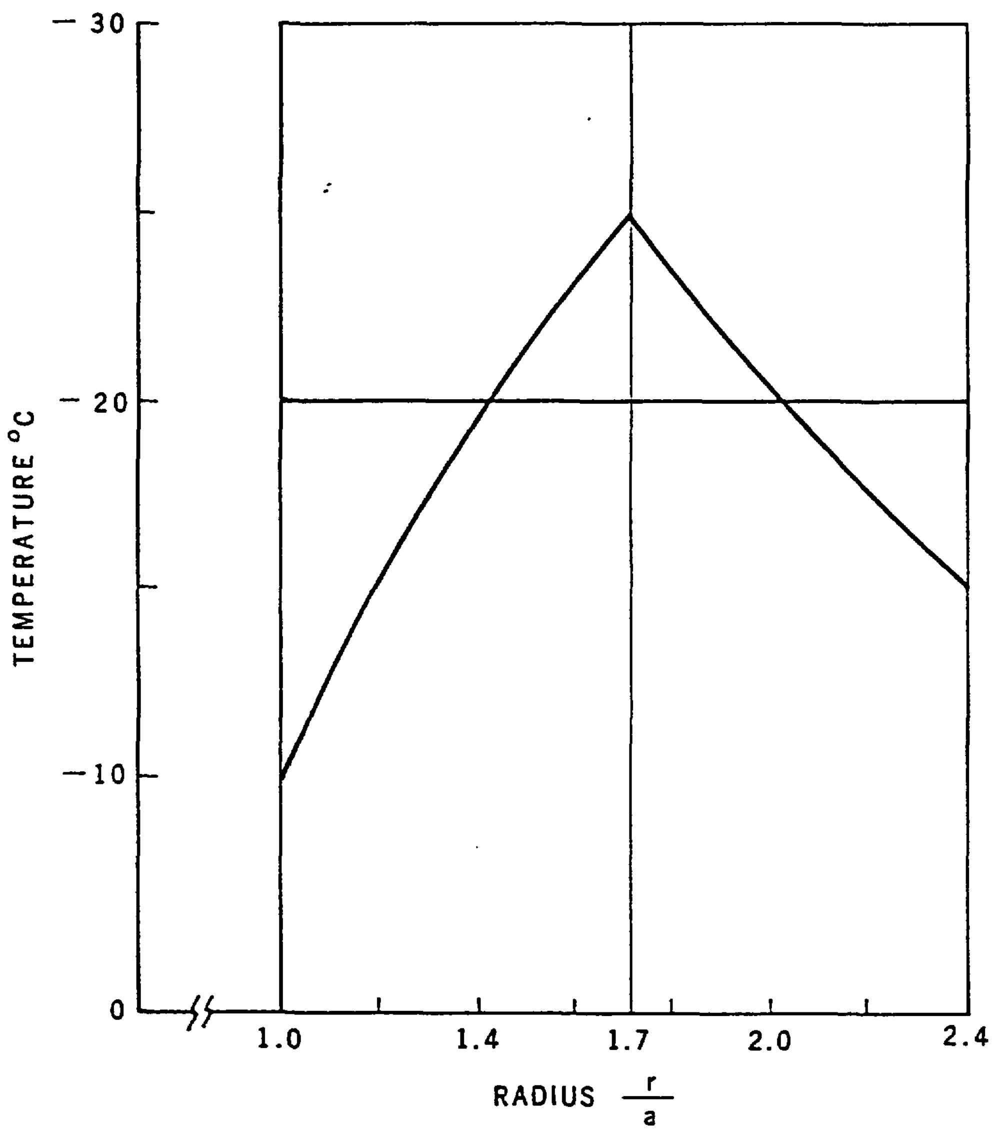


Figure 9. Conceptual Temperature Distribution
in Frozen Shaft Walls for Design
r = radial distance from shaft centerline
a = excavated radius
(after Klein, 1980)

(Hegemann and Jessberger, 1985). The greatest amount of heat is withdrawn from the region outside the freeze circle, and the frozen soil volumes inside and outside the freeze circle are about equal.

Placement of the temporary or primary lining against the frozen ground must be carefully done. If the concrete is too warm, a temperature differential develops that in effect acts like thermal cracking. Altounyan et al. (1983) report a temperature distribution that rose rapidly to 154°F (68°C) at the center of the lining. If the concrete is too cold, it may freeze before it attains its full strength and will not strengthen to the design value even after thawing.

Frozen ground behaves like a viscoplastic material and will creep after excavation. The colder the ground, the stronger and less creep-prone it is. Temperatures of 104°F (40°C) are possible in calcium chloride brine but lithium chloride brines can be much colder. For typical shaft diameters of 14 to 22 ft, the creep may have a radial component of 6 - 10 in. The initial deformation is exponential so that the walls are normally left unlined for a period of time after initial exposure to allow most of the deformation to take place without loading the primary lining. However, the creep never entirely stops, and the freeze wall will continue to grow at an ever-decreasing rate, so that for long freezing periods, the time-dependant deformability of the frozen wall becomes important.

Placement of the concrete lining will result in heat transfer from the concrete to the frozen wall. A rule of thumb, which was borne out at the Wulfen shafts (Kampschulte, Lehmann and Link, 1964) is that the depth of thawing is nearly the thickness of the lining placed. Placement of inner liner components can cause additional freeze-thaw cycles. The pouring of the 176°F (80°C) asphalt interlayer at the Wulfen shafts caused a thaw to 2.6 ft (80 cm) into the frozen wall (Kampschulte, Lehmann and Link, 1964). It is doubtful in this case that the placement of the inner layer of concrete had much thawing effect since there was an air gap between it and the primary lining. Also, the primary lining will have some insulating effect; nonetheless, the potential exists for multiple freeze-thaw cycles during composite lining placement.

Section 4.5.3 discusses in greater detail the concerns for void occurrence arising from thawing of ground.

Freezing projects are extensively instrumented to detect creep deformations, brine temperature changes, pressure buildups behind linings, water levels outside the freeze wall and within the ring (during freezing), and temperature profiles of both the frozen earth walls and the linings. In some cases ultrasonic profiling has been done to verify the dimensions of the freeze wall prior to sinking. The thermal response of the instruments, particularly the function of pressure cells at low temperatures, should be verified before drawing firm conclusions. Samples of the in-situ frozen material may also be taken and tested to verify that the design strength assumptions are being realized.

3.2.3 Depressurizing and Dewatering Wells

As a temporary measure of dealing with water pressure, and to a lesser extent, volume, aquifers may be drawn down and kept that way for the duration of the sinking and lining. This was used at the Selby Riccall shafts (Black and Auld, 1985) and at Nose Rock (Greenslade and Condrat, 1979). High water pressure is deleterious to stability and to grouting effectiveness. High water volumes may be impractical to reduce.

The dewatering procedure is described in the literature (Fortheringham and Black, 1983, and hydrogeology textbooks). At Riccall, dewatering holes were used to depressure the aquifer to aid in backwall grouting, using holes spudded near the shaft collar and angled outward. The initial yields from the holes in the Basal Sands for Riccall No. 1 were 420 gpm (26.6 ℓ/sec) initially and stabilized at about 111 gpm (7 ℓ/sec) after several days of pumping; only minor water was encountered on sinking. The yield for Riccall No. 2 was less (13 gpm decreasing to 5 gpm; 3.0 ℓ/sec to 0.31 ℓ/sec).

At Nose Rock, the objective was again to reduce aquifer pressures for grouting. Well yields were in the neighborhood of 1,000 - 2,000 gpm from three formations (Gallup, Dakota, and Wastewater) and residual pressures were on the order of 8 - 12 percent of initial pressures (Greenslade et al., 1981).

3.3 SHAFT EXCAVATION

This subsection provides an overview of shaft excavation methods. As indicated at the outset of this chapter, shaft excavation and lining are treated separately.

This discussion is limited to brief descriptions of shaft excavation methods. Discussions of the ground mechanical behavior, rock/support interaction, blasting technology, equipment, and so on, are limited to those aspects that are directly relevant to voids in linings or the efficiency of grouting.

The discussion is also necessarily generalized. It can be truthfully said that there are nearly as many shaft excavation methods as there are shafts, if all the detailed combinations of the techniques are considered. Descriptions of shaft sinking methods are always grouped according to the purpose of the description; in this case, the groupings are along the lines of likelihood of rock mass disturbance as related to excavation and temporary (construction) support Gonano et al. (1982, NUREG/CR-2854) discuss shaft sinking methods with a somewhat different focus and in slightly more detail. Other discussion may be found in the Mining Engineer's Handbook (Society of Mining Engineers, 1973) or in the extensive technical literature on the subject.

This discussion identifies three basic methods as being most relevant for repository shafts: blind drilling with top drive, pilot-and-slash with bottom access, and conventional full-face drill-and-blast. There are, of course, others, and these will be given brief mention in Section 3.3.4.

Gonano et al. (1982) analyzed these and other sinking options for repository shafts, from the stand points of construction time, reliability (predictability), sealability and rock mass damage, opportunity for rock inspection and testing, safety, alignment, and cost. Despite higher cost and longer construction time, the ratings suggest that drill-and-blast is slightly more desirable than blind drilling for both salt and hardrock sites for the diameters needed. It should be pointed out that sealability/damage, which for most cases may superficially favor blind drilling, is largely a matter of workmanship in excavation, which is more readily controlled where there is direct access to the rock. As the case histories in this report attest, prevention of annular voids and rock mass damage in blind drilling is not a foregone conclusion.

If the ratings in Gonano et al. (1982) that are most relevant to void prevention in the shaft/liner system (damage/sealing with respect to long-term performance, predictability of the construction outcome, and opportunity of direct inspection and testing) are summed separately, drill-and-blast still emerges as the preferred method.

3.3.1 Blind Drilling

Blind drilling as described in this subsection refers to top-drive equipment. Down-hole full-face blind drilling (Blind Shaft Borer (BSB) is not yet a state-of-the-art method for constructing repository shafts, and requires stable, dry conditions that are not realistic for most of the contemplated sites. Up-hole reaming or down-hole reaming (as with the V-moles) make similar demands of the geologic setting as the BSB although they have been proven successful in a number of cases. Were the geologic conditions suitable and bottom access available, these could offer an optimal combination of minimal rock mass damage and maximal construction control.

Top-drive blind-hole drilling evolved from oil field drilling technology in the 1960s and 1970s. Currently, efforts are underway to push the capability to 20-ft-diameter shafts with 3,000-ft depths in hard rock. At the Nevada Test Site (NTS), holes are routinely drilled 120 in. diameter to depths in excess of 3,000 ft and similarly-sized holes have been drilled considerably deeper. Drilled shafts larger than 12-ft-diameter at repository depths have been drilled in multiple passes with the exception of a 14-ft shaft in Australia.

The rigs in common usage differ little from large oil field rigs, except for modifications to the rotary tables, pipe make-up and handling equipment, and hook load limits. Hughes Tool Company has a redesigned rig (the CSD 300) intended for 20- to 40-ft diameter shafts that has been successfully used (after working out a few bugs in the system) on a large diameter shaft in Australia. Drill pipe is larger (13 3/8 in.) than most oilfield applications (the Hughes rig uses 20-in. pipe) and the bits are, of course, radically different.

Bits used in the United States for big-hole drilling are generally of the flat-bottomed type. This choice has evolved after trying most other configurations -- concave, hemispherical, and V-shaped. The tapered bits exhibit hole alignment problems and the concave bits have penetration and cuttings removal problems. Bits are constructed with a massive steel body and incorporate rolling elements that either chip or pulverize the rock under tremendous vertical stress. The 3 most common bit designs incorporate milltooth cutters for softer formations, carbide button cutters for hard formations, or disccutters for various formations. Disc cutters are more energy-efficient in that they are designed to lift out chips of rock rather than grind it, but they are more subject to longevity problems. Bits also contain reamer-rollers to maintain the gauge of the hole.

The rest of the down hole assembly (Figure 10) consists of the mandrel, donut weights, stabilizers, and a hold-down clamp. For added hole control, a stabilizer can be added at the top of the downhole assembly, Non-rotating stabilizers are more effective than rotating stabilizers (Carone and Whitley, 1981). The plumb-bob effect of the large weight down the hole tends to keep the hole vertical, although there is a tendency, as with all rotary drilling, for the hole to be slightly helical. The Basalt Waste Isolation Project (BWIP) Exploratory Shaft drilling concept (Morrison-Knudsen, 1984) specifies that no more than 60 percent of the drilling assembly weight be run on the bit, thus sparing the other 40 percent for drill string stabilization.

Hook load capacities of rigs can be in the neighborhood of 2 million pounds. Good drilling practice dictates that up to 25 percent of hook load capacity be reserved for wall drag, sloughage, and other problems when tripping out of the hole (Morrison-Knudsen, 1984).

Bighole drilling is almost always faster than conventional sinking for shafts of comparable diameter. For the standard 10-ft-diameter shaft, penetration rates can be as little as 0.5 ft/hr to 20 ft/hr with milltooth cutters in soft formations.

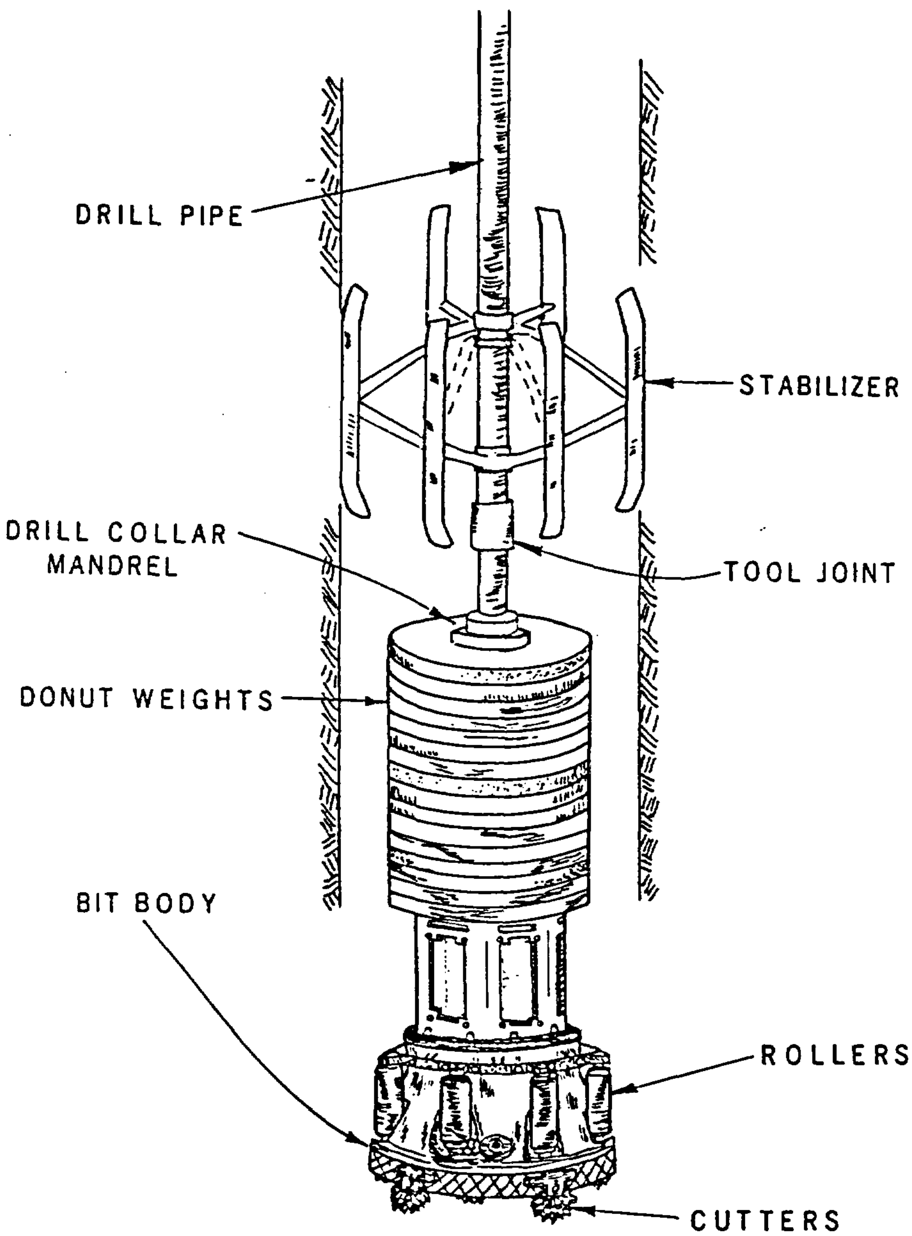


Figure 10. Down-Hole Assembly in Big-Hole Drilling

A major limiting factor of drilled-shaft diameters is the ability to develop sufficient torque. As the weight on the bit increases in order to sufficiently load each cutter at larger diameters, the net rolling resistance, and thus the torque, requirements increase. This in turn requires a larger rotary table and makes greater demands of the drill pipe, which slows pipe handling, make-up and tear-down time, and changes the hook load requirements.

Another major factor is effective hole cleaning. Large holes must be circulated in reverse: the flow is down the shaft and up the drill string. Circulation is developed through air assistance, that is, air incorporated into the fluid in the drill-string provides a density difference that draws fluid up and out of the hole. In single-string air assists (Schlage and Smith, 1981), Figure 11, the injection takes place near the top of the hole, and with dual-string (Fenix and Scission, 1983) it is injected close to the bit. The larger the diameter, the slower the fluid velocity at the perimeter of the hole. Insufficient cleaning results in extensive regrind and poor penetration as well as reduced cutter life and increased torque requirements. Increased bit weight will enhance penetration rate, but at the expense of hole deviation. Fluid velocities at the perimeter of a 120-in. hole at 3,400 gpm will be around 10 ft/min. The transport of sand in water typically requires 10 times that velocity, whereas gravel needs up to 50 times.

Sweep pick-up, cutter pumps, and jet-assist (Figure 12) are circulation aids that can help move cuttings (Schlage and Smith, 1981). Sweep pick-up incorporates baffles in the bit that mechanically shove cuttings and create turbulence that draws cuttings towards the higher velocity zone at the center of the bit. Jet assist (Carone and Whitley, 1981) involves jets of drilling medium directed at the cutters (Figure 13) that accomplishes the same purpose. At the Nevada Test Site (NTS), where the need to keep dry formations from becoming saturated results in as little as 200 ft of mud in the hole, it is difficult to develop sufficient circulation with air-assist reverse circulation alone. Table 8 summarizes some common circulation procedures.

It is important to drill a straight, plumb hole if casing centralization and a high degree of cementing effectiveness are to be assured. The roles of stabilizers, bit design, and bit weight have been mentioned. Plumbness is also affected by strata attitude, uniformity (hardness-softness) and rotation speed (rpm). Common practice is to specify hole vertically better than 9 minutes of arc. About the only operational controls on alignment are the use of top stabilizers, control of weight and rotation, bit design and maintenance, and adequate circulation. Frequent surveying (using a gyro through the drill string or some similar means) will allow the control of these parameters.

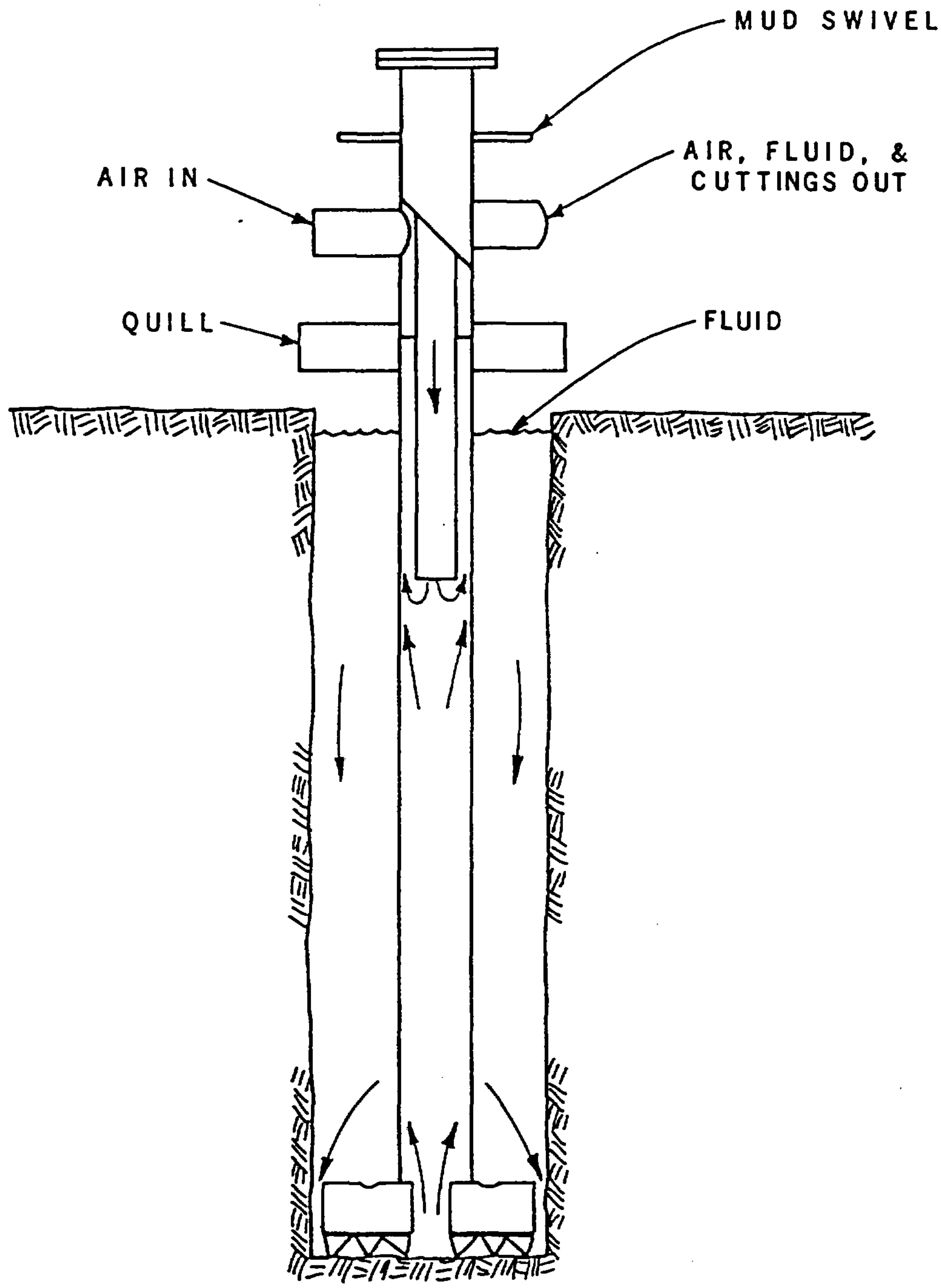


Figure 11. Single-String Air-Assist, Reverse Circulation

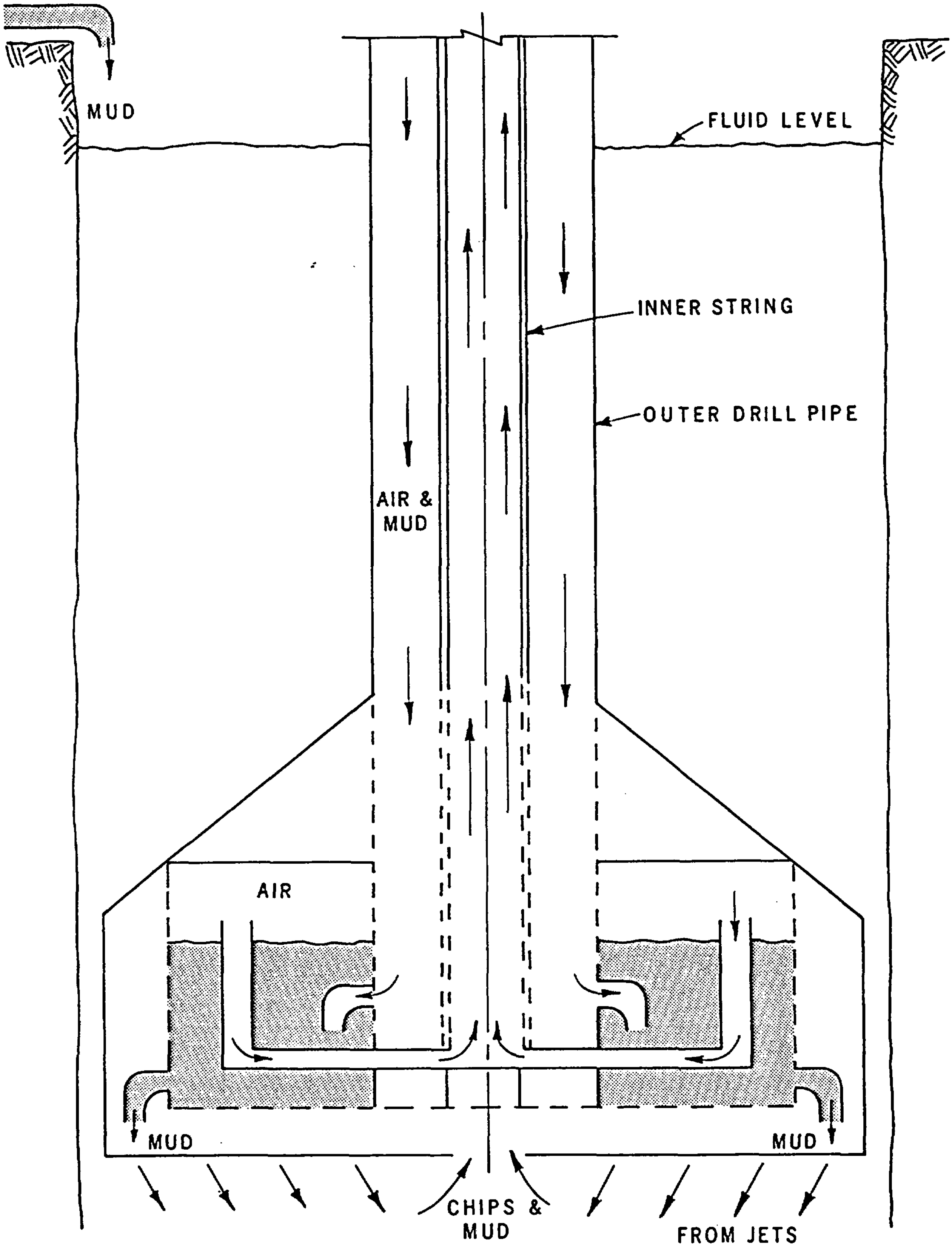


Figure 12. Reverse Circulation Dual String with Jet Assist

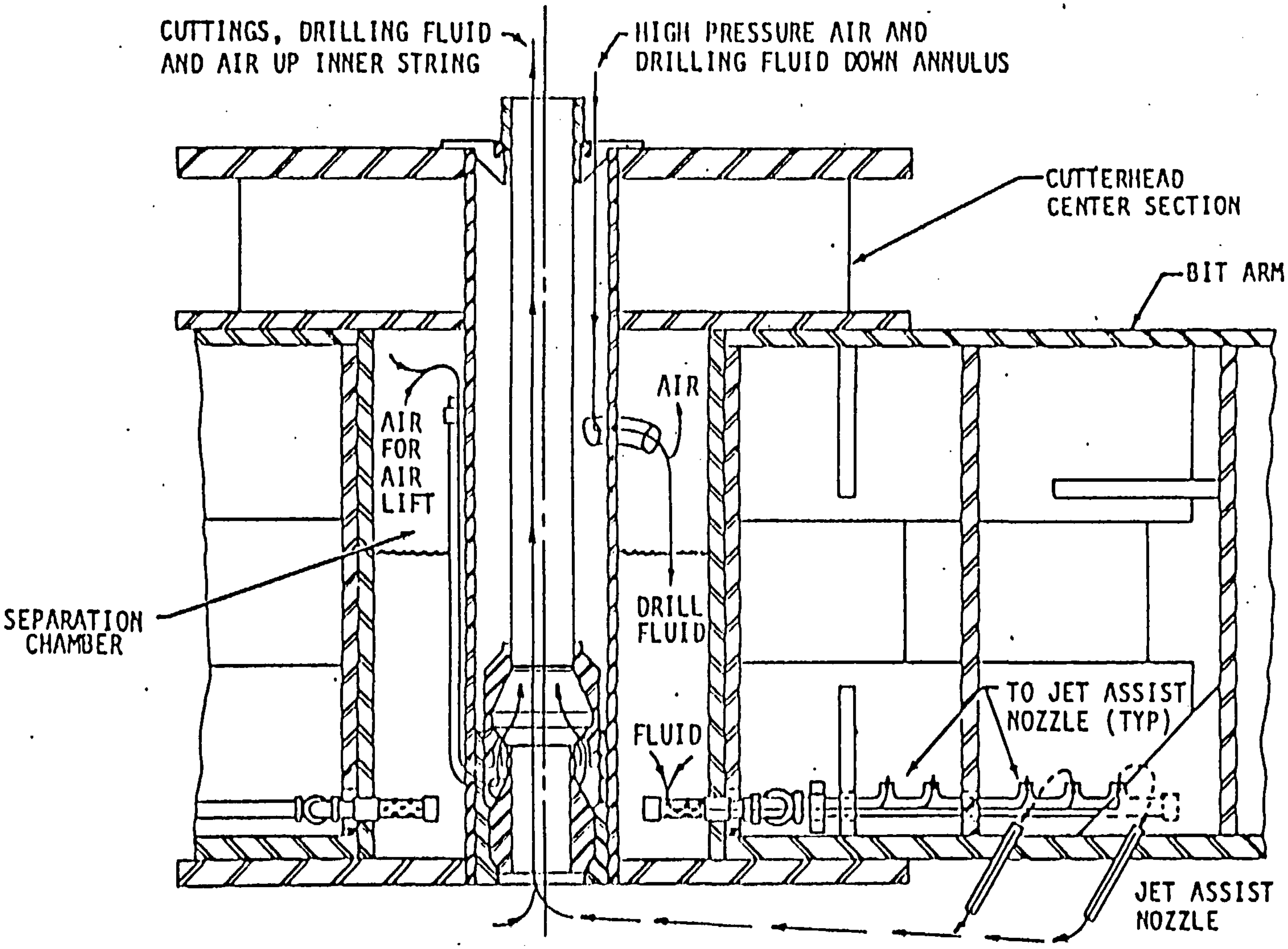


Figure 13. Bit Body Construction for Jet-Assisted Circulation

Table 8: Blind-Drilled Shaft Circulation Procedures

Condition	Procedure
Normal--mud fills hole, mud head greater than formation water head, large holes.	Reverse circulation, density difference created by injection of air in drill string.
Normal--mud fills hole, mud head greater than formation water head, small holes.	Direct circulation, drill string flow is pressurized with respect to mud head.
Loss to formations; alluvium, low water table, mud head greater than formation water head, hole about 10% full of mud.	Reverse circulation, air-lift by injection of air near bit, through annulus of double drill pipe.
Formations do not yield or must not yield water; dry formations and formations below static water level.	Reverse circulation, air-lift with water-air injection as above.
Dry or nearly dry formations that must be kept dry.	Reverse circulation of high-volume, low-pressure air. Direct circulation of air-water/soap-foam (used in large holes).

Practically every large-diameter drilling project has a fishing job of some kind. The most common causes are rotary table failures or drill string failures that release rod torque causing them to detach in the hole (wind-up/backlash), dropped cutters, or droppage of the bit body due to bolt failures. Mostly, these are unexpected occurrences although wind-up problems can be due to inexperience.

Improvements in design are reducing the incidence of fishing jobs. Specially-fabricated tools are usually necessary to fish debris out of the hole. The lost time and expense associated with fishing can be substantial and in some cases rock wall deterioration has resulted (Section 4.2).

Of direct relevance to lining voids is the formulation of the drilling fluid (mud) and hole wall stabilization. Mud engineering is a science unto itself and, like grouting, is highly site-specific. Accordingly, only the most relevant points will be addressed here.

Some of the objectives of the mud engineer may be contrary to shaft lining efficiency and have to be reversed just prior to cementing.

One is the formation of filter (or mud) cake. Filter cake arises from the need to provide a net positive outward hydrostatic pressure on the shaft walls for stability, circulation control, and control of water or gas inflow. Weight differential, according to Schlage and Smith (1981) should be about 0.0321 psi/ft. In porous, permeable, or underpressured formations, the particulates in the mud filter out against the shaft wall and are compressed by the hydrostatic pressure. Although the objective is to build the thinnest impermeable filtercake possible, variation in mud consistency in the hole and formation characteristics occasionally produce cake buildups of several inches. The filter cake at Beatrix was up to 6 in. thick. At Asse No. 4, it was designed and held to a few tenths of an inch using polymers. Aside from constricting the hole during trips in and out, filter cake takes up space intended for cement and prevents a direct rock-to-cement bond.

Because of pressure, time, and mud makeup dependencies, it is difficult to pin down the permeability or density of filter cake. Smith (1976) gives a permeability range of from 5 to 0.009 md at 1,000 psi. Filter cake is about the consistency of stiff modeling clay. However, its placement is not controlled or monitored and could be quite irregular.

Mud additives are used to control fluid loss, inflows, and wall sloughage promote cuttings removal and suspension, add density, lubricate and cool cutters, and reduce the swelling or plasticity of shales and clays. The density of mud is typically slightly greater than water -- about 9 lb/gal -- but it should not be so heavy that it breaks down the formation and causes a large fluid loss. There is an obvious problem in balancing the mud weight required for a deep high-or pressure or saline aquifer if there are weak, underpressured formations overhead. In severe cases, there is no choice but to case off the hole and reduce the diameter before continuing a circumstance that can in most cases be identified during site exploration and planned for in advance.

Other similar procedures could be required due to mud compatibility problems. For example, the use of potassium chloride to protect a thick section of montmorillonitic shale may be incompatible with deeper, freshwater aquifers.

Certain common mud additives are worth mentioning. Swelling shales are commonly inhibited with potassium chloride or sodium chloride; the latter is obviously preferred in the case of halite interbeds. Sodium hydroxide is often added to raise the pH and thereby reduce corrosion of the drilling equipment. Polymers are superior for stabilizing shales -- in addition to retarding hydration and expansion, they penetrate, support and seal the shale section and tend to develop reduced filter scale thicknesses (McLaurin, 1986). At the NTS, drilling sometimes begins with water but a native bentonite mud (3 percent) is formed from the cuttings (Rowe, 1986). Bentonite is commonly used as a water-loss preventive and dispersant. Bentonite muds are thixotropic, with a high gel strength (Haute and Cook, 1979). Lost-circulation materials such as cellophane flakes may be carried along with weighted muds to guard against fluid loss. In big holes, weighting of mud is normally not done, partly because the larger perimeter increases the chance of breakdown governed by discontinuities, and partly, because the need for weighted muds is infrequent at the normally-shallow big-hole depths, where gas pressures are seldom great.

Despite careful mud control, hole sloughage in big-hole drilling does occur. At the NTS, alluvium and poorly-cemented sands slough. Sloughage can also occur in lost circulation zones and fractured horizons. If it is severe, it may be necessary to pour a concrete plug and drill back through. Several such applications might be required in vertically-extensive zones. From a drilling point of view, minior sloughage can be tolerated, unless problems with tripping in and out or bit weight are experienced. From a cementing point of view, mud replacement in sloughed, open zones can be ineffective, and concrete plug placement may not positively seal either, trapping lost circulation materials if placed before hand.

Some large holes (for example, the Beatrix mine shafts) have been drilled with multiple passes, the Beatrix No. 1 and 2 shafts, completed in the late 1950s, were historic achievements at the time. Their construction is described by Weehuizen (1960). There was one incident of caving involving a solid clay layer after 15 months' open hole time. The shafts were enlarged in several passes from 6 1/2 ft diameter to 25 ft diameter, to a depth of 1,700 ft. The hole was kept full of soda ash mud at all times. A composite concrete-steel lining was floated into place and cemented.

There are numerous single pass blind-drilling examples, most of which were routine and successful. Some representative ones are described in the following.

Three shafts were drilled for Wyoming Mineral-Conoco's Crownpoint, New Mexico project, as described by Hunter (1983, 1986). The only other alternative, freezing, would have been 4 times as expensive. The No. 1 shaft was drilled 2,243 ft (referenced Kelly bushing, or RKB, measurement) 10-ft diameter (the other two were drilled 6-ft diameter to 2,188 ft). Potassium chloride mud (4 percent) was used to protect the bentonitic Mancos Shale. The final shaft deviation was 16 in., as determined with a gyro survey every 30 ft, and penetration averaged 16.9 ft per day. Prior to cementing the 85-in.-i.d. casing in the hole, the mud was conditioned and a 2 percent KCl wash was used. There was only one side sloughage (Hunter, 1985), measuring 6 to 8 ft high by 10 in. deep. The strata were principally sands and shales. There is only indirect information on filter cake buildup, as evidenced by a lower-than-expected fill-up volume (difference between caliper log hole outside diameter and the known casing displacement) which indicated that the caliper was riding on filter cake. The cement used was even less, indicating that filter cake continued to build up during cementing. During drilling, filter cake was controlled with a sand separation circuit in the mud return (Hawes, 1985). Aquifer communication prevention was part of the cementing design, which used Chem Comp in the breakouts and aquifer zones and either 2 percent prehydrated gel or neat cement tailed off with Chem Comp for filler. There were 8 grout ports provided at each of 3 locations. Had breakout taken place, the aquifers would have been depressurized and the casing grouted, to effect a seal. However, the mine was never developed due to the downturn in the uranium market.

The Asse No. 4 shaft in West Germany was blind-drilled into a salt dome occupied at its upper levels by a salt mine; the new development is for the storage of intermediate-level nuclear waste. The shaft was blind-drilled to 2460 ft (750 m) depth in 1973 - 1975. It is lined in the non-salt sections. The diameters ranged from 8.7 ft (2.64 m) (lined 7.15 ft i.e.2.18 m i.d.) in the 0 - 164 ft (0 - 50 m) interval in loose gravel, to 7-ft (2.13 m) (lined 4.9 ft i.e. 1.5 m i.d.) in the 886 ft - 1,322 ft (270 - 403 m) interval in salt. The remainder of the shaft was unlined to 2,460 ft (750 m). The annulus was thus as little as 4.5 in. (11.4 cm), far less than normally used today. Sperry-Sun surveys were run every 29.5 ft (9 m) and the deviations showed the typical spiral pattern:

Depth	Deviation	Direction
298 ft (91 m)	2.0 in. (5 cm)	NW
649 ft (198 m)	3.9 in. (10 cm)	W
800 ft (244 m)	2.8 in. (7 cm)	SW
1,312 ft (400 m)	6.3 in. (16 cm)	E

The deviation was in excess of the annular space and the casing did become stuck when run in the hole. During drilling, it was necessary to keep bit weight down and rotation speed up to minimize the hole deviation, which was critical due to the narrow annulus. The average penetration was 8.3 in./hr (21 cm/hr). The experience in cementing the Asse No.4 shaft is quite thought-provoking and is covered in Sectopm 3.4.1.

Adamson (1972) describes the drilling of outfall and intake shafts for the Alcan smelter in Great Britain that were successfully drilled and lined beneath the ocean floor. A steel casing (8.1 ft i.d. i.e. 2.46 m) cemented into place in 3 stages and fitted from below with precast lining segments. When the shaft was broken into from the out-fall tunnel (the shaft collar had been capped by divers at the ocean floor) the annulus was dry. The precast lining segments were backgrouted to seal the steel casing from the seawater.

The AOSTRA project in Canada is a pilot plant for in-situ oil recovery from tar sands. The ground will be heated to stimulate oil flow and the shaft design and layout had to consider thermal effects arising from 400°F temperatures in the oil recovery area (Greenwell, 1986) the expected displacement in the shaft are a total 3.9 in. (10 cm).

Two shafts were drilled 13-ft diameter to 732-ft depth, and cased 10-ft i.d. The construction method had to ensure that water invasion to the tar sands would not occur. The problem strata included thick sequences of "gumbo" clay with welling potential (Whitley and Pliska, 1985). The shales were controlled with an anionic polyacrylamide polymer. Only 1 washout occurred. Mud had a tendency to "heavy up" due to accumulation of solids, so sand separation circuits were used. Casing was floated into place and the breakout interval (bottom 20 ft and the 150 ft second stage) was cemented with Chem Comp run through 4 grout tubes. The remainder of the shaft cementing used a 2 percent prehydrated gel as a filler. Strength of the cement had to be in excess of 2,900 psi at 28 days.

The first shaft had been broken out and drifting toward the second shaft break-in area was in progress at the time of this writing. The first breakout was accompanied by a slight methane release, but no water (Greenwell, 1986).

3.3.2 Pilot and Slash Methods

Pilot and slash methods of excavation involve enlargement of an existing smaller-diameter hole to a larger size by drilling and blasting. Mechanical excavation (upreaming or downreaming) in a single pass is limited for repository depths to shafts less than about 20 ft in diameter, with present technology. The pilot hole or raise serves as a muck pass and sump and aides in ventilation and shaft drainage. It also constitutes a large blasting relief hole, thereby reducing the overall level of vibration damage to the wall rock. Rock damage can be further reduced with maximal use of hand cleaning, controlled blasting, or feather-and-wedge methods. Savings in mucking time and cross-sectional area that must be drilled out and shot result in a significant time and cost savings relative to conventional drill and blast methods. The shaft must be connected to an opening at the bottom for muck removal, and the ground must have favorable stability and ground water characteristics, or be treatable to establish stability and reduce inflows, because of the unlined pilot hole.

The pilot hole can be small, so as to provide for an intermediate, upreaming or downreaming stage, or it can be drilled to a large diameter in a single pass from the surface. The WIPP Waste Handling shaft is an example of the second method. For that shaft, the former ventilation shaft (blind-drilled 6-ft-diameter and left unlined) was enlarged by drilling and blasting from the surface down, in a single pass. The WIPP exhaust shaft is an example of the former method. It was begun with a small pilot hole, upreamed 6-ft diameter, and then slashed out to the final diameter. Both the WIPP shafts projects are discussed in detail in Section 4.1.

The initial opening to be slashed out varies from about 4 ft to about 10 ft. The size depends primarily on muck handling and ventilation requirements as weighed against the expense of the raise excavation. The size must be sufficient to prevent hang-ups of muck in the raise. Other considerations that can weigh heavily on raise or pilot hole size selection are the size of the final shaft, rock stability, and water make in the raise. In heavily waterbearing rock, it is necessary to improve the ground by freezing grouting, or both.

The presence of the raise, which stands open for some time, offers an opportunity for monitoring of the water, and for mapping of the rock mass.

Gonano et al. (1982) discuss several case histories of this type of shaft excavation method. Shafts similar to those built on many projects were sunk at Northfield Mountain, where a 15-ft diameter and a 31-ft diameter shaft were slashed out from raises drilled 6-ft diameter. The Dinorwic pressure shaft was slashed out to 33-ft diameter from an 8 ft drilled raise. The 39-ft-diameter Rocky Mountain pressure shaft, discussed in Section 4.4 was slashed from an 8-ft-diameter raise.

One difficulty with raise and slash shafts is maintaining satisfactory alignment of the pilot hole. They use the same temporary support and lining techniques as drill-and-blast shafts. Water control can be effected through the use of backsheets although it is not likely that the water problems during slashing would be very great since the method is not often useable in heavily water-bearing ground.

3.3.3 Conventional Drill-and-Blast

The majority of shafts today is excavated by drill-and-blast methods. The shaft excavation is performed full-face or in benches that are drilled out, shot, mucked, and as necessary, supported, round by round. The work is supported by a galloway or multi-stage work deck that hangs in the shaft (Figure 14). Storck (1968) describes a Canadian sinking-lining operation that uses a 9-deck stage.

Conventional sinking is still the preferred method for large-diameter shafts (those greater than about 20-ft diameter). Although some recent shafts have made considerable improvements in advance rates, the method tends to be slow and costly.

Drill-and-blast operations entail six separate steps: drilling, loading, blasting, mucking out, scaling, and as necessary, installing temporary supports and water control.

Drilling can make use of air tracks, jumbos (usually the 4- or 5-arm type), hand held sinker drills, and pneumatic rock breakers. Air tracks have been used to pre-drill holes for shallow lifts from the surface. Jumbos are generally used only on larger diameter shafts (30 ft and more). Pneumatic rock breakers are used in weak or frozen ground. Hand-held sinker drills are by far the most common.

On larger shafts there may be as many as 300 blast holes drilled. The center holes in the pattern are oversized to provide relief when the shot is initiated. Typical blast patterns involve a central V-cut. If the V-cut (or in rectangular shafts, wedgecut) is repeated in concentric outward rings or holes, it is referred to as a double or triple V-cut. If a V-cut is drilled in vertical orthogonal planes, the result is a pyramid cut. The angle from the vertical decreases toward the outer holes. It is difficult to control outward deviation of the holes on the outer ring. The pattern used depends on the delay sequencing and the necessary burden per hole, as well as on the anticipated rock breakage.

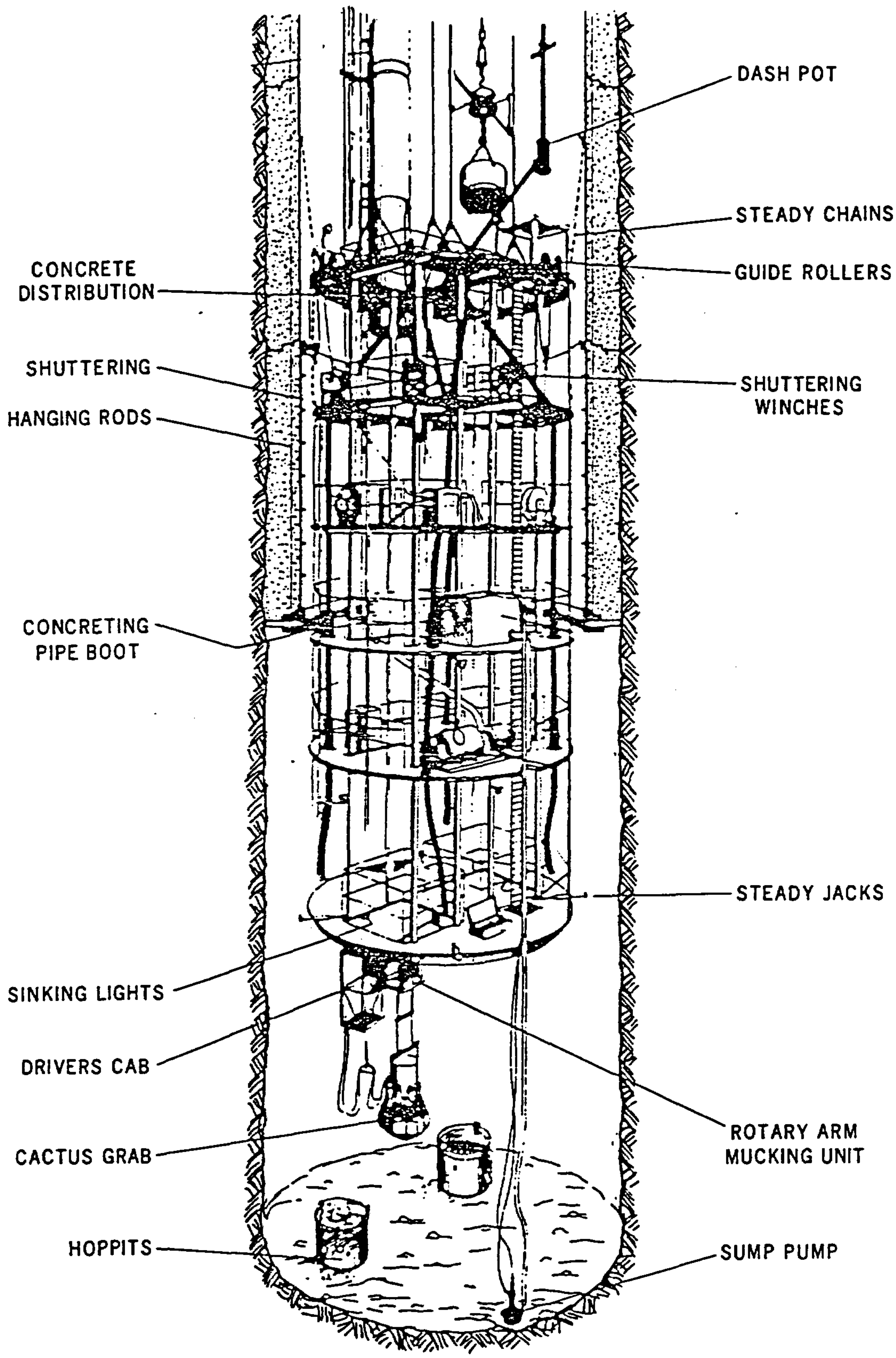


Figure 14. Conventional Sinking Stage, Using a Cactus Grab for Mucking (after Auld, 1983b)

Blasting generally involves electric detonators and high explosives; ammonium nitrate-fuel oil (ANFO) is difficult to use in water-filled holes. The outer ring of holes detonates last. Control of wall rock damage (overbreak and cracking) is aided by the use of light charges (spaghetti powder) in the outer ring of holes, somewhat closer spacings, better hole verticality control, and reliable explosive delays. Very close control of blasting is necessary in frozen shafts, for the reasons outlined in Section 3.2.2. Tunnicliffe and Keeble (1981) discuss the charge weights used in the frozen sections of the Selby shafts.

