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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-2-3506

Manual
No. 1110-2-3506

31 March 2017

Engineering and Design
GROUTING TECHNOLOGY

1. Purpose. The purpose of this manual is to provide detailed technical guidance and criteria for civil works grouting applications.

2. Applicability. This manual applies to all HQUSACE elements and USACE commands having responsibility for construction of civil works.

3. Distribution Statement. This publication is approved for public release; distribution is unlimited.

4. References. References are listed in Appendix A.

5. Discussion. This manual provides detailed information on state-of-the-art technology, practices, procedures, materials, and equipment for use in planning and executing a grouting project. Different grouting applications are discussed, including preconstruction foundation treatment and post-construction remedial measures. Methods of grouting that have proven to be effective are described, and various types of grout, their proportions, and their best-suited uses are addressed. The manual discusses grouts composed primarily of cementitious suspensions and additives, although other types of grout are mentioned.

FOR THE COMMANDER:

PAUL E. OWEN
COL, EN
Chief of Staff

4 Appendices
Appendix A References
Appendix B Pressure Calculations
Appendix C Glossary of Terms and Abbreviations
Appendix D Acronyms

This manual supersedes EM 1110-2-3506, dated 20 January 1984.

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TABLE OF CONTENTS

APPENDICES

LIST OF TABLES

LIST OF TABLES (CONTINUED)

LIST OF FIGURES

LIST OF FIGURES (CONTINUED)

LIST OF FIGURES (CONTINUED)

LIST OF FIGURES (CONTINUED)

LIST OF FIGURES (CONTINUED)

LIST OF FIGURES (CONTINUED)

LIST OF FIGURES (CONTINUED)

LIST OF FIGURES (CONTINUED)

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LIST OF FIGURES (CONTINUED)

CHAPTER 1

Introduction

1-1. Purpose. This manual provides technical criteria and guidance for civil works grouting applications. Information on procedures, materials, and equipment for use in planning and executing a grouting project is included, and types of problems that might be solved by grouting are discussed. Methods of grouting that have proven to be effective are described, and various types of grout and their proportions are listed. The manual discusses grouts composed primarily of cementitious suspensions and additives, although other types are mentioned.

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1-5. Changes. Users of this manual are encouraged to submit recommended changes or comments to improve it. Comments should reference the specific page, paragraph, and line of the text for which the change is recommended. Reasons should be provided for each comment to ensure understanding and complete evaluation. Comments should be prepared using DA Form 2028 (*Recommended Changes to Publications*) and forwarded directly to HQUSACE (CECW-CE) Washington, DC 20314.

1-6. General Considerations. Grouting in civil works activities is performed as: (1) an element of permanent construction, (2) a post-construction remedial treatment, or (3) an element of expedient construction or repair. Examples of permanent construction are curtain grouting in foundations for dams and ground stabilization of foundation materials for large buildings. Examples of post-construction remedial treatment include grouting voids under concrete structures and reducing leakage through dam foundations or abutments. Grouting is used for both temporary and permanent treatments. It should be considered in combination with other appropriate types of treatment for best results. Other types of treatment may include excavation, compaction, concrete cutoff walls, slurry trenches, impervious blankets, drainage blankets, and filter zones. Treatments also include relief wells, drilled drains, sheet pile cutoff, dental concrete, drainage tunnels and galleries, underpinning, and structural foundations. Purposes of expedient grouting include repair of roadways, cofferdams, and stability and groundwater control during construction.

1-7. Brief History of Injection Grouting.

a. Origins. Injection grouting originated in Europe. The first known case history of deliberate pressure injection occurred in France in 1802, where it was used to repair the foundations of a timber weir and a sluice structure. A crude type of piston pump was used to inject clay grout into alluvial materials and mortar grout into voids. Subsequently, injection grouting was used for the repair of a number of lock structures and repair of masonry structures in the period from 1802 into the 1850s. From the mid-1850s into the early 1890s, there were a number of significant advances in grouting that led to more frequent use in foundation repair, mining, and hydraulic structures. Compressed air pressure vessels were developed to facilitate increased injection pressure, stirring systems were developed to keep grout in suspension, and improved piston pumps were developed. The first known application of rock grouting for dams occurred in England between 1876 and 1878.

b. Dam Foundation Grouting in the United States.

(1) Systematic grouting of dam foundations in the United States appears to have started at New Croton Dam in New York in 1893. A limestone foundation was treated on a large scale by drilling holes with compressed air drills, washing out major fissures, and grouting under a gravity head or with a hand-operated pump and/or a piston-like rammer. The first major application of cement grouting occurred in 1912 at Estacada Dam in Oregon, where three rows of grouting totaling 34,120 ft (10,400 m) of grout holes were used in its construction. Publication of the details of the grouting program by the American Society of Civil Engineers (ASCE) in 1915 led to great interest and discussion within the profession on grouting procedures, problems, and results.

(2) Grouting at Hoover Dam in the 1930s marked the development of a systematic approach to dam grouting and the acceptance of cement grouting as a normal feature for treatment of dam foundations. Grouting at Hoover Dam was far less than fully successful, and excessive seepage occurred on first filling, requiring extensive remedial grouting between 1938 and 1947. From the 1930s forward, dam construction and dam foundation grouting was a prominent activity within the United States. Agencies such as USACE, the Bureau of Reclamation (USBR), and Tennessee Valley Authority (TVA) all constructed a large number of dams from that point into the early 1980s.

c. Recent History of Advances in Grouting within USACE.

(1) Advances have been made based on available technology in computers and instrumentation. The Waterways Experiment Station's (WES's)[*] CAGE program (Computer Applications in Geotechnical Engineering) developed a computer program for monitoring grouting that was used at Buffalo District's Black Rock Lock in 1975 and at Center Hill in 1984. The program was crude by today's standards and lacked the capability for graphical outputs so interest in it wanted. Interest by USACE in using better technology on site for grouting was revived by the Jacksonville District during the grouting of the Portuguese Dam foundation in Ponce, Puerto Rico, in 1997–1999. Efforts led by the Jacksonville District and supported by geologists and engineers from 10 other districts resulted in using the best available technology and practice in many areas, including: geologic surface mapping, core borings, exploratory adit mapping, borehole video mapping to determine fracture frequency, orientation, and aperture thickness, water pressure testing, and grout pressure tests. The results of these investigations and other field experimental efforts led to the determination that ultrafine cement was required for effective grout penetration, which marked the first use of ultrafine cement grouts on a USACE

[*] Now known as the Engineer Research and Development Center, Waterways Experiment Station (ERDC-WES).

project. Additionally, special project-specific computer programs were developed to track all grouting data collected in the field. These programs permitted input of grout data into a central database to allow continuous tracking of certain critical parameters.

(2) Concurrent with the USACE efforts, the private sector was in the process of implementing other major advances in technology. In 1998, Penn Forest Dam in Pennsylvania was the first project to use real-time automated data collection and display technology for grouting, along with balanced stable grout formulations. A USACE Grouting Workshop was held at that site in 1998 with attendees from many districts to demonstrate the application of the new technology.

(3) Since 1998, USACE has actively supported and embraced the latest developments in grouting, including: (1) design of grouting as an engineered feature, (2) use of balanced stable grout mixes, (3) advanced computer monitoring, control, and analysis for controlling grout injection, production of project records, and performance verification, and (4) Best Value Selection for grouting projects. These efforts were led by the Louisville District and Headquarters through use of these approaches in projects and by organizing on-site USACE workshops for dissemination of the information. In 2000, the Patoka Lake Project, where a grouted cutoff was constructed in karstic limestone between the dam and the emergency spillway, was the first USACE project to successfully incorporate and integrate all of these elements. Following the success at Patoka Lake Dam, the same general approach with substantially more advanced and more powerful computer technology was used effectively in the Chicago McCook Reservoir test grouting program (Chicago District), the Mississinewa Dam cutoff wall pre-grouting program (Louisville District), the sinkhole remediation project and cutoff wall pre-grouting project at Clearwater Dam (Little Rock District), and the cutoff wall pre-grouting project in karstic geology at Wolf Creek Dam (Nashville District). Many of the specific approaches and techniques for best application of the new technology were developed and refined on these USACE projects. Headquarters determined that this update of the *Grouting Technology Manual* should include all of the advances in equipment, methodology, materials, and technology that have been accomplished on these exemplary projects. At the current time, due to the progressive and diligent efforts by staff from these districts, USACE is at the forefront of best practice in grouting.