The round can either be taken full face or in benches. Benching is likely to be slower, all things being equal. However, the benching method provides a sump, which aids water handling, reduces the incidence of plugged holes or misfires resulting from the difficulty of loading holes under water, and provides a reference plane for drilling, which aids in blasthole alignment.

The length of the round depends on the lining support method, permissible unsupported length of shaft, shaft size, and drill-muck cycle. Ten (10-) and 20-ft lengths are common. Rounds occasionally "pull short", that is, rock breakage does not reach to the full depth of each round. Bootlegs (undetonated or misfired holes) are a hazard in shaft sinking and can more easily be identified in dry shafts or where benching aids in water control.

Mucking, or removal of broken rock, can be accomplished by hand, using a small loader, Cryderman mucker, backhoe, clamshell, or cactus grab (Ridell mucker). The world record sinking rates were established using cactus grabs. The Cryderman mucker has a bucket similar to the clamshell but the cables are replaced by hydraulic cylinders. Crydermans, cactus grabs, or clamshells, are mounted beneath an operator's platform at the lowest floor of the galloway stage. Small loaders (actually overshot muckers as well as loaders) are limited by clearance and are not practical in a benched shaft. In most cases, muck is hoisted to the surface in buckets.

Scaling involves the removal of loose rock from the shaft walls after mucking out, either with a pry bar or using pneumatic hammers or chisels. In some shafts, seal areas have been further prepared using hand grinders.

Temporary support and water control may involve liner plate, timber rings, bolting, mesh installation, or shotcrete, singly or in combination, together with provision for weep holes, backsheets, French drains or water rings, and extension of drain pipes. It may include bringing the full liner down or installing a primary lining of poured concrete. Temporary linings are for support, water control, and safety during the sinking. In most deep, large shafts at least the primary lining is placed concurrently with sinking. Slipformed linings will require excavation to full depth, in which case the shaft walls overhead must be supported.

Water control most commonly uses backsheets or liner plate. Backsheets for years meant corrugated metal sheeting pinned to the rock and overlapped to confine the water flow to the shaft wall surface and keep it away from the concrete linings. Recently, other materials for backsheets, particularly woven plastic fabrics with tough, durable backings, have been introduced. These will not corrode, and provide an acceptable standoff from the rock that aids backwall grouting effectiveness. If the wall rock is unstable, it may be necessary to use wire mesh secured with rock bolts or, in severe cases, liner plate bolted together and wedged into place with ring beams. Liner plate may use filter fabric behind it if the strata are erodable. If the rock is susceptible to air slacking (atmospheric deterioration is common in some shales and siltstones, as well as evaporates such as carnallite), liner plate or a shotcrete cover may be necessary if the period until final lining will be significant. French drains are gravel rings with an impervious layer beneath, fitted with weep pipes, that collect the water running down the shaft walls and vent it to collection pipes.

Lining is discussed separately in Section 3.4.

3.3.4 Other Methods

There are other methods for shaft sinking that are not considered the best-suited for repository shaft sinking for reasons of cost, or techincal limitations. For completeness, these are mentioned below.

The calyx drill is a coring machine using a downhole operator. Essentially the method is one of kerf cutting; when the barrel has been filled or the core broken off, the core is lifted to the surface. The method was uses to bore large-diameter holes from about 1936 - 1960. Its limitations are the types of formations that can be drilled and the sizes possible (about 6 ft maximum).

Downhole full-face boring is similar to the use of a tunnel boring machine on end. This was tried in Alabama in a demonstration project sponsored chiefly by the Department of Energy. The machine performed well in terms of penetration rate, accuracy, and safety, but muck handling was a serious problem. The Robbins Company, which built the original machine, has a new shaft excavator design based on a drum fitted with cutter wheels rotating about a horizontal axis, that permits a more reliable muck handling system to be used. The method is limited by the need for nearly dry conditions and stable shaft walls.

Grieves (1979) discusses the V-mole 4 borer, which has drilled shafts over 20 ft in diameter, full-face from the surface. Muck removal is from the subsurface. The V-moles are a series of boring machines with V-shaped cutter heads. All rely on bottom access for muck removal, but have posted some excellent performances (Table 9). V-moles using hydraulic cuttings removal systems for blind headings (Figure 15) have been under development. Since bottom access is available, it is practical to line behind a V-mole although the water make and ground stability requirements are similar to those of the blind-shaft borer described above.

Shafts can be reamed downhole using moles or reamers or uphole using raise borers. V-mole 3 is designed to ream shafts to 26-ft diameter and to 4,000-ft depth. Bruemmer and Wollers (1976) discuss the Turmag reamer, a multi-stage shaft reaming system (Figure 16). Downhole reamers can be equipped with provisions for on-cycle bolting and temporary support. Uphole reaming of shafts has been done for limited depths up to 30-ft diameter (Monterey Coal Company No. 1 Mine near Carlinville, IL). Completed shafts reamed uphole are open to total depth, so slipforming often is a suitable means of lining with the method. Despite the impressive diameters achieved, present technology does not support upreaming of shafts 20 ft diameter at repository depths (3000 ft). Shaft alignment can be difficult to maintain, especially at greater depths, with the uphole reamer. Heavy ground water inflows or unstable shaft walls require pretreatment of the ground, since opportunities for remedial action to alleviate such problems once reaming has begun, are poor.

Nash (1985) discusses the construction of the shafts at Carlinville by upreaming. The ground only made 2 gpm. The coping section (surface to 90 ft depth, in glacial overburden) was slipformed from the bottom up and the rock portion (90 ft to 310 ft) were step formed from the top down.

3.4 TYPES AND PLACEMENT OF LININGS

Linings described herein are classified according to their types and modes of emplacement. A distinction is made between remotely-placed linings (such as the casing and cement involved in blind-drilled shaft sinking) and direct-placed linings.

Ostrowski (1972), Auld (1979), and Black and Auld (1985) describe methods of designing linings and the structural calculations pertaining hereto. The lining design hinges on whether the full hydrostatic head is to be resisted, as is the case with requirements for watertightness or whether the water pressure is to be dissipated at various points with water collection provisions and drainage. For some applications, particularly in Germany where mining of the shaft pillar is practiced, and in the Gulf Coast salt mines where ground movement is common, the lining must be designed to retain its watertight function under appreciable ground deformation.

Table 9: V-Mole Data (from Grieves, 1979)

Shafts Bored with V-Mole 2 (5 - 6.5-m-diameter)

Mine	Firm	Year	Depth m	Advance m/d	Time days
Ibbenburen	Preussag	1977	467	10.0	47
Gottelborn	Saarberg	1978	414	13.8	30
Victoria	Ruhrkohle	1978	106	7.0	15
Walsum	Ruhrkohle	78-79	285	10.5	27
Prosper	Ruhrkohle	1979	180	10.0	18
Erin	Evb	1979	200	11.0	18

Statistical Data on Shaft Borers

Machine	V-Mole 1	V-Mole 2	V-Mole 3
Drive	Elec/Hydr	Elec/Mech	Elec/Mech
Drill diam	4.5-5.0 m	5.0-6.5 m	6.5-8.0 m
Cutter head	3 x 75 kW	4 x 110 kW	6 x 110 kW
Total kW	230 kW	490 kW	745 kW
Torque mkp	27 000	76 000	141 000
Thrust	300 Mp	550 Mp	740 Mp
Load/disc	App. 4 Mp	App. 12 Mp	App. 12 Mp
Cutters	Two discs	One disc	One disc
Disc spacing	48 mm	70 mm	70 mm
Head angle	30°	45°	45°
Gripper pressure	12 kp/cm^2	12 kp/cm^2	12 kp/cm^2
Weight	App. 100 Mp	App. 150 Mp	App. 237 Mp

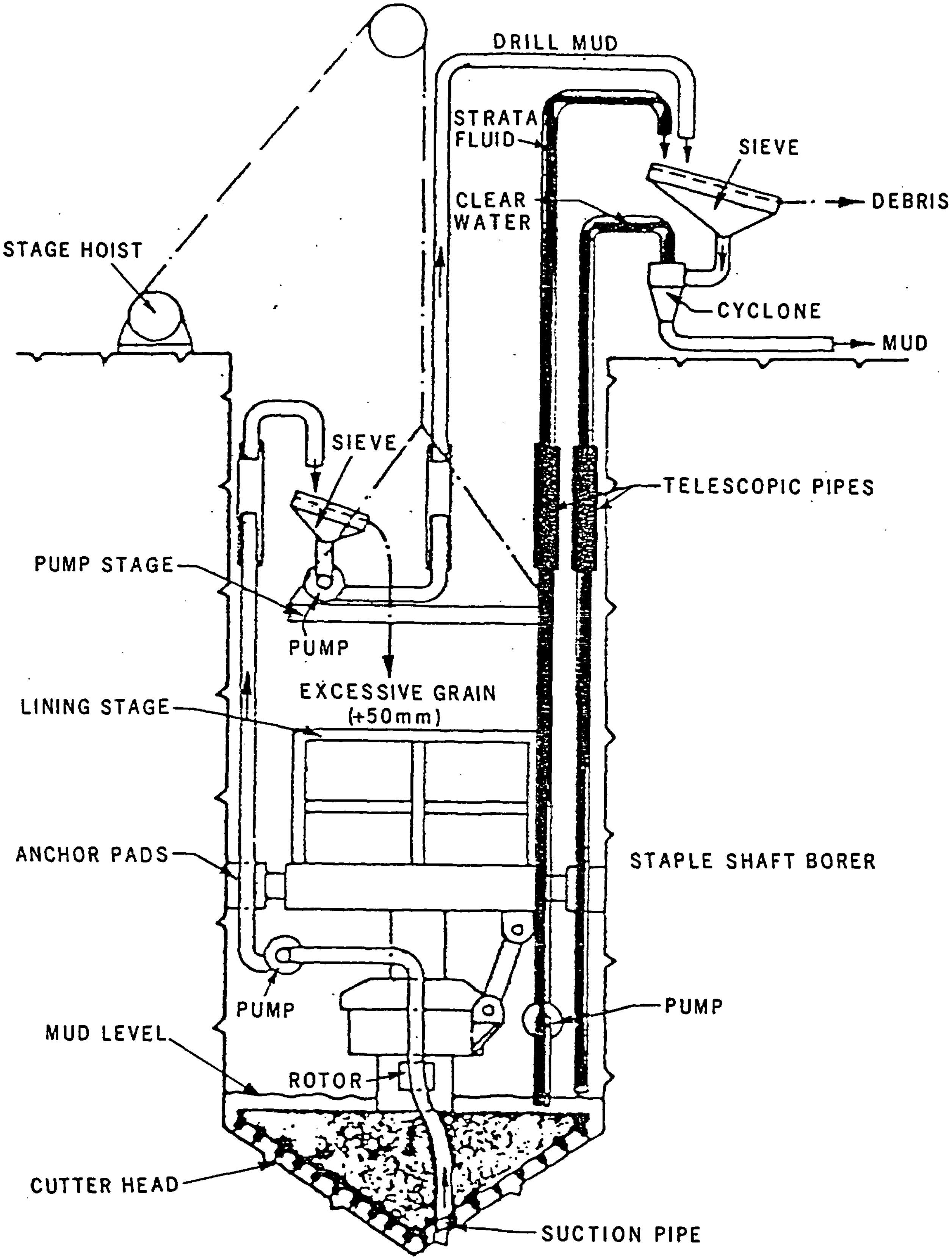


Figure 15. Blind Shaft Borer (full-face) with Hydraulic Muck Disposal (from Grieves, 1979)

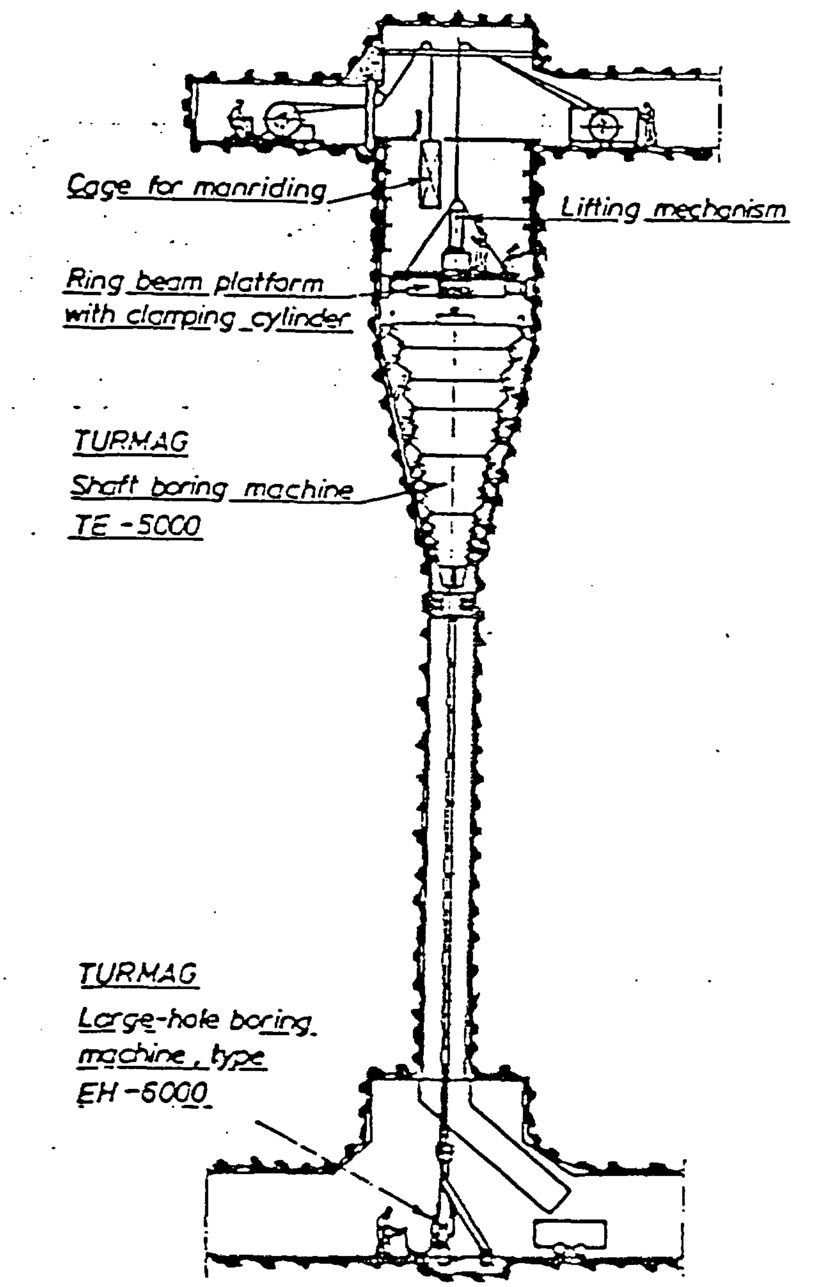


Figure 16. Turmag Shaft Boring System
(from Bruemmer and Wollers, 1976)

In competent rock with little water production and where the water make can be tolerated, a minimal lining of mesh and possibly shotcrete may be all that is provided, for safety reasons only.

Salt sections in many shafts are left unlined. This is a considerable savings in lining cost in that a lining resistant to creep does not need to be provided, but this savings is partially offset by maintenance problems in shaft equipment, which is attacked by the salt environment, and the degradation of the salt itself, due to the passage of air.

Linings that are not grouted, such as timber lagging and timber sets, shotcrete and mesh, and so on, are not discussed in this section.

Basically only two shapes of shafts are usually considered: rectangular and circular. Competent ground and good environmental conditions, will allow the use of rectangular, unlined shafts equipped with timber or steel frames. In the last few decades, the increasing need to sink deeper shafts combined with severe geotechnical conditions being encountered in many areas has resulted in a general acceptance of circular shafts with their inherent advantages, for example, Hecla's Lucky Friday Mine Silver Shaft at Wallace in the Coeur d'Alene district of Idaho. The major reason for this is that the resistance of a circular opening to lateral forces is superior to that of a rectangular opening. Also, the variety of linings and the methods of installation are considerably greater with a circular shaft than for a rectangular shaft. A rectangular shaft would not be suitable as access to a nuclear waste repository and has therefore not been considered here. Likewise the elliptical shaft section, practically abandoned in modern mining, has not been considered.

The sizes of conventionally constructed circular shafts vary from 12-ft i.d. to 34-ft i.d. The depths of conventionally constructed shafts often reach 8,000 ft or more. The deepest shafts with depths to 11,500 ft are in South Africa.

The ground surrounding the lining structure will exert both horizontal and vertical forces. These may result from tectonic disturbances in the rock mass, anisotropy of the ground materials, disturbances caused by excavation in the vicinity of the shaft, thawing of the frozen ground after application of the freezing process for sinking, drainage of the ground surrounding the shaft and backwall injection operations.

Water pressure will be applied to a shaft lining which penetrates an aquifer. The amount and time scale for which this pressure is applied will depend on the nature of the waterbearing rock formation and the type of aquifer.

3.4.1 Remotely-placed Linings

The most common remotely-placed linings are those for blind-drilled shafts. These are really composite linings using a steel inner shell with cement behind and in contact with the rock. The steel inner shell (casing) is provided with stiffener rings on the outside and a pre-selected number of slotted grout line guides welded vertically to the outside. The steel shell is floated into place with the hole full or nearly full or fluid, so that the casing weight is borne in part or entirely by its buoyancy. The rate of placement is adjusted by pumping water into the inside of the casing, which allows the casing to either float or sink. When the casing is in position, an initial cement stage is poured to take the casing into place so that it will not refloat when slurry is pumped in around it.

During the cementing, the casing interior is kept full of circulating water both to control the cement temperature during its set and to counteract the external hydrostatic pressures developed by the column of slurry before it sets. Even so, the height of each stage is limited by casing collapse pressures; the stages get higher as the annulus becomes filled, since the weight of the overlying mud is reduced with cement height. Typical pumping time per stage is 1 - 3 hours, so the gelling of the cement does not reduce the stage slurry head. The hole must normally be kept full of mud during cementing to maintain wall stability. The cement at a density of 13 - 16 lb/gal. (Table 10) displaces the mud, which is (typically) 9 lb/gal. The effectiveness of mud replacement, as is shown in this section and in Section 4.5.1, is critical to preventing voids in these types of linings. Essentially all grouted-in-place steel casing have used cement as a grouting medium. However, Einstein and Barvenick (1975) point out that other materials such as foamed plastics, foamed gypsum, foamed glass, or foamed sulfur, could be used.

It has been well demonstrated in both small and large diameter casings that Portland cement grout, being semipermeable, offers no reinforcement of the casing from hydrostatic water or gas pressure. Thus casings are designed for 1.5 - 2.0 times the full hydrostatic head. The sheath of grout, can slightly reinforce the casing insofar as loading from lithostatic forces is concerned. This is particularly important in materials which have a fairly high creep rate such as salt.

The clearance required to obtain the desired sealing effectiveness is a function of the straightness of the shaft, the depth of the shaft, the shaft diameter, and the size of the casing.

In the great majority of cases where the casing will be installed in a mud-filled hole, the mud exhibits thixotropic properties and will achieve a very firm gel structure after the installation of the casing and before grout placement begins. Although this is

Table 10: Typical neat cement slurry weights
(from Suman and Ellis, 1977)

Class	Percent water	Water per sack gal	Slurry density ppg*	Slurry yield ft^3/sk*
A	46	5.19	15.6	1.17
B	46	5.19	15.6	1.17
C	56	6.32	14.8	1.32
D	38	4.28	16.4	1.05
E	38	4.28	16.4	1.05
F	38	4.28	16.4	1.05
G	44	4.96	15.8	1.14
H	38	4.28	16.4	1.05

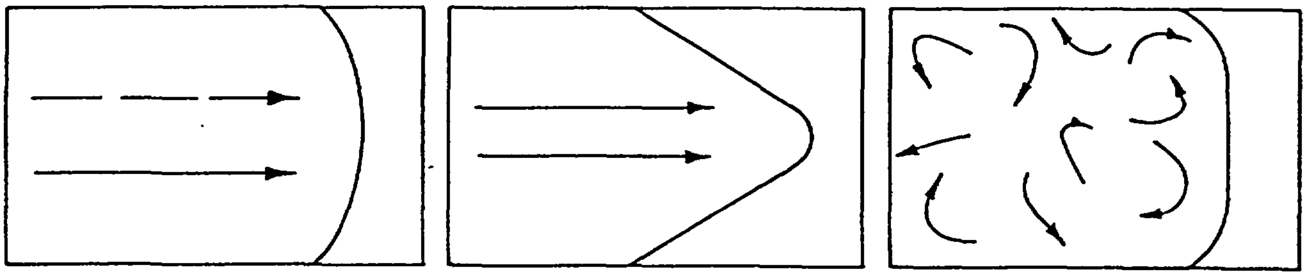
* Based on absolute volume per sack cement equals 3.59 gal

desirable for cuttings suspension, it is highly disadvantageous for mud displacement. It is absolutely imperative to break the gel structure of the drilling mud prior to its displacement by the grout. This is accomplished by conditioning (circulating, cleaning, and adjusting) the mud. As a minimum, a large volume of water should be pumped into the shaft prior to placement of the grout. This will dilute the drilling mud, reducing its gel strength and viscosity. A preferred preparation for grouting, not practiced on all projects, is pumping a dispersing wash followed by water prior to placement of the grout. A number of chemical washes is available that can be matched to the particular nature of the drilling mud to chemically attack the gel structure of the mud so that is more readily displaced by the grout. Washes also have the capability for reducing filter cake thickness. Typical washes include acids such as HCl, phosphates such as sodium hexametaphosphate, water, and KCl.

In small diameter holes the displacement of the drilling mud by the grout can be more readily accomplished by pumping grout at a rate sufficient to maintain turbulent flow in the annulus. In large diameter holes with large annular area, this becomes economically infeasible if not physically impossible. Large diameter holes, rather, are cemented in plug flow (Savageau, 1984). There is no opportunity to reciprocate or rotate the casing, due to its weight and the grouting hardware, as there is in oil-well casing cementing, to disperse the mud. Plug flow (Figure 17) allows greater mud-cement contact for better displacement.

To insure an uniform sheath of grout around the casing, an adequate number of grout lines must be placed around the perimeter of the casing to establish the uniform rise of the grout around the casing. Some early attempts to use stab-in shoes for cementing from within the casing, as is done in oil-field casing cementing, failed for casings over about 48-in. diameter. It is difficult to establish a uniform cement sheath in a large annulus from only 1 or 2 discharge points. There is no way to determine analytically the number of grout lines necessary. This will depend on many factors, including pre-treatment of mud prior to grouting, minimum annular thickness and nature of the grout. The use of the 3 grout lines is common in diameters up to about 72-in. i.d., though the use of 4 is more common. In diameters larger than about 72-in. i.d. casing, 6 or more grout lines have been used.

Obviously, to prevent casing eccentricity in the hole, the hole must be drilled straight and plumb, and the casing sections must be accurately aligned. To insure sufficient annular thickness, a centralizing device can be attached to the outside of the casing to maintain a minimum standoff between the casing and the wall of the hole. If the hole is severely out of plumb, or the casing is not straight, the necessary clearance can be lacking and the standoffs



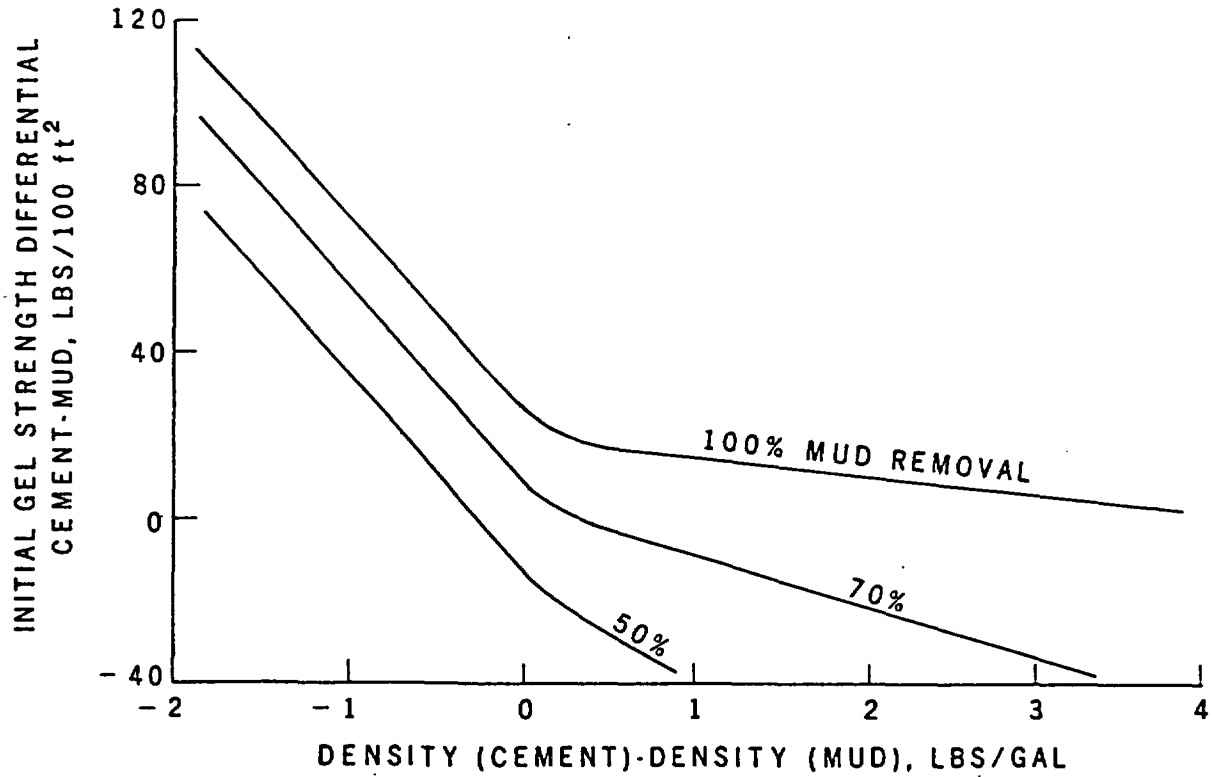


Figure 17. Mud Displacement in Plug Flow
(after Sauvageau, 1984)

can exert adverse stresses on the stiff casing or break off entirely. Grout line guides can serve as standoffs but their own contact with the shaft wall is not desirable.

The importance of sufficient annular area from a void occurrence point of view arises from concerns of cement bridging or channeling. A thixotropic cement in plug flow against a gelled mud may not enter a confined area, such as the point of tangency between the casing and the shaft wall or the grout line guide and the shaft wall. Even with limited confinement, the upper surface of the cement can develop sufficient gel strength to develop local breakthroughs and columns, which can slump and trap mud. Channeling can cause this cement to climb up the shaft annulus without displacing mud, and introducing voids or entrapped mud pockets.

The plug flow regime corresponds to very low fluid velocities, which are not sufficient to remove filter cake. Most of the filter cake on the shaft walls at the start of cementing will remain in the finished lining.

To insure that the grout lines are appropriately located, the usual practice is to attach a guide for each grout line to the outside of the casing. A separate guide is reserved for the cement fill-up logging tool. These grout line guides are larger diameter pipe which has some pattern of windows cut in it for the flow of grout out of the guide line into the annular space. To be effective, the grout line guides should be essentially continuous from the top to the bottom of the shaft. There have been a few instances where the guides had large gaps in them. Then, when the hole is a little crooked it is possible for the grout line to miss a portion of its guide with undesirable results. There have also been instances where insufficient clearance coupled with weakening of the grout line guides due to the slotting arrangement have resulted in guides crushing, also with undesirable results. Grouts themselves can take a number of forms, depending upon the service requirement. The use of Chem Comp and prehydrated gels has been mentioned. High-heat cements are normally avoided.

When absolute watertightness is a requirement, a chemical seal ring (CSR) can be used. The application extremes have ranged from the sealing of a casing string in a shaft where the casing was to be subjected to cryogenic temperatures, in this case -28°F (Cobbs, 1986) which can cause the casing to shrink away from the Portland cement grout with a major possibility of leakage. This shaft, in New Jersey, served as a storage cavern for anhydrous ammonia and the chemical seal ring effectively sealed the casing at the bottom of the shaft so that there was no leakage of ammonia vapors or water up the shaft into the cavern beneath the shaft.

Another extreme example was the sealing of casing for an emplacement hole for a nuclear device, at Tatum Dome, near Hattiesburg, MS. This was in a salt dome with high capacity aquifers above the dome and the caprock over the dome also contained a high capacity aquifer. Absolutely no entry of water to the emplacement hole could be permitted. In this particular installation there were two nuclear devices detonated in the hole drilled and sealed with the chemical seal ring. Drill-stem tests detected no water inflow. The chemical seals in this case were 100-ft long.

The objective of using expanding cement is to maintain a tight seal of the annulus. All Portland cements, when they react with water, are exothermic and some rather large temperature increases are possible during the reaction process. The great preponderance of the heat evolved is while the cement is still plastic, which produces a thermal expansion of the casing and this reduces the annular space somewhat. This is offset slightly by the slurry weight, but is augmented by the water head in the casing interior. After the cement reaction is completed and the heat evolved is removed from the grout either by transfer to the ground surrounding the shaft or into the shaft itself, the casing and grout shrink as the temperature decreases. In the absence of some expansion by the grout, an annular space with potential for leakage can be created either between the casing and the grout or between the grout and the borehole wall or at both places. The use of expansive grouts must take into account grout plasticity and casing strength to predict stress buildup and load on the casing.

Where seal integrity is not critical, it is common to use bulking agents in a Portland cement grout. The most common bulking agent is bentonite; expanded perlite, gilsonite, ground coal or sand can also be used. These bulking agents can reduce the cost of grout but sealing effectiveness is also reduced.

When a stage is initiated, the grout lines are "tagged on" to the top of the previous pour round then withdrawn some distance. This distance varies from a few feet to 20 ft. The less distance the cement falls through the mud, the better. During the placement of a stage of grout the usual practice is that a quantity of grout sufficient to give a rise of, say, 50 or 60 ft will be pumped through all of the grout lines and then one grout line will be closed off and a joint of that grout line will resume. This procedure is followed for each of the grout lines in turn so that the grout lines are being withdrawn almost continuously as the grouting proceeds. Even so, grout is always being injected beneath the upward-migration grout surface.

Prior to cementing, caliper logs are run to compute the cement volumes expected for each stage. Generally these are 6- or 12-arm mechanical calipers, although sonar calipers for submerged conditions or laser calipers for open hole conditions do exist.

Cement rise per stage is monitored with nuclear devices (Section 5.2). After WOC (waiting on cement) time for each stage has elapsed usually 20-24 hr) the top surface is plumbed with the grout lines. Often, the cement top surface is irregular. Hunter (no date) provides tag-on depths that indicate relief of up to 78 ft between adjacent grout lines and average slopes in excess of 80 degrees, in sections of 120-in. diameter hole cemented with Chem Comp at Crownpoint. Grouting systems are usually equipped with manifolds so the lowest point can be poured first (Boughton, 1986).

The casing design is critical, as the cost of the casing itself can be 1/3 of the cost of the entire project. It is important that the casing be welded straight. The casing at the Asse No. 4 and the Beatrix shafts were marked for welding at night so that differential thermal effects caused by sunlight would not compromise the precision of the welds. At Asse No. 4, because of the narrow (4.5 in. i.e. 11.4 cm) annulus, casing end parallelness had to be within 0.04 in. (1 mm) for each 39.4 ft-long (12-m) section. Pacific Northwest Laboratory (1984) has proposed a laser-based scheme for welding casing sections to match the hole alignment.

Albrecht (1976) describes cementing the Asse No. 4 shafts. The casing bottom area was left uncemented (by blocking out with a barite mud) to facilitate later removal of the casing shoe. The cement was run in 2 of 3 equidistant grout lines (the third was for monitoring); the casing i.d. was 59 in. Cementing between 164 ft (50 m) and 1,322 ft (403 m) depth was done in 3 stages, using API Class B slurry (salt-saturated in the salt sections) at a density of 1.95. About 164 cu yd (125 cu m) of cement had been pumped at a a rate of 1.3 cu yd/min (1 cu m/min) and a pressure of about 2.5 psi at the surface, and the cement top had reached 1,089 ft (332 m) in the initial stage, when the casing suddenly displaced upward 23 ft (7 m) over a 15 sec period, doing considerable damage at the surface. The casing set in this position and one grout line had to be abandoned. Further cementing had to use the guide reserved for monitoring. Despite all this, the casing was successfully cemented to the surface and, upon breakout, no water was evidenced.

Other remotely-placed linings include bitumen an pre-formed segments. Lackey (1983) describes the use of a composite steel and concrete lining floated into place in Canada.

The Beatrix shafts (See Section 3.4.2.3) were lined in a similar way.

Alternatives to the high cost of the steel casing have been investigated. Skonberg (1981) discusses pre-cast concrete lines for blind-drilled shafts. Each segment would typically be 10- to 15-ft in height and have a wall thickness of 10 to 24 in. The shaft is drilled 10 ft beyond the desired depth and the drill bit is then pulled 10 ft off the bottom of the hole. A grout plug, about 8-ft thick, is spotted on the bottom of the shaft through the drill pipe. Once the plug sets, it is dressed with the drill bit to ensure that the surface of the plug is perpendicular to the shaft wall. The drill rig is then moved off location and a steel frame with an overhead crane is assembled to install the liner segments. Drilling fluid remains in the borehole throughout the entire liner installation operation.

The liner running tool is positioned within the first liner segment and the retractable rams expand into steel ports cast in the concrete. Three 1-1/2-in. steel guide ropes are fastened to the outside of the first segment. These 1-1/2-in. ropes will be used to properly align the subsequent segments with those already situated in the shaft. Also, six 5/8-in. steel ropes are fastened symmetrically around the first segment, which will be used to guide grout lines for grouting the liner in place. Three (3) or more centralizers are attached to the outside of the bottom liner segment to ensure that it is truly located in the center of the drilled hole.

The first segment is lowered into the hole suspended by a single 2-1/2-in. diameter cable as the other cables are played out under slight tension. Once the segment is seated on the grout plug, the liner running tool is returned to the surface and all the peripheral cables are tied off at the surface. Six 3-in. diameter slotted pipes are run to the bottom using the 5/8-in. cables as guides. These slotted pipes will guide the tubing used in placing the grout. The first segment is then grouted in place.

Subsequent segments are positioned over the shaft, complete with the running tool. The 1-1/4-in. guide cables are clamped to the outside of the segment. The tool and segment are lowered into the shaft and the segment mates with the prior segment.

As mentioned, grout is placed behind the first segment once it is positioned. Grout is then placed behind the liner after every 10 segments are placed. This helps maintain centralization of the liner and also minimizes vertical compression forces by transferring the liner weight to the shaft wall in shear.

Once all segments are stacked in place and grouted, the drilling fluid is removed, and the shaft is commissioned. Since "lifts" of grout in the annular space are stopped at the midheight of a liner segment, a continuous grout sheath covers each cold joint of the concrete segments to shut off potential water inflow.

Bottom-to-top slipforming in mud-filled shafts has also been proposed (Schlage and Smith, 1981). However, high-strength concrete placement under water is not well-understood.

Somewhat different procedures can be used when it is not necessary to maintain the hole full of mud to stabilize the walls or where the casing weight is not such that the flotation effect is needed to place it.

The lining types associated with dry competent ground (except the thin steel lining) are installed from a stage hanging in the shaft in the same manner as for a conventional shaft sinking. Lining would normally progress from the top downward so that any bad ground would be supported before men had to work below it.

Thin steel linings are normally installed from the surface. The pre-radiused rings of lining are bolted or welded together as they are lowered into the hole by a crane. A grout seal is then usually installed at the bottom of the lining between the steel and the rock and the annulus filled with grout or pea gravel or sand. (Pea gravel was used in the ventilation raise at Anamax, Twin Butte Mine). In the case of a grout infilling it is placed either very slowly or in short lifts so that the grout head does not build up to one greater than the lining can withstand. This technique of backfilling a steel lining with a compressible material is also used to reduce lining stresses and is used in ground where stresses would normally be too high to be withstood by an economic lining system.

3.4.2 Direct-placed Linings

Direct-placed linings are those where there can be direct supervision of the lining installation at its point of contact with the rock. From a sealing point of view, this has its advantages and disadvantages. Linings placed directly can be plain, formed concrete or prefabricated, or can be composite linings for specialty applications.

3.4.2.1 Formed-in-place Concrete

Plain Concrete is the most common most economical, and most successful material for lining all types of shafts. The concrete that is used should be of good quality with a low water-cement (w/c) ratio. Concrete has a high compressive strength, the most important property for lining structures. Compressive strengths of concrete used for shaft linings may be as high as 8,000 psi.

The choice of concrete as shaft lining material is easily justified. Relatively small areas of wall can be constructed systematically in phase with the shaft sinking process. Transport to any position in the shaft is simple by means of pipeline or bucket. Concrete is a convenient material to handle and place within the limited confines of a shaft. When placed the concrete moulds itself to the excavated profile of the shaft providing and interlocking action with the surrounding rock. Testing procedures for concrete quality control are straight forward. Resistance to sulfate attack is achieved by using sulfate resistant Portland cement (Type V), or by adding flyash, producing a structure which requires little maintenance. Unreinforced concrete can withstand most loading conditions normally encountered and can, with care produce an essentially dry shaft. The benefits of using concrete, particularly unreinforced concrete, for shaft linings are therefore substantial.

During installation of the concrete lining, attention should be given to volume change and shrinkage. Internal stresses caused by expansion and shrinkage result in cracks which may exceed acceptable limits. At the Wistow shafts at Selby, the water temperature of the concrete was themostatically controlled to assure proper curing conditions (Tunnicliffe and Keeble, 1981). A knowledge and understanding of the technology of pouring and curing concrete are essential for meeting the design quality of the concrete.

Plain concrete shaft linings are not watertight and should never be considered as such. Even so, some monolithic concrete structures under low hydrostatic water pressure may prevent water seepage. In concrete-lined shafts, water under a pressure of a few hundred psi will seep through both the concrete itself and through the construction joints. However with good construction techniques, backsheets where required, water stops between concrete pours, and backwall injection, the majority of water seepage can be prevented.

In some instances, particularly in competent, reasonably dry shafts, it is possible to slipform the concrete lining upwards from the shaft bottom after having completed the excavation. In China, some shafts are stepformed from the top down and then slipformed from the bottom up, followed by grouting of the annualar space (Harrison, 1986). Slip forming techniques or leapfrogging of forms allows the concrete to be poured monolithically thus avoiding the necessity for construction joints. Care must be taken to avoid the formation of shrinkage cracks when using this method particularly when the walls are thick and a great deal of heat from the hydration of the concrete is formed.

<u>Reinforced concrete</u> has been used with great success for shaft linings in modern applications, steel-reinforced concrete offers versatility, excellent strength properties, and good economics. The strength and elastic properties of reinforced concrete are well known in the field of structural design and the relevant design criteria can be readily adopted for shaft structures.

Reinforced concrete lining should be used where the degree of nonuniformity of the rock pressure is high and tensile stresses caused by bending are anticipated. For the same reason, reinforced concrete should be used where longitudinal forces along the lining tube may occur.

A modification of the standard reinforced concrete lining has been used to accommodate vertical and horizontal movements without severely damaging the shaft lining.

This consists of 15-ft long independent concrete rings which are separated by a 12-in. (vertical) gap against the rock wall. The gap allows any horizontal or vertical displacement to take place and ensures better repair conditions. The rings require high structural strength and so they are heavily reinforced with steel bars.

As with the plain concrete lining, the reinforced concrete lining is not waterproof. However, the same precautions may be taken as used for the plain concrete, to keep water leakage to a minimum.

Figure 14 (previous section) illustrates the basic elements involved in the lining of shafts with concrete. Lining is normally carried out downwards, as sinking proceeds. The face of the excavation in only advanced as far ahead of the lining as the ground conditions allow, the sides being secured by temporary support as necessary. Abel et al. (1979) found that an elastically shielded zone may exist for about 2.9 shaft radii upwards from the shaft bottom. Placement of the lining above this zone will ensure that "green" concrete is not subjected to the full elastic load redistribution. However sloughage and dilation of the rock within this zone can require close proximity of an accelerated concrete to the shaft bottom.

Pours are normally 10 - 20 ft, except in unusually good ground. In wet conditions, backsheets are installed against the rock, for the concrete to be cast against and are used to channel the water behind. The outflow is controlled via pipes to the relief holes in the formwork for pumping to surface.

All sinking, lining and formwork handling operations in the shaft are carried out from a multi-deck scaffold (galloway) suspended in the shaft from the headframe. Winches mounted on the surface raise or lower the scaffold.

Shaft lining formwork is supported, during concrete pouring, by means of hanging rods suspended from the lift above. These rods eventually are covered by concrete. A kerb ring, once it is lined and leveled, acts as a stop-end to the bottom of the lift being constructed and provides a seating upon which to lower the complete set of formwork rings from the previous pour. U.S. practice is to use expanded-metal mesh at the kerb base, which provides a rough surface to match the next pour below. Other blockout materials include sand bags and plywood. The lowering of the formwork is achieved by means of chainfalls or winches on the scaffold or by winches mounted on the surface.

Figure 18 illustrates the concrete lining sequence. As many as 8 pour doors in the formwork assist with the placing of each lift of concrete and the specially-shaped joint permits matching up to the previous pour. Some designs call for troughs at the point of entry in the kerb ring or blockouts in the kerb itself, to provide positive pressure on the pour to complete the contact with the underside of the pour above. Concrete is transported from the surface either by slickline, via a dashpot and distribution box or by concrete bucket. A remixer hopper maybe used in the subsurface. Passage from the distribution box or the bucket into the formwork is by means of flexible hoses, called "elephant trunks".

Discontinuous lining may be step-formed or jumped-formed. The difference is that in jump-forming, there are multiple forms that are leap frogged downwards, resulting in more rapid progress.

Some minor differences exist in the method of lining construction from country to country or even from shaft to shaft, but the method described above is generally accepted. For example, in South Africa and Australia it is more usual to use chains to suspend the kerb ring rather than hanging rods.

In reinforced concrete linings the steel reinforcing is usually delivered down the shaft in mats of rebar of a size which are easily handled by two men. After the lowering of the kerb ring and prior to lowering the forms, the mats are wired into place (usually attached to the hanging rods). The pour then proceeds as for an unreinforced lining. Concrete may be vibrated if reinforcement is used to prevent air gaps and promote good bonding to the reinforcement.

Slipforming, as mentioned previously, is done at present, from the bottom up, and only as shaft wall, conditions permit. Temporary support may be required by regulations. Most slipforming is single-wall, meaning the rock provides the second source of restraint for the concrete. In double-wall slipforming, two concentric forms are used, leaving an open annulus adjacent to the rock. This space can be backfilled later with a compressible material such as crushed salt or gravel, a viscous sealant such as asphalt, or grout. Wall stability for slipforming can be improved by pregrouting or freezing. In Section 4, the performance of double-slipformed shafts at the Weeks Island, LA, mine, is discussed.

Pond (1979) describes a typical downwards step-forming procedure used on the Monterey shafts. The concrete was placed in 20-ft lifts and allowed to set for 24 hours before the forms were stripped. In this case, the shaft had been raise-bored and no contemporaneous excavation was in progress in the shaft.

Ferrari (1966) describes the use of a grouted gravel backpacking in lieu of the formed concrete at the Thorne Colliery shaft in England.

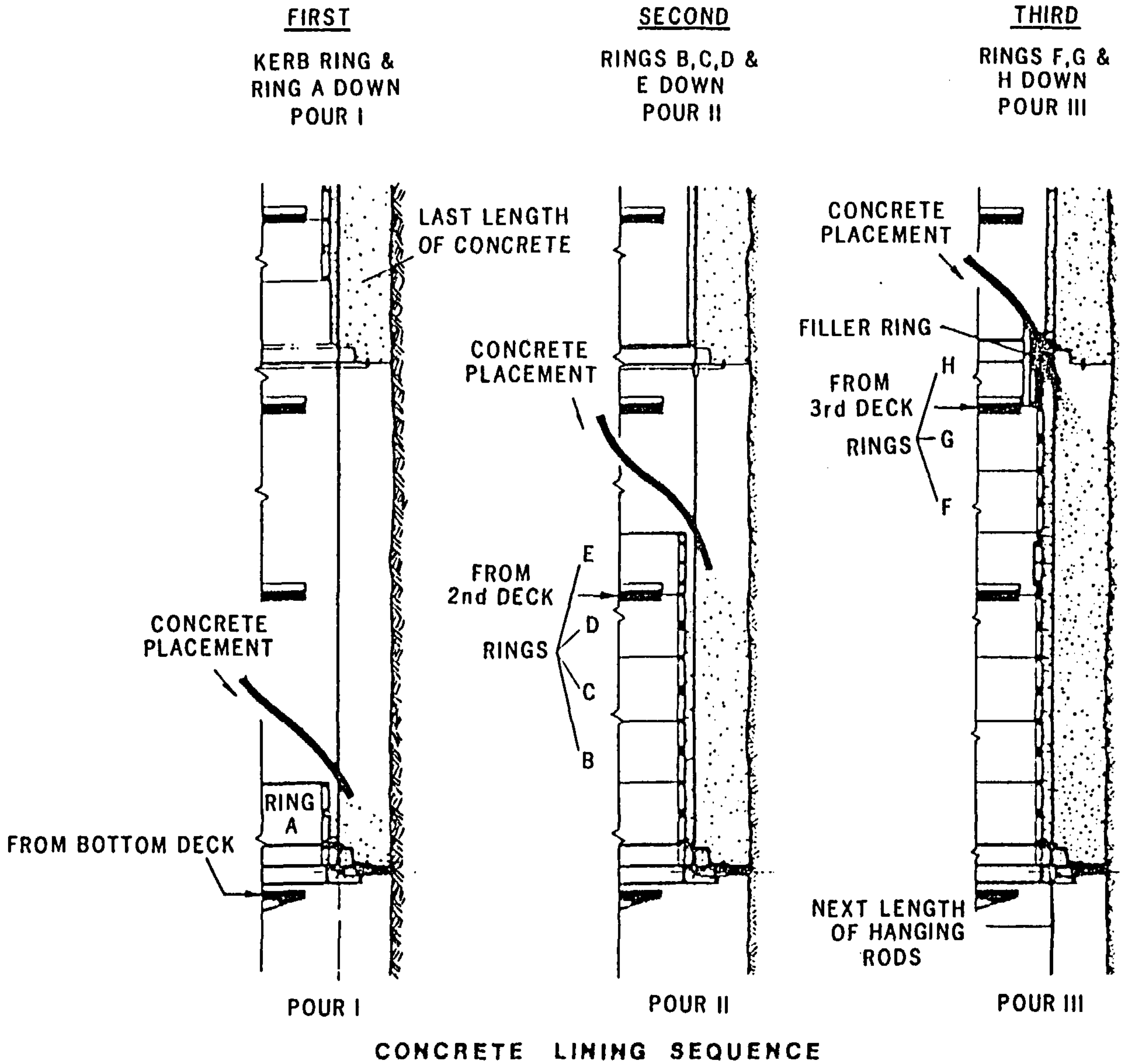


Figure 18. Conventional Shaft Lining Sequence
(after Auld, 1983b)

A Continuous Shaft Lining System (CSL) is under development by Foster-Miller, Inc. and Dravo Engineers (Torbin, 1982). The basic concept is to slope form the shaft lining downwards. The CSL is being developed to line shafts behind a Blind Shaft Borer (BSB).

The forms are similar to the conventional shaft-sinking forms. A lip type seal is used to seal the gap between the bottom of the forms and the rock wall. Concrete is pumped into the bottom of the forms and under pressure it spreads out and fills the complete void behind the forms. So far the system has not been used in a shaft but tests have demonstrated that a concrete lining can be placed by continuous slip forming from the top and progressing downwards.

3.4.2.2 Prefabricated Linings. Prefabricated linings components include precast segments, caissons jacked into place, hand-placed concrete block, tubbing, liner plate, and steel skins. Most of these are used in combination with other liner elements to form a composite lining, as discussed in the next section.

Precast concrete segments can be reinforced with steel fibers or fiberglass (Grieves, 1979). The Drumbo gypsum mine (Hartvikson, 1983) used a precast concrete and steel lining in a 12.5-ft i.d. shaft. Schlage and Smith (1981) discuss procedures for joining precast segments at the surface and floating them into place. Pakes (1976) describes the use of precast sections in a shallow 25-ft-diameter shaft in Edinburgh.

The use of concrete blocks for shaft linings is not typical and is limited to plastic or semi-plastic rock conditions where the behavior of the excavation wall is unpredictable in the magnitude of creep and unsymmetrical deformation. The outer ring of block at the Prosper 10 shaft was designed to withstand 3.9 in. (10 cm) of radial convergence before loading the outer liner (Hausler, 1972). Concrete block linings in these conditions create a flexible supporting ring. This type of lining cannot be consisdered permanent because of its vulnerability to displacement and deformation. It is used as a primary lining in German composite linings to bear earth pressures from weak rock and thawing, and is detailed by Stoss and Braun (1983). Standard structural design procedures cannot be applied for concrete block lining. The dimensions and strength of the blocks can be determined only from previous experience. The blocks used should be tapered and made of high-quality concrete since they are exposed to point loads which result from the deformation of the lining. They are held together mainly by friction and so the use of mortar to bind them together is of secondary importance. They are placed in lifts held in place by a compression ring. Chipwood boards 3/8- to 1/2-in. thick, inserted between the blocks, make a resilient structure. The Voerde outer shaft lining used a hot-pressed flax and glue pressboard for optimum flexibility even when submerged. The space between the excavation wall and the concrete blocks (approximately 2 in.) can be filled with sanded grout.

Recently, Deilmann-Haniel of West Germany introduced a new type of self-locking concrete block. These blocks are installed simultaneously with the shaft excavation by suspending them from the blocks above (Figure 19). This type of block may have wide application for lining drilled shafts.

The Cote Blanche service shaft in the Gulf Coast was built inside 8-ft-diameter caissons jacked through weak, water bearing sediments (ONWI-255). The caissons were fitted with a cutting shoe at the lower edge. The caissons remained as the primary lining. The relief valves in the non-salt sections occasionally show leakage.

Alignment is a problem with this method and the depth limit is probably 200 ft. Some water inflows up through the bottom of the caissons probably can be controlled with compressed air.

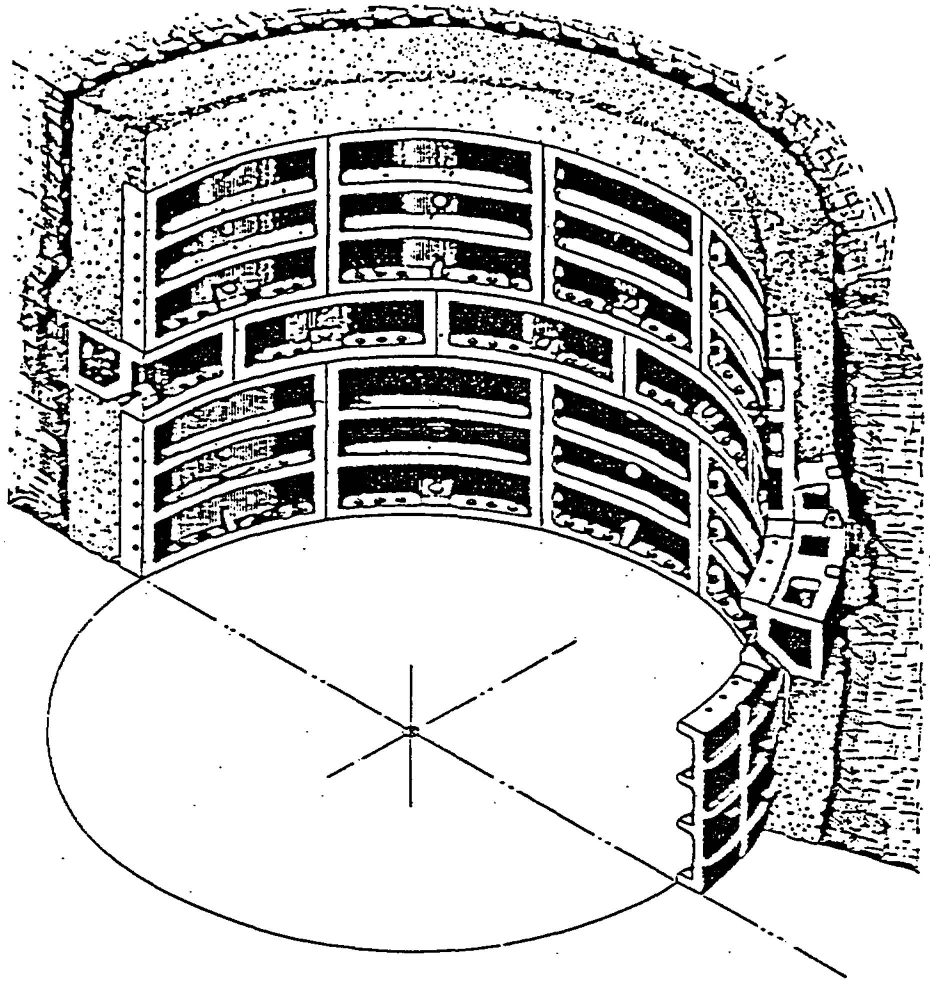
Tubbing plate (Figure 20) is made of pre-formed, cast iron sections that are bolted together with lead or resin gaskets. It is normally installed as the shaft is deepened. When sufficient excavation has been completed to make room for the next ring of tubbing plate, a leveling or jacking ring is installed on the shaft sump. The tubbing ring, complete with the lead gaskets, is constructed on top of this leveling ring and then jacked up and bolted to the ring of liner plate immediately above.

The leveling ring also acts as a stop-end to the rock wall at the bottom of the tubbing plate. After scribing to the wall, concrete is poured behind the tubbing plate. The entire tubbing zone is pressure grouted later to effect the filling of voids. Tubbing has been the historically-preferred method of lining water-bearing strata. Most of the Saskatchewan potash shafts use tubbing in the Blairmore section.

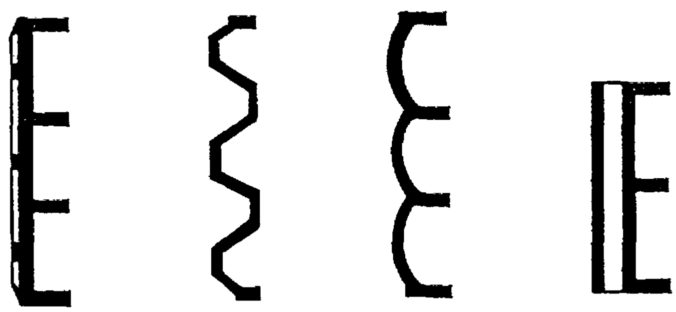
Other methods have been used to install tubbing where a long length of shaft was excavated making room for several rings of tubbing plate. The jacking-ring is located very accurately at a predetermined distance below the last installed ring of tubbing. Several rings of the tubbing are then built up on top of this until the top one can be bolted to the lowest ring previously installed. The whole length of tubbing is then concreted. Whether or not this system can be used in any particular situation will obviously depend upon the existing ground conditions.

Tubbing requires maintenance. Most tubbing eventually begins to leak and the bolts must be tightened. Historically watertight lining sections have been used in conjunction with picotage seals. In competent, dry rock, above and below the aquifer section, a wedge ring is installed with a seal of some kind to be installed later between betweem the ring and the aquifer. The picotage (Figure 21) consists of wooden wedges and steel needles that are forced tightly into the annulus the wedge ring and the rock. The transfer of weight

Figure 19. Self Locking Concrete Blocks



a) Shows wedge ring bearing support, picotage, and concrete backfilling



b) Typical tubbing sections

Figure 20. Cast Iron Tubbing Lining

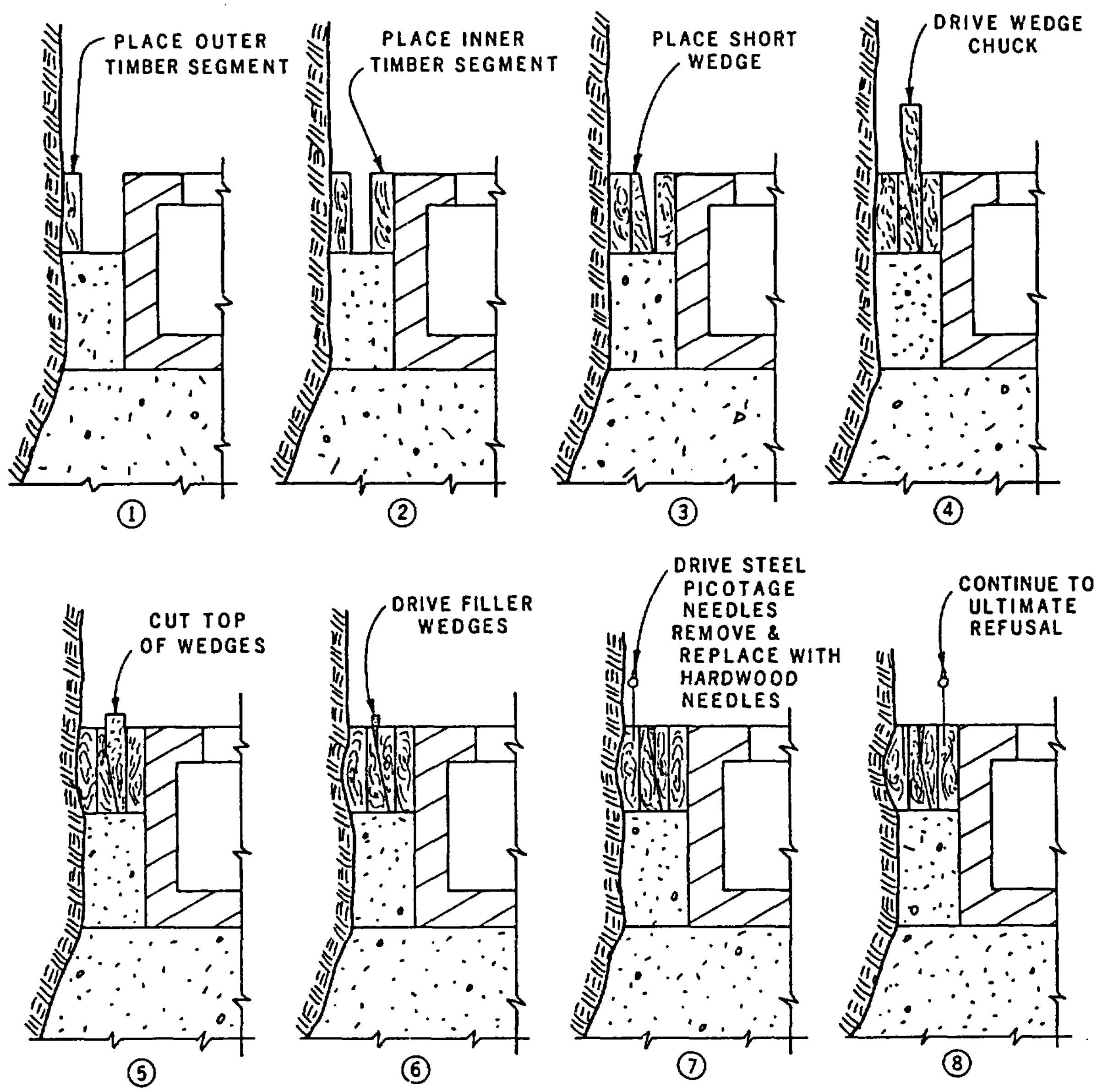


Figure 21. Construction of Picotage

from the wedge ring to the rock is thus effected and the picotage, which swells on contact with water, forms a seal. Picotage takes about 2 days to complete if done correctly, by experienced miners.

3.4.2.3. Composite Linings. In shafts used for the exploitation of evaporate deposits minimal water inflow is imperative. Watertightness is desired in other shafts to prevent ice buildup or for other reasons. True watertightness is difficult to achieve because water may enter the shaft through the lining, and also at the top and bottom of the watertight lining sections. Therefore, the lining itself, any micro-annulus formed during the construction and to some extent the rock wall, must be watertight.

As previously mentioned, a plain concrete lining is not watertight. The following general types of composite shaft linings have, therefore, been developed in order to achieve a watertight lining with adequate structural strength:

- steel and concrete
- reinforced concrete with bitumen envelope
- cast iron tubbing with concrete envelope.

Steel and Concrete lining may consist of the following variations:

1) A plain or reinforced concrete lining with a steel sheet welded in place on the outside. Cement grout is place between the steel skin and temporary lining (Figure 22).

2) A welded steel sheet skin with stiffener rings on the inside of a plain or reinforced concrete lining. Cement grout is placed between the steel skin and temporary lining (Figure 23).

3) A double steel lining consisting of two welded steel sheet skins inside and outside with concrete placed between them (Figure 24). A good example is the Alwinsal Shaft, described by Link (1971). Cement grout is placed between the outside steel skin and temporary lining. By substituting bitumen for the cement grout a watertight lining which is also resistant to vertical rock movement may be constructed.

   To prevent buckling of the inside steel skin, should leakage occur through the outside steel sheet skin, a special reinforcement or anchorage system for the inside skin must be provided.

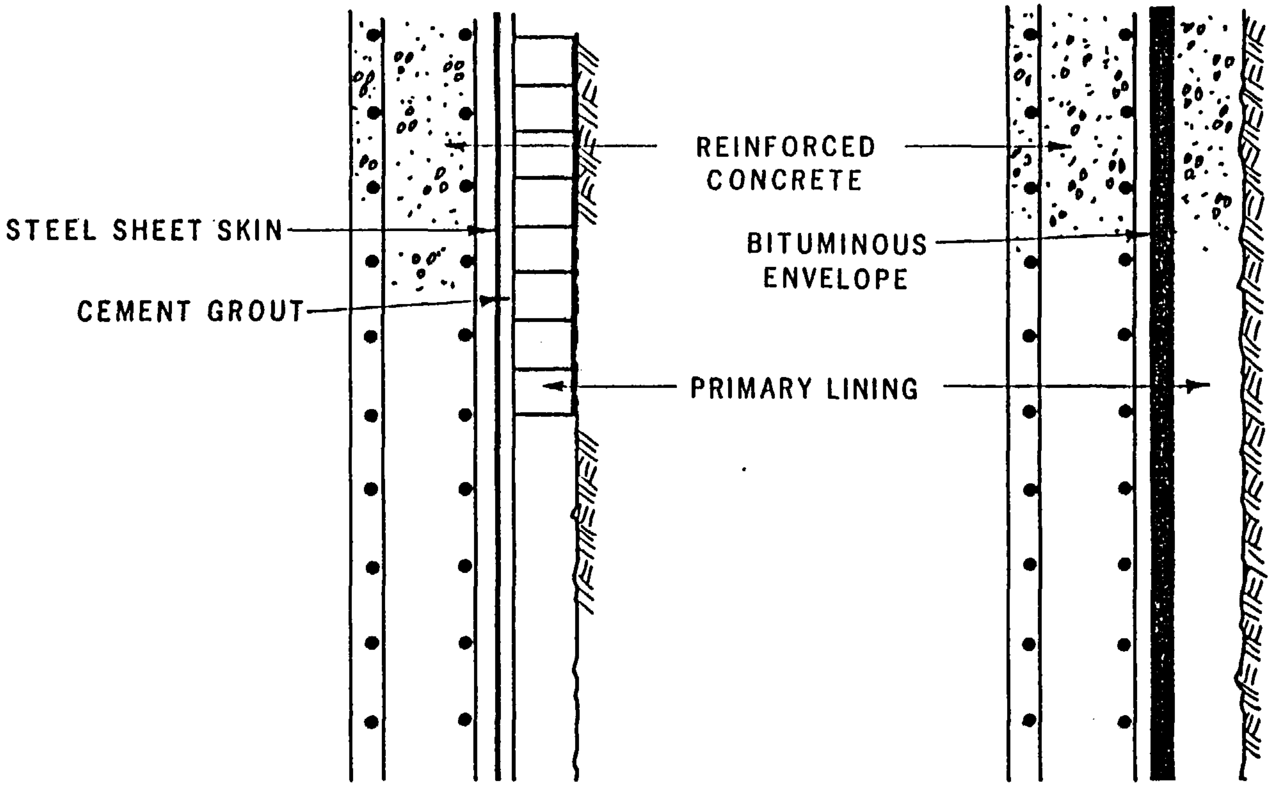


Figure 22. Composite Lining Concepts.
These use a grout interlayer (left)
and bituminous gliding layer (right)
(after Ostrowski, 1972)

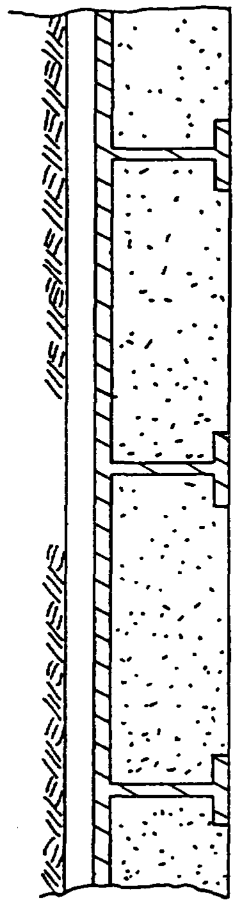


Figure 23. Watertight Lining Single Steel Sheet Skin with Stiffner Rings on the Inside

Absolute watertightness of the above linings as far as preventing water ingress into the shaft is achieved by the steel sheet skin that is butt-welded in place. The outside surface of the steel linings is not exposed to significant corrosion forces. Free oxygen in the water immediately behind the lining reacts with the steel plate, but a fresh influx of oxygen-bearing water is prevented by the watertightness of the steel skin. Without a supply of oxygen, corrosion cannot progress.

Vertical water migration in any micro-annulus formed within the lining is achieved by grouting the pre-formed annulus between the steel and the temporary support. In some cases where this type of lining has been constructed, the micro-annulus between temporary lining and the rock apparently was not grouted, although there is no reason why it could not have been. The shaft lining seals (picotage, chemical seal, or grouted seal) were relied upon to revent any vertical movement of water behind the lining.

For shafts using this type of lining and depending upon the existing ground conditions, it is usually excavated through the zone requiring the composite lining, by filling or grouting and with a temporary lining (either rockbolts and mesh, concrete or shotcrete). The final lining is then installed from the bottom upwards.

Starting from a wedge ring or key (Figure 24) accurately installed and adequately founded at the bottom of the composite lining section, the steel lining is built up and the pre-radiused sections welded together. Automatic welders are used and ultrasonic testing of the welds is carried out. As the steel lining progresses, the concreting follows up below and the grout filling of the annulus between the steel and temporary lining is carried out over the top of each completed steel lining length. All the operations are carried out simultaneously from the stage so that the completed lining is installed with one pass of the stage through the shaft. This requires a very long stage with many decks to accommodate the various activities which are being performed.

If a double steel lining is used, then the outer lining is installed from the upper decks of the stage, and the annulus between it and temporary lining is filled. The inner steel lining is installed at the bottom of the stage, and the annular gap between steel cylinders is filled from the middle decks.

The Beatrix shaft lining, described by Weehuizen (1960) (Figure 25), is a steel-and-concrete composite. The steel portions are liner plate bolted together. The lining was floated into place and positioned on a concrete plug at the hole bottom. A bitumen backing was poured behind it. The lining is 22-ft, 4-in. i.d.

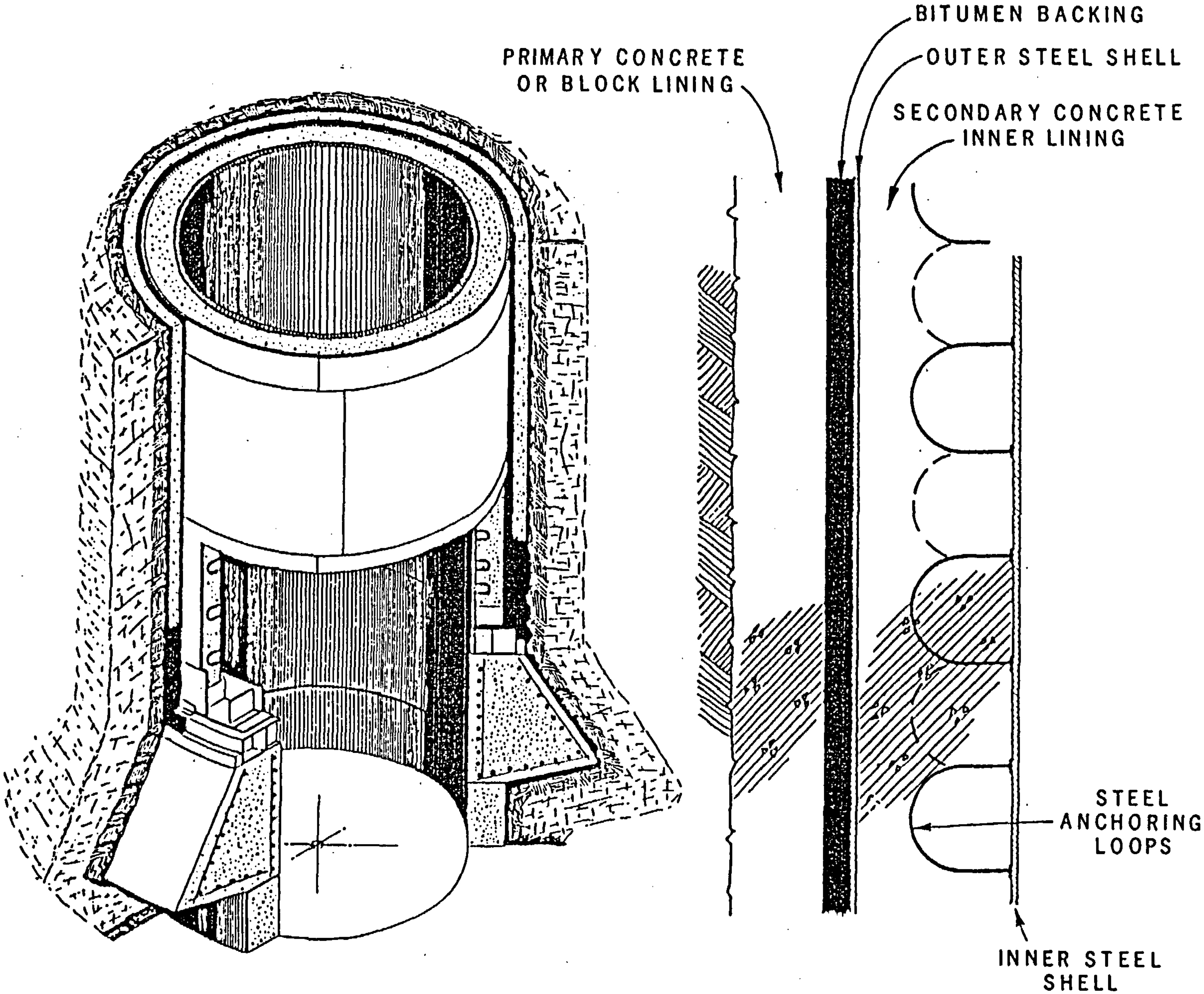


Figure 24. Composite Lining Section and Key Detail
(from Ostrowski, 1972)
Left - Key Detail
Right - Typical Lining Section

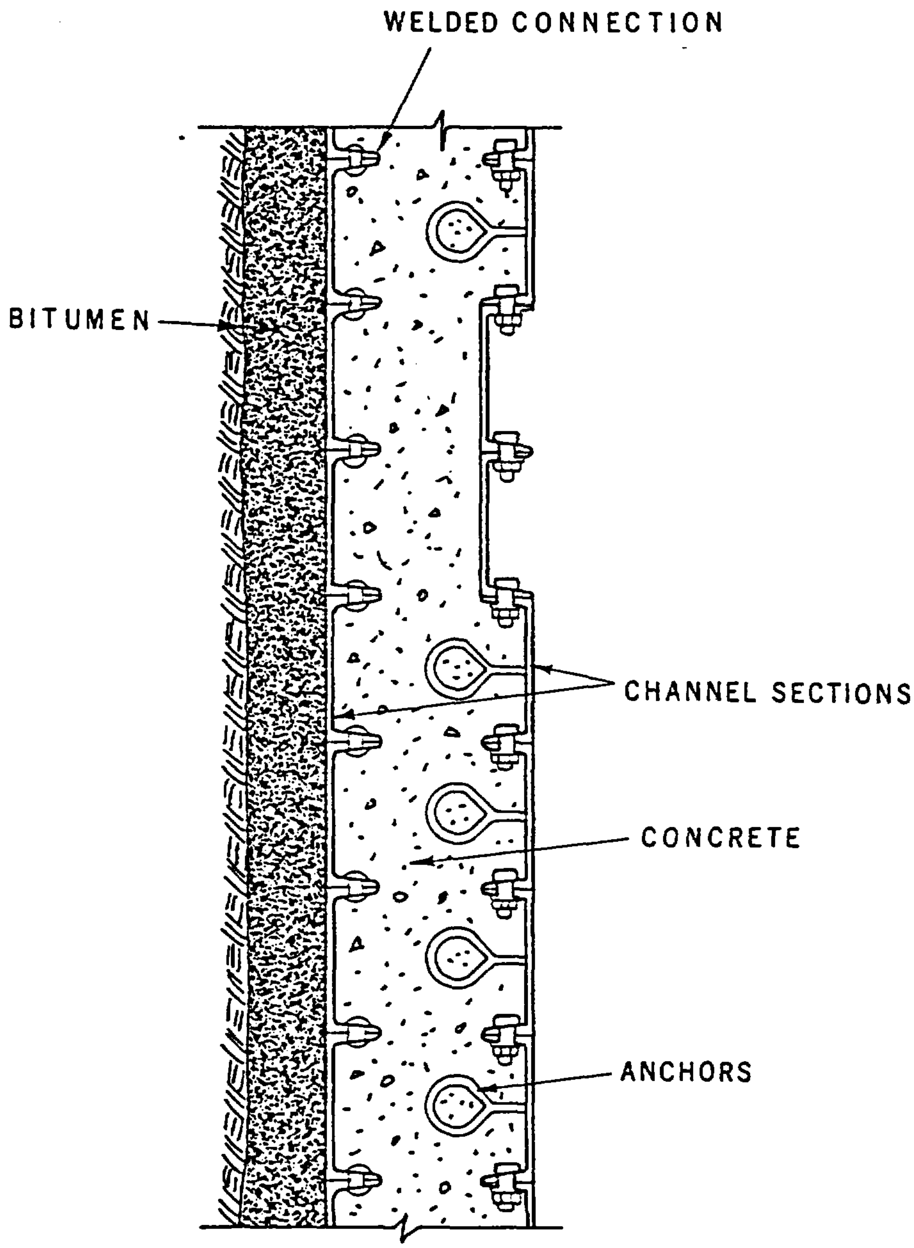


Figure 25. Construction of Floated-In-Place Composite Lining used on the Beatrix Shaft

Reinforced Concrete with Bitumen or Asphalt Envelope (Figure 26) is used if the shaft lining is required to withstand significant displacement. In this case hot bitumen would be pumped into the annular gap between the outer steel lining and the temporary lining. The Voerde shaft was designed to remain watertight in bending on a 9,840-ft (3,000-m) radius (Hegemann, 1981). The watertightness of this lining is achieved through the application of a bituminous envelope. These envelopes normally are 2 - 9 in. thick. The bituminous envelope can be installed in a from of frozen block welded together with a hot plate, or more commonly, poured in place in lifts as a hot mass into the gap between the primary and final lining tubes. The mix is typically placed at about 176°F (80°C) but is much hotter when mixed (302°F i.e. 150°C at Wulfen) (Kampschulte, Lehmann and Link, 1964). The asphalt is usually mixed with limestone flour to a density of 1.3 to be sure that hydrostatic heads are overcome. An exception was the Voerde shaft, where the lining design dictated a lighter (1.05 density) mix. As the bitumen cools in place, it contracts, and must be replenished. Normally there is a seal of some kind (asphalt-sand) at the bottom of the poured annulus to prevent asphalt leakage after pouring.

This type of lining has been used with great success in Holland and Germany as it is both watertight and allows some limited vertical rock movement without affecting the main lining structure. Braun and Stoss (1983) give an excellent description of this type of construction at the Halten Nos. 1 and 2 shafts in Germany. Owing to its flow characteristics, bitumen will inevitably penetrate all empty or fluid filled pours or cracks thereby acting as an active and automatic sealing medium. It also acts as a gliding layer to absorb ground movement due to nearby extraction. Shafts constructed this way in Germany include the Prosper No. 10 (Hausler, 1972), the Haltern Nos. 1 and 2 (Hegemann and Jessberger 1985), and Wulfen (Kampschulte, Lehemann and Link, 1964).

Each of these shafts was placed through frozen ground, and withstands the resulting ground movements. These types of linings have no bond to the rock and are designed to stand free in the shaft (Figure 27).

Cast Iron Tubbing with Concrete Envelope was first installed nearly 200 years ago in Great Britain and then later developed in Germany. This type of lining was designed for heavy water conditions in unstable rock. It has been the historical standard lining in frozen ground (Ostrowski, 1972) and has only recently been replaced by composite linings of the type discussed above. Various type of tubbing sections have been used in shaft linings, however for watertightness between the segments they all rely on the installation of a 1/8-in. lead or resin gasket between the flanges. Tubbing segments are held together by bolts and special lead washers.

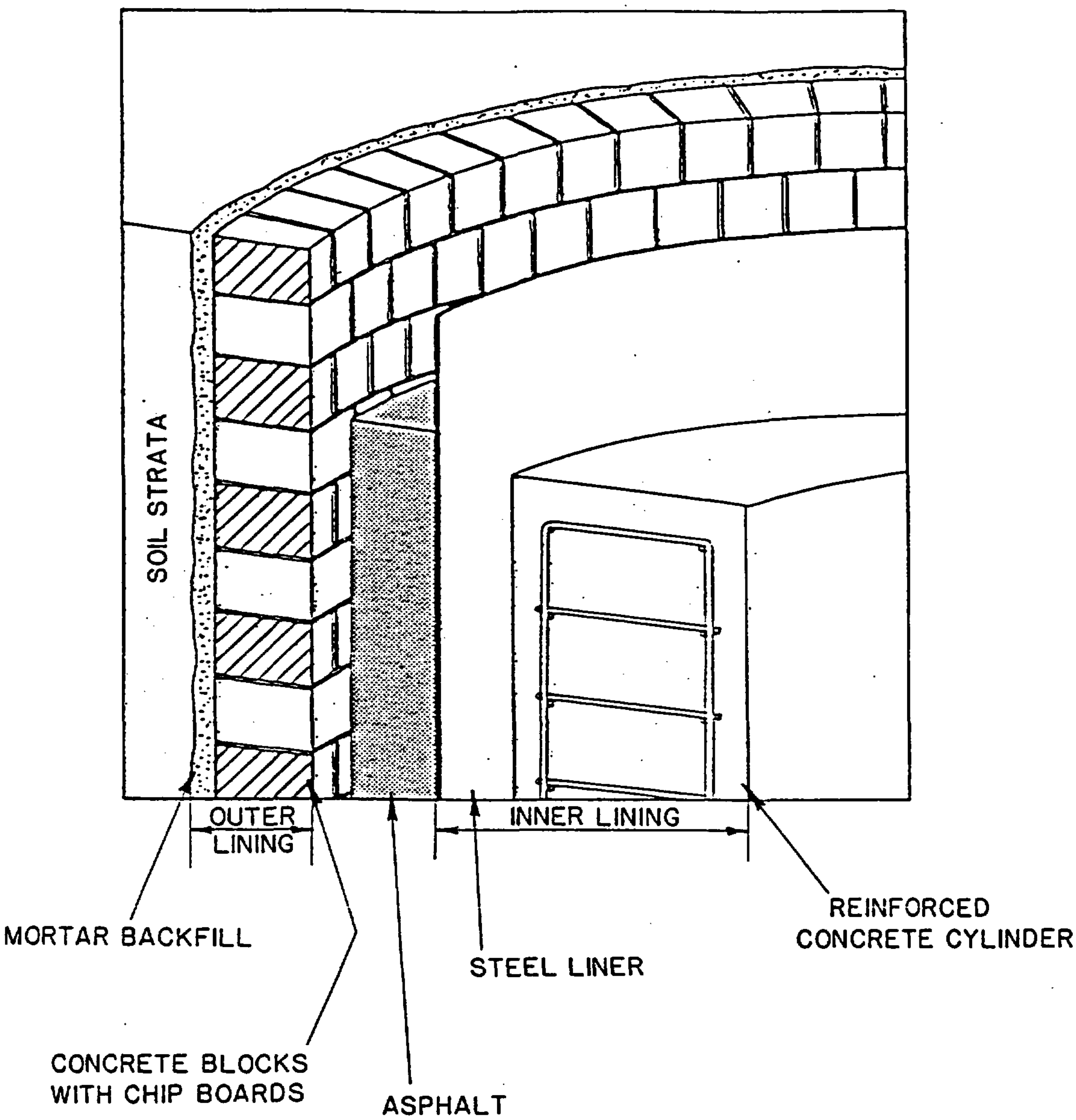


Figure 26. Composite Lining Designed for Flexibility
(from Stoss and Braun, 1983)

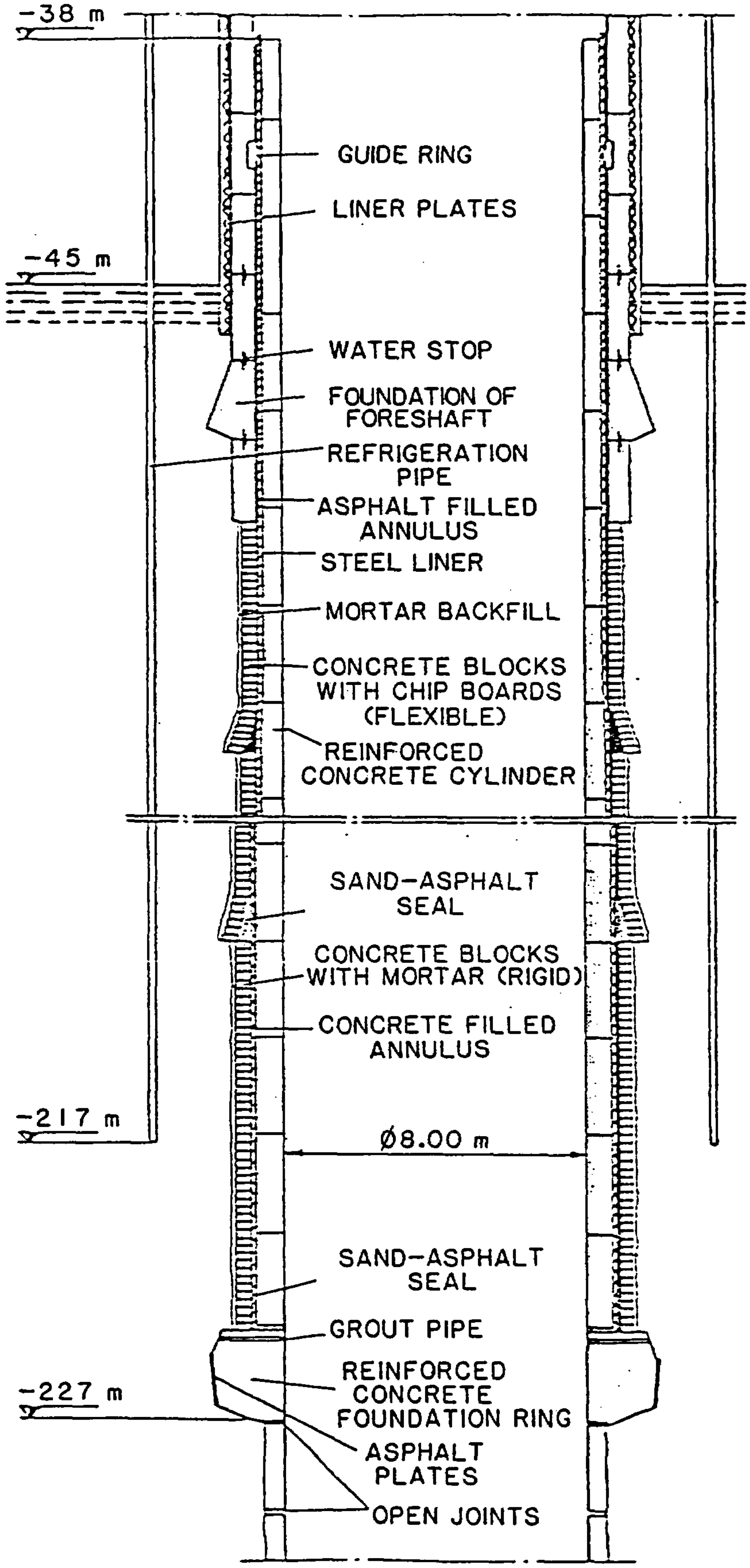


Figure 27. Shaft Design in Ground Stabilized by Freezing (after Stoss and Braun, 1983)

A pea gravel concrete envelope surrounds the cast-iron tubbing and fills the gap between it and the excavation wall. The concrete thickness varies from 12 to 24 inches depending on the method of installation. A thinner concrete envelope results with the suspended method of tubbing installation; the upward long-section method permits the use of a thicker envelope. Differential thermal responses of rock, tubbing, and concrete can interrupt bonding. The rock concrete contract is grouted through ports in the tubbing plate to ensure that all the voids are filled. Shaft lining seals are installed to prevent the vertical movement of water.

3.4.3 Summary of State-of-the-Art Lining System

A summary of the various shaft linings together with their advantages and disadvantages is given in Table 11.

It is important to note that although some of the linings are watertight, they are not necessarily guaranteed to prevent water movement within one or the other micro-annulus formed during construction. The closest approach to achieving this seal against any vertical movement is when a bitumen envelope is cast against the rock and the main lining is one integral unit.

All the linings require shaft seals to guarantee the prevention of large scale movement of water from aquifers.

3.5 GROUTING OF LINED SHAFTS

The grouting of lined shafts can be done during construction (pre-completion) as a planned operation, or in response to contingencies, or it can be done as remedial activity to arrest leakage after the shaft has been in operation. The need for remedial grouting either during or after construction evidences voids and is therefore taken up by means of example case studies. The operation where only the liner is involved, uses most of the principles described below. This section will deal with planned grouting of liners to fill voids and prevent water migration. This is known as drywall grouting or backwall injection. It may or may not include the grouting of backsheets (described in Section 3.3). The use of backsheets increases the chance of an effective seal. In some composite linings a planned annulus is left open for later grouting.

Techniques of placing a concrete lining in a conventionally sunk shaft, in water-bearing strata (where the water-make is within pumpable limits) entail the use of backsheets, which may be corrugated steel, plastic, or spun bonded polypropylene sheets, placed against the rock face prior to pouring the concrete. This allows the free water to track down in the cavity so formed between the backsheets and the rock, thus keeping the water out of the fluid concrete during the placing of the lining.

Table 11: Summary of State-of-the Art Linings

Lining System	Application	Advantages	Disadvantages
1.1 Plain Concrete - Pour from top downwards	Conventional and Drilled Shafts	a. Cheapest lining system available. b. Suitable for most shafts in relatively good ground conditions. c. Ideally suited to shaft sinking cycle and shaft environment.	a. Unsuited to withstand ground movements. b. Wall thickness becomes un-economic to withstand full hydrostatic ground water pressures below 2,000 ft. depth except when using specialized c. Not waterproof. d. Requires access to shaft. e. Does not ensure no water movement behind lining.
1.2 Pour from bottom upwards		d. No construction joints to leak.	f. Care required to prevent cracking of lining.
2. Reinforced Concrete	Conventional and Drilled Shafts		
a. Pour from top downwards		a. Cheap and effective for most ground conditions; can withstand some localized tensile bending stresses. b. As 1(c) above.	a. As 1(c) above. b. As 1(b) above. c. As 1(c) above. d. As 1(d) above. e. As 1(e) above.
b. Pour from bottom upwards		c. As in 1(d) above.	g. As in 1(f) above.

Table 11 (continued)

Lining System	Application	Advantages	Disadvantages
b. Pour from bottom upwards		c. As in 1(d) above.	e. As 1(e) above. g. As in 1(f) above.
3. Concrete block	Conventional and Drilled Shafts		a. High cost labor intensive. b. Limited to plastic or semi-plastic rock conditions. c. Cannot be considered permanent. d. As in 1(c) above. e. As in 1(d) above. f. As in 1(e) above.
4.1 Single Steel Plates and Concrete Composite	Conventional Shafts	a. Watertight.	a. High cost of installation welding expensive. b. Installation is slow. c. Requires installation from bottom upward.
4.2 Double Steel Plate and Concrete Composite	Conventional Shafts	a. Watertight. b. Very strong.	a. Very high cost of installation, welding very expensive. b. Installation is very slow. c. As 4.1(c) above. d. As 1(e) above.

Table 11 (continued)

Lining System	Application	Advantages	Disadvantages
5. Application of bituminous envelope between main lining structure and rock	Conventional and Drilled Shafts	a. Allows some rock movements to take place without damaging lining. b. Other advantages of main lining type apply.	a. Additional cost. b. Other disadvantages of main lining type still apply. c. Requires heavy lining foundation. d. As in 1(d) above. e. As in 1(e) above.
6. Cast Iron tubbing with concrete	Conventional Shafts	a. Suitable in unconsolidated or poorly consolidated ground where freezing is required for sinking. b. Is essentially watertight.	a. High cost. b. Requires continual maintenance to minimize water leakage. c. Installation is slow. d. As in 1(e) above.
7. Thin steel lining with drainage	Drilled Shafts	a. Relatively cheap. b. Does not require access to shaft.	a. Only suitable for competent ground. b. As in 1(e) above. c. As in 1(c) above.
8. Shotcrete Lining	Conventional or Drilled Shafts	a. Relatively cheap.	a. Only suitable for competent dry ground. b. As in 1(d) above. c. As in 1(e) above. d. As in 1(c) above.

Table 11 (continued)

Lining System	Application	Advantages	Disadvantages
9. Fabricated Steel Tube with Ribs	Drilled Shafts	a. Watertight. b. Strong.	a. Only suitable for competent dry ground. b. As in 1(d) above. c. As in 1(e) above. d. As in 1(c) above.
10. Double-steel Lining with Concrete Between	Drilled Shafts	a. Watertight. b. Very strong and flexible.	a. Very expensive. b. As in 1(e) above. c. As in 1(c) above.
11. Precast Reinforced Lining using Precast Rings	Drilled Shafts	a. Cheaper than fabricated liners with ribs for larger diameter shafts.	a. Not watertight. b. As in 1(e) above. c. As in 9(c) above. d. Most suitable for larger diameter shafts.
12. Slip Formed Downwards	Bored Shafts	a. Potentially capable of rapid production rates.	a. Untried. b. As in 1(a) above. c. Cannot change wall thickness. d. As in 1(c) above. e. As in 1(e) above. f. As in 1(f) above. g. Cannot be reinforced with steel.

To control the subsequent movement of water behind the lining and the leakage of water from the backwall cavity through weaknesses in the lining; (principally at the matcher joint between lengths of lining, where shrinkage of the concrete can cause a substantial gap, in cold joints, and in areas of honeycombed concrete) it is necessary to fill the cavity. This is usually carried out, using an appropriate cement grout mix, through ports that are either cast into the lining during the placing operations or are drilled through the set concrete. Grout ports (Figure 28) can be used in affording backwall grouting of steel casings. These ports serve two purposes; initially they are used to drain water into the shaft from behind the lining and subsequently to conduct the grout into the backwall cavity.

Backwall grouting can commence after the concrete in a particular length of lining has adequately cured, but depending on shaft water makes is normally left as long as possible to allow the maximum concrete shrinkage to take place prior to the injection.

To control the spread of grout and ensure that it moves upward, a seal must be formed at the bottom of the lining length to be grouted. This seal may be formed either by constructing a regular shaft seal (Sections 5.7), by grouting off a "French drain" or more simply, where possible, by pouring the lining against the dry rock located below the aquifer while the water is being controlled through drain holes above. Normally "French drains," gravel filled annular cavities, would be constructed against the rock to collect any large quantities of water moving behind the backsheets (which had not already issued through drain holes) and divert it through the lining. Grouting a French drain and directing the water to upper drain holes forms a seal behind the lining and allows backwall injection of the lining above the French drain to be carried out.

Backwall injection proceeds from the bottom ring of the lining length. As a grout approaching the mix consistency shows at each successive upper ring of holes, the injection points are moved up to that ring of holes and water relief is diverted to the next upper rings of holes until the lining length is fully grouted. Matcher joints, or cold joints in the lining which leak grout are caulked with oakum, non-shrink grout, or lead wool so that grout flow is reduced at these locations.

This procedure is repeated at the appropriate time for each lining length until the whole affected lining length has been treated. If complete sealing is not achieved it may be necessary to redrill the holes and repeat the whole sequence at higher grouting pressures using less viscous grouts, or even chemical grouts until leakage or water flow behind the lining is essentially completely eliminated. Identification of incomplete sealing is dependent on observations, either visually, or with instruments to measure water pressure. A certain amount of leakage can escape detection by these

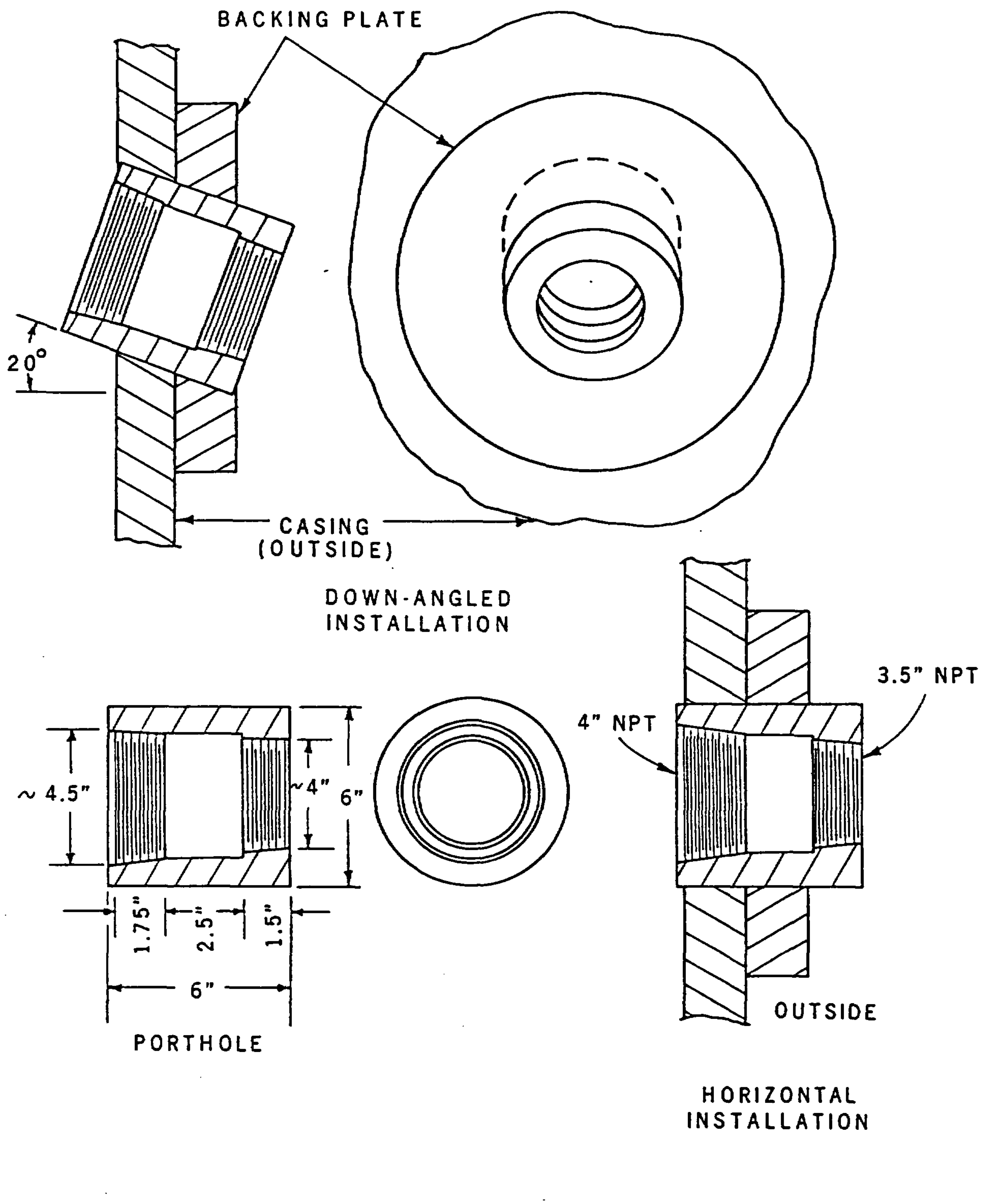


Figure 28. Grout/Porthole Concept for the Steel Shaft Casing at the BWIP Exploratory Shaft (from SD-BWI-TI-119, Basalt Waste Isolation Project)

means. The need for repeat drillings and injections is attributed to bleed incomplete penetration and to a lesser extent, shrinkage of the grout. This problem can be solved very largely by the use of the correct mix for the grout.