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CHAPTER 2

Purposes and Applications of Grouting

2-1. Purposes of Grouting.

a. General. Grouting is the process of injecting liquids, mixed suspensions, or semi-solid mixtures under pressure to achieve one or more desirable end results in terms of improving engineering properties. To accomplish this, the injected grout must eventually form either a gel or a solid within the treatment zone. Permeation grouting is the injection of high-mobility grouts (HMGs) into small voids within soil or rock masses, into small voids between these materials and an existing structure, and/or into small cracks or fractures within structures themselves. Void-filling grouting involves using low- (or limited) mobility grout (or grouting) (LMG) or other materials having properties suitable for effective filling of large voids. Compaction grouting is the injection of plastic, semi-solid mixtures to densify or displace deformable materials. In-situ modification or replacement includes specialized techniques such as jet grouting or hydrofracture grouting. The following paragraphs describe the engineering properties that can be improved by grouting. Depending on the specific application, grouting is used as either the primary or sole means of effecting property improvement, or it may be used in conjunction with other technologies and methods.

b. Permeability Reduction. Grouting is commonly used to reduce permeability, which might be necessary for reducing rates of seepage or leakage through or into new or existing structures and foundations, reducing hydrostatic forces acting on structures, altering flow gradients or flow paths to achieve specific design objectives, inhibiting internal erosion of foundation and embankment materials, and/or controlling water for excavations as required to facilitate dewatering or excavation stability. In any critical hydraulic application, grouting is normally one of several lines of defense.

c. Improvement of Mechanical Properties. Grouting can be used to improve the mechanical properties of soil or rock foundation materials for structure or excavation support purposes. Properties that can be improved by grouting include enhancement of bearing capacity, improvement in settlement-related properties such as elastic modulus and void ratio, improvement in shear strength, and elimination of voids that might adversely affect either loading conditions or the response to loads. Grouting can also be used as a settlement compensation method to prevent or repair damage to structures.

2-2. Typical Applications of Grouting.

a. Dams and Lock Structures. Typical applications of grouting for new or existing earth and concrete dams, and for lock structures include:

(1) Hydraulic barrier grouting to control leakage and pressure distributions (i.e., grout curtain construction).

(2) Foundation consolidation grouting to reduce foundation and structure deformations under load.

(3) Contact grouting of the interface between structures and foundations.

(4) Void filling in foundations beneath structures.

(5) Compaction grouting for densification of loose deposits or jet grouting to replace zones of loose materials.

(6) Pre-treatment of fractured rock foundations to enable cutoff wall construction.

(7) Grouting of leaking cracks or joints in structures.

(8) Abandonment or backfilling of exploration and instrument holes.

(9) Improvement of zones as required to facilitate dewatering and/or excavation stability.

b. Tunnels. Typical applications of grouting for tunnel work include:

(1) Grouting in advance of tunneling to reduce water inflows during construction.

(2) Grouting in advance of tunneling to improve excavation stability and/or reduce or to prevent ground loss during tunneling.

(3) Grouting between the tunnel lining and the tunnel excavation surfaces to reduce long-term tunnel loads, improve stress distributions, and reduce water inflows.

(4) Remedial grouting of joints and cracks to reduce leakage.

(5) Compensation grouting during tunneling to prevent settlement and to protect structures that would be adversely affected by ground loss during tunneling.

c. Other Grouting Applications.

(1) Rock and soil anchor grouting to develop anchor capacity and to provide corrosion protection.

(2) Lifting of structures by displacement grouting methods.

(3) Filling of abandoned pipes or other underground structures.

(4) Annular space grouting for pipes being re-lined.

(5) Environmental applications.

(6) Filling of mine voids.

2-3. Selection of Methods of Treatment. Grouting is one method of treating problematic subsurface conditions to reduce permeability or improve engineering properties of the foundation. However, other methods of treatment may be required in addition to or instead of grouting. Where structural safety is involved, multiple lines of defense will frequently be required. The selection of grouting as the method of treatment should be based on an evaluation of all pertinent aspects of the problem, including engineering needs, subsurface conditions, and economic considerations.

2-4. Potential Risk and Reliability Issues of Grouting.

a. Reliability of Grouting. Grouting will be a technically reliable means of accomplishing an intended purpose in any given application only if the following factors are present in the program: (1) geologic exploration that has been sufficient in scope and nature to allow proper characterization of the site specifically for grouting, (2) sound interpretations based on that information, (3) reasonable performance expectations for grouting that are consistent with the site conditions, the design, and the expected level of execution and quality control, (4) a thorough evaluation of the risks involved in pressurizing sensitive foundation materials, (5) well written and detailed plans and specifications that clearly define expectations, procedures, and QA/QC requirements, (6) and a thorough results verification program that is included as part of the construction. Adequate funding is essential to achieve these requirements. In the absence of one or more of these items, grouting frequently fails to meet expectations and/or performance requirements and can result in large expenditures with little improvement.

b. Grouting Risks. Historically, grouting has been one of the higher risk elements of construction, primarily because of the number of factors that can affect the overall outcome, but also because neither the actual features being grouted nor the behavior of the grout in those features can be directly observed during the course of the work. Frequently, grouting is executed in advance of other operations and is completed before the grout performs its intended function (e.g., before filling of a reservoir, during construction of a foundation, in advance of excavation). Grouting has also historically been an operation frequently subject to cost increases and/or extensions of time, sometimes due to unexpected subsurface conditions. Some factors that substantially reduce the overall risk of any grouting program are:

(1) Adequate and appropriate site investigations.

(2) Sound site interpretations as they relate to drilling and grouting.

(3) Appropriate program layout for the site conditions and intended results (hole spacing, depth, orientation).

(4) Proper sequencing and staging requirements.

(5) Proper material and mix selections.

(6) Proper pressures.

(7) Proper refusal criteria consistent with the program's intent and performance requirements.

(8) Modeling of the site using realistic expected performance parameters.

(9) Realistic project budgeting with appropriate contingencies.

(10) Contract documents and structuring of payment items that promote quality work and do not create a disincentive to quality.

(11) On-site contractor personnel who have highly relevant and successful recent experience in similar environments and in similar types of work.

(12) Appropriate, adequate, high-quality drilling and grouting equipment.

(13) A Quality Control and Quality Assurance Program that includes adequate inspection, control, testing, and analysis of the grouting work.

(14) Effective results verification program.

(15) Effective use of partnering to resolve technical and contractual issues.

(16) Adequate instrumentation to monitor project performance and distress during construction.

2-5. Permanence of Grouting.

a. Applications. Grouting can be used for both permanent and temporary applications. The durability of grouting depends on the grout mix design and rheologic properties, the environment into which the grout is placed, the quality of the final grouted product, and the service conditions. For example, the permanence of a grout curtain in fractured rock will depend on whether the rock fractures are clean or soil filled, the residual permeability achieved, and the gradient across the curtain. Where the grout formulations are properly designed, the rock fractures are clean, and the residual permeability is low, a completed curtain can be installed with a design life equal to that of the above-ground structure. When sulfate is present in the injected environment, durability can be an issue if sulfate-resistant cements or admixtures are not used. There are applications where, for expedience or feasibility, neat cement grout may be appropriate.

b. Long-Term Strength. Long-term strength is often a consideration for soil grouting applications. Permanence of grouted soil masses requires careful consideration of the grout materials and requires that the grout mix design be established by an experienced grouting professional. For cases where the rock fractures are not clean and are soil filled, future maintenance grouting may be needed to continue the level of protection.

2-6. Communication of Grouting Issues. Grouting, when executed properly, is often the best technical and/or most economically effective solution for a particular problem. However, it can also be a complex and expensive undertaking that is a critical element for the overall success of a project. Historically, it has been common that communication of critical information about grouting that is needed by project decision makers has been very limited and/or qualitative in nature. Throughout all phases of the project, from inception to completion, it is incumbent on the Project Delivery Team to clearly articulate all of the issues related to grouting. The project team must be able to: (1) technically substantiate the selection of grouting as an appropriate solution to a particular problem within the context of the site conditions, (2) present a rational basis for the types and amounts of grouting required, (3) quantify the expected performance results and benefits to be achieved, (4) identify the risks involved and the risk management strategy for each of those risks, (5) define the verification program, and (6) reasonable estimate the costs and contingencies.

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CHAPTER 3

Grouting Planning, Design, and Construction Process

3-1. Application of Design Principles. The level of technical oversight by the agency shall be appropriate for the level of risk and complexity of the project. The purpose of any grouting project, especially one designed for remediation of seepage problems involved with any USACE water retention project, needs to be fully defined and analyzed. The involvement of the Government is critical in assessing the measures that are required to address site-specific problems. The unique site characteristics and dam safety issues shall be considered when determining the most effective and economical solution, ensuring that the method chosen is the most suitable to achieve the desired results. The design and extent of the grouting program depends on the purpose for which it is intended, whether it is for avoiding the loss of slurry during cutoff wall construction or for installing a seepage barrier. The involvement of USACE experienced personnel is required from the start of the design phase and throughout the course of construction to provide technical expertise, guidance, and oversight to ensure that the grouting program satisfies the intended objective.

3-2. Effectiveness of Grouting. Grouting is an extremely effective treatment technology, but from a historical perspective, there have been many unsatisfactory project experiences. Unsatisfactory outcomes have included: (1) unreliable prediction of end performance, (2) unsatisfactory initial performance, (3) unsatisfactory long-term performance, (4) cost overruns, delays, and claims for additional compensation, and (5) ineffective communication about the need for grouting, the amount of grouting required, and the results of grouting. Each of these types of unsatisfactory outcome is the direct result of one or more shortcomings in proper execution of the planning, design, and construction process. This chapter outlines the general steps necessary for the successful completion and performance of grouting. More specific information related to particular applications and methods is covered in later chapters of this manual.

3-3. Reconnaissance Phase. The purpose of the Reconnaissance Phase is to identify an existing problem and potential solutions or to define a project to address a specific public need. Any evaluation of the possible applicability of grouting as an element of a project will be based on general geologic information, any site-specific information that might exist, and sound engineering judgment of conditions generally suitable for grouting and reasonable expectations of grouting results in comparison to other technologies that are available. An important outcome of the Reconnaissance Phase is an understanding of the scope of investigations and studies required for the Feasibility Phase.

3-4. Feasibility Phase.

a. Purpose. The purpose of the Feasibility Phase is to formulate a specific solution to address a specific public need. Work in this phase includes studying potential solutions, evaluating costs and benefits, preparing initial designs, and recommending a plan to solve the problem. An important objective is to develop the design of the recommended plan in enough detail that it can be authorized, implemented, and constructed without major changes in concept, cost, or schedule. With respect to grouting, the Feasibility Phase will include a site investigation and characterization, a preliminary evaluation of the technical suitability of the site for grouting, a determination of the potential benefits to be derived from grouting, and a preliminary estimate of the cost. Geophysical investigations can be used to define seepage pathways and to aid in the design of further investigations. The geotechnical investigations may include borings, which allow both water tests for estimating in-situ hydraulic conductivity and borehole imaging for characterizing the ability of the rock to accept grout. These boreholes may also be used for installing additional instrumentation in areas where information is deficient. The intended outcome from this phase is a rapid, but reasonably reliable assessment of whether grouting is likely to solve the problem at hand. Table 3-1 lists (and the following sections describe) elements of this phase.

b. Quantification of Site Conditions. Geologic and geotechnical investigation data need to be summarized in terms of engineering parameters and also in terms of the parameters needed to assess the applicability of the specific grouting methods being considered. Items that should be considered for rock materials include: (1) presence and locations of discontinuities, (2) orientation and spacing of rock fracture systems, (3) rock mass permeability, (4) rock quality as it relates to the ability to maintain open grout holes, (5) depth or thickness of weathered and infilled fractures, (6) compressive strength of the rock, (7) elastic modulus of the rock, (8) abrasiveness of the rock, and (9) effective stress conditions in the rock. Items that should be considered for soil materials include: (1) Unified Soil (USCS) classification, (2) density or consistency, (3) grain size distribution, (4) consolidation properties, (5) and effective stress conditions in the overburden. For the site in general, the piezometric levels and the nature of the flow regime must also be known with reasonable certainty.

c. Assessment of Grouting Suitability and Special Issues. Information gathered during the site characterization can be used to define areas that are suitable or not suitable for grouting and the probable degree of improvement. In addition, zones requiring downstage grouting and special treatment, such as zones with artesian pressures, the soil-rock interface, open and infilled voids, and alluvial materials, should be defined.

d. Establishment of Design Criteria and Objectives. Clear and quantitative design criteria and objectives should be established for the performance of the structure. Examples include such items as: (1) a specific maximum seepage rate, (2) a maximum foundation deformation and/or structure deflection, (3) a target density, (4) a specific pressure distribution pattern, (5) a specific shear strength, or (6) a specific permeability in a grouted zone.

e. Analysis of Foundation and Structure Behavior on the Basis of No Improvement in Properties. As a baseline condition needed to assess the benefits of the proposed grouting, the foundation and/or structure should first be analyzed on the basis of the unaltered properties. For a preliminary evaluation and assessment of the feasibility of grouting, detailed analytical models are not necessarily required. Simplified models can be adequate provided that they are fundamentally correct and that the input data are sound. The model should be used to define problem zones that require improvement. The results from the models should be compared to the design criteria and objectives to determine if grouting is of potential benefit for the specific site and application.

f. Development of Cost Estimates and Selection of Recommended Program. Preliminary cost estimates for alternatives that meet or exceed the performance requirements should be developed. Then the quantitative and qualitative benefits of each alternative and the incremental performance benefits derived from different concepts and configurations should be evaluated. Based on these evaluations, a recommended program should be selected. Once a program is selected, the details of the program should be expanded, including the development of a detailed list of work items. From this, a final cost estimate using quantities and units should be prepared and a cost-reality check using information from other successful projects and external information sources should be performed.

g. Preliminary Assessment of Grouting Applicability. If property improvement is of potential benefit or necessary for the performance of the structure, the next step is to perform a preliminary assessment of the suitability of the site for grouting. Some general guidelines for various rock and soil conditions are:

(1) Clean rock fractures are readily groutable at Lugeon values of 50–100 or greater, and it is possible to routinely achieve post-grouting permeabilities of 10 or less, depending on the sophistication of the grouting program.

(2) Clean rock fractures are marginally groutable at Lugeon values of about 10. It is possible to achieve post-grouting permeabilities on the order of 1 with a very carefully planned and executed program. If numerous special methods and materials are used, it might be possible to achieve post-grouting permeabilities on the order of 0.1 Lugeon.

(3) Clean rock fractures are barely groutable at Lugeon values of about 1. Rarely is grouting required at this level, but if it is needed for special applications, the permeability can be reduced to about 0.1 Lugeon if special methods and materials are used.

(4) Highly weathered and soil-infilled fractures in rock are problematic for grouting. In general, it is not possible to thoroughly wash erodible materials from the fractures before grouting. These materials have been successfully grouted by using multiple lines of grouting to confine the infilling and reduce the flow gradient across the zone. Provided all stages on all holes and on all lines are brought to full refusal at the desired grouting pressure, some level of confidence is possible that the zone has been adequately grouted. However, there are differing opinions on the longevity of the grouting effectiveness due to concerns about long-term erosion of semi-confined, erodible materials.

(5) Consideration must be given to the quality of the rock and/or the frequency of water losses, as those items affect the required drilling and grouting procedures and costs. In general, a Rock Quality Designation (RQD) less than about 40% may require downstage grouting rather than upstage grouting. Frequent water loss during drilling is also a strong indicator that either certain geologic zones or perhaps one or more series of holes in the grouting sequence will require the use of downstage drilling and grouting procedures. Section 3-5 provides further discussion of the applicability of downstage and upstage procedures. Additionally, consideration should be given to the presence of special zones at depth that may limit the amount of time a grout hole will stay open. Highly fractured units, soft or slaking zones, and other such conditions may require special techniques.

Table 3-1. Feasibility Phase elements.