Ideally the injection of a non-bleed grout with a suitable shear strength into the lining backwall cavity will form a completely void-free seal. Even so, experience has indicated that it may be possible to backwall grout the lining so that leakage and water movement is a minimum, but after a period of time (usually several years) water leakage will increase. Some instances of this are given in Section 4. This is particularly true for shafts which are sunk for the exploitation of evaporate deposits, where essentially zero-flow conditions are required. This increase in water flow is almost undoubtedly contributed to by relaxation due to rock and lining creep deformation forming a micro-annulus between the concrete and rock through which water can flow. Furthermore, grouts are not impermeable, they can shrink with age and be attacked by formation fluids, or crack and shear according to the creep deformation. Also, leakage is a time-dependent process that begins as soon as the former conduits are shut off.

In frozen shafts, leakage commonly begins immediately after thawing. Tunnicliffe and Keeble (1981) advocate three episodes of backwall grouting; after liner installation but before it freezes, during ice wall recession, and after complete thawing. A single injection against ground colder than 40°F (4°C) is not likely to be effective.

3.5.1 Grout Mix Design

The requirement of a backwall grout is to completely fill the lining-rock interface gap in order to ensure:

- an even distribution of stress around the lining. Hanis (1982) describes the use of grouting to equalize anisotropic horizontal stresses
- the prevention of leakage of water through discontinuities in the lining
- the prevention of water migration behind the lining.

The grout is therefore basically cavity-filling grout, not a penetrating grout and should be capable of displacing water in the backwall cavity evenly upwards to give a complete seal after a single application. Later applications may require more emphasis on penetrability. Annett (1969) describes a multi-phase program using cement, followed by silicate grout, and concluding with lignite injection, in the same holes. The grout will be required, particularly in wide cavities, to have a moderately high shear strength in order to resist erosion, and in some cases to resist penetration by flowing water. This assumes a certain lack of fluidity as well as the absence of bleed in the grout.

It is important to distinguish between the grout properties required for the backwall grouting application and the grouting of fissures in rock. The latter grout is required to have maximum fluidity and particle dispersion, both of which are essentially incompatible with the major purpose of the backwall grout, which is to fill the cavity. If fissure grouting is required it must be directed into the rock mass rather than the cavity and should precede or follow backwall grouting. Nevertheless, the fluidity of the cavity filling grout is important, particularly in view of the likely configuration of the backwall cavity. Although commonly represented as an annular cavity, it may be more accurately considered due to rock surface irregularities as a series of near-isolated cavities. Thus to completely fill the backwall space, the grout will be required to penetrate relatively narrow openings during the injection process.

To investigate the backwall grouting process a large scale mock-up of a backwall cavity was constructed by Cementation Research Co. Ltd. This was used to investigate the flow and bleed properties of several selected grouts under realistic conditions leading to positive proposals for the formulation of backwall injection grouts and for the mixing and injection procedures to be followed. The results of this testing were reported in September 1980 in a confidential report entitled "R5/80 Backwall Grouting" by R. M. Edmeades. The conclusions of this work are summarized here.

The series of tests determined that very fluid grouts with high water-cement (w/c) ratios will not successfully seal off a backwall cavity in one operation. The persistence of interconnected voids is most likely to occur in the narrower sections penetrated by the grout, i.e. where the width of the pathway approaches 0.4 in. or less. Unless large cavities are to be injected, there is little technical or economic advantage to be gained by the inclusion of flyash. A greater reduction in bleed can be obtained by using a plasticizer (water-reducing agent) at moderate dosage. This produces a well-dispersed grout capable of penetrating narrow cavities, giving moderate bleed (1 percent approximately) after the injection is complete.

Use of a superplasticiser at high dosage will produce fluid grouts with a very low w/c ratio and bleed potential. In order to prolong the fluid period of the grout, a 3:1 blend of superplasticizer: retarder can be used. Unfortunately, under turbulent conditions, intermixing of the grout with water in the backwall cavity appears to be facilitated by this admixture, presumably due to its high dispersing power. A grout more stable under such conditions can be produced by incorporating an anti-bleed agent, and a combination of plasticizer and cellulose either gives a very low bleed grout with good injection properties. At the Riccall No. 1 shaft, methocel was used as an anti-bleed agent (Fotheringham and Black, 1983). By careful choice of materials and dosage level it is possible to formulate this type of grout with an acceptable setting-time, i.e. less than 18 hr.

Based on these conclusions, the following recommendations were also made to optimize the effectiveness and void filling capability of backwall grouting operations:

- Where possible a high-shear mixer should be used, and a sufficient mixing period should be allowed. This produces grout with a lower w/c ratio and less bleed potential for a given fluidity. Penetration of fine cavities is also improved due to the higher degree of dispersion of the cement grains.

- Injection should be carried out smoothly in one operation, starting at the lowest point and switching to higher injection points as soon as thick cement grout emerges from the open valves. Steeps should be taken to prevent air leaking into the suction side of the pump, which can cause excessive turbulence within the backwall cavity.

- When grout finally flows from the upper relief pipes (or the joint itself) pumping should be continued until the consistency is the same as that of the grout being injected. Any diluted grout will thus be expelled from the cavity, and there should then be no reduced shrinkage caused by separation of bleed water.

- The grout should have a fluidity of 20 - 24 in. as measured by the Colcrete-Flowmeter, preferably at the lower end of this range. Bleed should not exceed 1 percent by volume.

- For a minimum bleed and maximum cohesion, the grout should contain a combined plasticiser/ anti-bleed admixture.

3.5.2. Grout Pressures

When backwall grouting first commences on a lining length, the pressure at the point of injection is just that required to overcome the shear strength of the grout as it penetrates the cavities. After a short time the pressure rises as the hydrostatic head from the grout behind the liner increases. As grouting continues, the back pressure continues to rise as the head increases, due to both increased grout head and water head, and the stiffening of the grout mix. As the sealing of a lining length is achieved, a grout pressure is applied which is normally 20 - 25 percent higher than the expected aquifer hydrostatic head at the shaft depth at which grouting is taking place. This pressure obviously must be less than that which the lining can withstand. However pressure relief of the aquifer (see Section 3.2.3) allows backwall grouting to be carried at lower pressures.

Backwall grouting of a direct-placed lining is analogous to cementing a drilled shaft casing. These are compared from a sealing standpoint in Table 12.

Remotely-placed linings may in-turn require backwall grouting to effect the blockage of annular flow, to fill mud pockets, or to seal a micro-annulus. In this case there are no backsheets and the discontinuities to be grouted may range from large, fluid-filled cavities to cracks that are at the limit of grout penetrability.

Table 12: Comparison Between Backwall-Grouting for A Direct-Placed Lining and Cementing of A Drilled-Shaft Casing

	Drill-and-Blast Shaft Lining		Drilled-Shaft Lining
1.	Very small annular gap.	1.	Large annular gap - although possible small gap if hole not straight and spacers not in correct place.
2.	Can be grouted in-shaft at close quarters directly from front of the lining (i.e., horizontally through the lining).	2.	Remote filling from surface via tremie pipe (secondary in-shaft backwall grouting possibly required at close quarters directly through the lining to ensure seal).
3.	Lower risk of voids due to small gap and close local control.	3.	Higher risk of voids due to large gap, distance of control from grouting discharge point, and presence of many potential void forming protuberances on lining, i.e., pipes, spacers, ribs, etc.
4.	Can be pressure relieved to reduce resistance to grout penetration (i.e., better penetration of grout possible).	4.	Displacement of drilling mud by grouting gravity feed system (penetration of grout resisted by prevailing mud head and mud cake on excavation sides). Choose drilling mud which will not prevent grout bond with rock. Break down mud before grouting. Available active pressure head for penetration is difference between grouting pressure head and mud pressure head.
5.	Chemical grouts can be introduced directly to penetrate low permeability zones.	5.	Chemical grouts can only be introduced as a secondary in-shaft measure.

Table 12 (continued)

Drill-and-Blast Shaft Lining	Drilled-Shaft Lining
6. Grout seals can be introduced at selected levels to isolate aquifer zones. Note importance of pressure gradient, i.e., pressure drop per unit length which dictates length for sealing purposes.	6. Difficulty in keeping hole clean. Ribs and pipes on exterior of lining scrub sides causing rock inclusions in grout.
7. Micro-fractures in surrounding strata due to blast damage need grout permeation to provide effective seal.	
8. Concrete lining design and construction details important: • construction joints require water stops • hanging rods, plus possibly additional vertical reinforcement, needed to prevent thermal cracking (also control pour length to suit) • avoid cold joints.	

Note: For the successful sealing of both types the surrounding strata must be competent and impermeable.

4.0 WATER OCCURRENCES DUE TO VOIDS IN LINED SHAFT SYSTEMS

The purpose of this section is to convey an understanding of how voids can and do form in shaft systems incorporating grout. The primary emphasis is on voids that occur in the linings themselves. Issues of how the water becomes available to the voids, such as induced fracturing and creep, release of brine pockets, and so on, are highly relevant to shaft performance assessments, but are beyond the scope of this discussion.

The discussion opens with case histories of void occurrence to support the hypothetical discussion that follows. Only those case histories that illustrate the occurrence of voids have been included, although a great many more were studied. All shafts leak. Some do not leak much, and others only appear to leak very little because the watertight lining obscures water movement that probably is taking place. Of those where the leakage is apparent, the degree to which voids in the linings themselves are responsible is often not clear from the available information, although it is clear that if the water is apparent, there must be some contribution from voids. We have selected a relative few of the hundred or so case histories analyzed as being the most informative on how voids can occur in repository shafts, what the severities of their effects could be, and how they could be mitigated.

Throughout the discussion, it should be borne in mind that none of the shafts discussed are constructed to the stringent performance and sealing requirements of a commercial HLW repository. Hard rock mine shafts are noticeably scarce in the case history discussion, primarily because the design objectives of these mine shafts in general do not include a specification that there be absolutely no leakage. In fact, shafts operating in arid regions, such as the San Manuel Mine shafts and the Miami shafts in Arizona, double as water wells. Such case histories tell us little of what to expect from licensed repository shafts. Evaporite mines, particularly the WIPP shafts, with their requirements for dryness to prevent dissolution, come the closest to simulating the HLW concerns for positive sealing, control of aquifer communication, and extended-term performance. Accordingly, these case histories are presented first and in the greatest detail.

4.1 EVAPORITE MINING

Salt mines have shafts that penetrate waterbearing strata, as do trona mines, potash mines, and gypsum mines. In all these, the solubility of the mine itself demands positive sealing of the shaft system and near-absolute watertightness.

In this discussion, primary consideration will be given to shaft behavior experience in the Gulf Coast of the US, the potash districts of Saskatchewan, Canada, and Carlsbad, NM, and potash and gypsum experience in the United Kingdom.

4.1.1 Gulf Coast Salt Mining Experience

Gulf Coast salt dome problems have been associated with caprock-salt interfaces and deep fracture penetrations below the salt upper surface, which can interact with shaft linings to admit water to the subsurface.

Taylor (1932) describes the sinking of the Grand Saline mine shaft, which became inundated after piercing a sand layer that had previously been intensively grouted. The sand was 1-ft thick and occurred above an anhydrite bed in the caprock. Section 3 of this report describes how the sand had been entirely replaced by the grouting, which was done from the shaft bottom. Nevertheless, when the sinking was finally able to resume, the contact at the upper surface of the anhydrite was leaking. Leaks also appeared at the bottom of the lining. Asphalt seals were placed in the salt, but the shaft has had recurring water problems.

Most Gulf Coast mines have experienced seepage problems in their shafts and these have recurred after periods of months or years following grouting programs. The Avery Island shaft has been grouted with cement-brine, bentonite-polymer, and AM-9 acrylamide grouts at various times, with the cement-brine seeming to be the most successful.

The Belle Isle mine production shaft has experienced repeated problems. It was built in 1962 and lined 14-ft diameter through alluvium and 160 ft into the salt, using freezing techniques for ground control. During construction, minor seepage occurred 30 - 40 ft in the salt. After the 1.5-ft-thick lining was completed, the shaft was post-grouted with acrylamide from 220 ft to 380 ft, the bottom of the lining. A minor seep had appeared opposite the salt-alluvium contact at 220 ft. Grouting was done again in 1974 - 1976 at various places in the lining and in the salt. In 1976, the lining was extended 200 ft to cover leakage zones. The lining was again extended, this time to the total depth of 1100 ft in 1982 on account of leakage below the 580-ft level. Convergence due to mining halted production in 1984, and late that year, the lining sheared off at the salt contact with a 2 - 3 in. vertical separation. The mining disturbance probably had an influence on the leakage.

The total collapse of the No. 2 shaft at Belle Isle occurred after grouting had failed to stop a leak that appeared from beneath the bottom of the lining (Mine Safety and Health Administration, 1979). After the leak was first detected, it grew to 10 gpm over the 10 days preceding the collapse. Clearly, a void existed, but its exact cause and nature was never determined.

One of the more interesting and informative Gulf Coast shaft leakage examples is at Weeks Island, LA. It allows a comparison of the performance of 4 shafts constructed over an 80-year period using different lining types (iron, concrete, and concrete-bitumen composite) and different sinking methods (cast iron caissons and ground freezing). Also, the episodes of grouting that have taken place are fairly well documented, and some new research is underway. Thus, Weeks Island was investigated in some detail for this report.

The Weeks Island salt mine of the Morton Salt Company at Weeks Island, LA, was acquired by the U. S. Department of Energy to become an oil storage facility as part of the Strategic Petroleum Reserve (SPR). Consequently, a high level of watertightness is demanded of the shafts accessing the SPR itself. One of the shafts there is 84 years old. There have been several different episodes of grouting using a variety of materials and methods. As will be shown, the older shaft seems to have exhibited the superior resistance to leakage.

Salt was discovered at Weeks Island in 1897, shaft sinking operations began the next year, and in 1902 production commenced from the present Service Shaft, located in a part of the dome where the depth to salt is the least. The Service Shaft, as originally constructed, was 535 ft deep and about 9 ft in diameter, and served as both the intake and the return. It was lined with concrete to a depth of about 150 ft, or about 41 ft into the salt. In 1956, it was deepened to the 800-ft level.

Also during the mid-1950s, a second shaft, the Production Shaft, was sunk to support mining of better-quality salt at greater depth. It is about 800 ft deep and concrete-lined to 18-ft diameter, up to a depth of 291 ft.

The commencement of operations at the SPR required that Morton sink two new shafts to support continued salt mining. While these were under construction, an incline was driven to access reserves as high as the original mining level in the dome, in order to maintain continuity of production; this incline has developed numerous episodes of seepage, and may be related to shaft seepage also. The seepage was thought to be connate initially, but its tendency to increase has led to several episodes of grouting, and slight seepage persists. Dye injection tests from boreholes drilled to the top of the salt confirmed that communication existed. The possibility that the dome top may be incised by waterbearing fractures is significant when considering the leakage that has developed in the shafts.

The two new production shafts were sunk between 1978 and 1980. The linings of both are 18-ft-diameter double-slipformed; ground control was achieved during sinking by freezing. The lining is bitumen-backed and extends to a depth of 297 ft. The salt contact is at about 140 ft. The shafts continue, unlined, in the salt to the mining level of 1100 ft. They are about 560 ft apart.

The overburden at Weeks Island consists primarily of weak sands saturated with salt water, and varies slightly from place to place. The water table is at about 60 ft. There is no caprock.

The Service Shaft detailed construction records have not been located. What is known about its construction has been interpreted from a drawing dated 1901 by a J. H. Hazelhurst and a 1907 report. The study of its construction is a continuing as part of an ongoing grouting contract. Acres American (1977) summarizes the Service and Production Shaft details that are known.

In cross section, the outside sections of the liner are two-inch-thick cast iron rings separating the overburden from the shaft. These were pneumatically driven from the surface. They are bolted together, like tubbing. The condition of this member is uncertain. About a foot of concrete covers the iron rings. The cast iron apparently ends about 10 ft into the salt, and apparently was placed in a pit excavated in the salt upper surface. The surface of the salt was evidently covered with asphalt brushed on after heating the salt surface, and concrete of uncertain make-up was continued downwards from the iron section to cover. The concrete is apparently much thicker near the salt upper contact. Hazelhurst's drawing shows four asphalt seals over about a 20-ft vertical distance that occupy shallow outward-flaring excavations in the salt. About the same thickness of concrete covers the asphalt seals as covers the cast iron above. Finally, the entire shaft lining was covered with 3 x 6 cedar wood lagging about 2 ft long, laid on its side circumferentially so as to form a series of offset, many-sided polygons. Beny (1986) reports that the concrete has little aggregate, more like a dry-pack grout.

Surveys have shown that the shaft is about 59 ft (18 m) out of plumb. It is unclear whether the shaft was built this way, or whether it has tilted in response to ground movement. Other shafts in the Gulf Coast constructed within driven caissons have suffered alignment problems during sinking.

Considering the nature of the ground penetrated by this shaft and the shaft's age, its performance in terms of watertightness is remarkable. Over the years, a number of damp spots has been noted but no action to mitigate them has been taken.

The shaft has been used for both upcast and downcast air, but most recently in the upcast mode. At present, there is a damp spot about 80 ft below the collar and some salt buildup at about 100 ft and 125 ft below the collar. There is also some salt buildup at the base of the liner. The salt buildup is of concern because of the potential it has to wedge against the petroleum pipelines that have been installed in the shaft. These same pipelines will hinder grouting, which is scheduled to begin during the preparation of this report. Together with this grouting, further information will be collected on the details of this shaft's construction.

The Production Shaft was lined from the surface down to 130 ft below the salt with concrete of the bell-and-spigot configuration. Bell-and-spigot linings are a series of outward flares at each new pour. Construction drawings show an average pour thickness of 1 - 1.5 ft in the non-flared portions; this thickness increases to 4 ft near the shaft bottom. The pour heights are 13 ft and 20 ft, and the top of each bell overlaps slightly with the underside of the pour above. The salt tip is at 161-ft depth. Nash (1984) reports that insufficient freezing temperatures had twice resulted in loss of ground on the organic silty clay immediately overlying the salt.

The Production Shaft was grouted in 1977 and again in 1981 to stop water intrusion between the shaft liner and the salt formation. In both instances, the water intrusion had reached the base of the shaft liner at 286 ft. Nash (1984) describes this grouting as including two rows of dry-drilled holes into the salt near the base of the lining. Numerous holes were drilled before a water-course was finally intercepted. The conduit was eventually plugged by using a progressively-thickened (10:1 to 2.5:1) cement grout.

Late in 1984, after a survey of the shaft, it wàs concluded that there was leakage through the shaft liner below the water table at about 111 ft depth and near the top of the salt at about 154 ft. In the latter instance, the concrete had been found to be about 2.5- to 3-ft thick and porous, with holes running 1 - 2 gpm. Leakage from some old mechanical packers was supportive of a conclusion that the water intrusion between the shaft liner and the salt extended down nearly to the base of the shaft liner.

Consequently, a grout program was instituted that consisted of 10 rings of grout holes with 18 holes per ring, with equal spacing between all the holes in a given ring. All holes were drilled before grouting commenced to a depth of 8 ft each. Drawings provided by the SPR show grout holes making considerable water: 6 holes at about 168.5 ft collectively produced over 50 gpm. Grout holes commonly found sand or mud behind the lining and the concrete was consistently described as "porous". All the rings except the top one were grouted with chemical grout; the top one combined chemical and cement grout. Following this work, the two most porous zones continued to leak, although more slowly. Several of the mechanical packers that were not removed and grouted off in the prior phase were also found to be leaking. The water had damaged the timber blocking points at the shaft station as well. There was evidence of the deterioration and possible erosion of the lining at places.

The shaft is presently undergoing a program of further regrouting and rehabilitation, together with attempts to seal the upper salt contact.

The New Mine Shafts employ a double-wall system that is a simplified version of the composite linings used in Germany to protect shafts against deleterious ground movements. The lining consists of an outside, 1-ft-thick unreinforced primary concrete lining of 24 ft i.d. and an inside, 2.3-ft-thick reinforced concrete lining, with a 9-in. annulus between them filled with a bitumen-limestone flour mixture blended to a specific gravity of 1.3 (ENR, 1979). The outer lining was jump-formed 10 ft at a time through ground stabilized by freezing. This lining extends only to 165 ft depth, 25 ft into the salt. The freezing was carried to a depth of about 200 ft and used calcium chloride brine at a temperature of -40°F.

Excavation in the frozen ground was performed with shovels and pavement breakers. The excavation proceeded in 12-ft lifts by hand to the top of salt, where the method was changed to drill-and-blast.

The double-wall slipforming took place after the shaft key had been formed. The inner lining uses No. 4 bars as reinforcing steel. The slip-formed faces were given a rubber float finish. Slipforming was performed at an average rate of 1.4 ft per hour in the first shaft and better than 2 ft per hour in the second shaft; both were continuous pours from bottom to top. Concrete was based on Type 5 cement, and was a 6.5-sack mix at 4 - 4.5 in. slump.

In the shafts whose grouting is described in the following, the only seepage of water observed at depth 285 ft. At the bottom of the liner, at depth 297 ft, there was salt build up that could indicate a "weep" under the wall. In the unlined portion of the shaft below depth 297 ft there was salt build up throughout but it was not continuous and no visible "wet spots" or "weeps" were observed. Consequently, an injection program was undertaken between 27 July and 08 August, 1985, to re-establish a seal (Cementation Company of America, Inc., unpublished report).

The visible leakage occurred at a horizontal construction joint at depth 285 ft which is approximately 12 ft above the bottom of the concrete lining. The thickness of the unreinforced concrete lining below the leak was believed to be between 6- and 18-in. This 12-ft length of lining was constructed solely to protect the salt interface on the underside of the heavily reinforced compression ring (bottom of structural lining) from erosion due to the high velocity flow of humid intake air which would make its first contact with the exposed salt wall at that depth. This 12-ft length of lining was not designed to withstand sufficient pressures to permit it to form part of the seal area.

The first reported indication of any leakage was a buildup of salt (salt plume) below the 285-ft construction joint. This was observed on Thursday, 18 July. On Tuesday, 23 July, a leakage estimated at 0.5 to 1 gpm was observed originating from a single location in the construction joint on the southwest quadrant of the shaft wall. On Wednesday, 24 July, the leakage reduced to 0.03 gpm. On Thursday, 25 July, the leakage further reduced to only a few drips per minute. However, the leakage had spread on the construction joint and seepage plus wet spots and damp walls occurred over a length of one half of the shaft circumference extending from the southwest through south and east to the northeast. The total volume of this latest flow probably did not exceed 0.25 gpm. The earlier, larger flows may have represented the draining of a reservoir in the salt or salt/concrete interface.

Below the bottom of the wall in the northwest quadrant there was a salt plume on the exposed salt wall. This may indicate that there was a weep of water through the interface between the concrete and salt from the leak at depth 285 ft to the bottom of the liner at depth 297 ft.

The chemical analysis of the water leakage showed that it was a saturated brine and that it was non-connate, therefore originating from above the top of the salt dome.

Construction records revealed that a leakage which totalled less that 0.25 gpm occurred through several of the 24 horizontal relief pipes in the foundation ring. These pipes are 2 in. i.d. stainless steel and 6 ft long, and connect from the salt/concrete interface to the exposed shaft wall. The source of this water is unknown, but it is not necessarily the result of leakage through the salt formation.

During construction the contractor had grouted water bearing fissures in the salt formation between depths of 165 and 175 ft.

The leak could have been caused by one or more of the following:

- voids, fissures, and micro-fissures that are naturally occurring anomalies within the normally impermeable salt structure

- construction related fractures and micro fractures in the salt in the immediate vicinity of the shaft wall

- fractures and micro-fractures induced by ground movement related to mining activities or to natural salt creep

- improperly abandoned surface borehole

- recharging of previously drained fissure patterns (as discussed above) due to the recent grouting activities in the wet incline area mentioned earlier.

In this shaft the risk of loss of seal by leakage through the shaft lining was thought to be extremely low. The structural strength and permeation resistance of the lining is superior to the normal practice in Louisiana mines.

The risk of bypassing the lining is the major risk in this shaft and all other shafts in Louisiana. The evident loss of seal in this shaft was caused by a bypass leak.

The immediate grouting program was designed to achieve a seal over a height of only 15 ft at the bottom of the structural lining. This short length of seal must resist a pressure of 130 psi in the interface between the concrete and salt. Experience indicates that this length of seal under these conditions may be inadequate. It was therefore expected that the immediate grouting program may not achieve a total seal (i.e., as an example, there could be a residual flow of 1 pint or less per day). Unless an additional length of seal is achieved it must be assumed that future maintenance grouting on a frequent basis will be required when the residual flow increases. There is a risk that salt dissolution in this process will increase the capacity of the flow path and increase the volume of succeeding leaks.

A significant reduction in the flooding risk could be achieved by:

- increasing the length of seal

- providing facilities to permit maintenance grouting for repair of the seal from top to bottom

- providing facilities to detect a leak before it reaches the bottom of the seal length.

In designing the grouting program, it was considered that there is a minimum thickness of 6 in. of bitumen within the concrete wall from the top of the foundation ring at depth 270 ft to surface. There are no casings in the wall in this section of the shaft and if grouting is required at any point the bitumen and massive reinforcing presents difficulties which would result in significant additions to the time and cost of drilling, installing casing pipe, and injecting grout effectively through the wall.

Below depth 285 ft at the bottom of the compression ring the shaft is not designed to withstand the hydrostatic pressure from any formation water; therefore no grouting was carried out below depth 285 ft.

The grouting was carried out, therefore, to form a seal within the zone between depth 270 to 285 ft by injecting through the existing 24 standpipes in the foundation ring.

The grouting operation consisted of:

- installing work platforms to give access to the concrete wall on the full perimeter of the shaft at the required horizons

- drilling out of the cement grout in the existing 24 stainless steel pipes in the foundation and extending the drill holes to a depth of 2 ft into the salt

- testing all 24 holes for acceptance of brine at 150 psi

- injecting with AV-100 chemical grout all holes that accepted brine

- drilling 12 holes through the concrete wall to determine thickness of the wall extension below the compression ring.

A total of 346 gal or 46 cu ft of chemical grout was injected through 10 of the 24 holes. The grout required a driving pressure of between 120 and 150 psi in most cases, even when coupling occurred to other holes. Because coupling to the other holes occurred during most injections it is probable that a significant amount of grout followed flow paths in the interface between the concrete and salt. Whether any grout followed flow paths in the salt is unknown. Because grouting during construction in 1979 employed cement grout only, it is possible that all the voids into which chemical was injected in 1985 are voids in the interface that existed at the time of construction. Unfortunately, this supposition, although logical, cannot be proven and it is possible that a portion of the voids represent solution or erosion channels in the salt which have resulted from a minimal flow through extremely tight fissures at high pressure.

The residual leakage following the grouting program is reported to be in the order of 40 drops per minute. The volume of leakage previous to grouting is not accurately known but was certainly not in excess of 0.25 gpm. It is possible but not probable that the current residual leak may dry up completely. It is probable, however, that the leak will continue and that it will increase. The significant fact about the leak is that the water is of meteoric origin and that it must be following a flow path from the top of the dome through the salt. The bitumen seal has, therefore, been bypassed. This type of leak, when permitted to continue and to increase to several gallons per minute, has been observed at other salt dome operations to dissolve or erode a flow path large enough to permit the flow of alluvium into a shaft.

None of the 36 standpipes and drill holes was dry-packed with cement, in order to permit detailed inspection and evaluation of any leakage development. Any leakage through valves or unvalved standpipes may represent an increase in the total leakage and may contribute to the rate of increase of flow path size.

4.1.2 Potash Experience in Saskatchewan

Potash mining in Saskatchewan, Canada, is carried out at depths of 3000 ft and more beneath overburden containing some of the most prolific aquifers in North America. Shaft sinking, sealing, and grouting practices developed in the district have played pioneering roles in the technology of shaft sinking in difficult ground. Indeed any one shaft sunk there could adequately be the subject of its own report. For the present purposes of describing grouting effectiveness expectations, the experience there will be discussed in the aggregate.

Problems with water control in the district have resulted in the use of double steel liners, freezing, tubbing, grouting, and abandonment. Pence et al. (1971), describe light potash projects requiring 15 shafts, six of which were flooded by water during construction. The Potash Company of America (PCA) No. 1 shaft was sunk by freezing, using a 5-ft-thick concrete lining (no steel or tubbing) to resist the hydrostatic pressure of the prolific Blairmore Formation. Water eventually migrated through a temperature observation hole and through a salt layer into the otherwise successfully-completed shaft and creating a large void. It took 3 years to effect sealing by grouting (Edwards, 1985). The Yarbo No. 1 shaft was flooded twice in 1958 - 1959 during attempts to pregrout the Blairmore, which is 200 ft thick and carries 400 psi. Eventually an underground freezing station was built and the shaft was finally completed in 1962. Calcium chloride brine had been lost to the formation, so lithium chloride was used for additional temperature depression. It was necessary to line with tubbing section-by-section, from the top down, backfilling with concrete each time. The Allan No. 1 shaft had reached 590 meters in 1966 when the ice wall ruptured in the Blairmore along a coal seam, resulting in a 106-cfm (3 cu m/min) inflow. The cause was attributed to freeze hole deviation (Edwards, 1985). The shaft was refrozen with additional holes and completed. Water originating in the Dawson Bay Formation flooded the K-2 shaft in 1967, which was down 2952 ft (900 m) at the time. The shaft was salvaged after a concrete plug was placed, and the leak was grouted off and secured with tubbing. Cominco's No. 2 shaft, and eventually the whole mine, was flooded in 1970 after a grout hole intercepted a pre-abandoned freeze hole just below the lowest tubbing ring at 2096 ft (639 m). The shaft and mine were eventually recovered after a relief hole drilled towards the breach and the injection of cement, sand, and bentonite into the shaft in the area successfully plugged the leak. There were 680 short tons (617 metric tons) of cement injected into the breach for a final repair (Prugger, 1979). The mine resumed production years later after a costly rehabilitation project.

Storck (1968) and Link (1971) describe the Alwinsal shaft, which cuts both the Dawson Bay and Blairmore aquifers. The Dawson Bay is high-pressure and low permeability (0.00019 - 0.000031 cm/sec), making it nearly impossible to grout. None of the 4 picotages withstood the total hydrostatic head that developed upon thawing. Cement grout was injected beneath the Blairmore section and Dowell Chemical Seal Rings were used. The shaft is now dry.

Another mine shaft completed to 18.5-ft diameter in 1967 is 3350 ft deep, with tubbing/concrete from 1224 ft to 1564 ft and picotage at 1237 ft and 1549 ft. The shaft was frozen to 1470 ft and chemically grouted for construction below that. The concrete, which is up to 42-in. thick, has required periodic regrouting, and water from the lowest aquifer at 2960 ft is now known to be tracking behind the lining to at least 3010 ft, only 65 ft above the salt.

4.1.3 Carlsbad, New Mexico, Potash Experience

Numerous shafts have been developed in the Carlsbad district to exploit potash in the Salado Formation. Although the ground is generally drier and the mining depths shallower than in Saskatchewan, some prolific aquifers have been dealt with. Of particular interest is the Waste Isolation Pilot Plant (WIPP) nuclear waste disposal project in the area, which will be addressed at length, because of its obvious relevance.

4.1.3.1 WIPP Site Shafts. Three shafts have been constructed at the Waste Isolation Pilot Plant (WIPP) site near Carlsbad, New Mexico. The shaft sinking at the WIPP site is of particular interest because different construction and grouting procedures have been used on shafts in essentially consistent geological and hydrogeological conditions, and because the construction and maintenance of the shafts have been in the context of a nuclear waste isolation facility.

This site was selected for its favorability for waste disposal. The same trouble-causing aquifers at the IMC No. 5 shaft (Section 3.2.1) are far less severe at the WIPP site, as will be shown, yet leakage problems do persist.

For the Site Preliminary Design Validation (SPDV) phase, an exploratory shaft and ventilation shaft were constructed. These were later enlarged, and a ventilation exhaust shaft added. The present complement of 3 shafts now consists of the exploratory shaft (now the Construction and Salt Handling Shaft), the Waste Handling Shaft (an enlargement of the former ventilation shaft), and the exhaust shaft.

WIPP Geology is complex, but the underground workings of the WIPP are developed in a salt bed at a depth of about 2,150 ft in the Salado Formation in southeastern New Mexico. Only the most salient and pertinent geological aspects of this well-studied site will be outlined here.

The upper Salado Formation contact is in the neighborhood of 850 feet deep. The Salado consists almost entirely of halite, polyhalite, and argillaceous halite, with other evaporites (anhydrite and carnallite) and minor subordinate clastic interbeds.

Directly overlying the Salado is the Rustler Formation, which contains the only two apparent aquifers in the section. The Rustler consists of fine-grained clastic rocks, mostly mudstones and claystones, with evaporites and carbonates. The two aquifers are the Magenta and Culebra dolomites, separated by about 85 ft. Both are about 25 ft thick. The Magenta top occurs at about 590 ft depth.

Neither aquifer produces much water; their designation as aquifers in chiefly a relative matter, since practically all the remaining section is quite dry. The Magenta, when intercepted by shafts, proved to be only damp to wet, and the Culebra proved to be wet to dripping. Testing in the vent shaft revealed only 0.3 - 0.9 gpm of total water make, averaging 0.6 gpm, with about 0.4 gpm being contributed by the Rustler. However, this was enough to wet the Magenta surface in the unlined vent shaft; the flow from the Culebra ran down the walls and dissolved evaporites, developing a fluted surface in the affected sections.

The ground water table is about 250 ft deep at the site.

The Waste Handling Shaft was an enlargement of the SPDV Ventilation Shaft. The Ventilation Shaft was blind-drilled by Challenger Drilling between 24 December 1981 and 10 March, 1982. It was 72-in. in diameter, cased to 96.9 ft depth, and unlined to its total depth of 2,196 ft.

Upon excavation, the Ventilation Shaft was wet where it intercepted the Magenta and wet to dripping in the Culebra section. Seepage from the Culebra could not be attributed to specific cracks or voids, but was sufficient to wet the walls in the evaporite section and erode them. Seepage was measured by observing the rate of rise in the sump and also by direct collection from the Rustler section alone. The sump rise method generated calculated inflows for the whole shaft of 0.3 to 0.9 gpm, averaging 0.6 gpm, but this figure may reflect some non-seepage components also. The Rustler seepage by direct collection was found to be 0.3 to 0.4 gpm, with the Culebra contributing the greater portion.

Liner plate was installed after drilling at various sections to protect the shaft walls from seepage and weathering. These included the intervals from 566.3 - 576.4 ft, 679.0 - 690.7 ft, 727.2 - 753.3 ft, and 744.5 - 758.9 ft, among others. These intervals do not include the aquifers themselves, but are chiefly weathering-susceptible claystones and evaporites.

A Sperry-Sun downhole survey of the bored shaft showed more than 2 ft of deviation at 2,177 ft depth. This conflicted with a later, precise engineering survey that showed a 1.3 ft deviation at the station at 2,150 ft depth.

Slashing began after the foreshaft completion on 19 November, 1983. The shaft final diameter is 19 ft in the lined section and a minimum of 20 ft in the unlined section. Controlled blasting was used, but overbreak still ranged up to 1 ft. Most rounds pulled 10 ft, with few exceptions.

Lining was by step-forming, following excavation. The lining was normally installed in 24-ft lifts to within 10 ft of the bottom, except above 125 ft depth or so, where erection of the sinking plant caused a 40-ft section to remain unlined for almost 3 weeks.

The Magenta was encountered from 596 ft to 621 ft depth, and is a gypsiferous dolomite, producing minor seepage. The Culebra was found to occur from 706.5 ft to 728.5 ft depth, as a brown, massive dolomite with unfilled or gypsum-filled vugs up to 1-in. diameter. It dripped moderately saline water, but produced no continuous flows. Both zones were overexcavated 6 in. and covered with 0.25 in. liner plate to protect the concrete from hydrostatic pressure buildup (Figure 29). The liner plate and concrete were provided with 6 grout ports at each of 3 levels. After lining, the liner plate was back-grouted with a thick (1:1 water-to-cement) Type V Portland cement grout by the contractor.

Lining thicknesses are 10 in. minimum to 462 ft, 14 in. minimum from 462 ft to 580 ft, 18 in. minimum from 580 ft to 762 ft depth (thicker at the aquifers) and 20 in. minimum from the 762 ft depth to the top of the shaft key at 836 ft.

The shaft key (Figure 30) extends from 836 ft depth to 900 ft depth and is provided with 2 Chemical Seal Ring (CSR) seals poured into blockouts at the tops of planned construction joints. The seals are against the rock and are 4 in. thick, and 3 ft long (Figure 31).

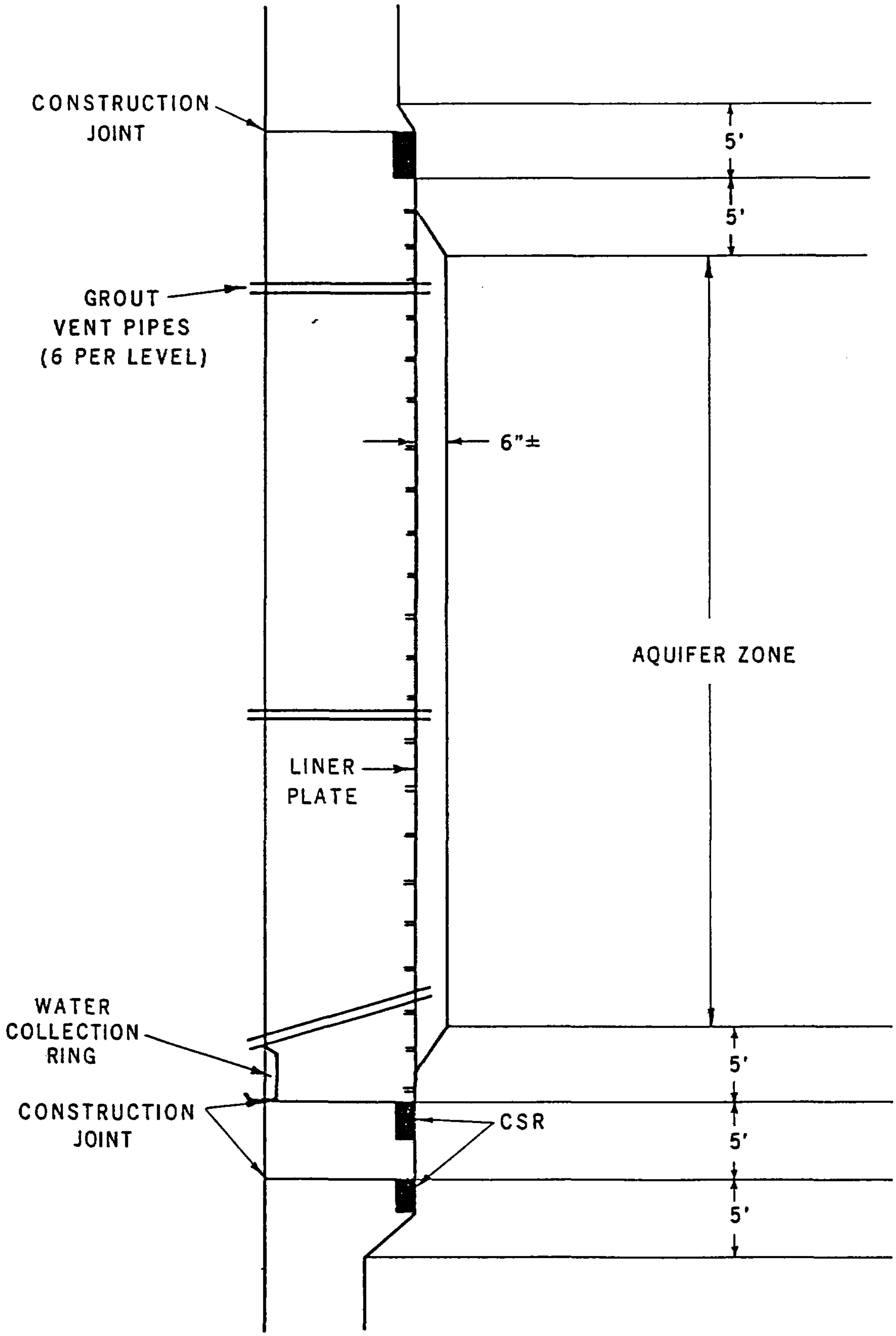


Figure 29. Aquifer Zone Shaft Lining Provisions for Exhaust and Waste Handling Shaft at WIPP. Chemical Seal Rings were Planned but not Acutually Built.

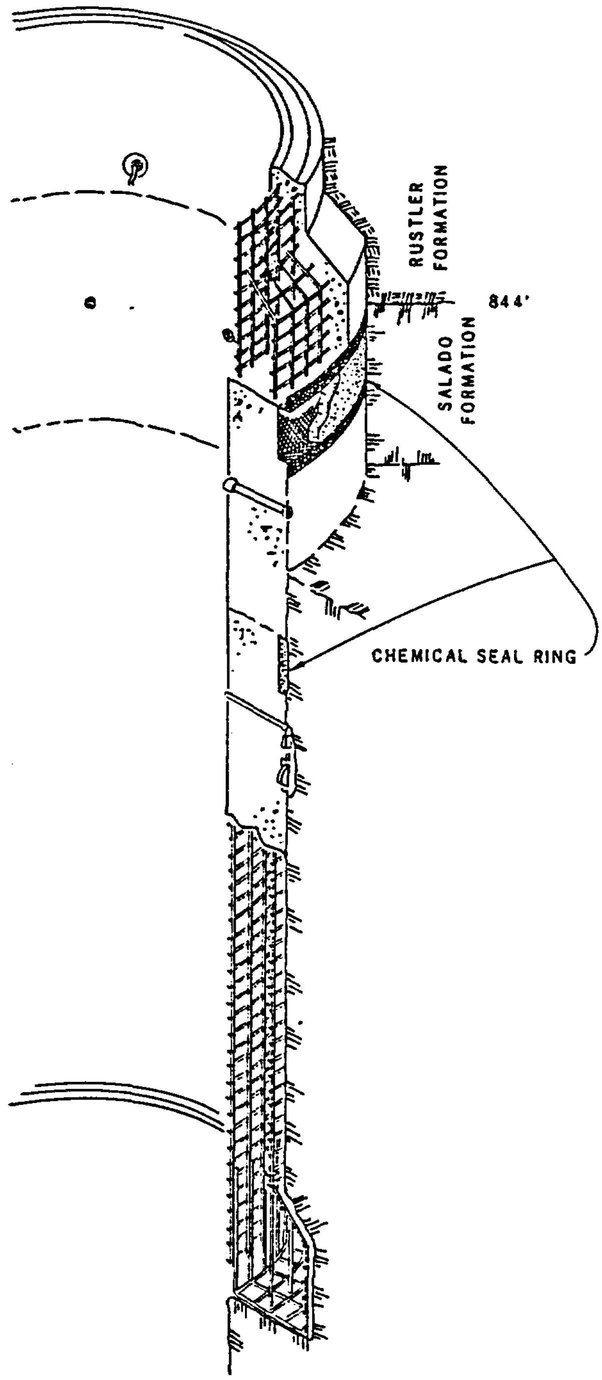


Figure 30. Construction of the Waste Shaft Key at WIPP the Exhaust Shaft Key is Similar

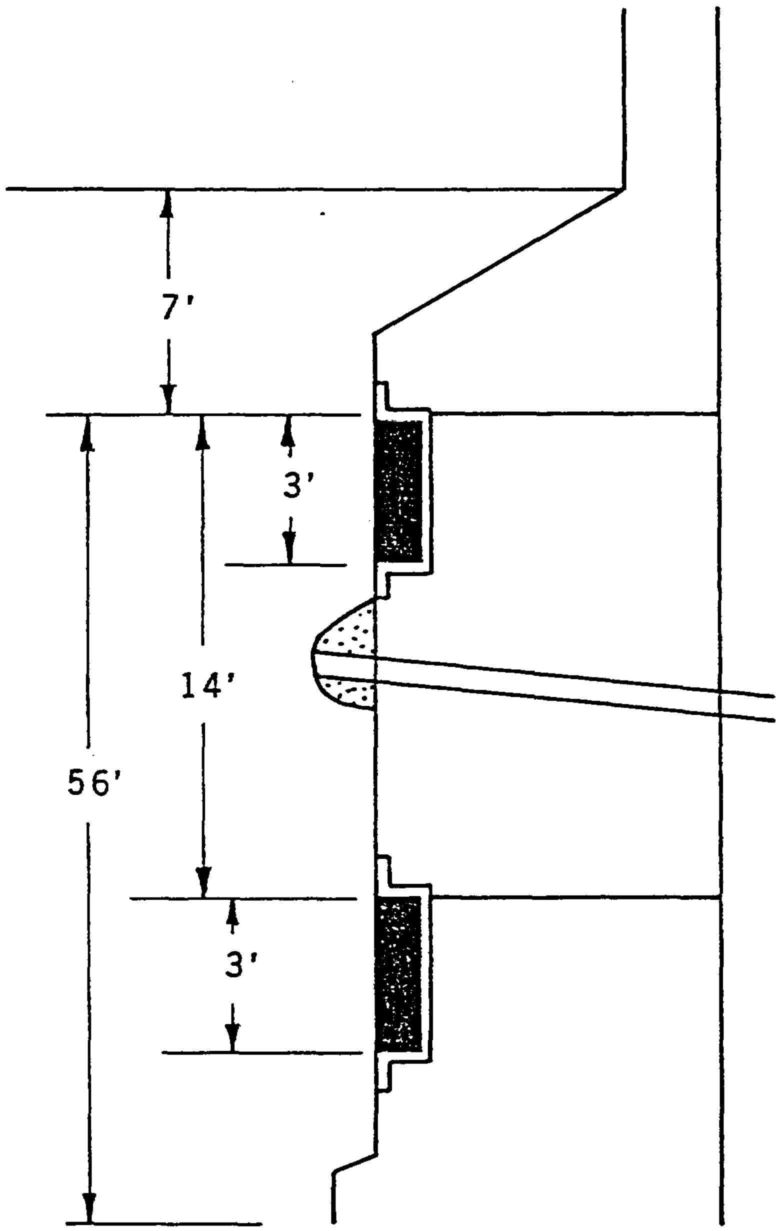


Figure 31. Chemical Seal Ring Applications on the Exhaust and Waste Handling Shaft Keys at WIPP

The lining was begun on 30 November, 1983 and completed 03 April, 1984, prior to sinking the unlined section. The liner plate sections were grouted in March and April of 1984, 3-4 weeks after coverage by the concrete.

Matcher joints in the lining were specially designed to promote complete filling and sealing (Figure 32).

Despite the precautions and the grouting, the waste shaft lining began to evidence seepage (estimated less than 0.5 gpm) within weeks after completion. The leakage was saline (up to 100,000 ppm) and issued from cracks, cold (unplanned) joints, minor honeycombing, and construction joints.

In the case of the construction joints, it is apparent that the bentonitic asphalt sheet sealant was being bypassed, possibly because the salinity of the water may have impeded proper bentonite swelling and possibly because of potential irregularities in the matcher surfaces, or poor contact due to settlement. Of the 12 construction joints between 556 ft and 759 ft depths, which includes the Magenta and Culebra intervals, 9 were sources of, or were associated with, weeps or wetness. Three months later, inspection found the concrete within and at the edges of many of the construction joints to be severely deteriorated and in need of dry-packing.

The leakage sources seen were essentially confined to the interval 560 ft to 760 ft depth. This interval extends about 37 ft above the upper Magenta contact and 33 ft beneath the Culebra lower contact. There was no seepage mapped in the shaft key area.

Accordingly, a contact grouting program was implemented between 11 and 24 August, 1984. It eventually involved excavating and dry-packing with non-shrink grout, where necessary, the deteriorated construction joints and cracks, and grouting through a total of 293 holes. Holes were both confined to the lining and drilled into the rock from 6 in. to 4.5 ft. The shallow depths reflected the objective of restoring intimate lining-rock contact only, not grouting of the rock mass itself. The program was impeded somewhat by lack of available time.

Grout holes were horizontal if for crack filling or, if for indirect filling of cracks or the annulus, angled up slightly or at 40 degrees.