Task	Details
Site characterization for grouting	• Research, review, and summarize available data. • Perform supplemental geologic mapping, field investigations, and testing. • Quantify all site conditions into usable design parameters for grouting (i.e., soil stratigraphy and characteristics, rock stratigraphy, fracture system spacings and orientations, permeability profiles, rock quality, weathering profiles, rock mass properties, etc.).
Establishment of quantitative design criteria and objectives	• Develop project-specific definitions of performance requirements for the completed work such as maximum seepage rates, maximum foundation deformations or structure deflections, density requirements for soils, specific pressure distributions, and required shear strengths.
Simplified modeling and analyses based on no improvement in properties	• Establish the extent to which improvement in properties is required for project performance. • Define problem zones that require improvement.
Preliminary assessment of grouting suitability and special issues	• Define groutable zones and probable degree of improvement possible. • Define non-groutable zones. • Define zones requiring downstage grouting and zones suitable for upstage grouting. • Identify special treatment zones (e.g., artesian zones, soil-rock interface zones, open and infilled voids, and alluvial materials).
Assignment of expected improvement in properties in grouted zones	• Assign properties that can be reasonably expected to be achievable in consideration of all factors that can affect grouting results. • Define possible alternate treatments for zones that are not good candidates for grouting.
Configuration of alternate designs and models and re-analysis of behavior with improvements	• Conduct a trial and error process to evaluate various alternatives. • Compare results from each trial with design performance requirements.
Development of preliminary cost estimates and selection of recommended program	• Prepare rough comparative cost estimates for alternatives that meet or exceed the performance requirements. • Evaluate total quantitative and qualitative benefits for each alternative. • Evaluate incremental performance benefits derived from different concepts and configurations. • Select a recommended program.

1:43 PM. (Continued).

Development of final cost estimate for recommended program	• Expand the details of the recommended program. • Develop a detailed list of work items. • Prepare a final cost estimate using quantities and units. • Perform cost-reality checks using other information (e.g., final total costs for similar successful projects, and external information sources).

(6) Known artesian zones may require special construction techniques.

(7) The ability to effectively grout soils by permeation into the pore spaces is highly dependent on the grain size characteristics of the soils and the uniformity of the soil deposits. While chemical grouts have a viscosity similar to water, grouts may penetrate into soils extremely slowly due to very low permeability created by even a small percentage of silts, clays, and very fine sands.

(8) The effectiveness of compaction grouts is a function of soil density, soil permeability, and groundwater levels. In a semipervious material, the desired densification will be inhibited by the buildup of pore pressures in the soil, which might also create problems in their own right.

h. Assignment of Improved Properties for Grouted Zones. After the determination of what zones can be effectively grouted, the properties that can be reasonably expected to be achieved in the grouted zones should be assigned. An equally important part of this step is to determine how zones that are not good candidates for grouting will be treated. Techniques for zones that are not good candidates for effective grouting include removal of materials and/or application of other technologies such as cutoff walls.

i. Configuration of the Dimensions of Grouted Zones and Re-analysis of Behavior with Improved Properties within Those Zones. A trial and error modeling process should be used for a preliminary evaluation of the benefits of various grouting configurations. The results from each trial configuration should be compared with the design criteria and performance objectives, and with each other, to understand both the total performance of a particular configuration and the incremental benefit/cost relationships of different alternatives. The analyses should also include sensitivity analyses for various grouting outcomes. For hydraulic applications, the sensitivity analyses may need to be based on order-of-magnitude variations, since that is generally the fundamental unit of variation in both natural conditions and potentially in grouting performance. For other properties and applications, the sensitivity variation may be less than an order of magnitude.

j. Development of a Preliminary Cost Estimate for Grouting. After one or more trial grouting plans are established, a rough estimate of the quantities and costs should be prepared. At this level of assessment, the simplest method is to use bulk total grouting costs obtained from reasonably similar projects converted to a simple, single unit. When using data from other projects, it is important that the data be obtained from project completion reports and records rather than from initial unit pricing, since there can be a significant discrepancy between the two. The project completion reports represent data generated from actual conditions, whereas the projected values are based on the initial pricing. Costs may vary greatly, depending on the application, project size, and injected volumes.

k. Development of a Final Cost Estimate for the Feasibility Phase. After the recommended grouting program is developed, a final cost estimate should be based on actual estimated quantities and units. Costs may vary greatly, depending on the application, project size, and injected volumes.

3-5. Preconstruction Engineering and Design Phase.

a. General. If the Feasibility Phase evaluation indicates that grouting is a technically effective and economically viable project element and part of the recommended plan, it will move forward to the Preconstruction Engineering and Design (PED) Phase. In the PED phase, the design is finalized, the plans and specifications (P&S) are prepared, and the construction contract is prepared. The intent is to have a grouting plan and contract documents that can be expected to achieve the design goals, is readily constructible, and is free from any major technical, contractual, or financial flaws. Table 3-2 lists (and the following sections describe) elements of work in this phase.

Table 3-2. PED Phase elements.

Task	Details
Outline of the major project execution elements required in context of project difficulty, criticality, project performance requirements, and site conditions	• Consider the fundamental nature of the project site conditions and the grouting performance requirements. • Assess the criticality of the grouting with respect to the performance and safety of the project. • Assess the sensitivity of the end performance of the program to potential grouting effectiveness outcomes. • Determine whether the grouting program can be highly defined by prescriptive methods or if it might need to evolve during the course of the work. • Determine the nature and degree of performance verification required. • Assess the experience and knowledge of the Corps project team providing program oversight.
Acquisition of supplemental design data	• Perform additional site investigations and testing. • Review existing instrumentation and determine if additional instrumentation is required. • Perform full-scale test sections.
Detailed assessment of equipment and methodologies	• Identify all necessary equipment, methodologies, and procedures necessary for execution. • Identify all special requirements, problems, constraints, and workable remedies.

Table 3-2. (Continued).

Definition of the minimum verifiable end results that must be achieved	• Define the minimum required geometry and properties that must be achieved for acceptable performance. • Define the target minimum permeability requirements.
Determination of Impacts to Grouting due to Construction Activities and Sequencing	• Plan sequencing of grouting with respect to other aspects of the construction schedules to achieve the best technical result.
Development of detailed program layout	• Determine the surface from which grouting will be performed, the site preparation requirements, the hole and line spacings, the hole orientations, the grouting depth, the stage lengths, the downstage and upstage zones, any special treatment zones, any sequencing requirements, etc.
Design of field monitoring, control, analysis, and verification programs	• Determine what results will be obtained and how the results will be analyzed, presented and verified.
Design of QC and QA organization and responsibilities	• Identify all staffing required for observation, testing, recordkeeping, reporting, and analysis. • Identify staff qualification requirements. • Determine the respective functions and responsibilities of the Government and the contractor.
Development of units for measurement and payment	• Review final list of units for adequacy, fairness, and flexibility. • Review units for potential impact on quality of work (i.e., units that create quality disincentives).
USACE District Quality Control/Quality Assurance	• Conduct all appropriate levels of review: District Quality Control (DQC), Agency Technical Review (ATR), Biddability, Constructability, Operability, and Environmental Review (BCOE), Independent External Peer Review (IEPR), Compliance with Environmental Operating Principles (EOPs), and Policy and Legal Review.
Preliminary Design Cost Estimate	• Combine results of all previous elements into a revised cost estimate based on estimated quantities for the specific contract line items selected. • Develop quantities based on directly estimated plan amounts, with appropriate contingencies.
Preparation of P&S and final cost estimate	• Tailor all elements of the specifications to the project. • Understand the basis for every requirement in the documents.

b. Assessment of the Fundamental Nature of the Site Conditions, Project Criticality and Difficulty, and Performance Requirements. A critical first step in the PED phase is to consider the fundamental nature of the project site conditions and the grouting performance requirements for the project. Information that must be considered is: (1) whether the grouting is being relied on as a critical or non-critical element with respect to the performance and safety of the project, (2) the sensitivity of the end performance of the program to potential grouting effectiveness outcomes, (3) the vulnerability of the project to potential negative impacts from the grouting, (4) whether site conditions (and/or knowledge of the site conditions) permit the grouting program to be highly defined by prescriptive methods or whether conditions or knowledge are such that the specifics of the grouting program might need to evolve during the course of the work, (5) the nature and degree of performance verification that is required, and (6) the experience and knowledge of the Corps project team providing program oversight. These factors will control the choice of many items relating to successful contract preparation and successful field execution of the project, including: (1) the extent of supplemental exploration and testing required as part of the grouting contract, (2) the choice of grouting materials and methods, (3) the choice of contracting method, (4) the level of technology required for monitoring, control, and analysis, (5) the measurement and payment items and units, (6) the quantity estimates, (7) the QC and QA organizations required by the contractor and by the Government, (8) the need for full-scale test sections, (9) the methods to be used to verify the grouting outcome performance, (10) the recordkeeping requirements, (11) the need for special equipment and techniques to be available for the project, and (12) the decision responsibility during execution.

c. Acquisition of Supplemental Data. If the Feasibility Phase or early consideration of the site conditions and performance requirements during the PED phase discloses data deficiencies or gaps that would prevent conclusive evaluations and proper continuation with the PED, additional site investigations and testing must be undertaken to remedy the data issues. This frequently includes investigations into localized areas, special investigation and testing techniques, or filling general data gaps. As part of the design phase, the existing instrumentation should be evaluated to determine if additional instrumentation is required to monitor the project response to grouting operations. Full-scale test sections can be used to gather this data.