Grouting was begun at a depth of 832 ft depth and moved upwards, to the 560 ft level. Initially, a Type V Portland Cement grout was injected behind the lining, at an initial water-cement (w/c) ratio of 1:1, gradually thickening to 0.75:1. The mix included a fluidifier. The lining itself was grouted with MC-500 microfine cement, using NS-200 dispersant, at w/c ratios of 3:1, thickening to 1:1. Finally, a chemical closed-cell foam grout, full-strength, was injected in

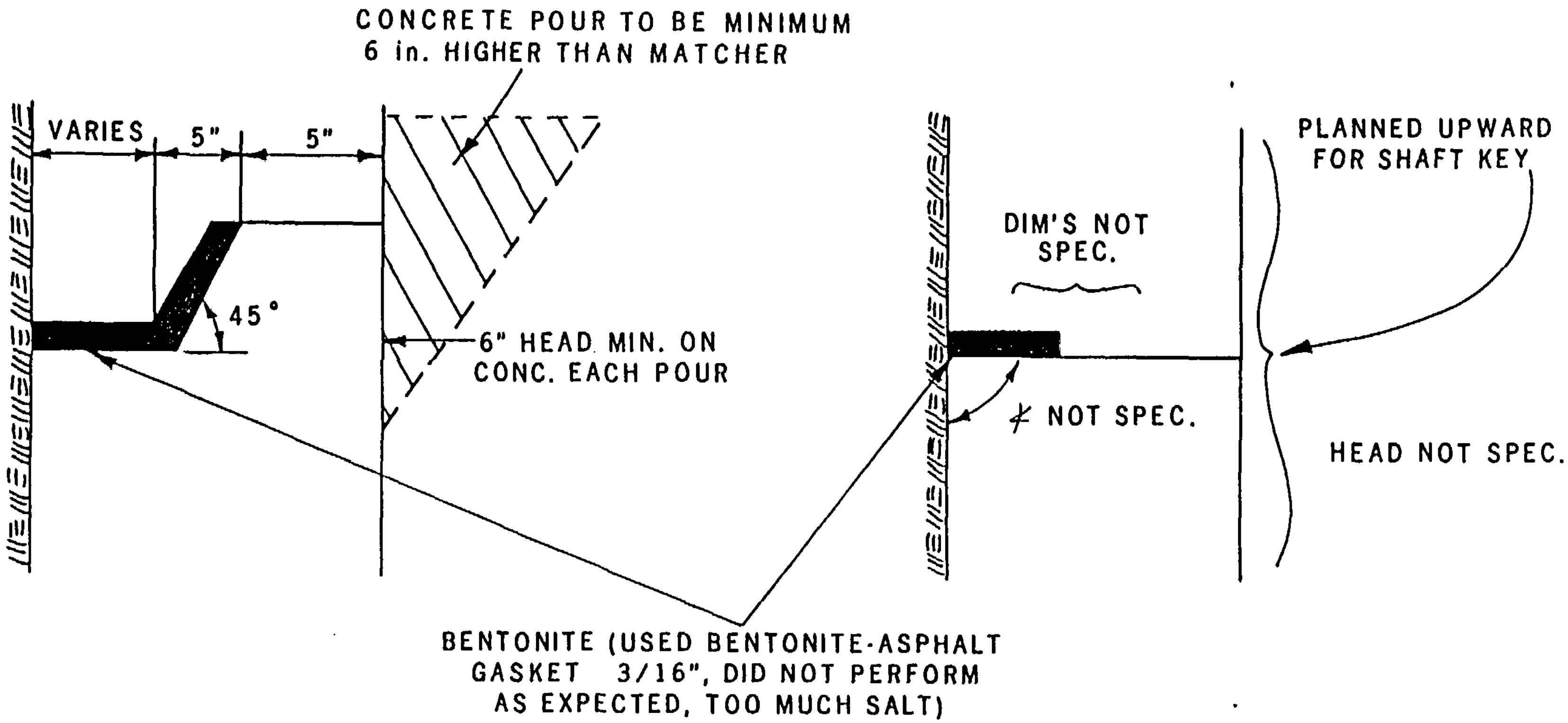


Figure 32. Matcher Joint Construction for
Poured Concrete Shaft Linings at WIPP

areas where seepage continued. This foam, on contact with water, swells to 9 times its original volume. However, its viscosity ranges from 200 to 700 cp.

Grouting was done in 18 rings, 6 holes per ring, and on a spot basis. Hole-to-hole communication was generally good, with grout sometimes appearing 20 ft above. In some cases, the cement grouts appeared to bridge, since water would be flushed out, but no grout. Some cracks issued chemical grout and water, the water continuing after grouting was shut off. The maximum single-hole take of Type V cement grout was 165 bags, 17.3 bags of microfine cement, and 5 gal of foam. Between the 758- and 759.5-ft levels (below the Culebra), over 500 bags of cement were consumed. In all, some 628 bags of Type V cement, 103 bags microfine cement, and 76 gal of foam were injected or used to fill grout holes.

The grouting was successful in reducing the shaft seepage, but did not eliminate it. Over the next 2 months, waste-shaft water-make was measured on 16 occasions. On 4 of these, the shaft was dry; the average inflow for the remaining 12 was 0.0085 gpm, ranging from 0.02 gpm to 0.0004 gpm. Some of the measured seepage reduction was due to evaporation associated with a change to exhaust ventilation in September, 1984. Collected water has continued to fluctuate and generally increase. By September, 1985, inflow was 0.05 gpm and it is now estimated to be about 0.3 gpm (whole shaft). Some of this may be due to discharge from near-surface overburden, due to construction.

Piezometers (12) measure the water pressure behind the shaft lining in the key area, and above, within, and between the aquifer sections. The highest pressures (100 - 125 psi) have occurred, predictably, in the Culebra section; the Magenta section pressures are distinctly lower (20 - 25 psi) (DOE-WIPP 221). Although both sections register increasing pressure, the Magenta zone increases are slow, whereas the Culebra section has apparently been building pressure at an approximate average rate of 8 psi/month since the summer of 1985. Immediately beneath the Culebra the pressures vary with time and the last reported readings (September, 1985) were 100 psi and beginning a stabilizing trend after a 6-month period of general increase. The pressures above the Magenta are near zero and are low also in the shaft key area (approximately 20 psi). Thus the piezometer data corroborate the observations to date.

As of July 1985, the salt creep had not yet made up for the concrete shrinkage and begun to load the shaft key (DOE-WIPP 221).

<u>The Exhaust Shaft</u> at the WIPP site is a pilot-and-slash shaft. As far as the lining and wall rock condition are concerned, it is quite similar to the waste handling shaft in its construction.

The pilot hole was spudded on 22 September, 1983, initially 7-7/8 in.-diameter, and was reamed to 11-in. diameter. Upreaming to the 6-ft-diameter began on 31 December, 1984, and was completed on 10 February, 1984. The shaft collar excavation began 15 July, 1984, and the shaft was completed and lined to its final 14 ft diameter 29 November, 1984. The shaft unlined section reached its final depth of 2,146 ft on 15 January, 1985.

Slashing was done using controlled blasting, and minor stability problems were presented by the Fortyniner claystone at 575.5 to 586.5 ft, the Tamarisk claystone at 589 to 595.5 ft, and the upper 9 ft of the unnamed lower member. Overbreak was slight throughout and negligible in the Rustler Formation anhydrites. Blast-induced fractures were found, during a detailed geotechnical study, to be subtle, except in the sections not to be lined.

The mapping revealed that the stratigraphic section was somewhat deeper than extrapolated from the waste shaft intercepts. Thus the Magenta and Culebra aquifer zone treatment depths were revised downward, by 9 ft, and the key was relocated downward by about 7 ft.

The pilot hole (11-in. diameter) made about 0.45 gpm. The Culebra and Magenta appearances were similar to those in the waste shaft, except that the estimated water production of the Culebra, as mapped during sinking, was 3 to 6 gpm. The greatest production issued from a fractured zone between 724.5 and 735.5 ft.

The exhaust shaft was lined with a nominal 10 in. of concrete to 500 ft and 14 in. from 500 ft to the top of the liner plate section surrounding the Magenta. The liner was thickened to 18 in. in both aquifer zones. Both aquifers were treated with liner plate and later backwall grouted as in the waste shaft. In the exhaust shaft, the liner plate backwall grouting was performed using 1:1 water-to-cement Type V cement at 30 psi. This consumed 2,336 bags in the Magenta and 1,871 bags in the Culebra, which are similar to the waste shaft takes.

The exhaust shaft key was instrumented with pressure cells but no data have been present (DOE-WIPP 221). The shaft key construction is similar to that of the waste shaft (see Figure 30).

Other instrumentation (piezometers) in the shaft lining were installed in December, 1985. There are 21 piezometers, 3 at each of 7 levels, with 2 in the shaft key. No data have been published, but the measured pressures are not much different in the Culebra than for that zone in the waste shaft.

The exhaust shaft was lined concurrently with slashing in 24 ft stages to within 40 ft of the bottom. Lining concrete was 5,000 psi (minimum) Type V sulfate-resistant. The lag distances generally reduced with depth.

After completion, the exhaust-shaft water-make was estimated at 0.35 gpm (in January, 1985), consisting of weeping at construction joints (same design as the waste shaft, Figure 32), cracks, and cold joints. Accordingly, a grouting program was instituted and included dry-packing to correct deterioration and cracking, using oakum and lead wool. Because there were fewer time conflicts, this program assumed a more leisurely pace than in the waste shaft. It extended from 01 June - 31 July, 1985.

Because small fissures were expected, the program was designed to include a low-viscosity acrylate polymer grout at full strength and Class C cement with a plasticizer. The w/c ratios for the cement grout were begun at 3:1 and reduced to 1:1. The cement and chemical grouts were expected to be more compatible with brine, which was planned for use as the carrying medium. Holes were drilled up to 10 ft deep and showed good communication in many cases. Pressures were up to 300 psi. The polymer set times were 30 minutes. The depths covered were from 595 ft to 870 ft in 32 levels.

The cement used totalled 164 bags and the total chemical used was almost 827 gal. Cement takes ranged from none to 41 bags (depth 748 ft) averaging 5.3 bags per level. Most of the cement was injected from depth 775 to the top of the key, from depth 615 through 675 ft, and from depth 730 ft through 750 ft. Grout communication was commonly 10 to 20 ft. Chemical grout takes ranged from none to almost 146 gal at depth 739 ft, averaging about 31 gal per level. Chemical grout takes were highest from depth 730 ft to the shaft key, but levels with high cement acceptance were usually not levels of high chemical acceptance. In many cases, the level previously grouted with cement would take little or no chemical, but the next level up would accept little cement and considerable chemical. Thus it may be that the low-viscosity (2 cp) chemical was effective in remedying cement grout bridging.

The high grout-takes do not seem justified by the surmised voids behind or within the liner. The Culebra liner plate zone took the maximum amount per stage (41 bags) but the total take in the Magenta liner plate zone was much lower (total of a little over 1 bag of cement and 67 gal of chemical) and most of the cement-take (84 bags) was between the Culebra and the top of the key. Apparently, the grouts were penetrating fissures, cracks, and vugs in the rock as well.

This may have had to do with the favorable outcome of the grouting program in the exhaust shaft. The flow is presently unmeasurable to absent. Although evaporation due to exhaust air passage may play a role in the low collected seepage quantities, there is very little salt encrustation. There is no observed leakage from the shaft key either. Because of the reduced exposure to seepage, the construction joints were in better condition, and dry-packing was not necessary.

The Construction and Salt Handling Shaft (Exploratory Shaft) can be instructively compared with the experience outlined above. It should be kept in mind that the construction and salt handling (C & SH) shaft was blind-drilled and contains a solid steel cemented-in-place lining, and that the context of this assessment includes aquifer communication in addition to shaft water-make. Furthermore, it should be remembered that the C & SH shaft has been subjected to potential disturbánce, at it has supported the passage of men, materials, and muck during the underground development.

The C & SH shaft was blind-drilled 142-in.-diameter to a total depth of 2,244 ft in a single pass. Drilling mud was about 10 lb/gal using saturated KCl/NaCl brine obtained from nearby wells. At 427 ft, the water was increased from 1:4 to 1:2 to aid circulation. The drill string was stabilized throughout and bit weight and rotation were controlled, to reduce deviation. The bit weight was 150,000 lb in the salt and 500,000 lb in the upper non-salt sections. Advance rates averaged 1.12 ft/hr, ranging from 0.55 to 1.83 ft/hr. Rotation speed was at 10 - 12 rpm. The water level was kept 200 ft off bottom in non-salt, and 400 ft off below 1,000 ft. Hole alignment was checked through a centralized 7-in.-diameter inner pipe by taking cluster shots (4 at 90 degrees) each 50 ft of advance. Drilling was performed between 04 July and 23 October, 1981.

The ring-stiffened, 1.5-in.-thick (maximum), 10-ft-i.d. steel casing was run to a depth of 842 ft. A specially-fabricated metal-rubber petal basket was used to contain the cement, which was placed beneath the mud in the hole. The casing was plugged with concrete at the bottom to permit flotation.

The cement mix was initially a heavy (17.4 lb/gal Class H, plus sodium silicate and other additives, designed by Sandia and the Waterways Experiment Station, to bridge at the petal basket, and upwards for 150 ft. This mix has a 7-day strength of 10,000 psi and is slightly expansive. The rest of the casing was cemented with Class C cement, accelerated with 2 percent calcium chloride, at 14.8 lb/gal.

Post-cementing Nuclear Cement Top Locator (NCTL) logs did not reveal any voids in the cement.

To date, no remedial grouting has been performed. Of course, there is no leakage through the liner, except for minor seepages at the instrument locations.

Piezometers were installed along with other geomechanical instrumentation in April and July of 1982. Unlike the waste shaft, where instrument terminals in the lined section can be set into alcoves, the instruments in the C & SH lining protrude from the shaft casing and are, therefore, subject to frequent and debilitating damage.

The February, 1983 Geotechnical Field Data Report contains details of the piezometer installation. There are 12 piezometers to a depth of 830 ft. Piezometer holes in the casing were opened with a cutting torch and the instrument heads are threaded into plates welded to the casing. The piezometer boreholes were drilled 3-in.-diameter through the cutouts and to varying depths into the rock surface, ranging from 6 to 9 in., which could be due to casing eccentricity or to wall sloughage. The piezometer holes are in pairs and where both pair members were drilled through distances in excess of the planned annulus of about 1 ft (for example, at depth 580 ft, where the distances were 31.5 in. and 44 in.), sloughage seems likely.

All the piezometer installations in the lined section leaked after installation and those in the aquifer zones, particularly within the Culebra, leaked prior to installation. It is not known how much of this was due to annular flow. However, it is noted that even piezometers 40 ft above the Magenta leaked continuously following excavation, and the largest pre-installation flow (1.3 gpm) was found in a hole 68 ft below the Culebra lower contact. These formations did not produce nearly this much water when exposed in the other shafts. The flow was severe enough to hamper piezometer installation in many cases and some installations were redesigned, to control the leakage.

Piezometer data in the C & SH shaft lining sections have been difficult to interpret due to numerous reinstallations, repairs, and failures. Also, the leakage has continued and the effect of this on the pressures measured up through June, 1985 are the highest of the 3 WIPP shafts, reaching 150 psi at a depth of 801 ft. The higher pressures may be due to the confinement afforded by the steel casing; the pressures in the Culebra in the C & SH shaft are comparable to those in the Culebra in the exhaust shaft, which is apparently not leaking in that zone. Nonetheless, data presented in DOE-WIPP 221 show no pressure increase with depth until the general area of the Culebra is reached; below this, the pressure increase parallels the hydraulic gradient, and does not drip until the shaft key is reached. This could indicate that there is little communication due to the Magenta zone but that water from the Culebra is connected to at least 21 ft above the Culebra top contact. Plotting hydraulic profiles from 30 April, 1984, 30 November, 1984, 28 February, 1985, and 30 June, 1985, reveals similar pressure distributions in each case, allowing for local perturbations that could be due to instrument problems, to hydrologic testing, or to natural fluctuations in the piezometric head within the Culebra.

There is no proof that these presumed connections are through the annulus, and if they do exist, they are doubtless small. However, because units that are dry elsewhere appear to carry water in the C & S H shaft, annular communication cannot be ruled out.

DOE-WIPP 221 describes aspects of the shaft key and seal behavior that also are relevant to the annular flow question.

The shaft key was constructed after completion of the drilling, by another contractor. The shaft key is shown in Figure 33. The first phase of the construction was to remove the ring of concrete which still remained on the inside of the lining and which acted originally as the plug for the bottom of the casing during installation and which had subsequently been drilled out.

The petal basket which was used to contain the grout behind the steel casing was then removed and the bottom surface of the grout was cleaned off. Inspection revealed no seepage in this zone at the time. The rock throughout the seal and key length was then excavated out to the desired diameters (i.e. 2 ft 6 in. concrete wall) including the excavation of the gravel trench (tell tale) and the key itself. It was initially intended that this excavation be carried out using heavy chippers, but as this was time consuming and inefficient, approval for very light blasting was obtained. Careful moiling and scaling was used to remove all the loose rocks so that the minimum of blast damage remained in the rock on the shaft walls.

Brackets were then resin-bolted to the shaft walls immediately below the bottom of the proposed concrete on which the concrete formwork would later be supported. The blockout for the lower shaft seal (Dowell Chemical seal) was installed. This consisted of 20-gage aluminum sheet pre-formed to the required shape, held in place with ramset bolts. The lowest mat of rebar was then installed and the lowest segment of forms and the water-ring blockout aligned and installed. The pipework for filling the seal was installed in the blockout and brought out through the forms. Then, this length was poured with 5,000 psi freshwater concrete. It should be noted that this first pour extended from the bottom of the key at 880 ft, upwards to the top of the chemical seal ring at 876.2 ft; the Salado-Rustler contact was at 850.7 ft.

After the pour the tell tale was constructed. The annular excavation in the shaft walls for the gravel was covered with 1/8-in. steel plate with holes to admit the weep pipes, and held in place by ramset expansion bolts. The pipework was then installed and gravel (3/8-in. plus) placed through holes in the plate to fill up the annular ring. The weep pipes are 2-in. PVC perforated in the gravel section.

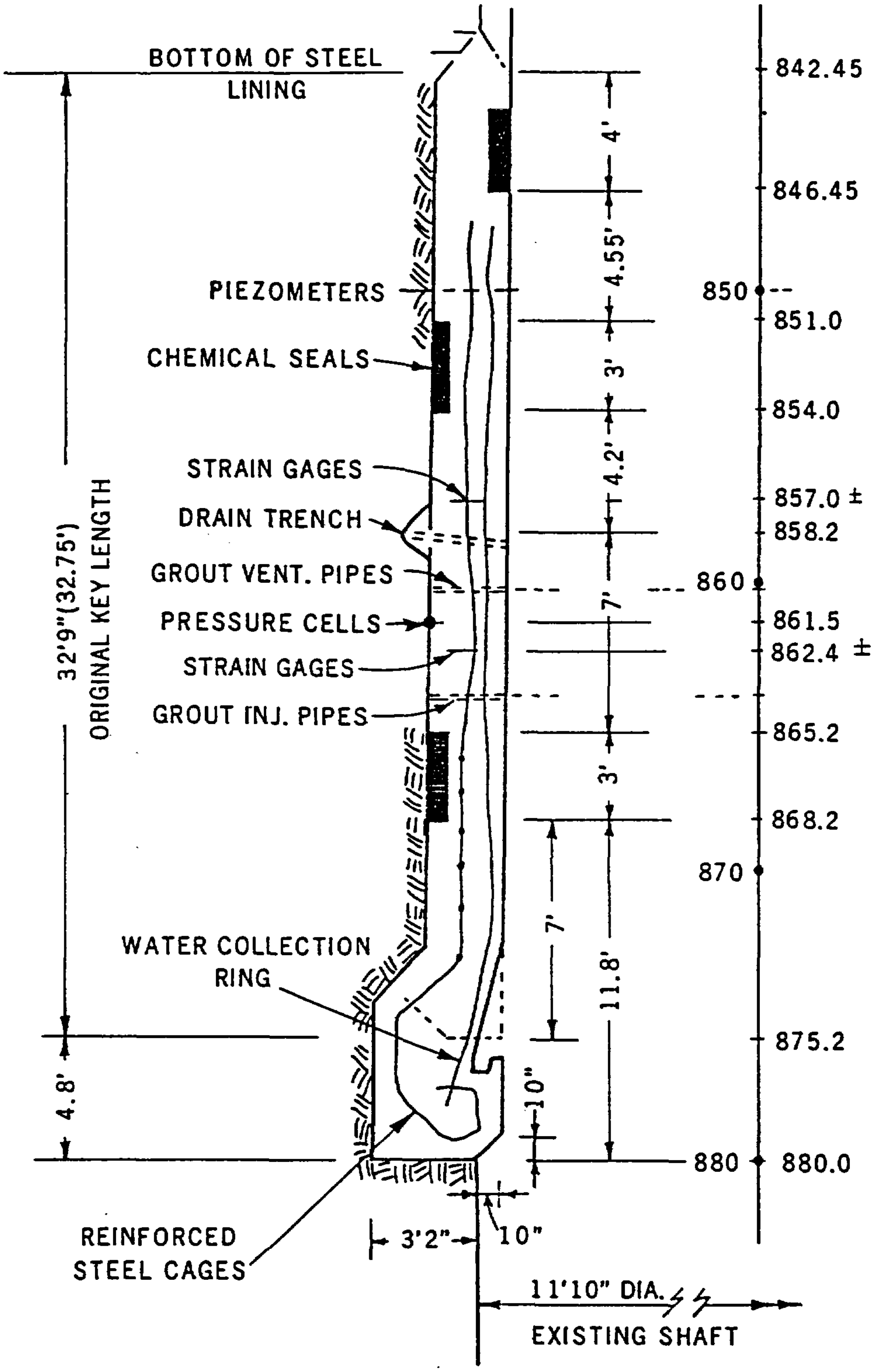


Figure 33. Shaft Key Construction and Seal Location, WIPP Construction and Salt Handling Shaft

The second lift of rebar and the forms were then placed and this section was poured with concrete.

For the upper and final concrete lift, the upper shaft rock-seal blockout was first installed and the rebar placed. Finally the steel liner skirt, complete with seal blockout, was welded on to the bottom of the existing steel lining and the forms installed to match up to this skirt. After the vent/grout holes were drilled through the liner at the very top of the pour where the petal basket had been removed, concrete was pumped into this space. A sanded grout (5,000 psi) was then pumped into any void which existed at the top of the pour to ensure a good, tight seal.

All three shaft seals were of the Dowell Chemical seal type. Representatives from the manufacturers mixed the component chemicals on the surface and final product was transported down the shaft in buckets. The liquid was then pumped through the lowest set of pipes into each seal in turn until the blockout was full and the material flowed from the upper pipes. All the pipes were then plugged (except the one used for pumping) and the pressure was taken up to 100 psi. The volume of chemical was almost exactly that expected from blockout measurements, i.e., no chemical connected to fissure in the rock or leaked inside any micro-annulus that might have existed between concrete and rock.

Inspections of the C & SH key were made during 1983, 1984, and 1985, and reveal that, although the key appears to be stable, there is leakage that is not being prevented by the chemical seals (DOE-WIPP 221). The tell tales drip intermittently, as do some other capped penetrations (grout pipes, etc). Liquid chemical seal material has appeared in the slightly brackish water collected in the grout vent, tell tales, and drain trench. This indicates that the upper, and possibly the middle, chemical seal rings, are not preventing this flow. However, no leakage has been observed from beneath the key, so the lower chemical seal appears to be functioning. It is not clear whether the seepage is due to the seeps noted during construction at 844 ft and 857.5 ft or to annular flow, or to some combination. The piezometers at 850 ft have consistently registered small, but finite (5 - 10 psi) water pressures behind the key. Also, as of 30 June, 1985, over 3 years after completion of the key, no pressure buildup in the key had been measured that would indicate salt contact and application of creep loads (DOE-WIPP 221). Embedment strain gages in the lining indicate from 0 to 0.01 in. of corresponding radial displacement at the concrete outer surface. Welded strain gages on the steel reinforcement show considerable scatter and some unusually large (0.05 in.) corresponding radial displacement. These do not include an initial reading (unreported in DOE-WIPP 221) of the original concrete shrinkage between 16 April, 1982 and 22 April, 1982. ("Corresponding" here means the conversion of tangential strain data to radial strain data with the assumption that the radial strain component is associated with the entire tangential strain component).

The shaft key was completed during April, 1982. The first piezometer readings were taken following the instrument installation in July, 1982. No reliable estimates of the C & SH water-make have been reported. Doubtless much seepage is hidden by the steel lining.

When the foregoing is viewed from an aquifer communication and annular-flow standpoint, it is not at all clear that the drilled C & SH shaft is now or ever was superior in performance to the step-formed waste and exhaust shafts.

4.1.3.2 <u>Other Shafts</u>. Mine shafts in the Carlsbad area do not leak through the bottoms of the linings, but about half of them make 1 - 3 gpm, the remainder being dry (ONWI 255). One that has been monitored over time, Duval's Wills-Weaver No. 3 shaft, is of interest.

The Wills-Weaver No. 3 was completed 14-ft i.d. in 1962, and lined with concrete, in 20 - 30 ft stages, 9 - 24 in. thick that is reinforced in the collar and aquifer (Culebra and Magenta dolomites) zones. Both these zones were pregrouted during sinking. In 1979, the United States Geological Survey (USGS) found during an inspection of the shaft that sulfate attack had deteriorated the lining in a 30-ft section opposite the Culebra, and that a 20 - 30 gal/hr flow was occurring. Radar profiling detected no voids or separations. It is possible that the grout had also deteriorated.

The pregrouting of the IMC No. 5 shaft was described in Section 3.2.1. of this report and required tremendous volumes of both chemical (Geoseal, 15%) and Type V cement-flyash, (66%) grout, both from the shaft bottom and the surface. This shaft is 875 ft deep and lined to 18-ft i.d. Even after the first pregrouting stage was completed in the dolomite aquifers with a 1:1 mix, some grout holes continued to run water prior to deepening. Six stages of pregrouting were used, with a salt-cement grout in the lower, salt section. When sinking continued, the inflow had been reduced from a predicted 5,000 gpm to 75 gpm.

After lining, seepage occurred through construction joints and backwall injection with cement and caulking were performed. This lowered inflow to around 0.1 gpm; however, seepage was noted shortly afterward from a tell tale at 325 ft, between the two seal zones. In 14 days it had increased to 2 gpm.

To examine the leak, 4 rings of 8 holes each were drilled 10 ft deep at 315, 310, 307.5, and 300 ft, and intercepted brine but no voids. They were grouted off with acrylamide, but 4 days later 6 gpm was entering the shaft. The leak was then sealed after pumping in 36 bags of cement at 320 psi.

4.1.4 Other Evaporite Mine Shafts

Two other salt mine projects are described here in reference to shaft inflow problems.

Morton Salt's Fairport Harbor Mine, Painesville, OH, has two 14-ft-diameter shafts, both completed in 1960 by drill-and-blast to a depth of 2,000 ft. They are fully lined with Type I or II concrete, 24-in. thick, and both penetrate the Oriskany Sandstone and Bass Island Limestone at 1,500 - 1,600 ft depth, which at less than 20 gpm are the only waterbearing strata. The Oriskany water is, however, quite acidic. The aquifer zones were pregrouted with chemical grout and post-grouted as well, with acrylamide. Nonetheless, a 1 gpm inflow persists, and the present strategy of the mine is primarily to keep this persistent flow away from the concrete face (Cementation Company of America, unpublished records, 1986).

International Salt's Whiskey Island Mine has two shafts of similar (16-ft-diameter) size, completed by drill-and-blast also in 1960, to 2,000 ft. The 12-in-thick Type I or II concrete lining thickens to 4 ft in the interval 1,300 - 1,600 ft, where the Oriskany and Bass Island have a reported combined capacity of 6,000 gpm at these excavated diameters. The aquifer section was pregrouted from the shaft bottom with over 100 holes of more than 100 ft each, using calcium-chloride-accelerated cement and sodium silicate. The interval was also post-grouted with sodium silicate, yet the residual make was several gallons per minute. The water is again acidic and attacks the concrete. Grouting has to be re-done about every 3 years. The seepage has reached 10 gpm and been reduced to 2 gpm with acrylamide (Cementation Company of America, Inc., unpublished records, 1985).

4.2 HARD ROCK MINING EXPERIENCE

Hard rock mine shafts are constructed with the objective of minimizing water handling costs and water interference in the shafts, but normally do not carry the imperative for absolute watertightness present in some other structures. Thus water inflows to hard rock mine shafts, though common, are not good barometers for judging grouting effectiveness. A few case histories are illustrative.

4.2.1 Metal Mines

Hawes (1985) described sinking the No. 11 (10.5 ft-finished-diameter, 3,359-ft deep) and the No. 12 (14-ft-diameter, 2,100-ft deep) shafts at the Miami copper mine in Arizona. Concrete strength of 3,000 psi was verified with 3 tests per pour and an engineer monitored all pouring operations. The ground consists of Gila conglomerate, dacites, schists, and granites, that are waterbearing and crossed by several major faults. Ground conditions limited the pours to 10 ft. The slump of 8 in. was thought to be high enough to remove any need for vibration; the concrete was brought to the galloway in a bucket and spread by hand. There were no major water problems, but the No 11 shaft makes 42 gpm. The water is recovered and used in the mill.

The No. 2 shaft of the Harmony Gold Mining Company, in South Africa encountered 43,500 gpm at a sedimentary contact that required 3,606 bags of cement to stop. The No. 3 shaft encountered a 22,560 gpm flow in a basalt unit and 8,120 gpm in the sediments. None of these leaks was plugged completely.

4.2.2 Oil Shale Demonstration Project

A blind-drilling project done for the U. S. Bureau of Mines in Colorado to access oil shale reserves for a pilot recovery program provided some examples of the effect of casing eccentricity and poor circulation on void occurrence (Utter, 1980; Amstutz, 1981; Amstutz and Hawkins, 1982; Fenix and Scisson, 1983). The end product was a 96-in.-i.d. shaft, 2,371-ft deep, bored in 3 passes. Circulation was via reverse-air, jet-assist, using a low-solids, 0.1 percent NaCl HEC polymer mud. Salt saturation and polymers were chosen for protection of shale, nahcolite, and halite beds in the overburden. There are also two high-pressure aquifers in the overburden that at the time were not well-known.

The surface hole was drilled 140-in. diameter to 195 ft and lined to 190 ft with 122-in.-i.d. steel casing. The casing was cemented with Class G in 3 stages, with varying calcium chloride and prehydrated gel contents. The cement volumes were slightly (3 - 9 percent) over theoretical. Drilling resumed at 120-in.-diameter to 845 ft, when two cutters were lost and a fishing job was needed. Due to bolt breakage, the bit was lost with the hole at 1,781 ft and it lodged at 1,245 ft. It required 67 days to fish it out. At the time, water from the aquifers and all the bedded salt had been cut by the hole and the water was free to degrade the salt, since the jammed bit overhead blocked circulation of the sodium chloride mud. A lost circulation zone had been encountered also, from 1,100 to 1,396 ft, thought to be due to breakdown of the leached zone, caused by the mud column being 100-ft higher than the water table. The zone was controlled with lost circulation materials and by lowering the mud level. Other mud losses had occurred at 360 ft and at 876 ft to 904 ft.

The casing used was 96-in.-i.d. with a wall thickness of 2 in. at the bottom grading to 3/4 in. at the top and 3-5/8 in. stiffener rings at 2-ft at the bottom and 10-ft near the top. During casing running, there was significant drag in the interval 1,284 - 1,564 ft (near where the bit had lodged) that crushed one of the 3.5-in.-o.d. grout line guides. The annulus past the stiffener rings, grout lines included, was only about 4 in. and the centerline was found to be off 1.68 ft at 2,343 ft.

The casing was cemented through three (3) 1.9-in. strings and one 2-7/8-in. string, in 9 stages, using Class H cement (Utter, 1980). Grout rise was monitored with a nuclear logging tool and grout lines were withdrawn 90-ft with each 90-ft rise. There was a 48 percent overall excess cement requirement but most of this occurred in a few zones:

Interval ft	Theoretical volume cu ft	Actual volume cu ft	Percent of Theoretical, by volume
753-1253			138
1253-1417			138
1417-1697	6,211	17,838	287
1697-1792	2,068	10,113	489

The extreme differential was caused by evaporite dissolution.

Breakout was initiated by means of a 0.25-in. hole drilled through the casing at 1,659 - 1,669 ft and resulted in an inrush of trapped drilling fluid under about 600 psi pressure. The fluid drained for 2 days. Drilling showed 7-ft of cement behind the casing. More mud was drained and some methane was vented. A grout curtain was begun with relief holes 20-ft and 40-ft above and below the grout holes.

One grout hole penetrated 60 ft and drained 45,000 gal of water with methane which apparently was contained within the soil shale rock mass. An estimated 7,000 cu ft of voids were encountered in this exploratory drilling. The voids were grouted with a salt-bentonite cement mix at about a 1:1 w/c ratio, with 0.25 percent defoamer, at a yield of 1.48 cu ft per sack. Some 3,550 sacks of cement were injected this way at a maximum pressure of 1,200 psi.

Investigations showed an excellent casing-cement bond, and that the casing was tight against the hole. Cement channeling was visible and extensive; dyed grout pumped in at 1,659 ft was detected 500 ft below. Stiffener rings trapped mud below them and the cement had numerous incipient vertical cracks that were concentric to the casing and were tightly closed until liberated by coring. These were interpreted to be cooling cracks. Apparently the cement had been effective at stopping the migration of formation water above the dissolution zone.

Amstutz and Hawkins (1982) suggest that 15,000 sacks of cement could have been saved had the hole been full of completely-salt-saturated mud when the bit was lost. Amstutz (1981) describes cementing problems including undersaturation of salt mud, poor caliper tool calibration, unavailability of brine and suitable cement, fluctuating fluid levels in the hole, and the crushed grout line guides.

4.3 COAL MINE SHAFT EXPERIENCE

Coal mine shafts, like hard rock shafts, do not require absolute watertightness although winter ice-up can be a headache since fans are not as readily reversed in coal mines for ice removal. Nonetheless water handling is a cost to be minimized. Also the extraction practices in some countries extend to the shaft pillar area and, therefore, shaft flexibility and watertightness on the presence of some major aquifers must be achieved simultaneously.

The Selby project in Great Britain has been alluded to frequently in this report and is truly one of the major underground mining and shaft sinking projects in recent years. Most of the shafts at Selby have experienced inflows during or after construction. In most cases the ground was frozen although some significant pregrouting was also accomplished.

While thawing naturally, the No. 1 shaft at Wistow (Keeble, 1984) developed a 15-gpm inflow in a zone adjacent to an 8-in.-thick halite bed that had not by then been backwall grouted, owing to the low (32°F i.e., 0°C) ground temperature. The No. 2 freezehole had deviated into the shaft-wall and had been disconnected. Another leak began 60 in. below and escalated to 500 gpm. Backwall grouting in seal areas nearby that had been completed had incorporated water flushing. Additional leaks developed. Eventually, the shaft was allowed to flood, the freezeholes were injected with concrete (reducing the water-make by 40 percent), and later backwall grouting was able to reduce the overall water-make to about 5 gpm.

The No. 2 shaft at Wistow (Black et al., 1982) penetrated ground with an inflow capability estimated at 12,000 gpm (750 l/sec) by the Pressure Recovery Test. Several of the primary holes made 600 gpm (38 l/ sec). The 32-hole primary and 16-hole secondary rings were injected with 204 short tons (185 metric tons) of cement, which reduced inflow to about 180 gpm (11.4 l/ sec). This was followed by a sodium silicate (Cemex A2) injection which reduced inflow to 12 gpm (0.73 l/sec).

Probe holes at Riccall No. 1 showed a capability of 33 gpm (2.1 l/sec) at 1335 ft (407 m) that were sealed with cement. At Riccall No. 2, one grout hole hit 71 gpm (4.5 l/sec) and 24 short tons (22 metric tons) of cement and 26,280 short tons (23,850 metric tons) of Cemex 2A reduced it to 4 gpm (0.25 l/sec). The Riccall shafts now make about 2 gpm (0.13 l/sec) (Fotheringham and Black, 1983).

At the Thorne Colliery in Great Britain, a trickle of water grew to 1,800 gpm over a 20-year span (Ferrari, 1966). Water was passing from the Bunter sandstone, through anhydrite and into the marls below. Abandoned freezeholes were suspected of causing the leakage through the normally impervious anhydrite. In the No. 2 shaft, the lining was stripped out, replaced, and grouted, and a permanent lining was installed in Shaft No. 1.

Considerable pregrouting and backwall grouting was done at all these projects.

At Monktonhall Colliery in Scotland, two 24-ft-diameter shafts were sunk through heavily water-bearing ground including old works and volcanic and sedimentary interbeds. The old works were pregrouted with cement-bentonite, the interbeds with neat cement, and underlying (1,520 - 3,200 ft) sandstone aquifers with AM-9. Heavy reliance was placed on backwall grouting. Currently, one shaft makes 367 gpm and the other, 418 gpm. Further grouting is planned.

4.4 CIVIL/MILITARY PROJECTS

Civil/military applications of shaft sinking include drainage outfalls, surge chambers, pressure shafts, and shafts as access for weapons tests. Most of the weapons tests shafts have been drilled at the Nevada Test Site (NTS) and in separate projects in Central Nevada, the Gulf Coast, and in Alaska.

Grieves (1975) describes the sinking of the Boulby potash mine shaft in Great Britain, which required freezing and a welded steel lining to penetrate the Bunter sandstone. This unit has a hydraulic conductivity of near 0.0001 cm/sec at 1,500 psi. The primary liner of the No. 1 shaft was within 295 ft (90 m) of the bottom when a leak began through a rockbolt hole and spread around the shaft, despite an observed shaft perimeter temperature of -8°F (-22°C) and confirmation by probe holes that the freeze wall was closed. Rather than pour a concrete plug and risk extension of thawing, a steel-concrete composite lining was installed with chemical seals at the top and bottom. The steel casing was backfilled with concrete. The No. 2 shaft was extensively pregrouted. Cement was injected first, followed by chemical grout to seal pores and cement shrinkage. Pretreated ground was lined with tubbing that was backfilled with concrete set against polyethylene sheeting. Dewatering outlets were provided from behind the sheeting. The lining was caulked and re-caulked but still leaked about 1.2 gpm (4.5 l/min.)

The Drumbo gypsum mine shaft in Ontario (Hartviksen, 1983) was sunk through 164 ft of artesian waterbearing glacial drift and an additional 291 ft of bedrock, that contains an artesian zone at a depth of 223 ft. The shaft was constructed by blind drilling and lined 12.5-ft-i.d. with precast concrete segments that were floated into place. The hole was drilled 14-ft-diameter. After an incident in which the coping section of the shaft caved in and damaged the drill pad, the mud density was increased from 8.8 to 10.5 lb/gal and the damage was repaired with concrete, the coping was grouted, and a cement plug was placed in the hole. After the shaft had been drilled to 250 ft, the liner was run. The hole was deepened by drilling to the final 455-ft depth but encountered a strong inflow near the bottom that filled the shaft to within 50-ft of the collar, through a vertical fracture. Placement of a concrete plug and grouting were unsuccessful. Finally, a 56-ft steel liner reduced the flow to 4 gpm.

4.4.1 Nevada Test Site Shaft Sinking Experience

Underground testing began at the Nevada Test Site (NTS) in 1961. This created a demand for experimental access to the subsurface via shafts. It is probably accurate to say that most of the adaptation of oil-field technology that has resulted in the blind-drilling and casing techniques in use today were developed at the Nevada Test Site.

There have been literally hundreds of shafts successfully drilled and lined at the NTS. In fact, over 750,000 linear feet of shaft between 36 and 160 in. have been successfully completed by the federal nuclear testing program. It was recognized that this experience could be very significant to this study, inasmuch as there have been numerous notable successes, as well as a few failures, from which much could be learned. However, the task of studying all the experience at the NTS in order to assess each significant case thoroughly would be a monumental one. A request for thorough background on a sampling of some of the most relevant experiences was made of the Nevada Test Site Office (NTSO). Inasmuch as the time required for the site shaft engineering contractor to locate and assemble the necessary information could have been substantial, the request became snagged. Ultimately, lack of time and funds, as well as institutional and security considerations, precluded even a thorough assessment of a reduced number of shafts for this study.

Consequently, reliance was placed on personal communications with knowledgeable personnel at the site gleaned from telephone contacts and a site visit, as well as on the published summaries and a few specific instances described in the published literature.

The early holes were of much smaller diameter than those presently being drilled (24 - 40 in. as opposed to 120 in. at present). As the holes became larger, advancements were made in penetration rate, casing design, cementing efficiency, cuttings removal, hole logging, and rig design.

It should be stated at the outset that shaft construction improvements at the NTS are made because of a need to improve production within a standard of quality, rather than to optimize the standard of quality at the potential expense of productivity.

This is not to say that the quality of construction is in any way poor; in fact every shaft lining system must meet the rigorous criteria of the Containment Evaluation Panel before approval is given to conduct an experiment. However, the objectives of construction are only to provide a dry, stable hole until the experiment can be completed. Radionuclide containment is provided by a separate system. In fact, once the shaft useful life has ended, there is typically very little if anything of the original lining remaining. So long as the shaft can fulfill the above purpose, there is little interest in annular flow or the occurrence of voids in the cement system that goes beyond that of most other shaft drilling projects. Holes are normally completed 8 - 12 months before use.

Thus shaft sinking at the NTS is an excellent example of high-rate production of shafts. (There are about 25 tests per year and 15 - 20 holes open at a time, according to Hammer, 1985). These shafts are drilled to considerable depths (commonly over 2,000 ft) and can encounter running sand and slough-prone alluvium, water under great pressure, and very hard rock. The geologic conditions are considerably more variable than might be casually assumed for a single site.

Lackey (1983) summarized the shaft drilling history at the NTS. In 1961, holes to 36 in. were being drilled using hole opening techniques (3 passes). The hole-openers were combined into a single tool and in 1963 the diameter capacity had increased to 64 in., with stabilization provided by other hole-openers uphole. Integral stage bits and weighted mandrels were in use by 1965, and in 1967 mandrel weights were up to 75,000 lb, which in combination with plate bits and bit roller reamers provided acceptable penetration rates with directional stability in holes to 96 in. Dual pipe air/water circulation came into use in 1969. Increases in the size of the drill pipe bits, stabilizers, and mandrel weights have occurred intermittently since, and along with circulation improvements such as jet assist, have resulted in excellent performance in holes to 120 in.

The dual-string circulation system (Section 3.3.1) was developed at the NTS to reduce shaft wall breakdown associated with fluid invasion. The dual string allows the drilling to be done with as little as 200 ft of water in the hole through the water-sensitive, unsaturated sections. Under considerable heads, an air-water mix is injected near the cutters to assist in moving the cuttings to the center of the bit.

The two principal drilling problems at the NTS are poor circulation that occurs when some loss to the formation occurs, and shaft-wall sloughage. When sloughage occurs, a decision must be reached whether to cement and redrill or to get through it so it is no longer submerged. The choice is largely based on experience. There are running sands in the alluvium on the flats, and some unwelded tuffs in the mesas can behave similarly. Plugging is costly, sometimes not effective, time consuming, and lends itself to deviation when redrilling. Sloughage is normally fairly superficial but can be up to 30 ft high and over 4 ft deep. Running sands can slough to an angle of repose, but this is very uncommon. It is usually desired to keep the alluvium as dry as possible (above the water table) when drilling. Under these circumstances, it stands well.

Normally, the unsaturated sections are not cased. Casing is fabricated according to the expected ground water table, plus 100 ft or so. In running sands, the section maybe cased off, and the hole below reduced to 96-in. diameter.

As stated earlier, there is normally not a great degree of attention paid to annular flow or to voids occurring behind the casing that do not affect the dryness of the shaft interior. Since the hemi-heads are not drilled out, there is no cause to check for annular flow unless a leakage develops. However, there have been some instances where voids have been evidenced when water has appeared through the steel lining.

In one recent example, drilling had proceeded normally to the final depth of about 2,150 ft. The ring-stiffened casing was run into the mud-filled hole (120 in. diameter). The water table had not been intercepted until slightly deeper than 1,500 ft, which was near the top of the casing at 1,497 ft. The bottom of the casing was about 50 ft off the bottom of the hole. Cementing was done in the usual way, to the top of the casing. The hole above the water table was left unlined. When the ballast was pumped from the casing, the hole began to refill with water. After a submersible pump had drawn the water-level back to the bottom, a leak of about 0.8 gpm was observed, with a television camera, to be entering the shaft through a failed weld just below where the hemi-head joined the tubing sections. The influx pressure was about 324 psi. The fact that the water had risen to above the initial water table -- and in fact above the level of the top of the casing -- indicated that the water entering the shaft was from a confined source and the 500-ft difference in elevation between the leak and the static water-level proved that the confined source was under considerable pressure. The television records showed conclusively that there was no water production from the strata above the liner top. Obviously, a void had somehow formed opposite the leak in the cement and had permitted water to enter through the weld.

The remedial treatment consisted of allowing the shaft to refill with water to the top of the liner, and then pumping a polymer sealant into the hemi-head. Allowing the polymer to cure under the water head in the shaft ensured that sufficient positive pressure existed on the polymer that it would gain intimate contact with the leak and possibly penetrate into it. Then, concrete was placed to hold the polymer in place and to resist the hydrostatic uplift forces.

The placement of the concrete required a considerable development effort (Rowe, 1985). It was considered essential to achieve a uniformly effective bond with the shaft lining, with no shrinkage or thermal damage. To minimize the volume of concrete to be poured in a confined space and subjected to potentially excessive temperatures, a large plug was cast around a mandrel under controlled temperature conditions on the surface, and allowed to cure for two weeks. After testing several types, it was decided to use a Type G cement on the mandrel, because of its heat tolerance and amenability to additives. It eventually withstood 800 psi of pressure with no leakage. The mandrel with the concrete plug was lowered into place and suspended above the polymer in the shaft bottom. Type G cement was pumped into the annulus, flowing around and underneath the mandrel. Both the mandrel and annulus cements were modified with plasticizers, water-loss additives, sand, and were expansive (0.12 percent at 28 days). The volume of cement was calculated from the depth, diameter of the mandrel, and the diameter of the shaft casing. Finally, a cap of an expansive Class A mix was poured over the top of the mandrel and the annulus cement, to seal any remaining pathways. It was thought that placing the plug under positive pressure and then dewatering would introduce a beneficial casing contraction. When the shaft was subsequently dewatered, it remained dry.

An example of the detrimental effect of eccentricity of the casing is afforded by another instance at the NTS. In this case, the shaft had been drilled and cemented without incident to about the same depth and under about the same hydrogeologic conditions as the previous case. However, when the casing was dewatered, a bulge in the steel casing was found about 1/3 of the way up from the bottom, and was leaking from a hairline crack just above. NTS hole deviations are about 1-ft per 1,000-ft, typically; the annular space is 1-ft. In this case, the television camera revealed that the casing was in contact with the drilled shaft wall at the top, so that the annulus was assymmetric. The bulge was estimated to be about 2-ft in diameter, roughly circular, and projecting about 6 in. into the shaft. No relationship between the eccentricity of the shaft casing, which is known to contribute to cement channeling, and the failure of the casing was established, but it is obvious that in this case, a hydraulic connection was established through the cement, and that this connection concentrated a large enough pressure on a small area to cause a loss of bonding to the casing. This would be true irrespective of any defects in the casing that would have permitted the failure itself. Had the casing not deformed and failed, it is doubtful that the existence of the hydraulic connection would have been noted.

Webster (1965) describes another shaft casing failure. This one occurred in 1965 (about 20 years prior to the ones just described). It was a 48-in.-i.d. casing in the process of being cemented. The casing was 3/4-in. thick and failed in a zone whose top was at 1,678 ft. The length of the failed zone was later found to be about 100-ft. The cause was later found to be a weld failure. About half the casing was involved in the failure. This half was deformed to produce a grossly contorted figure 8, with the two sides being in contact against the undeformed wall. Since the cement was still fluid at the time of the failure, there was obviously no hydraulic bond, and the pressure of the cement rushing into the hole tore the casing over a considerable distance. The failed section was removed by overcoring the casing and removing it piece-by-piece (the failure was above the water table). The shaft was subsequently relined and thereby recovered.

The standard cement used at the NTS is Class A, primarily because of its availability and low cost. The cement slurries are generally 15.6 lb/gal but can be as heavy as 17 lb/gal. The feeling is that the top surface relief at the end of any stage is no more than 8 ft, although cement channeling could extend this to 20 ft. (Rowe, 1985). The only additive commonly used is calcium chloride, at 2 percent, as an accelerator. In some cases, a flash set is needed; this is provided by gypsum cements. There are normally only two grout line guides for 96-in. casings, which probably is minimal for protection against channeling.

4.4.2 Early AEC Shaft Drilling Projects

Weapons testing and related underground detonations sponsored by the U. S. Atomic Energy Commission resulted in several episodes of shaft sinking by blind drilling in the 1960s and early 1970s. Among these are the Amchitka project, the Tatum Dome Project (Project Dribble), and the Central Nevada Project. Of these, Amchitka was the site of some inflows and will be discussed here. These have not been described in the published literature in any detail and much of the information is still classified. So, the following discussion may not be complete and relies in great part on the recollections of some of the involved personnel.

There were three large-diameter shafts and several smaller-diameter exploratory shafts drilled at Amchitka Island in Alaska. Amchitka was selected for its remoteness and for its proximity to seismic activity. The objectives of the Amchitka testing were for weapons testing and to provide data on how the shock from underground detonations could be distinguished from those associated with natural seismicity. At the same time, the tests provided sources of seismic excitation for investigations of the deep structure of the earth.