d. Detailed Design of Grouting Program. Whereas the Feasibility Phase elements did not require a comprehensive understanding of every element of grouting, the PED phase cannot be initiated effectively without a thorough knowledge of equipment and methodologies that are appropriate and/or required for project-specific grouting conditions and applications. After preliminary selection of appropriate methodologies and equipment, it is necessary to outline the entire drilling and grouting process for typical grout holes in the site’s geologic conditions, including all the steps and sequences that might be required on a hole. All special problems and issues must be identified, and workable solutions must be devised. Special problems can include such items as: (1) special requirements for drilling through existing dam embankments, (2) grouting of structure-foundation interface zones, (3) drilling and grouting in unstable zones, (4) dealing with voids, (5) drilling and grouting in weathered rock zones, the soil-rock interface area, and soil-filled fractures, (6) handling artesian conditions, (7) establishing environmental controls, (8) special deviation requirements, (9) protection of adjacent structures or features, and (10) site access and terrain considerations for the proposed equipment. When grouting through an existing embankment dam or other critical structure, it is essential to determine the allowable safe grouting pressure that can be used without causing hydrofracture or damage due to displacement. This may require the use of numerical models or other methods to estimate the principal stresses at the locations where the grout can be exposed to the soils (see Section 15-9). For grouting rock beneath a dam, a reliable method must be developed for isolating the grout from the foundation or embankment soils. This may include sleeve-port grouting of the soil-rock interface and downstage grouting at low or gravity pressures for the first few stages. After the complete process for a typical hole is verified to be workable, consideration must be given to the sequencing of holes, the work area requirements, and the outcome verification procedures. The realities of equipment, materials, and methods that are available must be carefully considered in this process to allow the program to be constructible in a production environment.

e. Final Design Criteria. In the PED phase, it is necessary to establish the minimum verifiable end results that can be achieved by grouting. The geometry and properties of the grouted zone need to be established with certainty. Because the satisfactory performance of the design is contingent on achieving these properties, they must be selected with care, conservatism, and due consideration of all the factors that might impact achieving them. After these values (such as the target permeabilities) have been determined, the construction process must result in attaining them. Grouting history unfortunately contains many cases in which either outcome expectations were unrealistic in the context of the design and/or the contract execution, or the design requirements were abandoned during construction as a result of unrealistically low cost expectations. In both scenarios, the result is the expenditure of a great amount of effort and funds with an unsatisfactory project performance outcome.

f. Constructability Evaluation. Considering grouting within the overall needs of the project should be done early in the design process to evaluate the advantages and disadvantages of sequencing scenarios. Sequencing the grouting operation with respect to other aspects of the construction schedules should ideally be planned and controlled for the best technical result. For example, on some jobs it may be advisable or necessary to delay grouting until other portions of the project are completed, but in other cases, the opposite sequence may be far more desirable. Grouting a foundation ahead of placing the embankment may reduce the pressures that can safely be used, potentially reducing its effectiveness; however, grouting earlier reduces the footage of drilling, and also the costs and risks associated with drilling and grouting through an embankment. Damage to the foundation that may occur during grouting is more apparent and can be addressed before an embankment is in place. In other cases, various constraints on construction such as cost, schedule and site geometry, construction season, or logistical considerations may force the construction activities into a less-favorable sequence. For example, grouting may have to precede blasting and excavation to control water, even though blasting may impact the grouted rock/soil mass. In these cases, damage, if any, to the completed works will be difficult to quantify and remediate, so these situations should be avoided where possible. Measures should be taken to protect previously constructed elements of a project and to minimize adverse impacts of one activity on another when less-favorable sequencing cannot be avoided.

g. Development of Detailed Program Layout. Work during the PED phase will include a detailed layout of the drilling and grouting program, including hole spacings, line spacings, and hole inclinations and azimuths. Figure 3-1 shows a typical grout hole layout. The detailed layout must also establish: (1) the surface from which drilling and grouting is to commence, (2) the surface and site preparation requirements, (3) the depth of grouting, (4) the stage lengths for grouting, (5) any special vertical or horizontal sequence requirements, and (6) a determination of whether drilling and grouting is to be upstage, downstage, or some combination of the two.

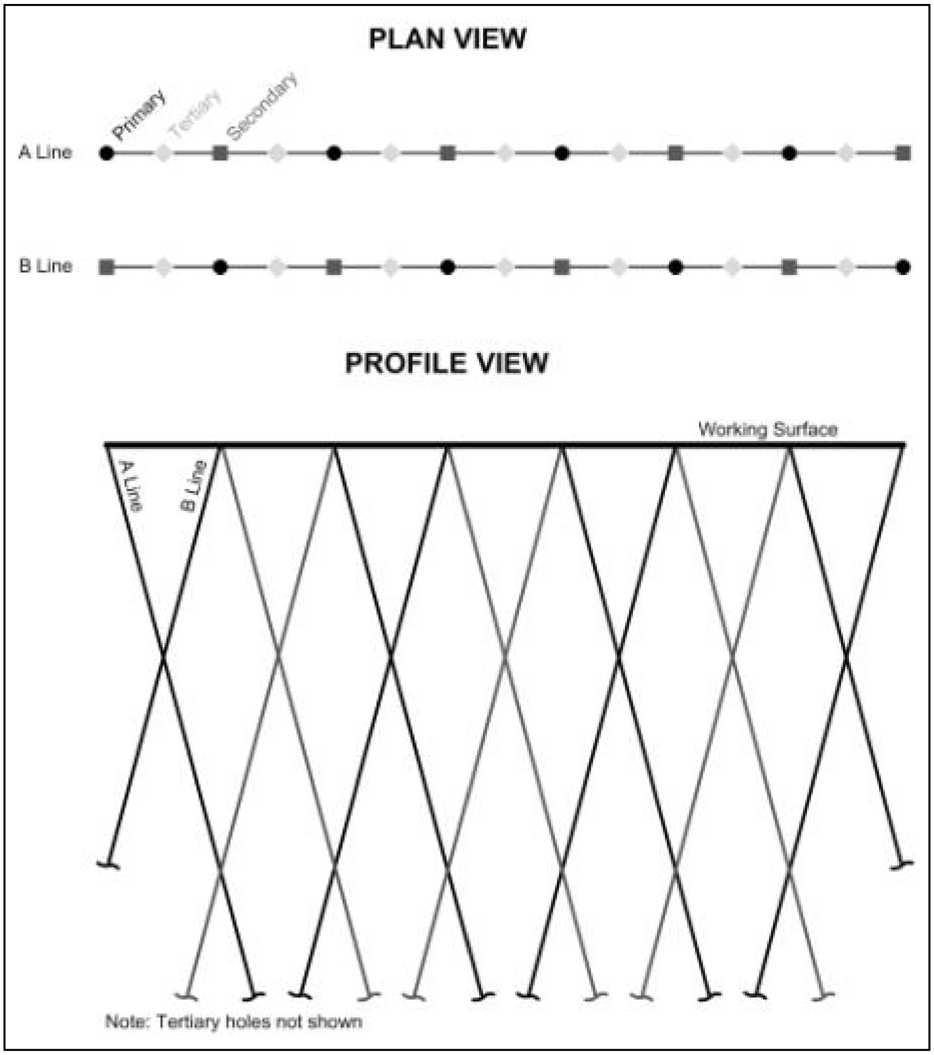

Figure 3-1. Typical grout hole layout.

h. Design of Monitoring, Control, and Verification Methods. Careful consideration should be given to how the results of the program will be analyzed as the work progresses in light of the project performance requirements. This includes the level of technology to be used, the records to be obtained, and how the records will be presented and used for verification. It should be anticipated that the grouting design will be modified as the work progresses. The design should be flexible to allow for the grouting program to be "engineered" in the field.

i. Preliminary Design Cost Estimate. The next step in the PED phase is to combine the results of all previous elements into a revised cost estimate based on the estimated quantities for the specific contract line items that are selected. If costs are to be extracted from another project, it is essential that a conversation occur with representatives who were directly involved in the work to ensure that their experience is captured in the data that are used. However, great care must be taken to ensure that the work is, in fact, actually comparable. For example, the same line item from another contract might include distributed costs associated with work that was required, but was neither specifically covered by the line item nor listed as an incidental cost item. Quantities should be based on the directly estimated plan amounts, with appropriate contingencies applied to each item. An adequate and appropriate budget is essential. One of the most problematic issues in grouting contracts has been underestimation of project costs and inadequate budgeting. Remedying this problem after the work is underway can result in claims and increased costs for both the Government and the contractor.

j. Contract Units for Measurement and Payment. A complete list of all contract line items, the units for measurement, and payment must be established with care as part of the PED phase. The selection of line items for measurement and payment can affect both project quality and project costs. The items must be appropriate for the work that is defined, the uncertainties that may be inherent in the plan, and any flexibility that may be needed. They must be workable in terms of field execution, and they need to be structured such that undue risk is not placed on either the contractor or the Government. The use of “a” and “b” bid items are encouraged as it provides significant flexibility in administration of the contract.

k. Plans and Specifications. Provided all the steps in the Feasibility and PED Phases have been executed properly, a sound blueprint for preparation of the P&S will be well established. The documents must provide an adequately detailed description of the materials, equipment, resources, methodologies, and results that are required by the contractor, and also a clear basis for acceptance, measurement, and payment for each element of the work. In some types of work, relatively standard specifications can be used as the base document. However, in many other applications, the specifications will need to be very carefully crafted to properly reflect the work. For more complex, critical projects, it is recommended that a geotechnical baseline report be included in the contract documents. When the contract documents are considered to be complete, they should be given a final review by the ATR team.

l. Quality Control and Quality Assurance Plan. A complete list of all personnel and requisite skill sets that are required for successful control and execution must be compiled. The list must outline their responsibilities and experience qualifications and detail when these personnel should physically be on the project site. The complete process of observation, testing, recordkeeping, reporting, and analysis activities should be diagrammed. The QC program required of the contractor should cover the entire process. After the QC program requirements and personnel needs have been thoroughly identified, the Government QA program should be detailed with the goal of assuring that the contractor’s QC program is adequate and continually monitored for contract compliance. It is essential that sufficient staff be provided and that they have appropriate specific experience for their assigned roles and responsibilities. Using inexperienced or junior staff is only permissible when they are functioning under the direct guidance of highly experienced senior staff. Experienced USACE personnel are required to be an integral part of the QA organization and should provide 100% coverage of the grouting operations.

m. USACE District Quality Control/Quality Assurance. Technical review is a continuing process from the start to the completion of the PED phase. After the grouting program has been detailed as outlined in the steps above, it is required as part of the design process that an in-depth technical review be performed in accordance with EC 1165-2-214, *Civil Works Review Policy*. The complete program should be presented to all appropriate levels of review (DQC, ATR, BCOE, IEPR, and Policy and Legal Review) as described in EC 1165-2-214. The presentation should include: (1) a detailed explanation of site conditions, special problems, and uncertainties, (2) a clear definition of the basis of the design and outcome requirements, (3) an explanation of the basis for the selection of materials, equipment, and methodologies, (4) a description of the monitoring, control, and verification methods, (5) an outline of the QC and QA organizational structure required to achieve the design requirements, (6) any contract forms necessary to achieve the desired results, (7) a proposed structuring of the measurement and payment items, and (8) the project schedule.

3-6. Downstage and Upstage Drilling and Grouting Procedures. In both the Feasibility Phase and the PED Phase, it is necessary to determine which portions of the work are expected to be conducted using downstage drilling and grouting procedures and which portions are expected to be conducted using upstage procedures (Table 3-3). This determination is important for structuring and preparing the contract documents and for estimating costs and schedule. Downstage drilling and grouting involves completing drilling, washing, water testing, and grouting of a stage within the hole and allowing the grout to set before advancing the hole to the next stage. Upstage drilling and grouting involves the advancement of the hole to full depth in one drilling operation, followed by washing the complete length of the hole in one washing operation and then pressure testing and grouting the hole in stages starting at the bottom of the hole and working upwards. More discussion on downstage and upstage drilling and grouting can be found in Chapter 15 of this manual.

Table 3-3. Advantages, disadvantages, and other considerations for downstage and upstage methods.

Consideration	Downstage Drilling and Grouting	Upstage Drilling and Grouting
Typical site conditions favoring use	• Rock of all types and conditions. • All water loss situations and/or sites with frequent hole connections. • Known problematic zones or reaches (e.g., highly broken or weathered zones, fault zones, weak rock zones, soil-infilled zones). • Karst formations and other void areas.	• Good quality rock (i.e., RQD>40%) that results in non-collapsing boreholes. • Sound rock suitable for sealing of packers on borehole sidewalls. • Minimal number of water losses during drilling (if water loss occurs, operations must cease and zone must be grouted before continuing). • Minimal number of hole connections.

Table 3-3. (Continued).

Advantages	• Shallow zones, often the most difficult and most important zones to grout, can be repeatedly grouted. • It is the most flexible method available to accommodate all conditions. • Drill cuttings from lower stages cannot clog fractures in higher zones. • It reduces hole interconnections that can result in incomplete or ineffective grouting.	• Stage lengths can be varied to fit conditions disclosed by drilling and pressure testing (e.g., short stage lengths can be used in problem zones and long stage lengths can be used in uniform, low-permeability zones). • Cheaper and faster than downstage grouting, provided conditions are favorable for upstage methods.
Disadvantages	• It is more expensive and time consuming than upstage grouting. • Potential for heaving surface rock when grouting without a heavy confining load if packer is set at surface, which can be avoided by setting packer at top of most recently drilled stage.	• Low-pressure grouting used in shallow zones. • Drill cuttings can contaminate fractures along entire length of hole. • Grout may bypass packers via the fracture system and re-enter hole above locations of packers. • Difficult to seal packers in weak or highly fractured rock, and water tests or grouting may cause loss of hole or drill tooling. • Connections with nearby holes may contaminate the holes before being grouted.
Other considerations	• It is common to use this method for upper zones of rock and known problematic zones. • This method is sometimes used to prepare site for upstage operations (e.g., sometimes for primary and secondary holes only, sometimes for first two lines, but not the middle line).	

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CHAPTER 4

Grouting Geology and Site Characterization Requirements

4-1. Impacts of Geology on Grouting. Grouting of soil or rock is an in-situ process resulting in the alteration of the natural geologic conditions. Given that grouting is an injection process, every aspect of a grouting program is either controlled or heavily influenced by the geologic conditions. Therefore, it is impossible to overemphasize the importance of obtaining sound qualitative and quantitative information on the geology and interpreting the data in terms of its relevance to the grouting program. The purpose of this chapter is to highlight some of the considerations in different geologic environments. A partial list of the multitude of factors that will be affected or controlled by the geologic environment includes: (1) design of the site investigation program, (2) site access and site preparation, (3) drilling equipment and methods, (4) hole orientation and depth, (5) hole pattern and spacing, (6) hole staging and sequencing, (7) hole stability, (8) choice of grouting materials, (9) rate of grout injection, (10) grout travel distance, (11) uniformity of properties within the grouted zone, (12) grout program quantity estimates, (13) need for special procedures, (14) verification program design, (15) interpretation of drilling and grouting results during the execution phase, and (16) reliability and/or permanence of the grouting program.

4-2. Generalized Grouting Geology Profiles.

a. General. Figure 4-1 shows a generic geologic profile structured to illustrate physical zones of interest for grouting. Depending on the particular grouting application, one or all of the zones may be of interest. In some geologic settings, not all of the zones may be present. Additionally, the nature and the characteristics of the zones will vary greatly according to the geologic environment in which the zones were formed. Figures 4-2 through 4-5 show a few examples of geologic profiles exposed through excavation.