Project Long Shot, a precursor to the main drilling activities, was a staged hole completed at 52-in. diameter from 260-ft to the total depth of 2,310-ft, and cased with 32-in.-i.d. surface casing cemented to the surface (Kemnitz, 1966). It was primarily for seismic characterization. The Project Long Shot 54-in.-i.d. casing was successfully cemented using a latch-in shoe to deliver 2 percent prehydrated gel cement with 3 percent calcium chloride, via a 5-in.-diameter drill-pipe internal to the casing. The 32-in.-i.d. deep casing was floated into place and cemented through 4 external 5-in.-diameter grout guides, with a 2 percent prehydrated gel slurry at 13.7 lb/gal density, and 3 percent calcium chloride. Although the hole deviation was large by today's standards (6.96 ft at total depth or TD), there were reportedly no problems running the casing, and the shaft was completed.

The problems with inflows at Amchitka occurred in later shafts, that differed in construction from Project Long Shot primarily in their dimensions. There were three large-diameter shafts drilled. One was drilled 60-in. and cased 42-in.-i.d. to 3,000-ft, and a 1-megaton device was detonated in it. There was no breakout in this shaft. There was also a 120-in.-diameter shaft, abandoned at about 4,000 ft. In this also there was no breakout, and this was never used for an experiment. The inflows that are known to have occurred were in association with a 90-in.-diameter shaft (UA-2) drilled to about 6,200-ft and lined to 54-in.-i.d. This was a record depth for this diameter at the time. There also were mechanical and operational problems associated with the sinking that are not apparently relevant to the void occurrence, that will not be mentioned further here.

The detonation in UA-2 was to take place in a free-air condition. To simulate a free-air condition, a 54-ft-diameter spherical chamber was excavated, at a breakout horizon. This was connected to a lower mining level via the shaft and a steeply-inclined muck raise used for greater ease of excavation of the sphere.

The intervening strata consists of volcanic breccias and basalts, some of which are prolific aquifers, carrying seawater. The mud used was a low-weight, low-solids mix (about 9 lb/gal) designed to produce 1/32-in. filter cake. Stewart and Kemnitz (1968) also mention an oil content of 4 percent. Since the reverse-circulation, air-lift method was used, the top of the mud column was below the shaft collar. This and the increased density of sea water could have produced a very-nearly balanced hydrostatic condition deep in the hole. On one occasion circulation-loss required placement of a cement plug over almost 100-ft of hole near 3,000-ft depth, and redrilling. There were several other instances of hole instability, chiefly attributed to boulders caving into the hole.

The hole was bored in 336 drilling days, at an average advance rate of about 18 ft/day. A 108-in. surface casing was used. The hole was directionally surveyed. The net deviation at TD was about 1.5-ft; there was line-of-sight to the bottom (Hunter, 1986).

The cementing stages were carefully designed, as the casing was already under substantial pressure at the bottom at that depth. An expansive cement (Chem Comp with 2 percent prehydrated gel) at a slurry density of between 13 and 14 lb/gal was placed from the casing shoe to well above the breakout horizon (Hunter, 1986). The remainder of the hole was cemented with a relatively light (12 lb/gal) 2 percent prehydrated gel slurry.

The problem developed when breaking out through a 2-in. groutport at the cavity horizon (Hunter, 1986). This was the first port opened, and was quickly controlled through a shut-off valve that had been installed previously as a precaution against such an occurrence. The port was provided with the necessary plumbing to run a grout line (2 in.) to the surface and the trouble zone was squeezed from the surface, using standard oil-field cementing trucks. Prior to squeezing, the shaft was refilled with water to protect the casing. The pumping rate was slow and the whole operation was done with great care so as to minimize the threat to the casing. In all, about 8,000 bags of cement were pumped in the first squeeze effort.

The hole was then dewatered, re-entered, and the valve cracked in the trouble zone. The flow was still 22 gpm, so the zone was squeezed again, using the same procedure. The take on the second squeeze was about 25 percent of the first.

When reopened for a third time, the port was dry. The other 7 ports were then opened, and were all found to be dry. A third squeeze cycle was then performed through the next set of ports up (about 30 ft) and the takes were in the neighborhood of 2,000 bags. These ports were checked, and were dry. Jackleg drill holes through the ports also were dry, so the remainder of the breakout and excavation of the 54-ft-diameter sphere commenced.

During the mining of the sphere, cement was found to be occupying a 2-in.-wide crack extending to the limit of the mined area (27 ft).

The experience was clearly not a casing-related failure, but it is worthwhile noting that water was in contact with the casing at the initial breakout port. It is not known whether water was in contact with the others, but it is probably safe to assume that it was.

Also, appreciable takes were experienced 30 ft above, after two episodes of grouting below. Obviously, a void of some kind existed in the breakout zone at the completion of cementing. How it formed is unknown. One could speculate that the cement could not have penetrated very far into the fissure during the initial cementing, since Chem Comp is well-known to be highly thixotropic and the cementing was done in plug flow, and since the available pressures would not be high due to the limitation on the stage height and the relatively light density of the cement slurry. It is also known that oil-based muds can be associated with poor hydraulic bonding.

Another flooding incident at the same shaft occurred later, but was due to pump failure (the inflow originating in the mined openings, in reaching the sump, accumulated an appreciable content of abrasives) and not to annular flow (Hunter, 1986).

4.4.3 Other Civil/Military Shaft Experience

Howes (1963) describes the construction of a shaft for a nuclear test in the Carlsbad area for Project Gnome. The shaft was constructed by drill-and-blast methods to 10-ft-diameter and 1,216-ft-depth, and lined to 725-ft with 8-in. concrete above 360-ft depth and 16-in. below. The Culebra was found to be vuggy, fractured, and brecciated and was pregrouted through 36 holes with cement at 400 - 800 psi when the shaft bottom was 36-ft above the Culebra top contact. The holes ran 0 - 20 gpm with 0.5 gpm being typical. During the first stage, the total inflow was reduced to 14 gpm. The shaft was deepened to just above the Culebra and a second stage of cement grouting from that level was able to reduce the inflow only to 13 gpm. Due to slow takes in the second stage, 12 outward-looking perimeter holes were pre-injected with sodium silicate. The inflow was eventually reduced to 0.5 gpm with backwall grouting. It apparently was not necessary to reduce it further.

The Rocky Mountain Pumped Storage Project's main intake shaft (Engineers International, Inc., unpublished data) was completed to a final inside diameter of 35-ft in 1985. The excavation was by slashing out an 8-ft raise bore to the excavated diameter of 39 ft, in 10-ft rounds. The ground consists of shales, sandy silts, and sandstones, some of which are moisture-sensitive, so the shaft was equipped with a preliminary lining of shotcrete, wire mesh, and rock bolts for protection from air-slacking and for safety. This shaft is designed to convey water both to and from an upper reservoir, which is near the center of an elevated, doubly-plunging syncline. The shaft connects through an elbow to a pressure tunnel of a diameter similar to that of the shaft, at a depth of around 600 ft. The shaft has an outward-tapering flare in the top 60-ft which has a finished diameter of 50-ft at the collar. The shaft is not required to be absolutely watertight, but the concrete must be on intimate contact with sound rock for structural integrity under changing internal pressures. The shaft must be watertight enough to prevent excess water pressure buildup behind the lining during operation.

The flare section was pregrouted from the surface (elevation 1,304) in 60-ft vertical stages to a total depth of 200 ft (below the deepest evidence of surface weathering), through a circular curtain 82 ft in diameter. The grout holes penetrated weathered shale at the top and a fine-to-medium-grained sandstone beneath that, terminating 20 ft into siltstone. There were two rings of holes; the second which split the primary hole spacing. Upon excavation, the flare section was essentially dry. The flare section was concreted inside a single-form constructed to the total depth of the flare at the time (60-ft), with 5,000 psi concrete in multiple lifts. During excavation, the flare section was supported with 12-ft-long bolts on 5-ft centers. These and spot bolts used in the shaft sections were effective at stopping localized sloughage both in the shaft and in the flare.

The rock throughout, except for the upper and lower sandstones and some thin shales, is laminated with varying proportions of sand, silt, and shale. Bedding varies from about 2-ft to about 20-ft in thickness. Natural jointing is spaced at from 2- to 15-ft and occurs in two primary, nearly orthogonal, steeply-dipping sets with various random orientations also represented.

During slashing of the main shaft sections, several water zones were intercepted, none of which was particularly severe. There was fracture seepage in most of the sandstones, and a thin carbonate in the center of the vertical section produced about 40 gpm. The wettest unit on the project, a silty sandstone, is present in the tunnel but dips beneath the shaft elbow invert.

The shaft was excavated full-face, in 10-ft lifts, using the pilot bore as a muck raise. The explosives were Tovex water gel, and Atlas Kleen-Kut in the perimeter holes; overbreak was normally 1-ft but locally reached as much as 3-ft due to fracturing and overblasting. Scaling was extensive at places, using compressed air and scaling bars. At the direction of the site engineer, some areas were spot bolted for stability against major sloughage or wedge failures, as at this diameter the rock perimeter can approximate a flat face.

The wet-mix shotcrete and wire mesh were applied throughout, to a nominal thickness of 4 in. This operation proved to be time consuming, the wire mesh commonly requiring 8 - 10 hours and the shot-crete cycle up to 2 shifts overall. Locally high rebound and open vugs were problems in the shotcrete. Below elevation 999, a requirement was strictly enforced that the shotcrete was to be brought to within 5-ft of the bottom before deepening.

Obvious seeps and waterbearing fractures were piped away from the surface, but there were no backsheets or water collection rings used.

The shaft was concreted from the bottom up in 10-ft lifts. The final concrete volumes suggested an average concrete lining thickness of nearly 30 in.

After lining, the shaft was backwall grouted starting at the bottom with Type II cement in rings spaced at 10 ft with 8 holes per ring, in 1-ft stages. The original pressure specification was hydrostatic (as measured with packers) plus 125 psi. Despite the depth, the highest hydrostatic pressure measured was only 80 psi at Ring 19, less than half the full head as would be predicted referencing the surface. Grout takes averaged 0.5 cu ft/ft from elevations 737 to 1,020. The grouting was not completely satisfactory in the first 23 rings, especially for the first 180 ft, and this interval was regrouted from the bottom up on a split spacing, using Type III cement. Takes were spotty. From elevation 737 to 1,020, the average take was 0.3 cu ft/ft, with some zones accepting less and others, considerably more. In the remainder of the shaft, the average take was 0.77 cu ft/ft. At Ring 24, the allowable injection pressure was increased to 250 psi or 1.1 psi/ft, whichever was less. The mixes were thick in both phases (w/c = 3:1 or thicker) to reduce bleed. The change to Type III was to permit a lower w/c to be used without too much gain in viscosity.

After the backwall injection program, some holes in the lower 180 ft and in the upper part of the shaft continued to weep; these were plugged and grouted. There were no final injections for complete sealing.

The difficult grouting can be attributed to the nature of the shotcrete-rock interface. This interface is thought to be irregular and poorly bonded in places as a result of the use of a wet-mix and water on the rock surface.

4.5 POTENTIAL MODES OF VOID OCCURRENCE IN SHAFT GROUT SYSTEMS

Voids associated with grouting of shaft lining have to do with the nature of the shaft components penetrated and how they behave in response to excavation and placement. Although this discussion will emphasize how voids can and have occurred in the grout placed between the lining and the rock, it is desirable to acknowledge sources of shaft leakage in the primary lining materials themselves and in the rock. Therefore, these phenomena will be given cursory mention.

4.5.1 Voids in the Principal Lining Materials

Principal shaft lining materials include the steel shells or casing, concrete, reinforcement, shotcrete, wire mesh, chipboards, backsheets, concrete blocks, and poured or blocked asphalt. Inadequate performance on the part of these components can threaten the performance of the grout by admitting water, with the potential for undercutting the grout by dissolution, weakening the grout by diluting the mix, attacking the grout chemically, eroding it either before or after it has set, or generating adverse pore pressures that make the grout more susceptible to failure by weakening. Adverse principal lining material performance can also include structural failure that can overstrain the grout.

Voids in directly-placed concrete lining materials can result from the natural permeability of the concrete due to its porosity and entrained air, as augmented by fracturing associated with shrinkage, heat of hydration, aggregate reactivity, or strain. Sources of concrete cracking were described in Section 2.1.7. Gilbert (1982) presents evidence of concrete shrinkage in a halite zone at Riccall. The larger the annulus, the more likely is thermal cracking. Concrete placement effects can completely override these, however, and include honeycombing, segregation, trapped air pockets, erosion due to ineffective water control (which preferentially removes cement and sand, leaving a honeycomb-like structure), cold joints (unplanned pour interruptions), and mix variation. Concrete delivered to the subsurface via slickline is believed by many to be more likely to segregate or honeycomb than bucket-delivered concrete. Slugs of separated concrete reaching the remixer may not be adequately reincorporated before delivery to the forms. Although gaps between poured concrete and well-scaled rock surfaces are infrequent, other very important sources of leakage that are addressed by grouting are pour boundaries (matcher joints), and seal blockouts. Mud and debris can adhere to these surfaces. Even with proper placement practice and adequate design of the construction joints, an improper mix that does not have sufficient strength, fluidity, or resistance to chemical attack by ground water can reduce the effectiveness of grout.

Concrete placed against frozen ground must be temperature controlled to prevent its freezing before it sets, and differential thermal cracking after it sets. The water generated by the local thawing of the ground adjacent to the fluid mix can add water at the contact, weakening or channelizing the bond.

Other primary lining materials that can be important in grout performance are backsheets, steel linings, reinforcing steel, steel ribs, tubbing, liner plate, asphalt, and seals. Backsheets should obviously be as chemically inert and physically durable as possible. Steel linings, reinforcement, and other metallic components are subject to corrosion and bonding problems, especially if coated with grease or mud. Asphalt with a high volatile content can densify after extended periods of heating, reducing the ability to flow and seal, and resulting in cracks. Although some asphalts from Utah have up to 85 percent asphaltines, most paving asphalts are not well-enough controlled for shaft work and may have higher volatile contents.

Differential thermal responses of the components of complex composite linings can interrupt bonding and create damaging stresses and displacements. Expandability of seal materials can enhance joint permeability in the directions perpendicular to the least horizontal stress.

4.5.2 Voids Occurring in the Rock Mass

The rock mass itself is an integral component of the shaft system and its performance should be assured also if watertightness is desired. The role of the liner-rock interface in annular flow may be subordinate to that of the increased-permeability zone in the rock (Kelsall et al., 1982). In evaporites, dissolution and grout effectiveness are involved in a circular interaction that can threaten the entire shaft system or bring on complete failure, as the case histories have shown. Rock mass disturbance (the Disturbed Rock Zone, or DRZ) results from direct blasting damage and local wall-rock failure (spalling, sloughage, and overbreak), which, together with stress relaxation, can produce deep-seated changes in rock mass strength and permeability (Kelsall et al., 1982; DOE-WIPP 221). A salt mass at 2,000 ft could experience an 8-fold increase in permeability even at 20 radii (Kelsall et al., 1982). Discontinuities such as naturally-occurring jointing, cleavage, or bedding, vesicularity or natural porosity, and brine or gas pockets, can challenge the effectiveness of grouts. The permeability of these will vary with their orientation on the redistributed stress field around a shaft (Kelsall et al., 1982). In particular, the release of stress, creep, and trapped brine may be responsible for recurrent and intermittent seepage in salt mine shafts. Fractures and cavities that are clay-filled or are coated with soluble materials such as gypsum, carbonates, or halite are susceptible to drastically-increased permeability if erosion or dissolution are permitted to occur. Poor rock-surface cleaning can affect concrete-rock bonding due to buildup of mud or loosened rock.

The phenomenon of ground thawing deserves special mention because of its potential use as a ground improvement method for repository shafts. The literature emphasizes the establishment of an adequate frozen annulus for safe construction but far less on the effect of thawing on the lining system and the surrounding rock or soil. When ground is frozen, ice lenses tend to form as a result of segregation of water, and ice expansion. This occurs in zones concentric to the freeze pipes that coalesce as the shaft perimeter is approached. Most of the water migrates towards the shaft centerline, where the excess flows out of the pilot hole and the remainder freezes, stressing the ground. Chamberlain (1981) found that permeabilities can be enhanced by 100 percent, relative to the undisturbed case, in clays subjected to freeze-thaw effects. Clays and silty clays are more affected by ice segregation than sands (Schuster, 1981). The clays become overconsolidated during the freezing process; upon thawing, they lose water up to the plastic limit, shrink, and settle. This type of phenomenon occurs during the thawing of freeze shafts (Figure 34). The ground adjacent to the lining undergoes multiple freeze-thaw cycles. Figures 35 and 36 show the effect of the heat exchange between the frozen ground and the lining materials in a large frozen shaft in Germany. Welding on steel casings will also produce local thaws. These multiple transient freeze-thaw episodes can enhance the segregation of water further as the thaw repeatedly advances and retreats both radially to the shaft and in the vertical direction with the progress of lining. When lining is finished and the ground is allowed to thaw naturally it does so from the perimeter outwards. Any free water in the system cannot escape outward because of the remnant ice wall, so it becomes distributed along the shaft-wall where it is free to enter the shaft liner wherever it can. Generally backwall grouting is initiated soon after the wall temperature warms past 32°F (0°C). Thus the grout is introduced into a system containing abundant free water constrained radially by a remnant ice wall.

Another highly relevant grouting issue is the effective abandonment of freeze holes. The case histories have pointed out many cases of inflows or failures resulting from water-bearing freezeholes, as well as deviated freezeholes. These present separate sealing problems.

4.5.3 Voids Occurring in Grout

Of direct relevance to this report are the potential sources of voids in the grout itself. Grout can be injected into shaft liners and behind liners, or be part of the liner itself, as in a cemented casing in a drilled shaft. In the latter case, voids can result from shrinkage, mud-cement incompatibility, channeling, bleed, hydration heat effects, poor hydraulic bonding, and poor placement practice. In injected grout such as that used for contact grouting of shaft liners or behind liners, voids can occur due to bleed, grout degradation, insufficient coverage/penetration, structural failure (cracking), dilution, erosion, inhibited set, and procedures used. The following discussion is developed from experience with various forms of grouting as well as actual shaft experience such as that summarized in Sections 2 and 3 and in the case histories.

Bleed probably heads the list as a mechanism for void occurrence common to all the forms of grouting addressed above. The w/c ratios (thickening to 1:1), that have been classically used to achieve penetration of the irregular voids behind backsheets in backwall groutings, are susceptible to bleed. Annular cement has relied on thixotropy and prehydrated bentonite to promote suspension and

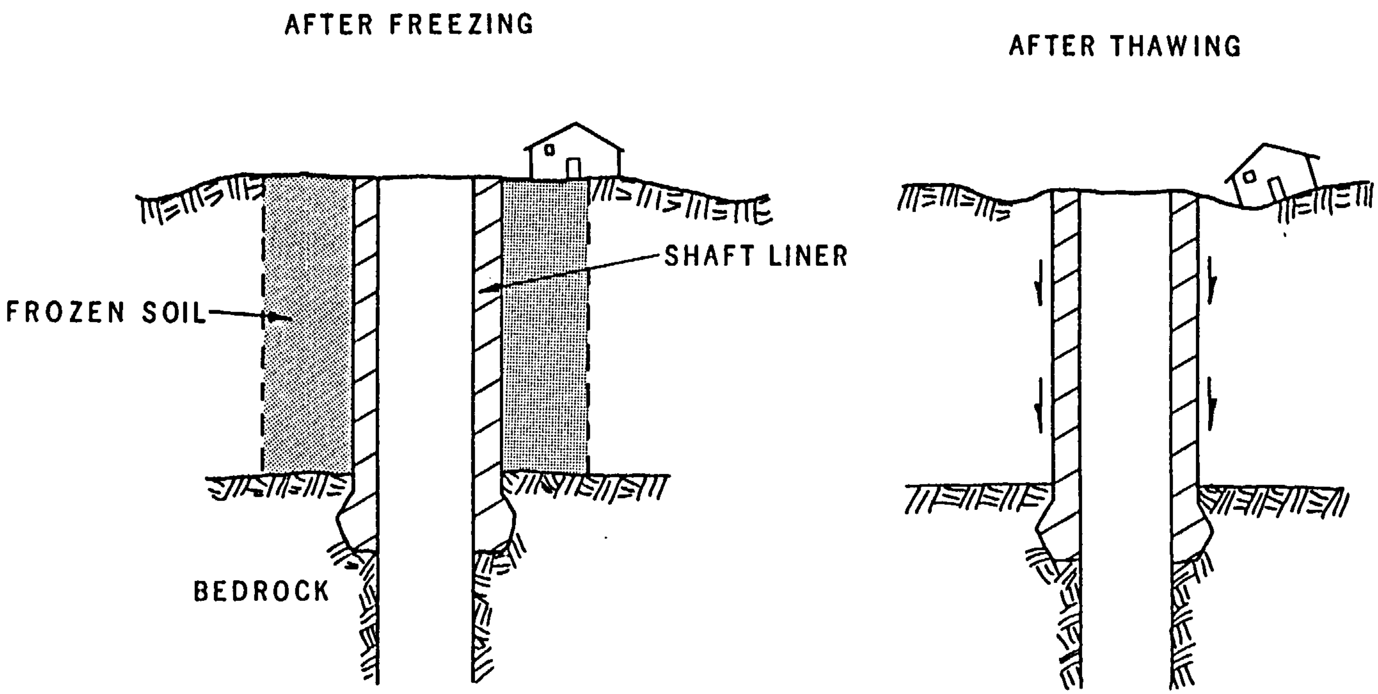


Figure 34. Surface Settlement After Thawing Due to Net Water Loss from Formation (from Chamberlain, 1981)

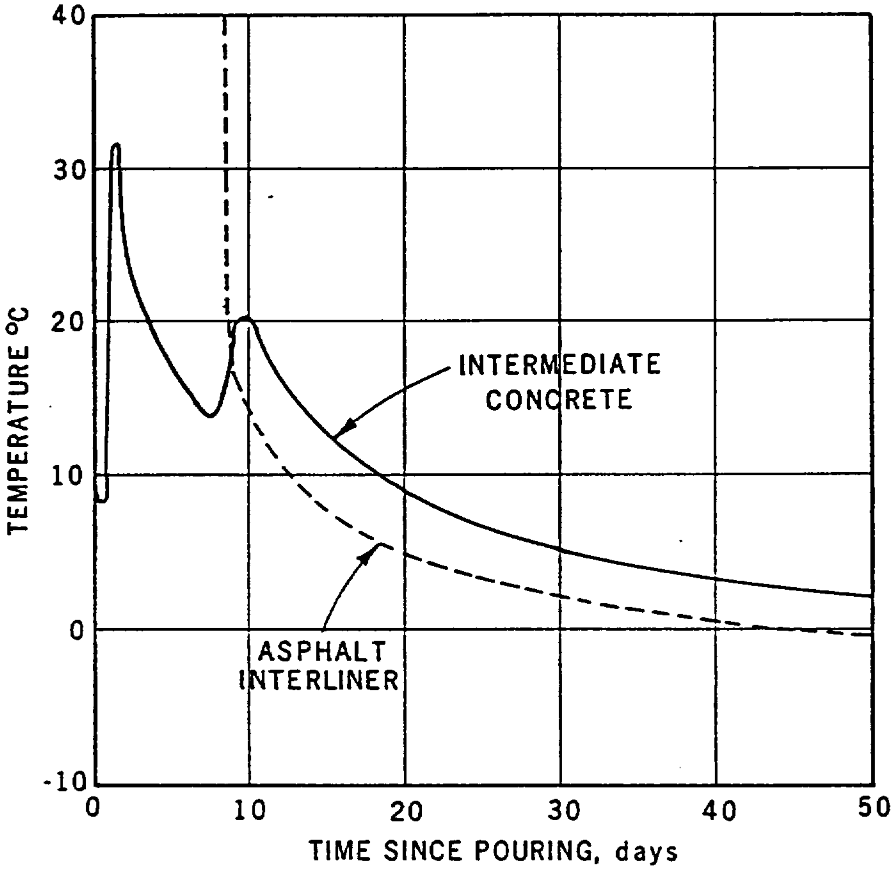


Figure 35. Effect of Frozen Ground on Asphalt Interlayer on Wulfen No. 1 Shaft Composite Lining (after Kampschulte, Lehmann, and Link, 1964)

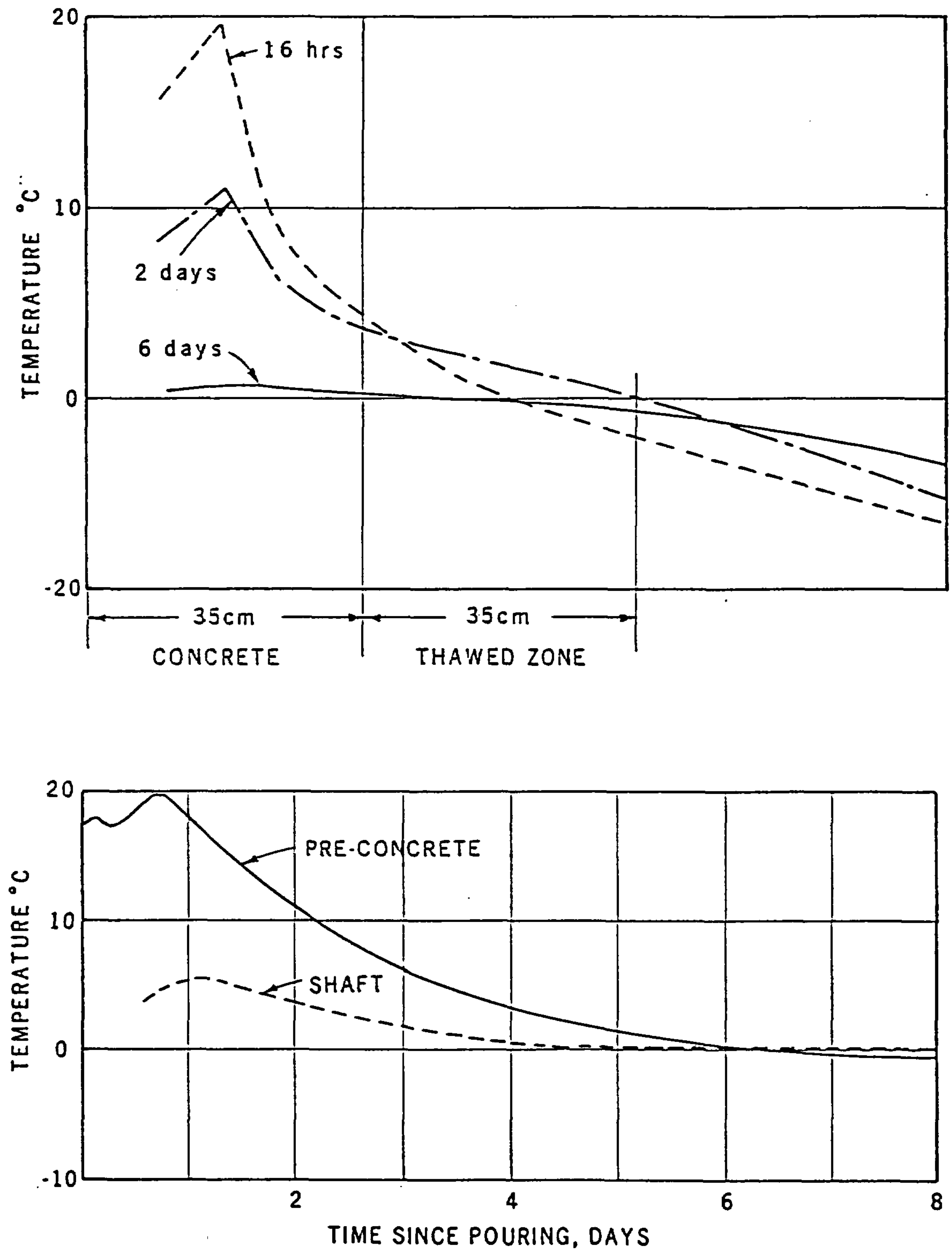


Figure 36. Interaction of Concrete Hydration and Frozen Shaft Walls, Wulfen 1 and 2 Shafts.
Above - Effect of Heat of Hydration on Frozen Ground
Below - Effect of Frozen Ground on the Shaft Concrete (after Kampschulte, Lehmann, and Link, 1964)

counteract bleed, potentially at the expense of strength and bonding. Because of the volumes involved, sanded annular cements are susceptible to settlement. Bleed can be effectively countered by admixtures.

Incomplete penetration of injected grout can result from viscous mixes, flash setting, and blockage due to too large a grout particle size or agglomeration of particles. Thus refusal may not be associated with the extent of water stoppage that was intended in the grout design. Grout has greater bulk than water, so that there are points near all crack tips where water cannot be displaced by grout. This has much to do with why rock of less hydraulic conductivity than 0.0001 cm/sec cannot be improved much by grouting, yet can pose isolation concerns. Annular grouts are not likely to penetrate fissures due to thixotropy and limited pressures, and the probability that fissures capable of accepting grout would be plugged with lost-circulation material anyway.

As discussed in Section 2, some grouts degrade chemically or physically or are unstable under in-situ conditions. Most chemical grout gels can dehydrate, shrink, and never recover their original sealing capabilities. The dehydration effect may take place in some saline environments, in the absence of actual drying, by changes in salinity of the water in contact with the grout in-situ. Chemical grouts may not be strong enough to withstand ground pressure disturbances, and once fractured, most cannot reheal. The stability of low-viscosity chemical grout gels at elevated temperatures that could reach repository shafts is doubtful. Cement grouts can shrink, particularly in the long term.

Grouting procedure can result in voids. Insufficient hole density, hole depth, pressure, or provision for water relief can limit grout travel or leave gaps in the grout coverage. Lack of sufficient mixing energy can result in poorly-hydrated, bleed-prone mixes that can be bypassed by water at the point of application (Houlsby, 1982b). The use of high pressures to promote penetration without water relief can induce dilation of fractures and entrapment of pressurized formation and bleed water near crack tips, that together widen the zone of increased permeability, If pressure is released too early, bleed water can be forced back through the incompletely-gelled grout mass and create channels. Chemical grout gels may not have the strength to withstand the differential pressures introduced if the grout lines are disconnected too early or the means of shutting in the grouted hole generates a transient negative backpressure. Grout penetration can be limited if hole communication is high. Pumping should continue until all water and dilute grout has been forced from the relief holes. The grout materials should be fresh and properly stored and handled. Old materials can partially hydrate and be difficult to properly mix, or experience setting and durability problems.

Flowing water in the grouted zone can erode, dissolve, or dilute the grout before it has set, or even (in the case of cement grouts), after the set but before full strength develops. Erosion of the casing annular grout may have been a contributing factor in the Amchitka inflow. Dilute chemical grouts may not set at all at the extremes of their penetrability, due to dilution or variation in the mix ratio by the time the grout reaches the crack tips. "Salt-saturated" cement grouts injected into halite or potash may be undersaturated at the injection pressure, or actually incompletely saturated at the outset, and therefore capable of accepting additional salt during injection, causing erosion and dissolution, retarding the set, and changing the chemical and physical properties.

The grout mix itself must be designed for compatibility of all admixtures among themselves, with the cement or chemical base, and with the environment being injected into. Gel times and thickening characteristics should be evaluated for the downhole conditions, not just those in the laboratory. The grout should be compatible with its own mix elements since some reactants may not be entirely consumed. The hazards that can be presented when drilling a shaft define potential adversities of the cementing environment. These include squeezing or swelling shales, caving or sloughing zones, lost circulation zones, artesian flow, and fishing jobs. Tight or deviated holes can cause problems tripping in and out and running casing, with the potential for additional rock damage and crushing of grout-guides, as well as cement channeling due to casing-wall contact. These are examined in terms of cementing in the following paragraphs.

There are significant difficulties that can occur in annular grouts placed beneath a fluid interface, such as is done when cementing blind-drilled shaft casings. Poor cement jobs result from improper casing centralization, improper mud conditioning, poor mud removal, mud contamination by cement, mud-cement incompatibility (excess interface viscosity), slurry fluid loss, gas cutting, lost-cement circulation, formation breakdown (cement fallback), and washouts (Kirksey and Warembourg, 1980).

The most important of these is probably effectiveness of mud displacement by cement, which is a complex question involving viscosities, flow regimes, geometries, and chemical compatibilities.

Chemical compatibility can be an intrinsic problem because most muds are siliceous due to added or acquired bentonite, and the cement is alkaline (calcic). Thus the interaction tends to form calcsilicate globules (Nelson, 1986). Muds with a high pH to prevent drilling equipment corrosion may require an acid prewash before cementing for reduction of filter cake and mud removal from the casing surface, so that the incompatibility can persist from this source also. Polymeric muds are more compatible with cements than bentonite muds.

Effective mud replacement in plug flow is hindered by a constricted annular area, which when combined with the mud gel strength can allow the mud to bridge and support the cement around and above it (Figure 37). The result is cement channeling and the formation of mud pockets. The case histories have some occurrences of this in drilled shaft lining. It is a common concern in oil-well cementing, and most of what is known about channeling mechanics is derived from oil-field experience.

The bouyant driving force resulting from the mud-cement density difference can have a diminished effectiveness in mud replacement (Clark and Carter, 1972) and can be easily overcome by the drag forces occurring at constrictions (Figure 38). Unless mud mobility can be improved by conditioning, washes, or dilution, the bouyant force is not efficiently utilized.

Mud that adheres to the casing at the time of cementing, and filter cake that has not been removed from the shaft walls, interrupt hydraulic bonding (Evans and Carter, 1962) and, even if initially of low permeability, represent components of the lining system that are not controlled and are of questionable long-term performance. It is possible to control to a limited extent the toughness and thickness of the filter cake by altering the mud fluid loss, strength, and density (Table 13). However, there is only one mud in the hole at a time, and there may be considerable variation in formation permeabilities, pressures, and strength, so it is not likely that the filter cake will be uniform or even uniformly controllable throughout the shaft.

The consistency of the mud seems to have more to do with the effectiveness of the mud removal than does the consistency of the cement (Haut and Crook, 1979).

A gelled interface can form a above high-early-strength or prehydrated gel cements, or due to mud-cement reactions, that inhibits the uniform rise of cement and causes penetration of the mud in a sporadic way. Haut and Crook (1979) noticed that once cement had found a pathway, it tended to follow the same path. Even thick cements can channel under the right conditions. In a supportive mud environment a thixotropic cement such as Chem Comp can receive enough support laterally from its own slurry strength and the mud to create columns of the type that apparently occurred near the tops of the cement stages at Crownpoint. Richardson (1986) described a cementing project in Alberta where the mud gelled and allowed cement channeling to occur to the point where grout lines could be lowered well below the top of the first stage. The mud pockets were filled by squeeze cementing. In oil-well cementing, the options for breaking down the gel strength of the mud, besides conditioning, pipe centralization, and density differential, are pipe movement (reciprocation and

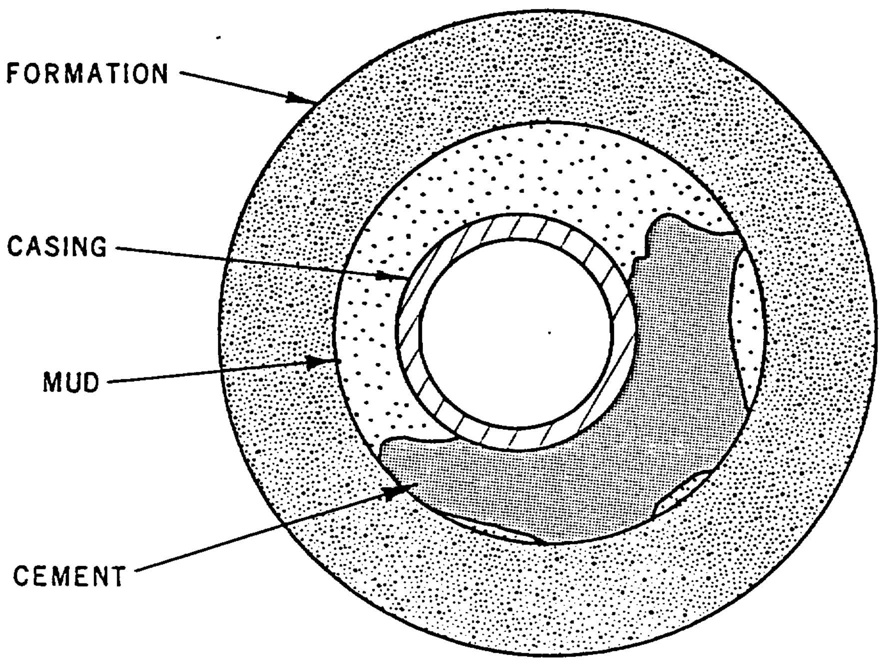


Figure 37. Occurrence of Cement Channeling
(after Haut and Crook, 1979)

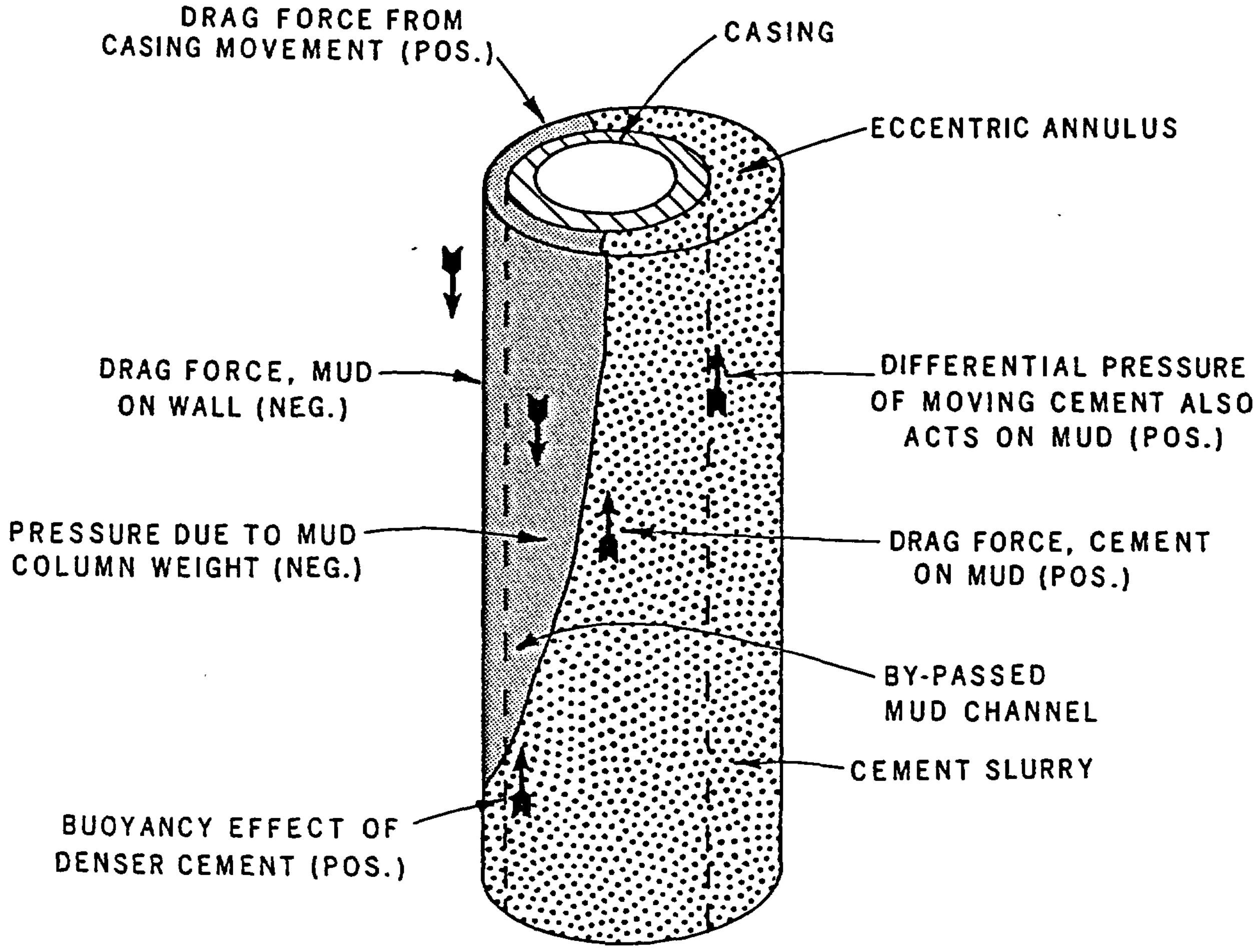


Figure 38. Mud Displacement Mechanics, Showing Potential for Channeling (after Suman and Ellis, 1977)

Table 13: Typical Filter Cake Characteristics of a Drilling Fluid

Drilling Fluid Properties			Filter Cake Thickness, mm, built up in 30 minutes at 83°C/6.8MPa (180°F/1000 psi) on core with permeability of:		
Yield Point	Plastic Viscosity	API Fluid Loss			
Pa	Pa·s	cc/30 min	1 md	15 md	200 md
96	.022	6.5	4.0	3.2	3.2
0	.032	5.2	4.8	5.6	3.2
48	.033	4.6	4.8	4.8	4.8
144	.029	6.2	3.2	5.6	3.2
96	.017	5.0	3.2	3.2	3.2
	AVERAGE		4.0	4.8	3.2

rotation), maintenance of turbulent flow regimes, and high flow fluxes. In shaft-casing cementing, where the annular space is large and the casing weight exceeds the suspended load capacity of the rig, the latter three options are not present.

Even if displacement of mud is generally effective, mud pockets are likely in borehole wall irregularities and around casing irregularities such as stiffener rings (see the Piceance Creek oil shale shaft case history in Section 4.2.2). Annular cements are cavity fillers, not penetrating grouts, and will not move into highly irregular caving or sloughage zones in the shaft walls. The role of these in promoting annular flow is entirely dependent on whether such pockets are extensive enough to link aquifers. In lost circulation zones, the lost circulation materials can trap mud within or behind them. Concrete plugs placed as a last resort in drilling through lost circulation zones are anchored in such materials and, since their placement is not directly controlled, there is potential for entrapment of mud within the concrete or at its interface with the rock.

To some extent, contamination will occur in the cement used behind shaft casings. Partially-gelled cement, filter cake, and other foreign material, can get caught up in the cement but the consequence of this is likely to be minor. However, practically all stage cementing jobs are likely to experience contamination at the top surface of the stage, which is due to a combination of mud-cement interaction, localized turbulence at the beginning of the stage, and fluid loss effects. Just before the stage is begun, the grout guides are full of mud, and the issuance of first mud and then cement is accompanied by turbulence as the cement falls through the mud to the top of the preceding stage. In so doing, a contaminated cement can be incorporated into the stage. As cementing proceeds it could well be carried upwards for some distance in the annulus. In pure plug flow, it is unlikely that much further turbulence would occur, but if channeling mechanisms are operating, slumping and coalescence of channeled cement can be accompanied by further turbulent mixing of mud and cement. Although the effect has not been studied first hand, it is unlikely that the initial contamination zone would be carried upward for the full 150 to 400-ft-length of a typical stage. More likely, drag against the rock and casing, augmented by irregularities such as stiffener rings and scribing from the gage cutters will result in a particle motion towards the sides of the annulus that would move most if not all of this contamination to the outside and inside of the annulus. In those few instances where it has been studied, stage tops show some evidence of mud-cement mixing. Mud-contaminated cement is less dense, and more permeable.

Rowe (1986) in studies of the tops of sanded cement plugs placed beneath mud, reports that a layered zone, about 14 in. thick, of poor-quality cement was noted. The top is generally chalky due to mud contamination and bleed and the cement gets a little better a few inches below. Beneath this is a sandy zone with little cement, gradually grading downwards to the design mix. The effect was observed in surface mockups where extensive vertical stage heights did not precede plug placement.

Annular cements behave exothermally in a way that is similar to mass pours. A sufficient annulus is necessary in minimizing channelling due to casing eccentricity resulting from casing straightness and hole deviation problems. The case histories have mentioned several instances of problems arising from a too-narrow annulus. To combat this and be sure the grout lineguides clear, special deviation and casing fabrication controls are practiced, and in addition, annuli of 1 - 1.5 ft are typically provided. Thus the adequate dissipation of the hydration heat from curing a 300-ft stage in a 10-ft hole becomes important. Sand can be added as a heat sink and fluid can be circulated within the casing to dissipate the heat, as well as to provide internal support. Nonetheless there is potential for concentric longitudinal cracks due to differential thermal expansion, radial cracking due to shrinkage on cooling, and the formation of a micro-annulus due to differential cooling response of cement and casing, coupled with casing relaxation when the head from the internal water is removed. Dehydration shrinkage is normally not a problem in blind-drilled casings, because curing conditions are typically moist (Griffen et al., 1979) but could be if fluid loss, incomplete slurry mixing, or insufficient water for hydration occur.

Hydraulic bond has been mentioned repeatedly in this report in connection with drilled-shaft-casing cementing. The establishment of a good hydraulic bond initially would greatly enhance confidence in the long-term performance of the casing-cement system. Hydraulic bonding is weaker with respect to gas than to water. The presence of mud on the rock or casing (particularly late, bentonite, and sodium silicate muds), and the presence of filter cake on the rock are unfavorable for hydraulic bonding. Obviously the casing surface should be free from grease, oil, loose rust, or scale, and mill-varnished surfaces are not conducive to good bond. Larger casings are more susceptible to the effects of thermal and mechanical expansion and contraction. If not compensated by cement expansion, a micro-annulus of 0.002 in. or more is theoretically possible (Griffen et al., 1979). Suspected poor bonding or a micro-annulus could be treated through backwall grouting.

Poor hydraulic bond without discrete void formation can result from fluid loss to the formation before the slurry sets. Because of the greater slurry weight, fluid loss from cement slurries can occur in zones that do not exhibit noticeable circulation loss during drilling. Fluid loss results in a poorly-hydrated cement that is week and porous and subject to shrinkage cracking. Unfortunately the best pallative for filter loss is a low-permeability filter cake, which will tend to form in precisely those zones most susceptible to filter loss. Total removal of the filter cake obviously would subject the slurry to loss of slurry-filtrate. It is doubtful that total filter cake removal would be possible because there are always some suspended solids in the mud. Nonetheless, some filter loss additive (Section 2) would aid in preventing filter loss.

A potential source of voids in deeper shafts or where highly pressurized gas is present is gas-cutting. The gas pressure need not overcome the full head of the slurry. Constrictions can lead to cement dehydration or bridging, wherein the slurry develops enough strength to support the overlying slurry, which decreases the effective back-pressure on gas-bearing formations. Levine et al. (1980) present a graphical method to predict gas invasion potential. Once it sets, cement will resist gas invasion. At most shaft depths, it is unlikely that sufficient gas pressure would exist to provide gas entry unless bridging augmented with dehydration below the bridge, were to occur. Bridging can result from invasion of debris, cement-top gelling due to reaction with mud, loose mudcake, accelerated cement set, or dehydration (fluid loss). Lindsey and Bateman (1973) mention augmentation of gas-cutting in overly-retarded cements (used mostly in hot, deep oil wells). Gas-cut cement can be difficult to post-grout.

Casing damage can occur from corrosion, and/or defective installation or fabrication. Generally casings with effective cement sheaths that are not in contact with the shaft wall are not subject to corrosion. However, if a replenished source of oxygen exists, it can combine with hydrogen sulfide in ground water to produce sulfuric acid. Electrolytic reactions can also occur if the casing is in wall-rock contact. Casing can be damaged during installation if the hole is doglegged. Fabrication problems include weld quality at the rig, mill damage, out-of-roundness, seams, laps, rolled-in slugs, pits, gouges, and cracks. Differential pressure sticking (Suman and Ellis, 1977), often a problem in oil-well casings, is unlikely in a ring-stiffened casing.

Dissolution can occur when undersaturated cements are placed into contact with soluble strata.

Procedural effects include batch variability, use of washes and mud conditioning, selection of stages and monitoring of cement rise, water circulation during cement hydration, numbers of grout lines, hole straightness and casing alignment, quality of pre-drilling investigation and frequency of caliper checks, elimination of need for fishing, and adherence to quality control standards. Batch control is particularly important so that the slurry density can be optimized to prevent fluid loss due to high weight, or fluid or gas invasion due to low weight. Often, stages of prehydrated gel or neat cement are tailed off with 10- or 20-ft of expansive cement for improved sealing.

5.0 MINIMIZATION OF VOID INFLUENCE IN SHAFT GROUTING

This brief section is a prelude to the conclusions to this report. It is intended to describe the precautions and practices that are available to optimize shaft liner grouting. The discussion focuses on control of lining materials and grout but, as indicated in Section 4.5, the lining and post-excavation rock function together as a system, to effect the overall performance. Thus, effective pre-grouting, minimization of rock mass disturbance in the shaft walls, effective abandonment of freezeholes, and adequate water identification and control prior to and during sinking are important contributors to the grout in minimizing the occurrence of voids initially and during extended shaft operation.