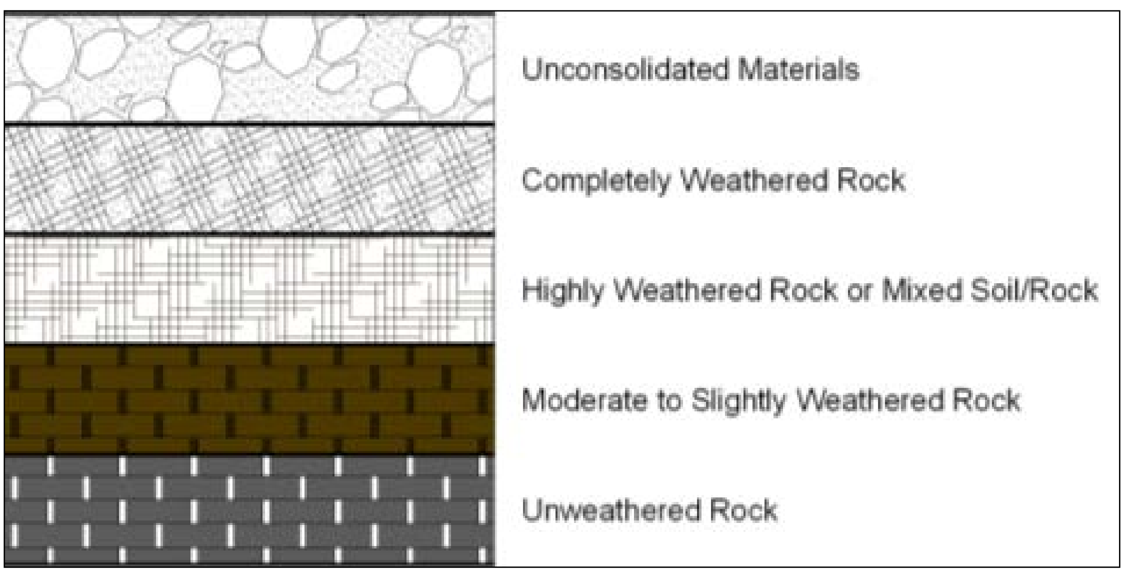
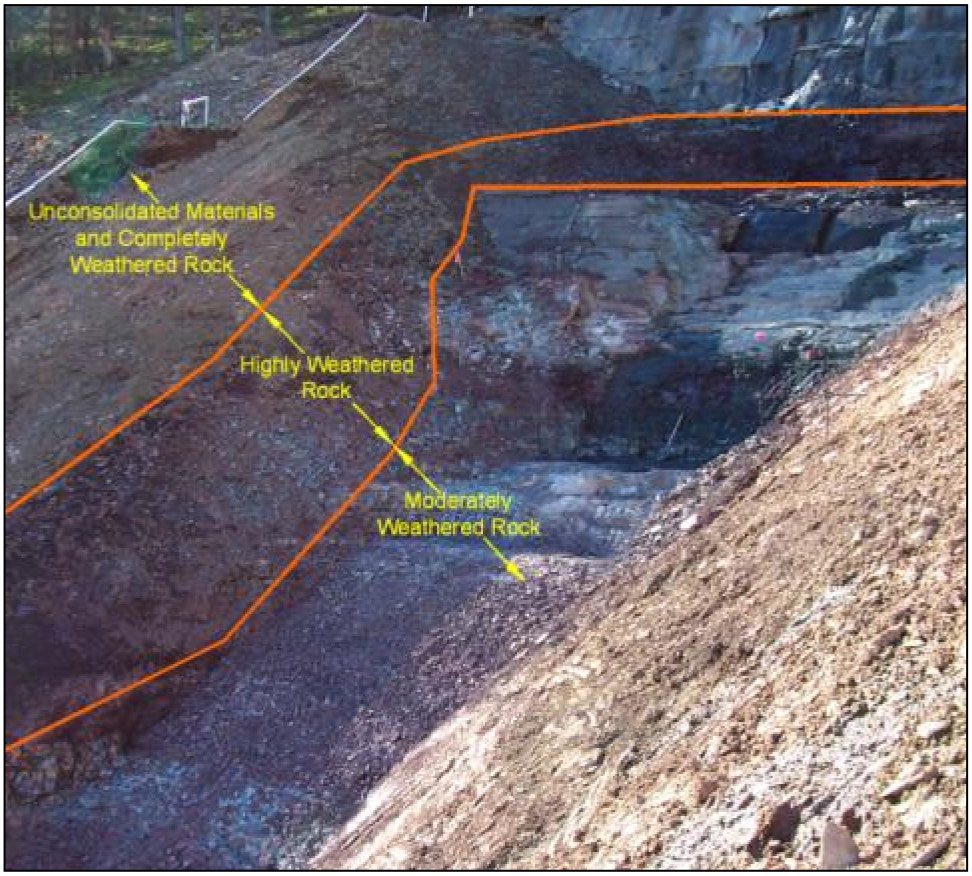
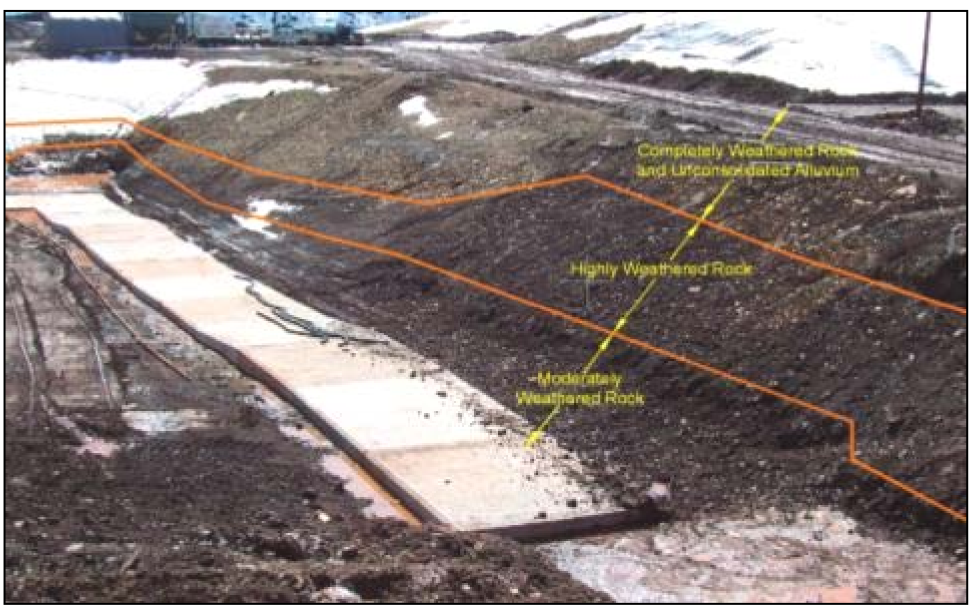


Figure 4-1. Generic geologic profile showing physical zones of interest in grouting.



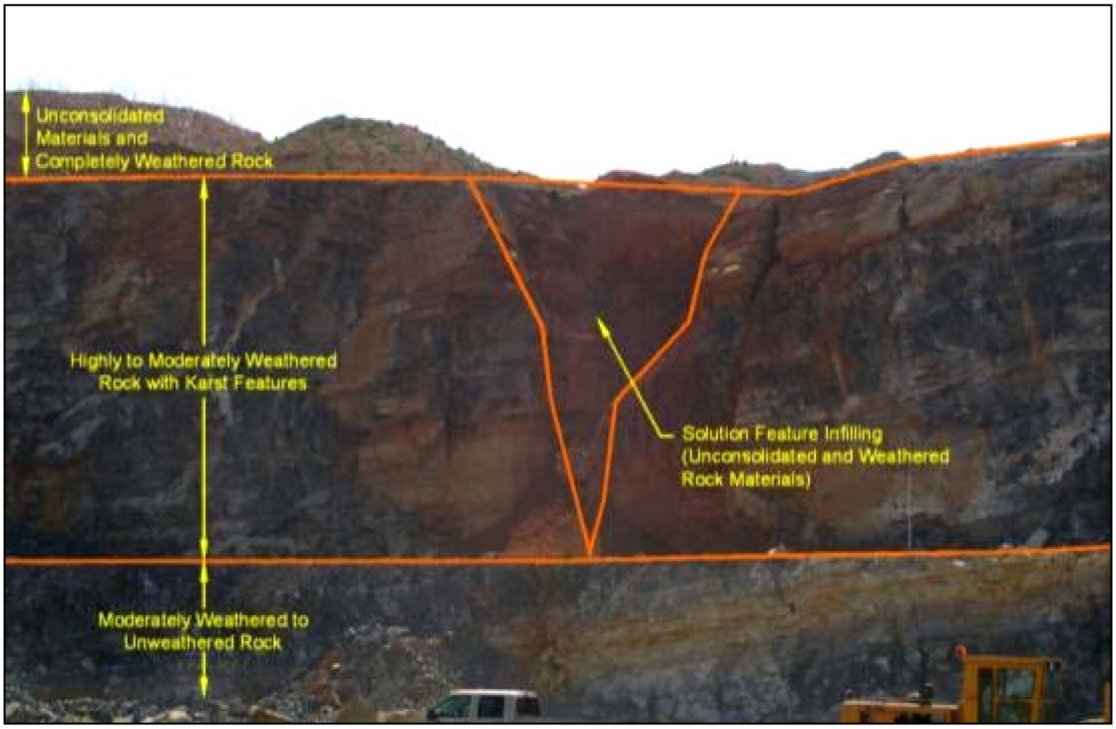
(Courtesy of Gannett Fleming)

Figure 4-2. Geologic profile in sedimentary rock at abutment, Lyman Run Dam, PA.