5.1 LINING PLACEMENT CONTROLS

The following measures are available to prevent situations that promote voids within concrete composite linings that, as indicated previously, may compromise the grout function:

- prevention of bleed/settlement, segregation, and air entrainment, through the use of plasticizers, vibration, remixers, and dispersants
- control of construction joints, but using sloping kerb rings, cleaning of and sealants in matcher joints, and positive pressure on concrete at the tops of pours
- construction water control with non-corrodible backsheets, depressurizing wells, pre-grouting, and so on. The less water in the shaft during construction, the better. Freezing should be considered critically as a method of water control because of the shaft wall disturbance that results
- mix-delivery control using sufficient delivery points for adequate concrete spreading and minimization of honeycombing and segregation, especially around reinforcement
- design of stage heights to minimize through-going pour boundaries. Offset pour boundaries between primary and subsequent linings
- sufficient curing times before stripping forms, to prevent shrinkage or plastic deformation
- protection of green concrete from blasting vibrations

- appropriate mix design for formation hydro-chemistry, dissolution, chemical attack of concrete, etc.

- quality control and inspection of lining operations including batch uniformity and adherence to design, pour procedures, pour preparations, water control effectiveness, and other activities. Thorough wall cleaning and scaling

- high cement and sand contents for reduced matrix permeability

- use of low-volatile bitumens, with sufficient mixing periods and density control, in composite linings

- use of washed aggregates of appropriate makeup

- control of dry-mix moisture contents

- temperature control of placement and curing

- high-quality mix water.

5.2 GROUT PLACEMENT CONTROLS

The preceding discussions and the case histories have illustrated numerous procedural and design measures that can be used in avoiding problems, when placing annular or backwall grouts. For annular grouts such as are used in drilled-shaft linings these include:

- maximizing compatibility of mud and annular cement. Use of mud decontaminants. Use of calcic or polymer muds

- frequent mud conditioning, minimally after the completion of drilling and after running casing and preferably after or more stages of cementing, with the use of spacers or scavengers to clean grout guides

- pre-flushing each stage before cementing and removing any debris

- filter cake thickness minimization or complete removal; with washes or even jet subs to induce turbulence, and/or with slurry loss compensated for by fluid-loss preventives in the washer slurry mix

- rational design of cement slurry, in terms of density, chemistry, expansiveness, heat control, thixotropy, fluid loss prevention, bleed segregation, and hydration (minimization of interference) on the basis of exploration data and drilling performance

- sufficient WOC time and verification of adequate set

- sufficient number of functioning grout lines

- adequate stockpile of properly-stored basic and contingency supplies

- minimal or no fall distance of cement when initiating each new stage

- assurance of casing quality; absence of mechanical damage, high welding quality, careful alignment of sections, surface preparation of casing conducive to good bonding

- chamfered or trapezoidal stiffener rings to minimize the trapping of mud

- straight, plumb hole; control of bit weight and penetration use of stabilizers surveys

- minimization of sloughage, spalling, washouts, debris falling during cementing, and so on, by careful mud design based on a focused exploratory drilling and downhole testing program

- circulation of water in casing during WOC period, with optimal base level of water for internal pressure on casing and acceptable casing deformabiltity

- provision of grout ports in casing for backwall grouting program after cementing is complete to mitigate potential channeling throughout the liner, as well as in the formations expected to cause the worst problems

- sufficient annular space, or capability to ream the hole to obtain it

- accurate knowledge of grout-line pressure losses to avoid overpressurization of grout at the outlet while controlling flow rate for best displacement

- quality control and inspection; beginning with drilling mud formulation and drilling practice and extending to the completion of cementing

- preservation of exploratory drilling information from the shaft site and the surrounding area

- support of hole preparation procedures and cement displacement behavior predictions with mock-up tests, especially in verifying slurry flow properties beneath the mud

- relevant mud formulation to formation-water chemistry and dissolution characteristics

- centered casing

- reliable measurement of formation and fracture gas and water pressures, especially delineation of overpressured, confined formations

- sufficient cement density (2 lb/gal density difference is probably minimal for displacement in plug flow)

- tailing of the upper, filler cement stages with expansive cements or use of expansive cements throughout

- choice of stages with respect to confinement of aquifers, not just the maximum slurry height the casing can withstand

- detection of borehole wall irregularities, shallow fractures, etc. prior to cementing and appropriate action, such as jetting.

Good backwall grouting practice would include the following:

- non- or limited-bleed cements; low w/c ratios (less than 0.4) that are viscosity compensated with superplasticizers

- avoidance of unstable or weak grouts; verification of grout stability with chemical, thermal, and mechanic overtests and petrographic studies

- verification of mix control, mix-materials quality, documentation, adherence to pattern

- multiple grouting phases tailing off with a micro-fine cement to effect optimal filling of bleed passages

- gradual increase to the design pressure limit for each stage, checking for problems throughout, but not slow enough to approach gel times in the hole before grout reaches the necessary limits

- in the case of freeze shafts, multiple phases of grouting with consideration to the possibility of flowing water and dilution, keyed to thawing behavior measurements

- sufficient maintenance of pressure after refusal has been reached

- well-cleaned grout holes

- adequate pressure relief for venting of displaced and bleed water (if any)

- redrilling for verification on spot basis

- improvement of penetration with pre-injection of sodium silicate or with water washing

- sufficient mechanical mixing energy (not just time) to break up flocs of cement particles, to improve hydration at low w/c ratios and reduce cement grout particle size

- design pressures sufficiently low that hydro-fracturing does not result

- prevention of vortex during grout mixing or other sources of entrained air in grout

- use of a permeability test standard to determing the completeness of grouting

- use of a standard to verify setting times in-place to afford regrouting of set times are too long

- use of pressurized grout seals to isolate aquifer zones

- avoidance of methods requiring packers, where possible.

5.3 VOID IDENTIFICATION

It would obviously be useful if there were a method that could unequivocally delineate voids in shaft linings of the dimensions that could pose a problem during operation of for long-term performance after decommissioning. Useful techniques would ideally identify void formation during grout placement so that steps could be taken to change the mix or the procedure and eliminate them. At the very least it would be useful to have a technology that would delineate voids in the completed shaft lining so that an appropriate remedial action could be taken and it's effectiveness verified.

5.3.1 Verification of Grout Quality in Drilled Shaft Casings

5.3.1.1 Monitoring During Placement. The present state-of-the-art of grout placement monitoring for annular grouts address mainly the pumping progress and the location of the top of the grout.

Parameters that are measured at the surface include flow rate, slurry density, feedstock quantities, and pressure. Flow rate is best measured with a turbine flowmeter. Slurry density is usually measured with standard oil-field downstream densitometers. On some jobs, only the truck line is monitored for density and pressure, because the flowmeters are expected to reflect any problems such as blockage in a single line. However, the chance of difficulties with a single-line that a better-monitored job should collect pressure data from each line. Generally the data are fed to a strip-chart recorder so that a continuous record can be obtained of the whole cementing stage. At Crownpoint, there were 4 grout guides and therefore 4 flowmeters where the grout lines issued from the manifold; for these and the single densitometer and pressure logs, a 6-pen recorder was required. Normally, additional samples are collected at intervals for verification of properties after laboratory curing at the bottom hole temperature (BHT).

Prior to cementing, a caliper log is usually run to estimate the fill-up volume for the stage. The theoretical volume can then compared with the actual to determine if there is excessive fluid- or slurry-loss. In variable ground, filter cake buildup in some zones could compensate for fluid-loss or slurry-loss in others, and the net change on the stage totals would be misleading. An instantaneous record of the fill-up volume versus theoretical would help in identifying these situations. Also, before cementing, the depths to solid grout (or bottom, if the first stage), are determined. During pumping, at the NTS, note is made of the depths to grout line terminations, cement upper level, start-and end-depth of pour, and (sometimes) mix-water volume.

The top of grout is usually monitored during the cementing with a Nuclear Annulus Investigation Log (NAIL) or Nuclear Cement Top Locator (NCTL). These tools are normally run down a grout line guide dedicated for the purpose. These are essentially nuclear density logs, responding to the differential radiation absorption of the grout versus the mud. As mentioned before, there can be a gradation at the top of the cement due to mud-cement co-mingling. Figure 39 shows an NCTL log response to the top of the cement at Crownpoint. These logs, if run in a single guide, do not detect cement top variability since they respond only to the interface nearest the guide they are in. They can be confounded if a sufficient density contrast does not exist between the cement and the grout. In addition to verifying the completion of a stage to its intended height, the NAIL or NCTL logs provide data to keep the injection point below the top of the grout surface and correlate with volumetric monitoring to detect slurry- or fluid-loss. They measure a gross density contrast and are sensitive only to relatively large voids. There is some potential for their use in post-cementing void detection if run down open utility lines, but the sensitivity would be poor and the interferences great. At the NTS, some attempts have been made to identify voids using multiple gamma logs at different energies.

Other types of downhole loggers can perform cement-top location functions. A temperature log can detect the temperature gradient arising from the heat of hydration of the cement. Inclusion of a radioactive tracer at the end of a stage can be picked up by a sensor lowered into the casing itself. Grout-bond logs are used extensively in-oil wells and are essentially an acoustic log. Not only will a grout-bond log detect the start of the free-standing portion of the casing at the top of a stage, but it will also evaluate the quality of the bonding of the cement to the casing and to the formation, after the cement has set.

Figure 40 shows the idealized interpretation of a grout-bond log. Like all idealized geophysical interpretations, the real-life situation is considerably more complex, and some very sophisticated routines have been developed by the logging service companies to interpret bonding in deep oil-wells.

Basically, the theory is that the free pipe with bond to either cement or rock will vibrate freely when tapped by the logger. If the cement is bonded to the pipe but not to the formation, there is a time lag while the input pulse travels through the cement and is reflected back to the sensor, resulting in a delayed reflected wave arrival time but no formation signal. In the case of good bonding, there is no pipe signal and the signal received is characteristic of the formation behind the pipe, as long as the formation is as stiff or stiffer than the cement. If the pipe is bonded at only one side, both pipe and formation signals are present. The situation becomes very complex when cement strength varies, where the pipe is bonded but is eccentric in the hole, where it is against one side of the hole, and so on. Furthermore, grout-bond logs have been best adapted for use in relatively thin-walled, smooth-sided casings. The complications introduced by stiffener rings, grout and instrument

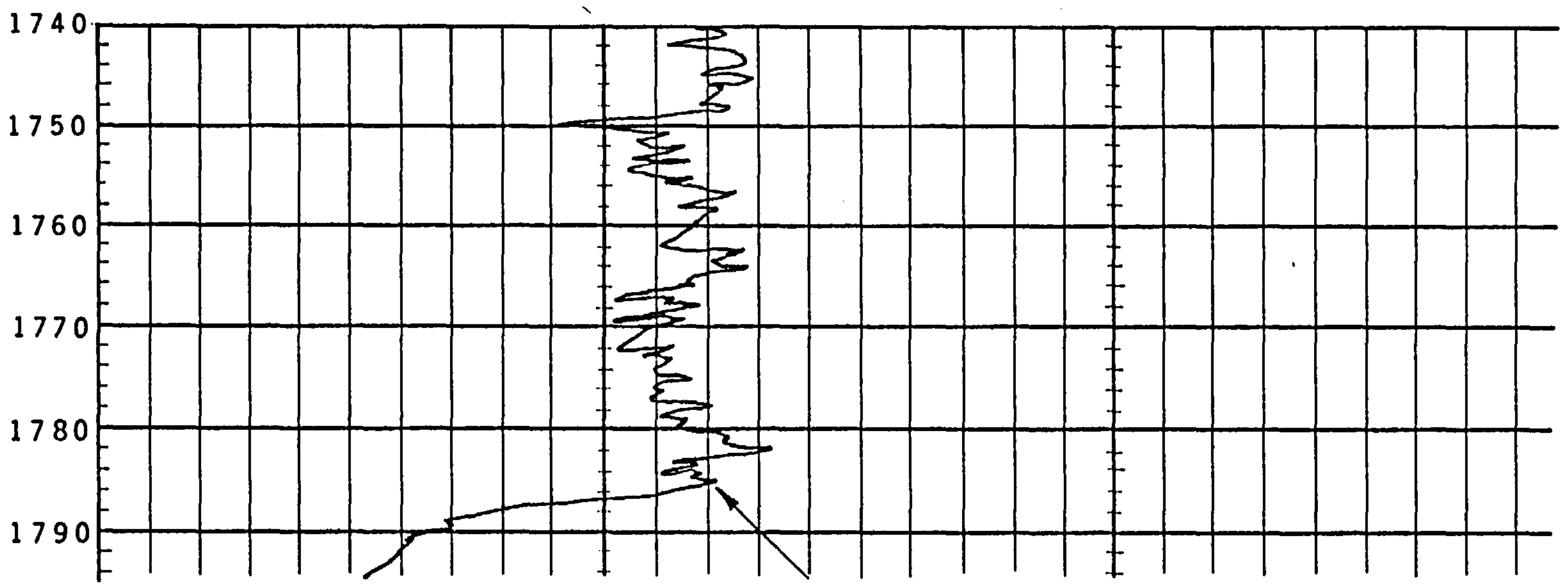


Figure 39. NCTL Log Response in Big-Hole Liner Cementing, Crownpoint Project (after Hunter, no date)

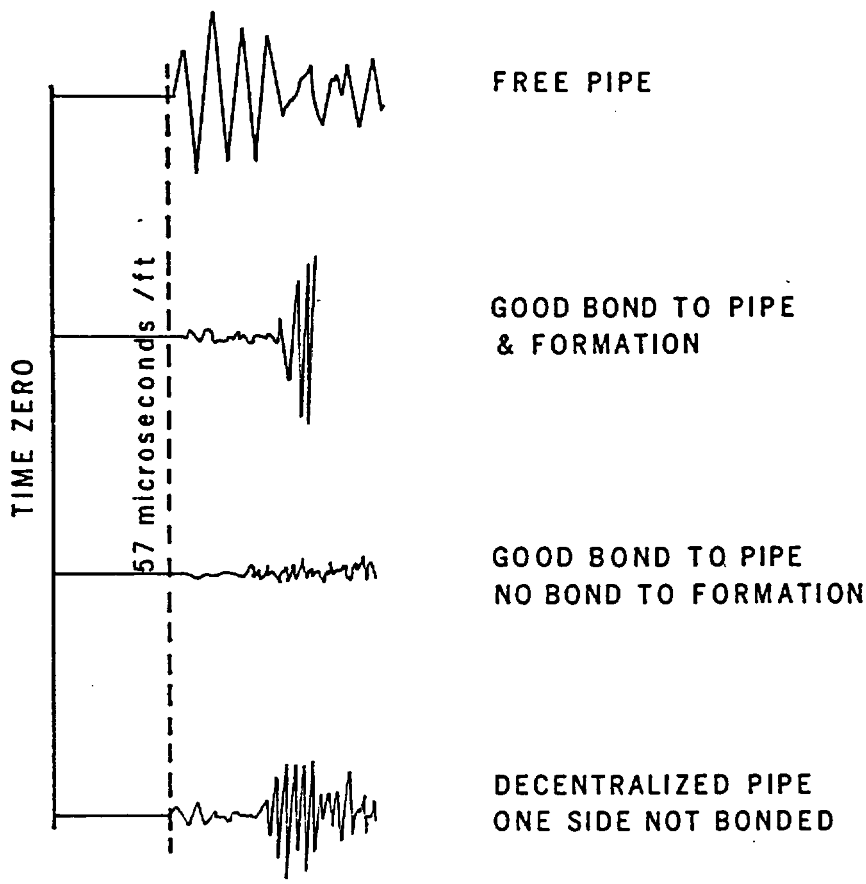


Figure 40. Grout Bond Log Interpretation (after Smith, 1976)

guides, and utility conduits (such as the ventilation pipes planned to be incorporated into the annulus for the BWIP Exploratory Shaft), in addition to the variability of the formation, the size of the annulus, the quality of the cement, the presence of water, and the void sizes that might be of interest, are well-beyond the present state-of-the-art of conventional cement-bond logging. There is some potential for the use of such logs in open conduits that are cast into the annulus if their modified frequency responses due to the one-sided weld to the casing can be accounted for. Considerable refinement is necessary, as described below, to use acoustic techniques for repository shaft grouting assessments.

5.3.1.2. Void Detection After Shaft Completion. If it is not possible to detect void formation during cementing then it would be desirable to conduct an inspection supportive of decisions to post-grout the shaft lining to assure compliance with the design performance.

The condition of the post-construction shaft annulus may be inspected by coring, seepage monitoring and various geophysical instruments.

Boreholes cored from the shaft interior through the disturbed rock zone (DRZ) could serve several important functions. The condition of the micro-annuli (steel/grout and grout/rock interfaces), the grout, and the disturbed rock could be inspected directly. This will identify conditions such as fracturing or the presence of mud cake at the micro-annulus. The borehole can then be packered off in segments to measure permeability and seepage inflow. Hydraulic gradient along the shaft and tracer tests can be made at adjacent borehole pairs. Tracer dilution tests at individual boreholes would also indicate seepage flow.

Reliance on this means alone has two serious and related drawbacks. First, unless voids were suspected in advance, there would be a good chance that the investigative holes would compromise an existing, favorable isolation condition. Second, without targets to drill for, the drilling may miss important zones of seepage or voids. Due to the difficulty of drilling a thick steel casing it is probable that drilling locations would be pre-selected by confining drilling to the breakout ports, or would be seriously limited by reluctance to penetrate the casing at other places unnecessarily.

Geophysical tools would, therefore, be preferable to initially search for voids and other grouting flaws, for moisture, or to detect seepage. The signals used must be able to penetrate the grouted annulus at frequencies suitable to resolve flaws at levels associated with overall shaft performance. Candidate instruments are based on acoustic and electromagnetic principles. Thermal instrumentation also can play a minor role as discussed below.

The hydration of the annulus grout can be monitored using temperature sensors to detect the heat of hydration. This test is relatively insensitive and indicates only that the cement is or is not hydrating generally as expected.

A better indication of annulus grout quality is the sonic velocity. The stiffness of a material is related to the acoustic velocity by well-known formulas.

Thus the velocity of an acoustic wave provides an indication of grout stiffness and density, and hence strength. Approximate compressive and shear wave velocities through the materials of interest are:

Material	Compressive Velocity ft/sec (km/sec)	Shear Velocity ft/sec (km/sec)
steel	19,360 (5.9)	10,170 (3.1)
grout	9,840 ∿ 13,120 (3 ∿ 4)	6,560 - 9,840 (2 ∿ 3)
rock	6,560 - 13,120 (2 - 4)	3,280 - 9,840 (1 - 3)
water	4,920 (1.5)	-

Thus, to monitor grout stiffness, one need only measure the acoustic velocity periodically and measure the time of flight. Acoustic velocity measurements may be made through the steel casing or one of the external utility lines using reflection techniques.

When an impulsive load impinges on the surface of an half-space, several acoustic waves are generated. Compressive waves, also referred to as dilatational, primary, or p-waves, radiate spherically at a velocity, ν_p. Approximately 25 percent of the energy from a vertical impulsive load goes into generating p-waves. Shear waves, also referred to as distortional, secondary, or s-waves, radiate spherically at a velocity, ν_s. S-waves transmit less than 10 percent of the energy from a vertical impulsive load. Rayleigh, or surface waves radiate along the surface of the half space at a velocity, ν_r, near that of the s-waves. Rayleigh waves include both shear and compression components. Being constrained to propagation in the surface, they attenuate less rapidly with distance than compressive and shear waves. Approximately two-thirds of the total available energy goes into generating Rayleigh waves.

The moduli of layered systems (airfield pavements) were measured by Heisey et al. (1982) using Rayleigh waves in an elegant experimental procedure. Two receivers at different distances from the source were used to develop "input" and "output" signals for the cross-spectrum phase function. The cross-spectrum is the Fourier transform of the cross-correlation function between the input and output signals. Wave velocities were calculated for each excited frequency using the cross-spectrum phase function. From this, it is a short step to derive the moduli of various layers using the relationships between wave length and velocity and between velocity and elastic modulus. It is necessary to assume a Poisson's ratio, but the method is relatively insensitive to the value assumed. The method of Heisey et al. (1982) achieved quite good results when compared to cross-hole acoustic methods. The spectral analysis equipment is readily portable and fast in operation.

As indicated in Section 5.3.1.1, reflection acoustic techniques are often used to measure the bond between the steel liner and the grout. The amplitude and sign of an acoustic wave reflected from an interface depends upon the impedance mismatch at the interface. If one were to tap the inside of the steel liner with a small mallet, one would expect a thud if the grout were well bonded and a ringing sound if the grout were not bonded. The technique used in a grout-bond log employs rather more sophisticated instrumentation to perform a similar function.

Acoustic instrumentation can also be used to search for voids, inclusions and anomalies within the grouted annulus. Reflection techniques using a mechanical or piezoelectric transmitter with a piezoelectric receiver would be appropriate. The grout-bond log should employ frequencies in the low megahertz range. Reflections from the steel grout interface will have round-trip travel-times of several to several tens of microseconds. The reflection from a well-bonded steel/grout interface will be relatively small since much of the acoustic energy will be transmitted through the interface. The reflection from a steel/mud-cake interface would be much larger, especially the shear component, and multiple internal reflections (ringing) would be expected in the steel. Modern acoustic systems can digitize and add signals. Thus the reflection at a well bonded location can be stored and added negatively to signals at other locations to assist in interpretation.

Acoustic probing for inclusions and to measure grout stiffness should use frequencies between perhaps 200 kHz and 2 MHz, which would permit detection of voids as small as several centimeters. For long-term performance higher frequencies would be needed to resolve the smaller features (millimeters or fractions of millimeters) that would be of interest. However, this scale of feature is an order of magnitude or more smaller than the irregularities that could be expected on the borehole walls form the gage cutters or minor sloughage.

Figure 41 shows the instrument set-up for an acoustic reflection test. If no void or obstruction exists in the grout, the p-wave reflection from the shaft wall would return in several tens of milliseconds and the shear wave would require about twice as long for the round trip. The wave velocities can be inferred from the travel times and assumed grout thickness. These may be compared to laboratory tests on grout samples to determine the general in-situ grout quality and strength.

Should a void or occlusion exist in the grout it also will reflect acoustic energy as shown in Figure 41(c). Note, that in this case the shear-wave reflection from the void overlaps the primary-wave reflection from the shaft-wall. To help interpretation of confused signals, the reflected signal at several points may be recorded and subtracted digitally by some systems. Actual data are typically more confusing than the idealized signal shown in Figure 41.

Figure 42 shows that a void or occlusion in the grout can be seen from several nearby transducer locations. If data from several points are stacked, the target reflection will appear to trace a hyperbola as shown in the lower half of the figure. Although actual traces are more complex than the idealization shown, the hyperbolic trace on a stack of several signals usually shows clearly in both p- and s-wave arrival times.

The character of the obstruction (void or rigid inclusion) may be inferred from the acoustic reflection. A compressive wave will reflect as a tension wave when passing through the interface from a stiffer to a less stiff material. Thus, reflections from voids will be reversed in sign whereas reflections from rigid inclusions will be the same sign as the incident wave. Applying this rule, the target shown in Figure 42 is seen to be stiffer than the grout, most likely a rock sloughed from the shaft-wall.

Schalge and Smith (1981) evaluated the utility of acousitc sensors for measurement of slipformed concrete liner thickness and to detect fractures and anomalies in a laboratory environment. They did not simulate a steel inner-casing. Although considerable difficulties were encountered with Rayleigh (surface) waves, they were able to:

- detect thickness variations corresponding to a 25-degree slope
- detect horizontal and vertical fractures
- detect air- and water-filled voids to 1.6 in. (4 cm)
- detect concrete sidewall intrusions
- irregularities on the order of a wavelength in size could not be detected.

This effort might have been enhanced by the following modifications:

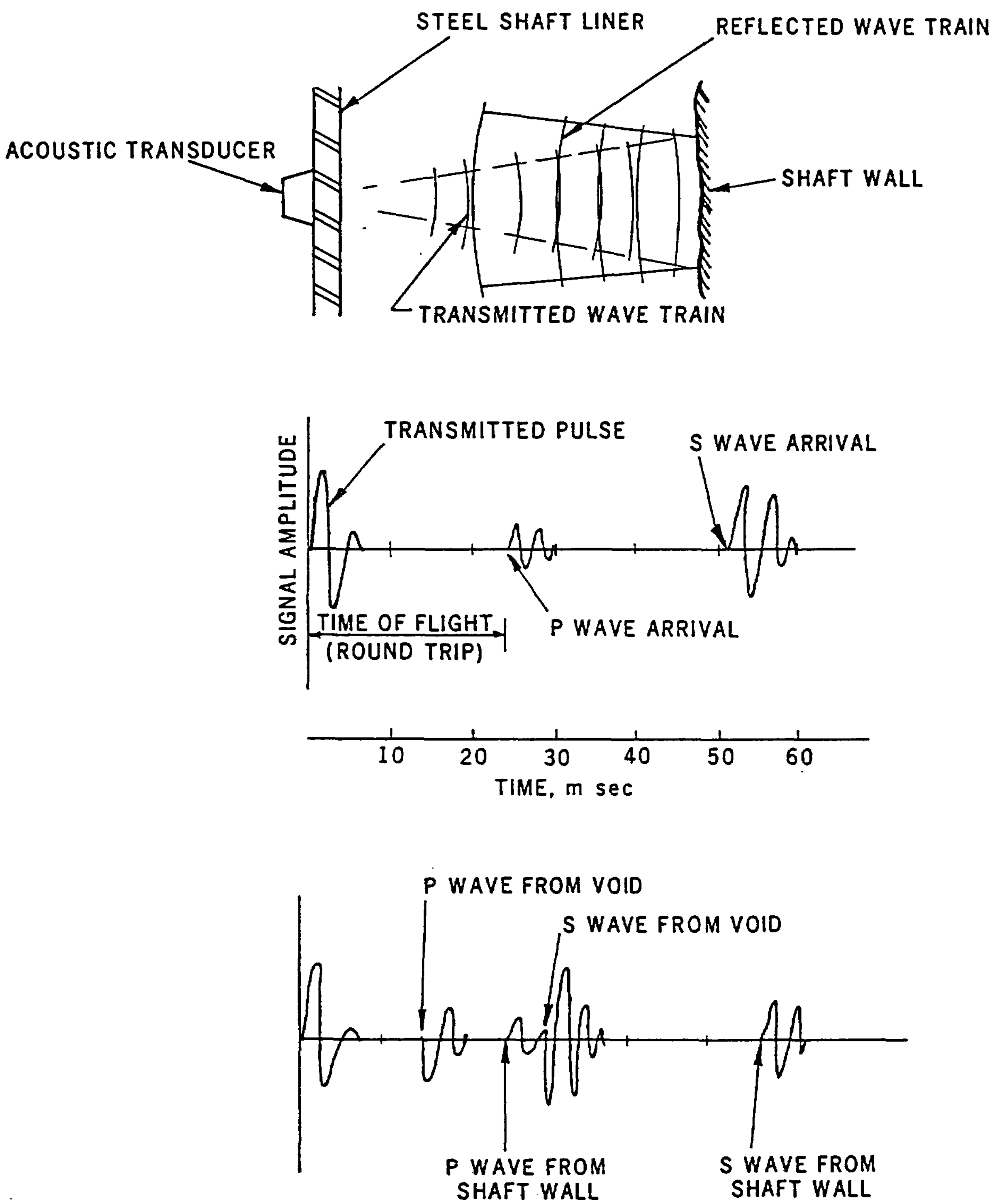


Figure 41. Idealized Acoustic Reflection Survey

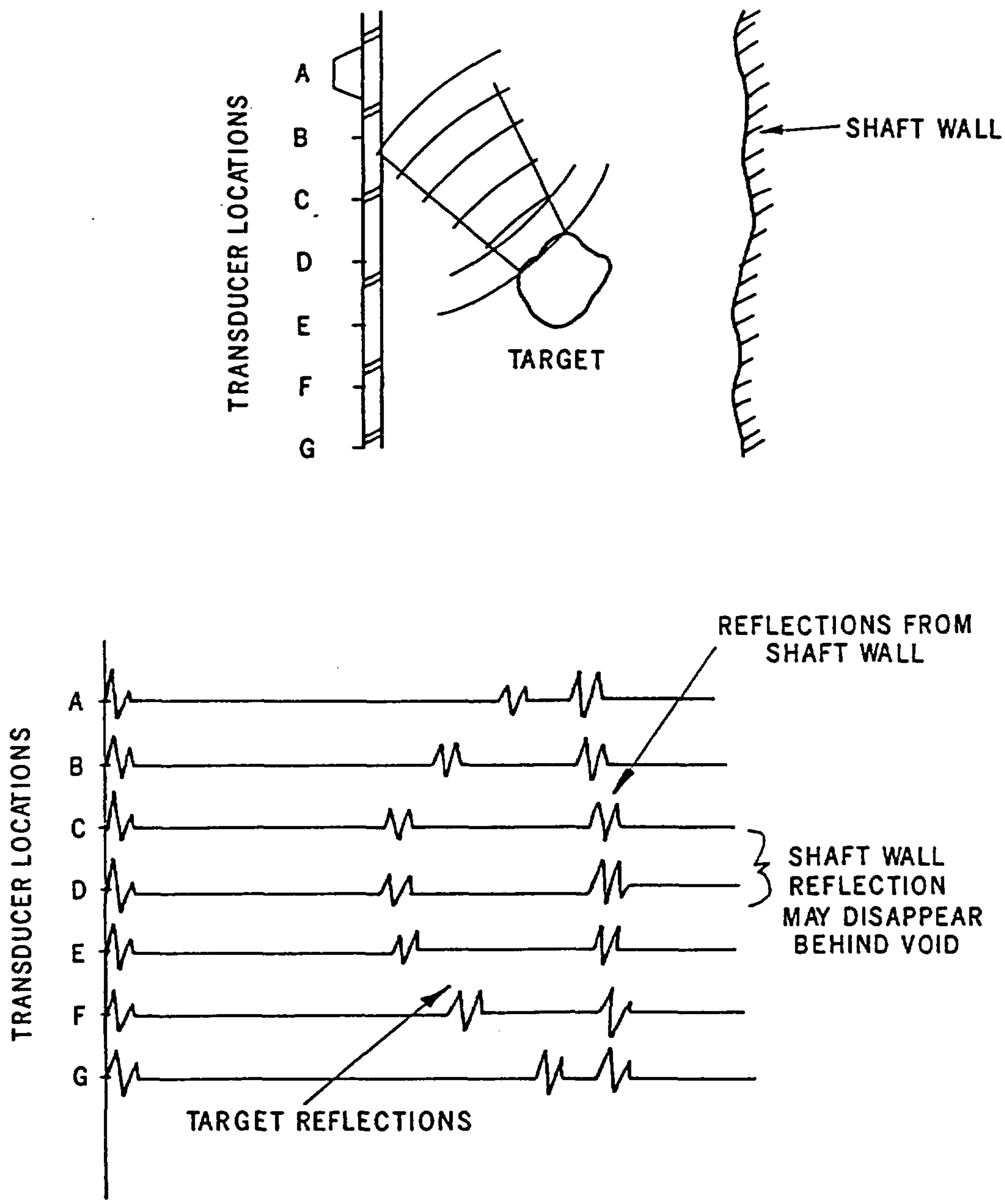


Figure 42. Identification of Target Location by Closely Spaced Survey

- The transducers used were large single ceramic crystals with resonant frequencies of 50 kHz. Mosaic arrays of smaller high frequency crystals can provide the same excitation energy at higher frequencies. Proper phasing of such an array can also be used to damp the Rayleigh waves.

- Signal enhancement was not attempted other than an effort to space two receivers as to cancel the surface wave arrivals. Signal addition and correlation were not attempted. Signal processing and pattern recognition can do much to enchance the recovery of information from noisy acoustic signals.

Electromagnetic instruments may be used to search for voids if a steel liner is not present. Candidate instruments would include frequency modulated (FM) radar and electrical resistivity.

Surface radar systems include pulse radar, FM radar, and continuous wave (CW) radar (Fowler, 1985). In the transmit/receive (T/R) mode, the transmitter and receiver are located in the same package, sensing reflections from changes in the electrical impedance of the medium. In the transillumination (TI) mode, the transmitter and receiver are on opposite sides of the target and sense transmitted signals. Since surface antennae must be used in a shaft liner, the T/R mode must be used to probe the shaft-walls.

The system best adapted for void detection would be FM radar. CW radar works best in the TI mode and pulse radar cannot operate at very short ranges where the pulse duration would be longer than the round-trip time of flight.

FM radar was used by Tranbarger (1985) and by Matzkanin et al. (1982) to probe brick subway liners. The transmitter antenna emits a "chirp", a relatively long burst of radio frequency (RF) energy at constantly increasing frequency. This chirp is reflected from interfaces with the body being probed and returns to the receiver antenna, located adjacent to the transmitter. This signal is delayed in time by the distance traveled and is, therefore, lower in frequency than the portion of the chirp being transmitted at the instant the reflected signal returns. Data are interpreted by mixing the transmitted and received signals to generate sum and difference signals. The frequency of the different signal is proportional to the distance (time delay) of the reflected signal. A spectrum analyzer is used to acquire and display the data. The system described by Matzkanin et al. (1982) was capable of detecting voids as small as several tens of centimeters in a brick wall 2 ft (0.6 m) thick.

A disadvantage of the FM radar is that phase information is lost in data reduction. Thus, one cannot infer whether the reflector represents an increase or decrease in the material impedance. Of course, no radar system can be used to probe behind a steel shaft-liner, although radar signals can penetrate a mesh of reinforcing steel if the bars are sufficiently widely spaced.

5.3.2 Annular and Backwall Injection of Conventional Shaft Linings

Backwall and annular injection present different monitoring concerns and utilize different techniques. Backwall-grouting pressures are limited by the lining strength and the communication between holes. Once backwall injection has been carried out, it may be desirable to re-grout with a low-viscosity mix (plasticized fine-grained cement or stable chemical) to seal bleed channels and smaller fractures, a process similar to penetration grouting. The monitoring effort includes design of the grouting program, process monitoring during injection, and post-injection evaluation. The achievement of mix design (viscosity, strength, bleed) should, of course, be verified for each batch.

The grouting effort can be self-checking. This is accomplished by dividing the injections into primary, secondary, and sometimes subsequent stages. Each stage of injection can thus serve as a test of the adequacy of the preceding stages. When the final stages refuse grout at high pressure, confidence in the grouting effort is enchanced.

Grout-process monitoring includes a variety of efforts. For pay quantities, the total volume of each grout component injected at each hole is usually recorded. The volume of grout injected at each hole should be plotted on a grout-take log. Large secondary and tertiary injections in holes can be identified that suggest a problem area that may require additional grouting.

Process monitoring should also include continuous recording of injection pressure and flow-rate at each injection point. Grout refusal at a point is indicated by a pattern of increasing pressure and decreasing flow-rate, as the effective permeability decreases around the grout-hole. Figure 43 shows this signature. Injection pressure and flow-rate should be monitored by strip-chart recorders or a microprocessor-based data acquisition system. Such data are sometimes collected manually, using pencil and clipboard. The tabular format of manual data make the desired signatures more difficult to see. The microprocessor-based system not only collects more data with fewer blunders, but the data can be presented graphically for easier interpretation. By collecting injection-pressure/flow-rates throughout the project, anomalous behavior at any particular injection point can be more readially identified. Jeffries, Rogers and Reades (1982) described electronic tranducers that are

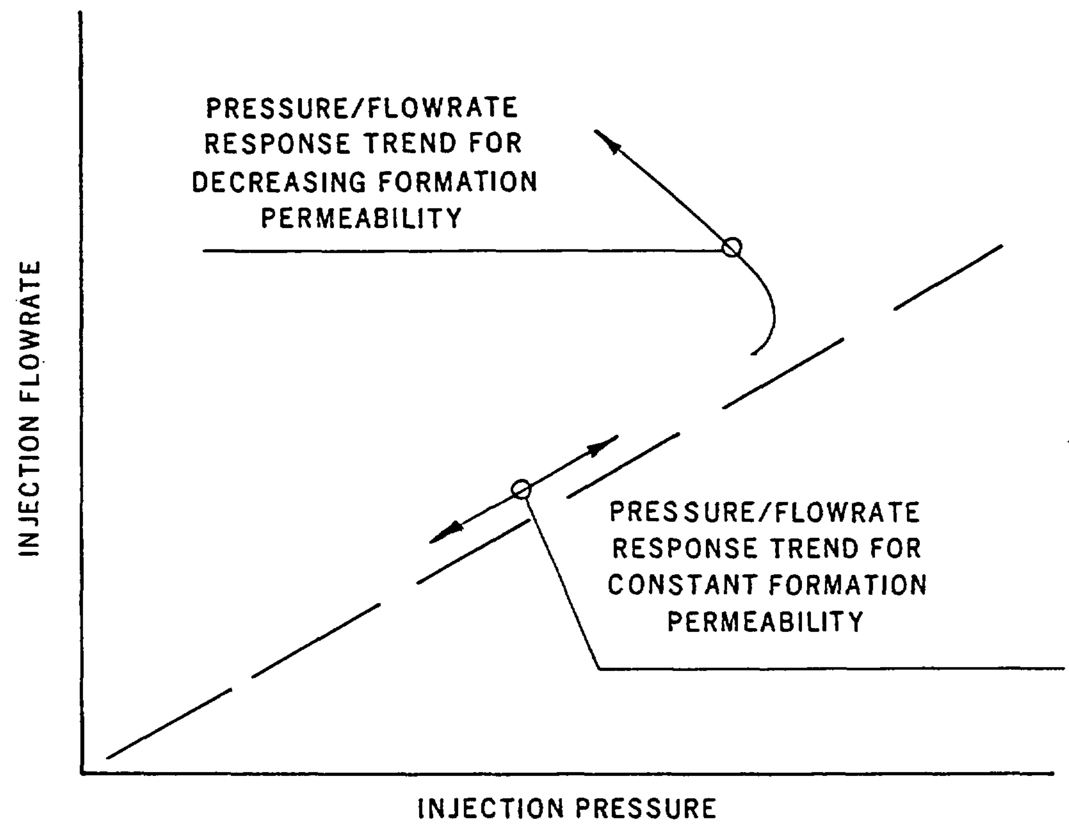


Figure 43. Grout Refusal Signature on Injection Pressure and Flowrate Cross-Plot

available for measurement of injection pressure, flow-rate, and volume. If recorded on strip-charts, water-injection test data of these types, can greatly aid planning the injection program.

A concern that is often raised during final, completion penetration-grouting is with regard to possible formation or lining damage, caused by excessive injection pressure. If rock or lining distress is be avoided, it is more reasonable to monitor the injection for possible damage rather than to rely on rules of thumb limiting injection pressure. Rock distress may be identified by acoustic emission (AE) monitoring. An AE sensor, usually a hydrophone, may be placed in a nearby empty grout-hole and monitored for anomalously high count rates to detect hydraulic fracturing or strata movement. Figure 44 shows the acoustic emission signature usually observed during hydraulic fracture in a porous medium. As the injection pressure gradually increases, at some level the critical fracture pressure is exceeded and a fracture is initiated, with an accompanying burst of acoustic emissions. As pumping continues, the fracture is held open, but may not extend. This can cause a quiet interval in the record, even while injecting at high pressure. Upon the cessation of pumping, the fracture will close, producing a smaller burst of AE activity.

Acoustic emission monitoring should use a down-hole AE sensor, in order to:

- place the sensor near the target event
- isolate it somewhat from equipment noise.

In addition, a high-pass filter is necessary to reduce the interference from construction and grouting equipment noise.

During injection, grout samples should be taken several times a day and stored in sealed glass containers. Nalgene or other plastic containers should not be used since some chemical grouts may be adversely affected. Nalgene, for example, can retard the gelation of sodium silicate grout. In addition to these permanent samples, frequent samples should be taken to measure gel time.

Post-grouting evaluation may include a variety of techniques depending upon, the site and method of shaft lining. Some remote void detection methods have been discussed. For a sealing effort in a saturated medium, this should certainly include hydrologic monitoring of ground water pressures, gradients, and flow. There is no more certain indication that a site has been sealed than observing a cutoff of ground water flow. Techniques also exist for determining grout location. Two excellent approaches are cross-hole acoustic testing and transillumination borehole radar. Surveys should be conducted in boreholes prior to and following grouting. An area that has been well grouted will display high acoustic velocity and will be

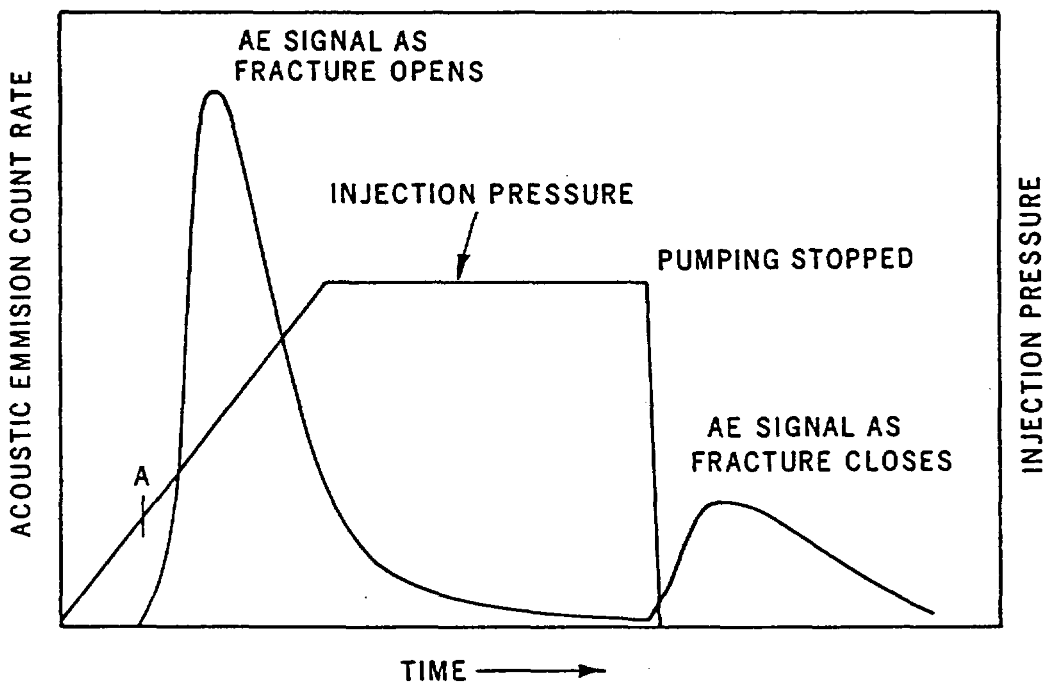


Figure 44. Typical Acoustic Emission Signature for Hydraulic Fracture

opaque to radar. Acoustic cross-hole shooting, borehole radar and geotomography have been used with good results (Huck et al., 1980; Baker, 1982). Because of the expense and specialized personnel required, such techniques are usually used to sample any given project rather than using 100 percent inspection.

Good Pentration Grout Monitoring Practice

Monitoring of penetration grouting should extend thoughout the project to provide maximum confidence in the quality of the work.

During project design, care should be taken to use successive stages of injections to help make the grouting effort self-checking.

During injection, the following activities should be included in addition to the usual measurement of pay quantities:

- preparation of a graphical grout-take log. This should be updated and reviewed frequently

- automated monitoring of injection-pressure and flow- rate at every injection point to detect grout refusal. The instrument used should display a graphical plot for the operator's use and subsequent review

- acoustic emission (AE) monitoring to detect incipient hydraulic fracturing

- shaft convergence, liner stress, and water pressure measurements may be appropriate.

Post-grout evaluation should include review of all grouting records. In addition, the following may be considered, depending on site needs:

- hydrologic monitoring to measure seepage

- borehole seismic and/or radar survey (before and after grouting) to locate the actual position of the injected grout.

The program outlined above will contribute significantly to the confidence that can be placed in the results of grouting. Additional details may be found in Huck et al. (1980).

In simple composite lining where grout may have been injected behind the primary lining or between layers, post-grouting investigations similar to those described in Section 5.3.1.2 could be used. The geophysical responses of composite lining with steel skins, either internally or externally (or both), would be easier to interpret in at least one respect because the steel is thinner and not fitted with stiffener rings or grout line guides. It may, however, have anchoring hooks or eyes in the concrete, and the concrete, if reinforced, may have a complex signature. The grouting quality in tubbing sections backfilled with concrete would not be amenable to remote monitoring. It would be useful to verify any proposed remote void detection apparatus and the technique for interpretations in a full-scale mock-up before it is used in a repository shaft. Various shapes, sizes, and positions of synthesized voids made from styrofoam blocks, could be cast into parts of a mock-up; or an actual shaft lining during its construction, and investigated with several technologies. If the site were an actual mine, the sections would have to be chosen in non-critical locations and be amenable to repair after the experiment had been concluded.

5.3.3 Detection Limits

The preceding discussion has pointed out that the ability of present methods to detect voids is limited by the rather gross material properties that are detected. Acoustic methods cannot resolve below about 1 wavelength. It is doubtful if presently-available hardware would be capable, except under very ideal conditions, to detect narrow, longitudinal voids as small as 0.2 in. (0.5) cm thick within a composite shaft-lining system, particularly if the configuration of the steel casing or membrane is at all complex. Yet, this scale of void has been duplicated in laboratory studies of oil-well casing cementing. Drill-and-blast shaft wall irregularities would introduce further complexities.

The basis for establishing the size limit for voids that would be of concern in repository shafts should be the performance expectations of the shaft lining system. At the present time, calculations have not been made that relate the expected performance (that is, performance level that would permit confidence in compliance with the regulatory mandates) to the permissible void level in the shaft lining. Such calculations would have to consider not only "watertightness" (measurable inflow to the shaft interior) but also aquifer communication that could modify radionuclide travel times or compromise the long-term integrity of the shaft-lining system. Establishment of such leakage parameters for the post- closure period will essentially depend on what credit is to be taken for the shaft-lining isolation capability. In relating the parameters to permissible design, the discontinuities in the shaft-lining may require a focused engineering effort. For the pre-closure period the decision may be based on desired water-handling limits, risks from flooding, and lining longevity. Until this is done, it will not be possible to provide a meaningful assessment of how satisfactory present or reasonably-developable, technology may be in assuring location of critical voids.

5.4 SEEPAGE DETECTION

The primary method of seepage detection is, of course, visual inspection. Seepage is evidenced by the appearance of water, mineral encrustations, or corrosion. With water-tight linings, however, seepage may be visually obscured, and it may only be suspected because of anomalous water-pressure readings with instruments, or by questioning the grouting or construction methods used. Even if seepage is visible, the source may be in doubt. It is, therefore, relevant to consider how seepage may be remotely identified as an aid to optimizing grouting efficiency.

The best system to detect seepage behind a shaft lining is a monitoring borehole. As this technique is expensive and destructive, it would be desirable to use a surveying instrument of the types indicated below,that could scan for areas of potential seepage. The potential seepage zones could then be drilled for confirmation, and sealed by penetration grouting if necessary.

Moisture seepage in a porous medium emits mechanical energy that can be detected using acoustic emission (AE) monitoring systems. Koerner et al. (1981) report preliminary laboratory investigations in which acoustic emissions were generated by clear- and turbid-water in sand. The technique was used by Huck (1981) in monitoring seepage beneath an earth-fill dam. Although a new, undeveloped technique, AE monitoring maybe useful is locating zones of general seepage. Because the technique employs what is normally considered background levels of acoustic energy, it can be confounded by construction noise or other sources of acoustic emissions.

It may be desired to monitor moisture content, although the presence of moisture does not necessarily imply actual seepage. Nuclear gamma loggers can be used to measure moisture and density at ranges of several tens of centimeters. It is questionable whether such tools could be used to make measurements within the rock.

A variety of moisture sensors is available that might be embedded in the rock behind the concrete lining. In drill-and-blast construction, such transducers might be emplaced prior to shaft lining. In blind boring, however, a borehole would have to be drilled, probably in a pre-selected breakout, after shaft construction. If this were done, it would be more reasonable to use the borehole for hydrologic monitoring than to place a moisture sensor.

Seepage paths can also partially tracked with radioactive- or dye-tracers. Obviously the latter are inferior in murky or discolored water.

It is important to stress that, from a long-term isolation point of view, visible seepage is not necessarily of more concern than hidden seepage. Visible seepage appears at tremie pipes, water-rings, tell-tales, around seals, and weep holes. What is seen may not reflect the full extent of the problem, but it at least originates from provisions in the lining, such as French drains or grout seals, that were installed specifically to detect and measure it. The same magnitude of seepage may occur in remotely-placed steel-cased linings, but is likely to appear to be far less significant if it is only observed at instrumentation points and in seal areas.

5.5 SEALS

Various types of seals have been used to retard annular flow in shaft linings and these interact with the grout to the benefit of each. The most common are bitumen, chemical (such as Dowell Chemical Seal Ring; CSR), picotage, and grout.

Whenever a water-tight lining is installed, an inherent problem that is encountered is sealing the top and the bottom of that section. The location of a seal is very important, and at both the top and the bottom of the waterproof lining section the seals must be located in impervious, competent rock formations. To prevent the vertical migration of water behind the lining, seals must be constructed between the impervious rock and the waterproof lining. In the piotage type of seal, wedge-rings are installed at the top and the bottom of the water-tight lining section. Although the wedge-rings do not fit tightly against the excavated rock, watertightness is effected by the seal that is installed between it and the rock wall. To keep the rock-fracture damage to a minimum in the area of the seal, the rock is usually chipped to the final excavation size rather than using blasting techniques. The seal is achieved by installing a picotage (Figure 21, Section 3.4.2.3) which consists of various sizes of wedges made from Nicaraguan pitch pine. These wedges are driven into the free space (6 to 9 in.) between the wedge-ring and the rock. The wedges must be driven uniformly and tightly around the entire wedge ring. When water then comes into contact with Nicaraguan pitch pine, the pine swells and creates an impervious barrier.

The wooden picotage in one of the oldest methods of constructing vertical seals in shafts to prevent water migration. It is a time-consuming method and requires skilled and experienced personnel for installation. Nevertheless, picotages have been used successfully throughout the last two centuries. It is not completely water tight, so in modern shafts, it is used in conjunction with other types of seals, and to bear the weight of tubbing or the free-standing composite lining.

Cement grouting of a pre-formed annular space behind the shaft lining is probably the cheapest form of shaft-seal. It is usually not 100 percent effective, so is used chiefly to isolate aquifer zones for construction water-control and grouting. An annular ring, 4-ft to 8 ft high, is chipped out of the rock outside the normal excavation line. This space is then filled with a single-size gravel (1 in. to 1.5 in.) which is retained by rings of liner plate, and the top covered. Conventional lining is then cast in front of the seal location, with pipes leading though the concrete from several locations around the shaft perimeter at both the top and the bottom of the seal. After the concrete has cured and gained adequate strength, cement grout is injected through the lower holes until it exits through the upper ones. The seal area is then pressurized so that the rock and concrete lining are post-strained at pressures in excess of the pore-water pressure at that level. A non-bleed grout-mix is obviously important for this application and often an expanding agent is added. Normally, at least two seals of this type are put in at any one location, with tell-tale drains below.

Chemical seals are essentially synonymous with CSR, which has been used extensively and successfully in the Saskatchewan potash district. Figure 45 shows a typical application. The material used is an elastic polymeric chemical compound. It is either pumped as a slurry or placed as a precast ring of jointed segments into a 3-ft to 8-ft high pre-formed annular gap behind the shaft-lining, against the rock walls. The slurry sets within 6 to 8 hrs to from a rubber-like material and the lining is the cast on top of the seal, thus confining it against the rock surface. CSR reportedly can expand to 150 percent of its original volume on contact with water. If water comes into contact with the seal, it will be absorbed and the swelling pressure produced thereby will enhance the sealing efficiency. The main advantage of the chemical seal over the picotage is that it is relatively simple and easy to install, although it requires skilled personnel familiar with the chemical to mix and supervise the slurry placements.

CSR has been mainly used in direct-placed applications in drill-and-blast shafts. The salt protection rings used in Canadian potash shafts (Pence et al., 1971) include CSR placed behind tubbing that is overlain by plastic sheets, gravel with tell-tale drains, and concrete. At the Boulby No. 2 shaft, CSR was placed into a hand-trimmed zone behind a wedge ring, in place of picotage.

In addition to water, the CSR can reportedly seal off high-pressure gas. The CSR at Tatum Dome, which was 100 ft long, reportedly withstood 2 nuclear detonations below it without failure.

As with all type of seals described above their adequacy depends upon the permeability of the adjacent rock. It is therefore normal practice to grout the ground around the seal after it has been installed, to ensure that any fissures or fractures that may already exist, or were induced by the excavation, are filled. The process will often require the use of both cement and stable chemical grouts.

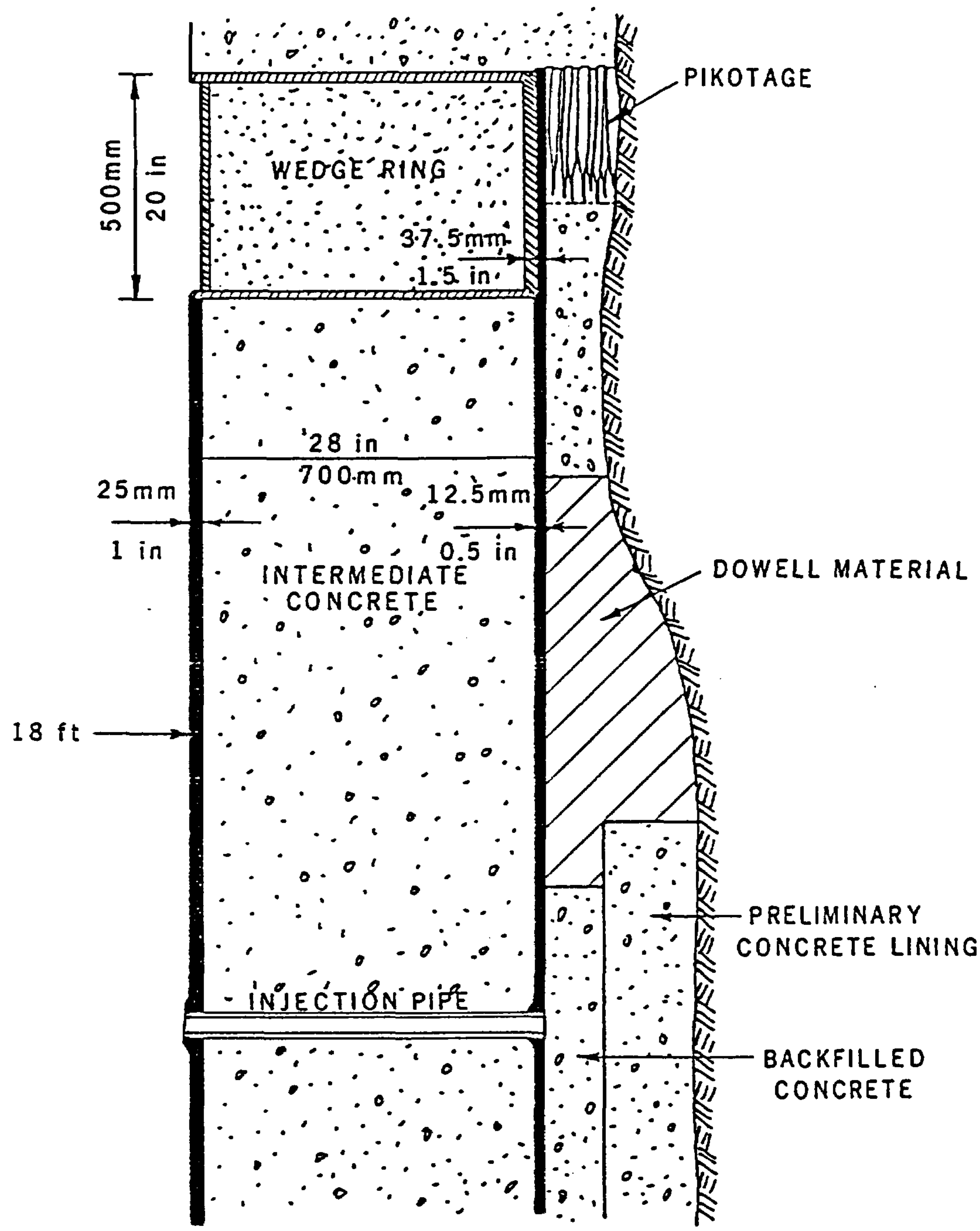


Figure 45. Placement of Picotage and Chemical Seal Material (from Storck, 1968)

CSR has been proposed for placement remotely, in blind-drilled shafts for nuclear waste repositories. A rigorous test of CSR in a blind-drilled hole was done at Tatum Dome (Boughton and Dellinger, 1965) although the hole was much smaller (28-in. drilled, 20-in. cased) which allowed for the use of standard oil-field casing cementing techniques, such as filter cake removal with scratchers, casing reciprocation, latch-in shoes, and the use of scavenger cement in front of the main cement stage and the CSR. There was also extensive use of salt-saturated, expansive cement; even the filler-cement stages were tailed off with an expansive mix. The casing outer-surface was resin-sand treated to enhance hydraulic bonding. In placing the CSR, several step were used: water pre-flush, chemical flush to remove water, placement of CSR beneath chemical, chemical flush on CSR to isolate CSR from water during setting, final water flush, and placement of next cement stage. There had been several unsuccessful casing cementation attempts at Tatum Dome prior to the one described above.

For a drilled repository shaft, some technical issues for CSR placement in the grout-annulus are:

- ability to remove filter cake and mud adhesion
- ability to adequately pre-flush with water and chemical in a large annulus
- stability of the larger hole under plain water
- technique for delivering CSR through grout guides that are under water
- integrity of the chemical wash and the rock response to the presence of the chemical
- ability to maintain the chemical on top of the CSR during setting
- water invasion from below
- cement dehydration at the CSR contact.

A jet sub could be used for installing submerged polymer seals, to aid in bottom-cleaning and replacement of fluids by the polymer. A principal concern for any seal placement is the minimization of the DRZ (disturbed rock zone) associated with the placement of the seal itself. In Kelsall et al. (1982), a grouted rock cutoff (Figure 46) is proposed. The grouted rock cutoff features a process of repeated stage grouting that limits the amount of stress relief at any one time. Several of the case histories of shaft grouting in salt mines are suggestive of fluid movement in very fine fractures within the salt that might have been related to stress relaxation. Such fluid migration could by-pass seals within the salt if the seals are surrounded by a relaxed zone, and threaten the integrity of the grouted shaft system.

It should be pointed out that the seals described above are operational seals designed for water control during the operation and retrievability periods. Operational seal requirements may not be

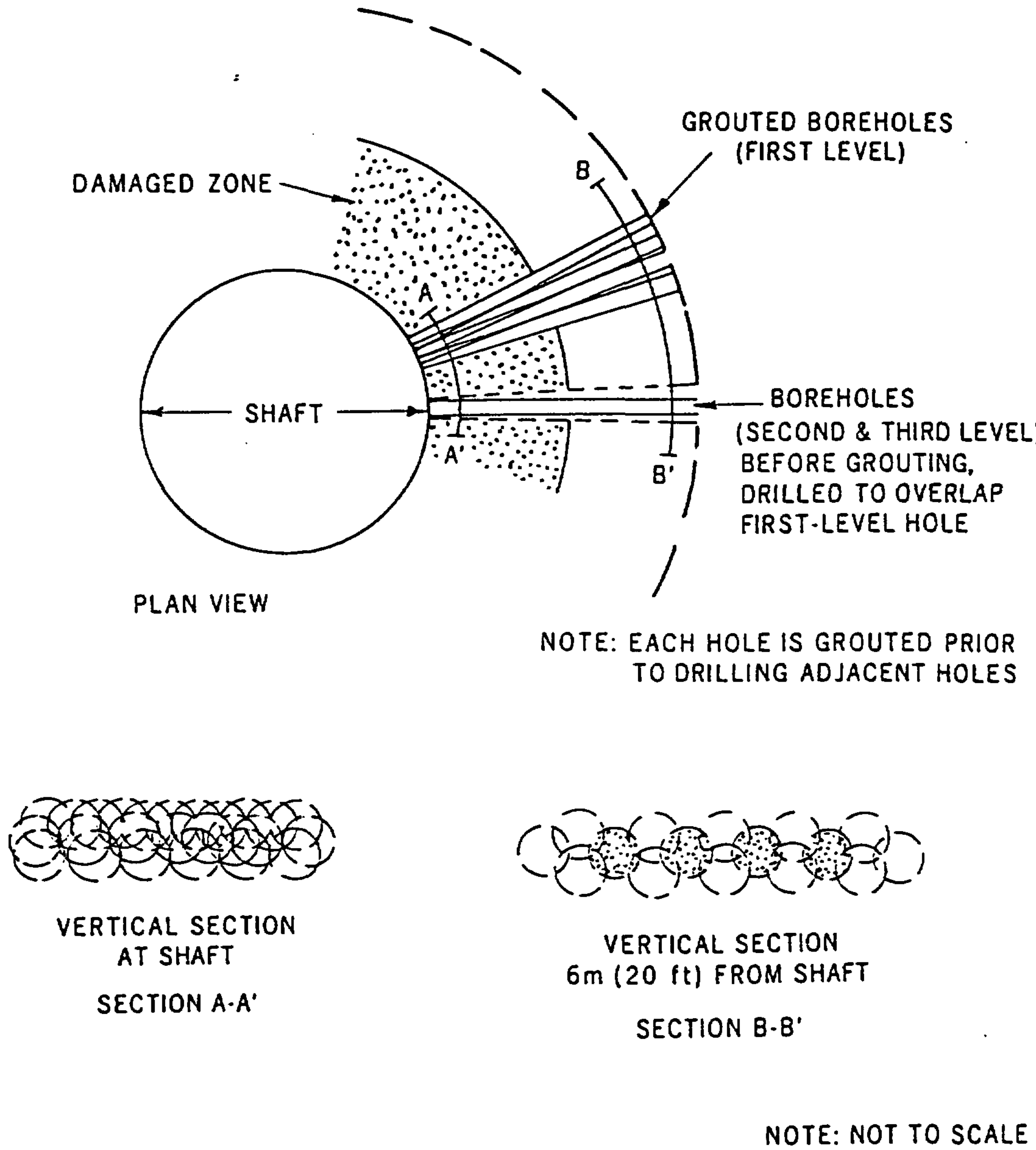


Figure 46. Proposed Grout Cut-off for DRZ Treatment
(from ONWI-411, figure 6-2)

congruent with decommissioning seal requirements. Post-closure (decommissioning) seal systems may not make use of any of the operational shaft-lining components.

5.6 WATER MOVEMENT THROUGH VOIDS

The mechanics of water migration through void systems in the shaft lining are not well understood and are affected by the wide variety of conceivable void configurations that could be involved.

No documented cases have been found where the transport of water through voids in shaft grout has been studied specifically, although it forms an implicit part of many studies of casing, liner, or grout failure. The occurrence of voids in grout is a common problem and it is common knowledge among drillers and miners sinking shafts which must be sealed or grouted in place. These voids present a problem in two ways: they affect the structural integrity of the grout, and if interconnected they can provide a conduit through which fluids can travel. The movement of fluid through such voids may case rapid erosion to occur and may also transport hazardous material.

Water under hydrostatic pressure impinging on a shaft liner will, as always, follow the path of least resistance, or in other words will flow down the pressure gradient. Flow through the shaft liner ran occur as <u>Darcian</u> flow through the grout matrix or as a stream through voids or along the boundaries of discontinuities. Diffusive movements through the grout matrix is extremely slow in the absence of fractures or voids (i.e. in the nanodarcy range) and thus does not institute a significant threat, since the permeability of the DRZ, lining-cement contact zone, or even the rock mass may be considerably greater. Unquestionably the most difficult problem is the flow or fluid along discontinuities which are represented by the interfaces between the grout and the shaft liner and between the grout and the wall rock, as well as along fractures and voids within the lining and grout.

It is very difficult to form a perfect seal at an interface between two materials of different density, structure, coefficient of thermal expansion, elastic modulus, and surface chemistry. It must be expected that the discontinuity or micro-annulus will exist and that voids will result from differential movement at the interface, from adjustments to temperature or pressure changes, or to stress. Once a void is created and water is admitted, propagation could proceed rapidly if dissolution and corrosion by the fluids are permitted. In the case of extreme temperatures, if a pressure drop is experienced by the fluid entering a void, it may <u>flash</u> to the gaseous phase and create a serious hazard as well as elevated pressures to further propagate the void.

In the case of a shaft, the interior is generally empty and filled with air at approximately atmospheric pressure when in operation. The path of water movements during operation then is from the formation, through the casing/liner, to the interior, since the water is not confined by a water-tight lining, this constitutes the direction of the pressure gradient. The first interface encountered by formation water then would be the interface between the wall-rock and the grout.

The surface of the wall-rock is generally irregular due to the effects of excavation, whether it drilled-and-blasted or blind-bored. The flow of fluid along and across this interface is a function of shaft wall preparation (cleaning and scaling) and grout quality (pressure, viscosity, penetrability). Because of the irregularity of the wall-rock and the nature of the grout it is not possible to prevent the occurrence of voids entirely, however, pre-conditioning the face of the wall-rock prior to injection of grout will minimize the amount of void space created.

The void space, if created, can enlarge due to dissolution, differential movement, and erosion. The benefit of irregularity of the grout/wall-rock interface is that the micro-annular pathway is tortuous. This tends to reduce the velocity of flow and thus minimize any mechanical advantage which may result from increased velocity and dampen any kinetic advantage in dissolution reactions which would result from more rapidly cleaned mineral faces being exposed to the water. In very small micro-annuli, the friction head loss maybe substantial. This helps to reduce the pressure gradient and, therefore, the flow velocity.

One cause of a micro-annulus is the contraction or shinkage of the grout as it hydrolyzes (reacts with its mix water during curing). Also, the hydrolysic reactions are exothermic and can generate fairly high temperatures (up to 300°F.). The net decreases in volume of grout during the curing process causes the grout to pull away from the wall-rock at the interface or to crack within its matrix. The differential thermal expansion is augmented by this behavior. Methods for controlling shrinkage, such as additives and curing environments, have been discussed earlier. Expansive cements also are in common use, as has already been discussed.

The preventive measures mentioned above are helpful, however, none of these will eliminate the formation of voids entirely. Probably the best solution to obtaining a good seal at the grout wall rock interface is to combine thermally-controlled, non-shrink formulation with the use of a "reactive grout", that is one which will react with the wall-rock and eventually form an integral bond with the wall-rock by forming a linked intercrystaline structure.

Problems of a similar nature exist due to the micro-annulus between the grout and the casing. Generally the casing is constructed of synthetic material having a smooth surface. The problems of different mechanical characteristics such as coefficients of expansion have been discussed previously. If the casing is intact, the hydrostatic pressure-gradient and flow-direction are different, because the fluid-flow would come through the matrix of the grout impinge upon the casing-wall. An adequate seal at this point is, therefore, important also.

Generally it is possible to find synthetic materials such as epoxy which will form a reactive-bond with the casing material. The casing superficies can be treated to provide a rough surface to which the cement can adhere effectively, and likewise the seal material is chosen which will react with the grout.

The grout itself is permeable and will contain some fractures. Movement of water along grain boundaries through the grout matrix will occur as a result of hydrostatic pressure and capillary action. Such movement is very slow but is documented; for example, in the passage of contaminated water through the concrete liners of radioactive storage tanks at Hanford, WA. Generally, movement through the matrix will become concentrated along the fractures, structural discontinuities, or irregularities. Many times such irregularities are imperceptible to the naked eye but become obvious as differential flow is observed.

The movement of water through unchanneled grout thus reduces to a question of Darcy or fracture flow. Dracy flow is readily analyzed but may not be realistic since fracture flow, if present, will govern the flow rates. Daemen et al. (1985) found from studies of cement plugs that Darcy flow assumptions break down for small flows and result in high uncertainties. As stated above, Darcy flow alone may not be relevant for real-life grouts considering the potential for fracturing and the chance that the DRZ or rock mass permeability will be greater.

Fracture permeability, on the other hand, is not easily characterized. Fractures can be small in grout, such as the incipient fractures evidenced during the coring of the Piceance Creek Shaft grout (Section 4.2.2) or can be larger if shrinkage occurs. An estimate of fracture permeability, which is presently being investigated by many researchers, approximated can be using the cubic law (Witherspoon, 1980; Zeigler, 1976), if the effective aperture or roughness are known. The cubic law (for laminar flow) or the Missbach Laws (for turbulent flow) can be used to estimate flow through <u>single</u> fractures if correction factors for roughness can be empirically derived. Zeigler (1976) gives an approximation for idealized, multiple sets of orthogonal joints. The more general non-orthogonal case requires computer simulation, and verification of those routines is being undertaken. Iwai (1976) found that fractures exposed to cyclic loading exhibit permeability hysterisis, suggesting that permeability of real grouts may increase after initial construction.

Transport of contaminated materials through the grouted annulus may occur by direct mass transport in migrating solutions, or via diffusion through the matrix of saturated grout. Frequently the rate of diffusion of such materials is orders of magnitude faster than the actual rate of migration. Diffusion through saturated voids proceeds much faster than diffusion through matrix material.

Another pathway for fluid movement which must be considered is the disturbed zone surrounding the excavation. The permeability of the disturbed zone can be expected to be up to 6 orders of magnitude higher than that of the undisturbed host rock. Conductivity increased of 1 - 2 orders of magnitude can significantly reduce flow times (ONWI411). In some stress fields, internal pressure, such as from expansive cement, could augment DRZ permeability.

The pathways discussed thus far are primarily of significance with regard to hydrostatic pressure exerted on the shaft system by the ground water. The pathways would remain the same for solutions trying to escape from inside the shaft into the surrounding ground water. For this to occur however the hydraulic gradient must be reversed and, of course, the seals must be permeated.

Large scale voids may occur during emplacement of the grout by a variety of mechanisims that have already been identified on this report. These include channeling, erosion, bleed-water voids, gas cutting, and entrapment of undisplaced mud at irregularities. Terine et al. (1983) discuss a method by which gas cutting or erosion might be predicted by plotting the hydrostatic pressure of the fluid column with depth on the same scale as the corresponding pore pressure. A reduction in the height of the cement column will reduce this effect proportionately, and multi-stage grouting may avoid it entirely. Should multi-stage grouting be found undesirable for some reason, other mitigating measure might include: increasing the density of the mix water, maintaining pressure on the system from the surface, or the addition of a foaming agent to the slurry which might help maintain pressure. The application of surface pressure may require the use of a quick-setting cement.

Such large-scale voids, if they occur in proximity to aquifers, can contribute to or govern the annular-flow mechanics, overwhelming the effects of Darcy flow through intact or even inferior (fluid loss) grout, and incipient thermal or shrinkage cracks. Channeled zones and mud pockets will act in combination with the grout material permeability, the permeability of the filter cake, or the DRZ to define the flow path. The rates of movement can range up to tens of centimeters per second. The flow paths could be extremely complex and their importance may only be realistically assessed in terms of a specific hydrogeological regime with which they can interact.

Any analysis of the water movement through shaft lining must therefore account for the spatial position of potential large voids with respect to linking fractures and sources of water. The analysis should also recognize that the excavation, heat buildup, and lining stress will affect the availability of water and the permeability of linking fractures. Estimation of gross hydraulic properties of potential flow paths must assume a conservative proportion of annular flow confined to a micro-annulus or tight fractures (shrinkage or thermal) as determined by water-injection test data.

The interpretation of such test data both for back-calculating the liner or formation permeability, and for estimating the degree to which grouting could lower it, should recognize the effect of the viscosity of the water used in the testing as well as that of the injected grout. As Figure 47 shows, water warmed by the heat from the waste could be significantly less viscous than that used to estimate formation or lining permeability -- before and after grouting -- with a resulting much more rapid and extensive annular flow. This reduced water viscosity will also increase the rate of saturation of the annular grout, which can enhance both flow and diffusion.

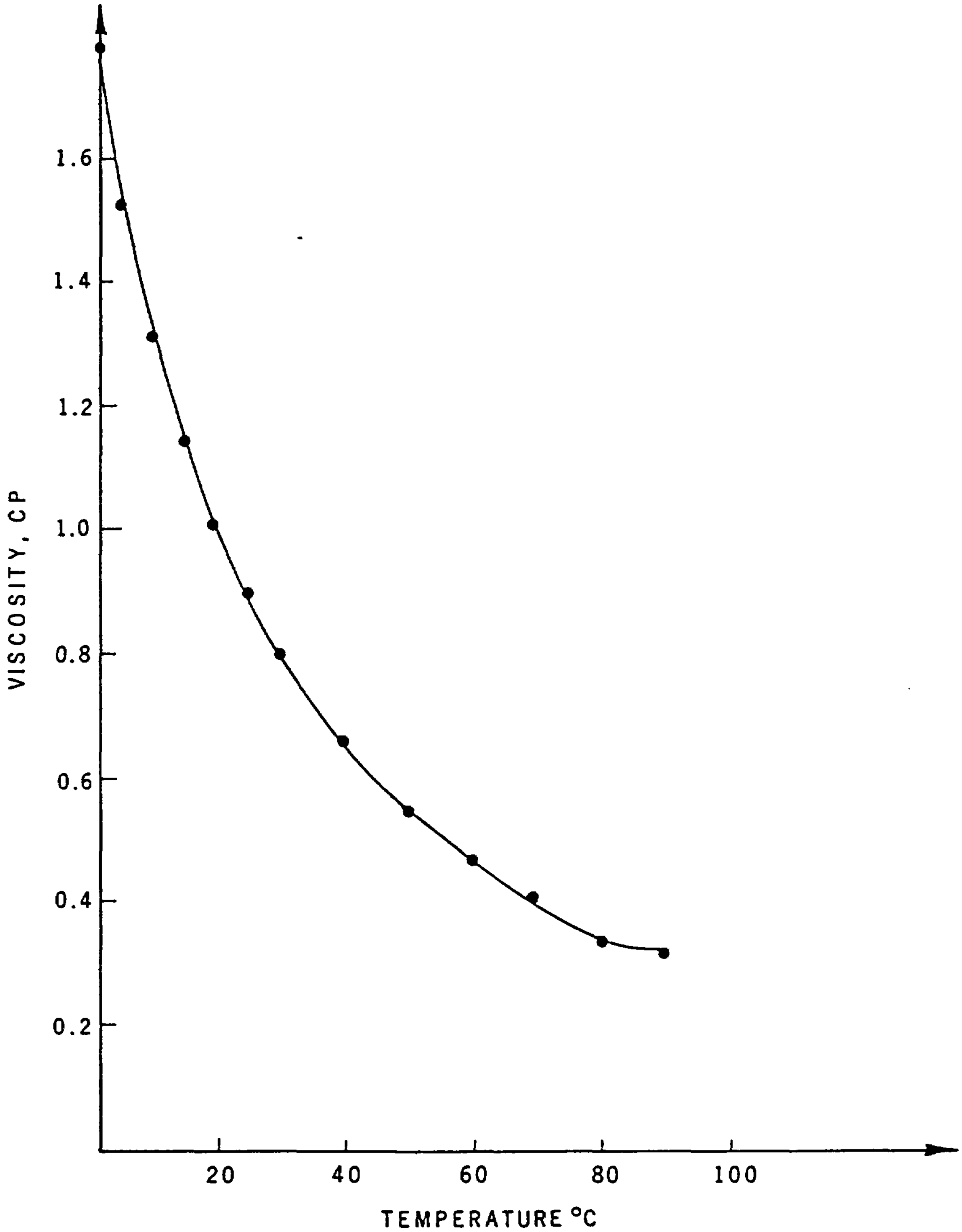


Figure 47. Effect of Temperature on the Viscosity of Water (from Daemen, et.al., 1975, p. 705)

6.0 CONCLUSIONS

This section relates the capabilities of shaft sinking and lining technology to the requirements of a nuclear waste repository for eliminating or reducing voids in shaft grout systems. The evaluation is necessarily subjective because there are no analogs among existing shafts. The relevance of the case histories is explored first, followed by a rough subjective evaluation of best expectation of void occurrence.

6.1 CASE HISTORY SUPPORT

If there is one fully-supportable statement that can be made about how actual modern shaft lining experience has to reduced or eliminated voids in the grout, it is that leakage could have been further reduced by the more extensive application of known control techniques, more thorough design, or the exercise of additional care in construction. Put another way, where both the need and the means exist, there are things that can be done to reduce the probability of occurrence and reduce the effects of any of the types of voids that were indicated by the case histories.

Modern shafts have had to contend with less-ideal conditions for shaft sinking and lining than their predecessors in the historical past: greater depths, higher water pressures, high rock temperatures, larger diameters, weaker ground, and more productive aquifers. Many modern shafts have been completed in conditions which would have been avoided, in most cases, by the shaft sinkers of 50 or even 25 years ago. Yet in modern shafts, water handling is typically less, and instances of maintenance or reconstruction are less frequent.

Nonetheless, the typical history of a shaft required to be water-tight includes mention of recurrent grouting, residual water-make, and increasing flows through time. This suggests that the longevity of shaft grout systems can be compromised. In addition to voids or just poor grouting, compromising influences include the effects of creep, relaxation of the rock, grout deterioration due to shrinkage or chemical attack, or the ability of the invading water to find new pathways through the rock mass and bypass its former course now occupied by grout. Thus the case histories that describe residual water-make and recurrent flows may or may not be referring to the effect of voids in grout systems. Conversely, some shafts where voids were known to exist have not exhibited such ongoing leakage problems.

While this type of leakage may not be considered to be acceptable performance for a waste repository shaft, it has not been cost-effective for most shaft operators to either completely eliminate these problems once they have started, nor to take the sometimes-extreme precautions that would be required to prevent their occurrence in the first place. Neither has there been sufficient need in most cases to undertake thorough scientific investigations of the water problems to pinpoint the exact cause, be it voids or some other mechanism. The understanding normally achieved is that which is sufficient to outline a grouting or remedial construction program that would yield cost-effective and sufficiently reliable results for continued operation.

There are a few shafts where some special requirements for prevention of annular flow, as well as watertightness, have resulted in a greater level of care in planning, design, construction quality assurance, and remedial investigations and action. In addition to those whose special features have been mentioned at various places in this report, the following have exhibited superior levels of assurance and actual performance to date.

Deep waste disposal and underground storage wells, drilled in some cases to 4-ft-diameter, must be positively sealed, and it appears that the level of assurance necessary is routinely achieved. Cobbs (1986) mentioned a cryogenic underground storage facility for anhydrous ammonia that subjects a drilled-shaft lining to low temperatures under a requirement for complete confinement of the stored product, that appears to have satisfied its designed function. (This shaft used a CSR seal, but is considerably smaller in diameter than the planned repository shafts.) It is not known whether voids occur in these types of shafts, but if they do, their effects are obscure or negligible.

The Gas Buggy project, near Rangeley, CO, was able to successfully isolate aquifers (no measurable venting or leakage) from a horizon that was artificially stimulated for gas production. Despite the depth (5,000 ft) the casing diameter was only 20 in. and conventional oil-field techniques could be used.

The AOSTRA project in Canada, a demonstration project in the underground stimulation of recovery of oil from tar sands, involved two drilled shafts of more repository-like diameters (14 ft) and will incorporate elevated temperatures in the stimulation procedure. The drilling program featured many of the desirable characteristics listed in Section 5.2, such as thorough mud conditioning, use of expansive cement, desanders to control mud weight to rigorous tolerances, use of polymer mud, and careful drilling procedures. The mud used (Alcomer-120 and MudLube) seems to have been a key to the success. Alcomer-120 is an anionic polyacrylamide. MudLube is composed of mono- or di-esters of hydroxyl-terminated alkaxide condensates, which act as friction reducers. Swelling of shales occurred at a pH of 10 (which is commonly the target pH used for drilling tool corrosion prevention) so the pH was reduced to 9 - 9.5. The mud weight during drilling was 9 - 9.3 lb/gal; for cementing, it was conditioned for 14 hr with neutralizers added for improved cement displacement. Upon breakout, the results were encouraging -- only minor and temporary methane release, good cement contact with both the rock and the casing, and no apparent voids. The inspection ports have been mostly dry; only one shows a slight weep (Greenwell, 1986). Since these shafts have only recently been completed, there is no long-term performance history.

High-level waste repository shafts have better, but still indirect, analogs in the shafts accessing other repositories for lower classes of nuclear waste. The Asse No. 4 shaft, whose construction was described in Section 3.4.1, experienced a severe abnormal event during cementing, but apparently did not leak. It is not known whether there are voids in the lining. The key in the Construction and Salt Handling shaft at WIPP shows evidence of fluid movement, and there is leakage through the rock, or the lining, or both, in the running shaft sections as well. Comparison of the grouting success in the waste and exhaust shafts at WIPP shows that there can be significant enhancement of the chance of success once some experience in the site conditions has been obtained. The ground at the WIPP shaft sites is relatively water-free, compared to some of the proposed repository sites. Being a pilot operation not involving permanent disposal of HLW, the WIPP shaft designs are not as rigorous in their water prevention measures as the commercial HLW repository shafts might be expected to be. The WIPP leakage is minor, but in some cases it has increased with time, and there is not a sufficient record to assess the long-range performance of the shaft grout systems there.

In summary, the case histories have served to identify potential modes of void occurrence in shaft grout systems, but the quality and quantity of information on void occurrence is not directly supportive of projection of shaft grout integrity for a HLW repository.

The observation that all shafts leak is true, if not a little trivial. This does not mean that repository shaft grout systems will contain voids. The case history data are uncertain as to the exact level of voids even in cases of severe leakage or shaft loss. The data do represent windows to a hypothetical continuum of grout system performance with respect to the level of reliability required. This extends from single-pass or no grouting for shafts where water-make can be tolerated or accepted, to cases (such as in the examples above where water-make must be minimal. The best-case assessment of void likelihood requires a subjective extrapolation of grout system performance to the point on that continuum corresponding to the level of reliability of a repository shaft grout.

In making a best-case assessment for a repository shaft from documented shaft experience, it is necessary to account for:

- a higher target level of reliability desired of the repository shaft grout system

- longevity of the repository shaft system (of the order of 100 years)

- eventual exposure to thermal and thermomechanical influence

- geologic and hydrogeologic conditions at the proposed repository sites

- a diminished budget restriction and an enhanced design- and construction-control philosophy

- the need to pass the scrutiny of licensing entities.

The grout will have to meet performance requirements over the operational-plus-retrievability period, ensuring acceptable water handling and maintenance requirements from the standpoints of nuclear and public safety. The disposition of the shaft grout system will also be affected by the decommissioning requirements, primarily sealability. A decision will have to be made as to whether it is best to leave the lining in place or to remove it. If voids can be preventable through the design and construction practices, and if their absence can be confirmed through a remote detection technique and projected by longevity studies, then the long-term performance of any shaft grout sections to be left in place will be more assured.

The design requirements of repository shaft grout, for meeting these requirements, are still evolving. It seems clear that the shaft design will require a degree of watertightness, since a 100-year period of voluminous water handling would undoubtedly be an undesirable characteristic. For a salt repository, complete watertightness will be required. In general, water-tight shaft linings, as described in Section 3, use steel-concrete composites. No case history examined indicated the occurrence of water through such a lining, which would indicate that if significant voids are occurring in the watertight sections they are being counteracted by the continuous steel membrane. The question of voids contributing to annular flow in a primary lining, however, is still open. The primary linings are not designed to be water-tight for many shafts using composite linings, because the primary lining is necessary only for safety-support of the shaft-wall during construction, for insulation of frozen ground, or to isolate the water-tight lining from ground movements anticipated during the shaft operation. An outer steel membrane is used to keep water away from the inner membrane, thus reducing the overall threat of corrosion.

Some steel-concrete composite linings are not backed by a primary lining but by a filler material such as bitumen or grout. Whether or not this backing material, contains voids is not indicated in the case histories; if there is leakage that would evidence voids, it tends to be hidden behind the water-tight lining. Some case histories do mention water pressure buildup at seals, or water appearing from tell tales or from beneath the key, but it is not clear whether this is due to leakage at the interface of the backing and the rock, through a relaxed zone or fractures within the rock, or within voids in the backing itself.

6.2 SUBJECTIVE BEST-CASE EVALUATION

The preceding sections of the report have described potential sources of voids as identified from experience with grouts in general, and in shafts not necessarily designed or built to standards congruent with those of a repository. Reducing the chance that voids will occur is a matter of recognizing their potential and accounting for them in the design, construction, and supporting research of the grout system. In Section 5.0 measures were listed that, if diligently applied, would result in a reduced potential for void occurrence. The best-case evaluation assumes that the shafts would incorporate the precautions listed, with the objectives of achieving acceptable operational watertightness, reducing or eliminating the chance of concealed annular flow through voids that could compromise decommissioning performance and shaft operational performance, and be sufficiently long-lived within the expected physical and chemical environment. Gonano et. al. (1982) consider that the techniques used to achieve or surpass these requirements would not be much different than are presently available.

Thus, it is the considered judgement of the authors of this report that there will be mechanisms similar to those that have occurred in the past, which will operate to produce voids in ways that are inherent in the construction method, and which will exist to some degree whatever level of present technology is applied.

6.2.1 Blind-Drilled Shaft Casing Cements

In blind drilling, there will remain a potential for voids associated with fluid loss, channeling, filter cake, and gas cutting. With proper site selection and slurry design, the chance of voids due to gas cutting is minimal and the consequences in terms of annular flow that would result from voids associated with gas cutting are moderate. Similarly, proper mud design and conditioning prior to cementing should effect an almost-total removal of filter cake, although remnant zones of thin, low-permeability filter cake should be expected. Most zones of filter cake buildup would be hydraulically isolated, at least as far as the cement-rock interface is concerned, although there could still be leakage through the rock mass that could connect buildup zones. Fluid loss (resulting in incompletely-hydrated, porous cement) can be controlled with additives, but if filter cake removal is important, filter loss will occur locally near porous zones. Channeling is another matter. Ideally it would not occur at all in a uniform annulus full of well-conditioned mud where a sufficient number of discharge points deliver a controlled slurry. Chances are that these conditions will not be satisfied everywhere in a 3,000-ft-deep shaft and that some channeling will occur. If blind drilling is used, channeling at a low level should be expected and mitigated to the extent possible with a post-grouting program.

Present methods of delivery of cement will result in some, if minimal, initial turbulent flow of cement through the mud and a contaminated zone at the top of each stage should, therefore, be expected. It should also be expected that the stage top surfaces will in general be slightly irregular, with a 1- to 2-ft zone of inferior cement at the top. The cement-rock hydraulic bond to porous formations and some shales, may be poor, which could be a source of problems for long-term performance of the grouted-shaft system and should be strengthened by post-grouting. The bond to casing may be expected to be generally good, and a micro-annulus should be preventable, at least in the short-term.

In complex stratigraphy, there will be differential responses to the effect of the mud. Some distress of the shaft walls should be expected although careful pre-drilling exploration should be able to detect conditions that could lead to large-scale failures. The anticipated wall damage should thereby be limited to a few susceptible formations. Where distinct horizontal structure occurs, the distress may occur as sloughage with brows or gently-inclined upper surfaces. Complete mud removal within these zones may not occur. Cement also will not penetrate all sharp-edged, waterbearing fractures.

Drilling the shaft will minimize mechanical damage to the rock mass but will not appreciably reduce the DRZ component related to stress relaxation, since the internal restraint offered by the mud will be only a fraction of the lithostatic relaxation induced. This and any voids or micro-annuli remaining from the cementing operations will have to be treated with post-grouting (contact and penetration grouting). Where a thick steel casing is used, this implies preselection of the grout ports so that they can be installed as part of the casing fabrication. Since it is impossible to predict with 100 percent accuracy the locations of the zones that will require post-grouting treatment, and how these will be most advantageously reached by the grout (spin and elevation angles), and because the grout breakout port locations will be partly constrained by non-geological requirements such as utility lines and available shaft clearance, the post-grouting program will not be optimal and cannot be expected to be 100 percent efficient. The degree of efficiency can be enhanced, however, if a potential need for post-grouting is recognized and considered throughout the shaft program.

The lower limit of cement porosity and permeability that will be higher than the laboratory ideal. Hence, the cemented annulus will saturate Ultimately, and flow will occur. A properly, mixed cement-slurry will probably have little potential for shrinkage cracking under in-place curing conditions, but there is a distinct potential for thermal cracking of the type observed in the Piceance Creek shaft (Section 4.2.2), particularly if the annular space is large. Permeability estimates for the annular grouts should consider the effect of these confined, initially-obscure cracks, and that they may enlarge with age, long-term shaft deformations, and temperature fluctuations. The occurrence of these cracks can be limited by water circulation within the casing <u>and</u> any embedded utility lines, throughout the WOC period, together with pre-cooling of the slurry. Retarders may lower the time rate of heat production, but at the potential expense of settlement and bleed.

It is worth stressing that there are no data that confirm the performance of drilled-shaft casing-cements over the periods of time that will be required by the repository shafts, and there will be no opportunity to directly observe such performance prior to licensing activities. Thus the assurance will have to be extrapolated from existing shafts and overtests of the cement formulations that are being proposed for use.

6.2.2 Backwall Contact Grout and Supplemental Penetration Grout

The efficiency of grouting in directly-placed linings (backwall grouting and placement of thin annular or interlayer grouts) depends to a great extent on the grout formulation. The hypothetical "ideal" grout is non-toxic, fine-grained, low-solids, has the viscosity of water and remains at this viscosity until it sets instantaneously; exhibits a wide and positively-controllable range of set times, and is sufficiently strong, durable, impermeable, chemically inert, and stable, for long-term reliability. The repository ideal grout must also, in addition to the above, be thermally stable both mechanically and in response to heat-induced changes in hydrochemistry and rock mass behavior. It does not exist. Approximations and compromises will need to be made in the grout properties, and these will result in less-than-ideal performance.

The ability to attain efficient grout sealing and watertightness in a directly-placed lining depends also on the elimination of water from the advancing shaft excavation. Shaft construction experience indicates, with few exceptions, that where no water is present in the excavation, the lining will leak little or not at all; where small amounts (1 - 10 gpm) of water are present the leakage and water-make will be minor to moderate, and where the excavation is wet (10 gpm and more) the lining can be expected to leak, even if it is backwall-grouted, and the shaft water-makes from drains and water rings will be appreciable. Thus the best performance from the grouted-liner can be expected if the pregrouting is as close to 100 percent effective as it can be. With considerable care in design and application, the pregrouting program could be quite effective (say nearly 98 percent).

From the standpoint of elimination of inflows into the shaft excavation, freezing can be completely effective (assuming that freeze wall breaches can be prevented). The likelihood of voids due only to the presence of inflows to the shaft excavation would therefore be minimal. The desirability of freezing from an overall annular flow prevention perspective, is, however, somewhat different because of uncertainties in the application of backwall-grouts in frozen or thawing ground. The disruption of the rock mass due to freezing and thawing add complexities to post-construction penetration grouting of the rock mass and augment the action of seepage on the shaft-grout system in ways that are potentially significant to the overall shaft performance expectation.

For directly-placed concrete linings in frozen or unfrozen ground, grouting requirements will be defined by the likely sources of leakage. As indicated above, it should be assumed that some water will be entering the shaft during its excavation and that this water will need to be conveyed away from the shaft-walls during concrete lining. Thus the matcher joints will leak slightly from minor accumulations of laitance and small gaps. Localized erosion of the concrete is possible, although not likely to be extensive, except in frozen shafts where total thaw control is not exercised and where pregrouting is ineffective. Honeycombing can probably be eliminated with careful placement practice. Thermal cracking can be avoided (although in frozen shafts it is a distinct hazard). Localized shrinkage cracking in a deep shaft lining should be expected. In frozen shafts, a micro-annulus will form upon thawing and its characteristics and those of the freeze-thaw zone will be material-specific. It is safe to expect, though, that sources of potential lining leakage will remain (and be accentuated) if the ground is frozen, both because of the inherent behavior of frozen ground and because ground that requires freezing tends to be heavily water-bearing originally.

For concrete-only linings, there will be fewer restrictions on grout-hole placement than with steel linings, so the grouting may be expected to be more effective at reaching the sources of leakage. However, grouting will not be as efficiently done in composite linings with steel membranes, for the same reasons as were mentioned for drilled-shaft casings. For concrete linings with extensive reinforcement, post-grouting in any but the grout-ports originally provided will be very difficult because of interference by the reinforcement.

Single-pass backwall grouting, even if carefully done, will not be fully successful at sealing all voids behind and within the lining. A second or third pass, possibly finishing up with a nonviscous acrylate or chrome-lignin grout, may be necessary to effect a high (over 95 percent) efficiency. The selection and placement of backsheets is critical. A regular, consistent annular space, in which all the voids are well-connected, will be amenable to good filling at the outset, and subsequent grouting phases will only be required primarily to compensate for imperfections in cement quality (bleed, shrinkage). Backwall grouts will not penetrate well into the rock mass, so separate grouting for high penetrability (again with a reduced-bleed mix) should be considered to reduce the demands made of the backwall grout. If the backsheets are subject to corrosion or other deterioration, their long-term performance could be adversely affected and, if so, would be evidenced by seepage appearing at matcher joints and in tell-tales.

The efficiency of penetration grouting is limited by the fact that the most durable and stable grouts (cementitious) are the most likely to exhibit penetrability and bleed problems. The best solution may be to use a microfine grout. At the NTS, keyed bulkheads of tunnels used for weapons test access are specified to be gas-tight at 1,000-psi pressure. The grouting program used emphasized microfine grout and the use of a rupturable Celtite interior plug to prevent void formation around the standpipes incorporated into the concrete bulkhead (Tibbs, 1985). Of course, this procedure has been aided by the general absence of water, so that there is little water displacement at fine crack tips. There is also minimal potential for dilution, and maximal potential for the slow outward release of any bleedwater, rather than the formation-water pressure pushing bleedwater back through the grout.

Regardless of the effectiveness of penetration grouting at the outset, water at high pressure will eventually find its way through the formation through ever-finer cracks and will eventually be available at the backwall-grout surface.

In spite of the care used in construction, the disturbed rock zone for a repository shaft will be influenced by excavation stress redistribution, grouting and maintenance procedures, thawing effects (freezing method), excavation damage (mechanical due to blasting, sloughage), inelastic deformation, temperature change, swelling and erosion of susceptible formations, chemical effects such as leaching, precipitation -- which may be beneficial -- weathering, or dissolution, gross movements such as subsidence and uplift, and penetrations of the rock mass for rock support, over-excavation for seals, and test boreholes. If, as suggested in ONWI411, the hydraulic conductivity of a disturbed zone is in the 10^{-6} to 10^{-8} cm/sec range for a rock mass whose undisturbed permeability is 2 - 3 orders of magnitude less, then it is highly unlikely that grouting would be able to restore the DRZ to the level of permeability of the undisturbed rock. The practical limit of fracture aperture for reliable penetration, even by low-viscosity chemical grouts, is 0.01 in. (0.2 mm). In terms of hydraulic conductivity, it is roughly 10^{-5} cm/sec under the most ideal conditions. Cementitious backwall-grout penetrability will be extended possibly to 10^{-4} cm/sec in the absence of voids and with the use of microfine cements and great care, although the generally accepted limit for cement grout penetrability is 0.01 cm/sec.

Thus it appears that current, state-of-the-art grouting technology will need to be extended if shaft-liner permeability (non-watertight portions) and that of the liner-rock interface are to match that of the DRZ. It is unlikely that the shaft lining non-watertight portions or interface permeabilities can be reduced to the level of a completely undisturbed rock mass whose hydraulic conductivity is 10^{-6} cm/sec or less.

Improvements in grouting performance expectations can be achieved through pregrouting improvements to lower the proportionality of the final seepage levels with respect to the initial formation water-makes.

Improvements in grout system reliability could also be attained with the application of proven void detection methods. At present, it is unlikely that voids on the order of 0.2 in. (0.5 cm) or less could be detected even under superior conditions in a real shaft. The simpler the geometry of the void detection problem (in other words, the simpler and more regular the shaft design and rock mass condition), the more effective such a detection method will be. Regardless, the assurance of the absence of voids will be obtained chiefly from the absence of detected seepage and from the inherent capabilities of the construction method.

In terms of longevity, apart from repository-unique conditions, it must be remembered that grouts have not normally contended with the demand of permanence that will be required in a repository. The case histories reveal very little about the upper time limit of acceptable grout performance, although there is ample evidence of relatively short lower time-limits. In one of the few systematic studies of this type, Petrovsky (1982) investigated four 16-year-old cement grout curtains in the Soviet Union and found that calcium oxide concentrations in the seepage water denoted dissolution of some of the grout hydration products. It was theorized that diffusion was responsible for the removal of calcium oxide from the more remote portions of the grout curtain, and that the concentration of the oxides in the ground water had been at both a constant- and low-level throughout the 9-year period of observation.

Chemical grout-gels present serious longevity questions for repository shaft applications, although their short-term performances can be quite good. If a chemical grout deteriorates severely, the deterioration products (porous, shrunken, fractured, or chemically-altered grout) will remain in the ground as a potential impediment to later remedial grout attempts. Chemical grouts could be useful, however, as to form a pervasive, wide, low-permeability zone to delay water contact to the more durable and stable cementitious grouts in the DRZ and the backwall areas. They may represent the only alternative to effect final penetration of the DRZ and backwall-grouts to remedy the effects of bleed or fracturing.

Grout performance in the post-closure period must account for the adverse effects of elevated temperatures. Heat will result in dilation of the rock mass at some places and contraction in others (by drying it), will reduce the viscosity of water, and can adversely affect the integrity of the grout itself.

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