(Courtesy of Gannett Fleming)

Figure 4-3. Geologic profile in sedimentary rock valley, Lyman Run Dam, PA.



(Courtesy of Gannett Fleming.)

Figure 4-4. Geologic profile in a karst quarry in PA.

(Courtesy of Gannett Fleming)

Figure 4-5. Geologic profile in metamorphic rock, Hunting Run Dam, VA.

b. Unconsolidated Materials. Unconsolidated materials include all types of non-lithified materials, regardless of geologic origin. Unconsolidated materials include all soil types identified in the Unified Soils Classification System (USCS), and other materials such as cobbles and boulders and mixed soil/cobble/boulder materials. Man-made fills constructed of these materials, such as embankments and other engineered or non-engineered waste fills, are also unconsolidated materials. In some applications, unconsolidated materials are the object of grouting; in other applications, they are a zone that must be successfully penetrated by drilling to reach lower-profile elements to be grouted. In remedial grouting applications for embankment dams, they are typically materials that must be meticulously protected from disturbance by using special drilling and grouting techniques.

c. Completely Weathered Rock. A completely weathered rock zone is present in some geologic environments. This zone is characterized by materials having soil-like characteristics, but retaining an in-situ relict structure from the parent rock material. Drilling in these materials can be difficult because they can be both erodible and prone to hole collapse. Water pressure testing can be problematic due to softening and disturbance effects caused by the introduction of water. Grouting effectiveness in this zone can be very limited, and in new construction, this zone is normally removed by excavation before grouting whenever possible. It is common for the permeability of this zone to be the result of very fine fractures. Accordingly, grout penetration into the fracture system may be very restricted. Additionally, grouting will not improve the properties of the materials themselves. Where grouting is necessary, sleeve-port pipes or very short down stages are typically required. The overall effectiveness of grouting completely weathered rock can be marginal. Benefits may result, but they may not be predictable.

d. Highly Weathered Rock or Mixed Soil/Rock. This zone is characterized by predominantly lithified materials with a readily identifiable rock structure and fracture system, but substantial portions of the fracture systems are partially or completely filled with soil-like materials resulting from either severe weathering of the fractures or infilling of fractures with overlying soils. At one end of the spectrum, the general types of problems similar to those in completely weathered rock zones may occur, necessitating short downstage drilling and grouting or the use of sleeve-port methods. At the other end of the spectrum, the material might be predominantly rock, and holes may be stable even when penetrating soil-filled joints. A common misconception is that joint infillings can be effectively removed by washing. Even with extended washing until the return flow from the top of the hole is clear, it has been found that the effective washing distance is either very small or that only a limited number of joints have been washed clean (i.e., removal of material only to point of accepting the wash water injection rate). Accordingly, in these materials it should be assumed that soil-infilled joints will remain within the grouted zone. When this zone cannot be removed by excavation and when grouting of this zone is required, it is common to use a variety of methods to improve the effectiveness of the grouting. Wider grouted zones created by additional lines of grouting are often employed to improve the containment of infilled materials and reduce gradients through those materials.

e. Due to the potential for structural damage, hydrofracturing shall not be considered an appropriate measure for remedial grouting at existing dam projects. For projects where the potential for damage is less of a concern, hydrofracture grouting of infill may be beneficial. If episodic hydrofracturing occurs within infilled fractures at pressures less than the desired refusal pressure, grout injection should be extended as necessary until those episodes cease and refusal at the maximum desired pressure is reached. Pressure reductions to prevent the hydrofracturing are, in general, not recommended. Provided that hydrofracturing is not causing other incidental damage, the hydrofracturing will be beneficial in assisting to compact and contain the infilled materials, and it can chemically alter and improve the erosion resistance of the infilled materials if they contain some percentage of clay minerals.

f. Moderate to Slightly Weathered Rock. This zone is characterized entirely by lithified materials. The fractures are generally open, but may show evidence of staining extending several inches into the parent rock. After drilling, holes typically stand open. Upstage grouting should be possible, but the presence of highly fractured zones may warrant downstage techniques at some locations.

g. Unweathered Rock. Most geologic profiles contain unweathered rock with clean fracture systems. Weathering of fractures, if present, is sufficiently slight that staining is surficial. Similar to moderate to slightly weathered rocks, holes stand open indefinitely and the formation readily accepts grout.

4-3. Regional and Site Geology. A thorough knowledge of the regional and site geologic history is essential for the design and treatment of the foundation in many grouting applications, particularly when constructing hydraulic barriers at dams. It is particularly important for planning effective site investigation and testing programs, interpreting site investigation data, and making field decisions throughout the course of construction. Available sources of published information and records from prior investigations should be collected, compiled, and used. The available data should be supplemented as necessary by original geologic fieldwork and additional investigations. A cautionary note is that, for many projects, investigations have occurred in a series of phases stretching over many years and perhaps for multiple purposes. Rarely has all of the information been compiled, re-interpreted, and integrated into a single coherent form for the specific purpose of grouting programs. Failure to fully organize and use the information properly as one of the first steps has, on some occasions, led to defective designs and/or defective contract documents, both of which were readily apparent after problems were encountered and the data were reviewed.

4-4. Structural Geology.

a. Structure. "Rock structure" is a broad term referring to the spatial relationships of rock features and their discontinuities. Folds, faults, and fracture systems are all common elements of rock structure that are important in grouting.

b. Folds. A common type of deformation is folding. The folded rocks often show considerable fracturing along the axis of the fold. The severity of engineering problems depends on the complexity of the fold with relation to the type and geometry of the proposed structures and could include excavation, stability, and leakage problems. Folded sedimentary rocks have the potential to contribute to severe foundation leakage if the fold axes cross the dam axis. Fractures associated with the fold axes develop in the more brittle rocks in the sedimentary sequence, but they might not extend through the intervening, less competent layers. Openings in these fractures, which are the result of tensional forces at the crests and troughs of folds, may not even totally penetrate the brittle beds, as compressional forces will be dominant at the inner part of the folds. Therefore, the potential seepage paths are, in effect, more nearly linear than planar, so they require a dense pattern of grout holes for effective treatment.

c. Faults. Faults are fractures along which masses of rock have been moved in a direction parallel or perpendicular to the fault surface. The movement may vary from less than a centimeter to many kilometers. Faults created by compressional tectonic forces are likely to be impermeable, but can be bordered by strongly fractured and/or jointed rock that may be permeable. Faults created by extensional tectonic forces may themselves be permeable, and down-dropped blocks overlying inclined faults are likely to contain permeable joints. Faults very rarely show a clean and uncomplicated break. The rocks normally exhibit folding, fracturing, crushing, and grinding. Sometimes the walls exhibit polished and smoothly striated surfaces called slickensides. The rocks on the opposite sides of the fault surface may occasionally be broken into angular fragments referred to as “fault breccia.” In addition to these mechanical effects, faults may result in channels for transmitting water or may be impermeable and form groundwater barriers. Recognition of faults is of great importance because faults represent zones of weakness in the crust of the Earth, and the presence of these zones affect the engineering properties of a site, including seismological considerations, excavation, tunnel support, dam stability, and leakage problems. It is prudent to “stitch-grout” fault zones in dam foundations, crisscrossing them at various depths with fans of grout holes.

d. Joints. Joints are almost universally present and are of considerable engineering importance for that reason. Joints offer channels for groundwater conveyance. Joints may also exert an important influence on weathering and excavation characteristics.

(1) Rocks formed at great depth, such as granitic and metamorphic rocks, tend to have strongly developed orthogonal joint systems as a result of stress release as erosion strips away thousands of feet of overburden. Weathering may proceed to substantial depths along steeply dipping joints, but joint openings below the zone of weathering may take significant volumes of grout.

(2) Sheet jointing, a form of load release jointing that parallels the ground surface, is particularly common in granitic rock. The orientation of these joints, together with the extensive grout travel that they may make possible, places a substantial constraint on the injection pressure that may be applied safely. It may even be prudent to consider installing grouted dowels or rock bolts to increase the factor of safety against hydraulic jacking. Where sheet joints intersect the face of a foundation excavation, pneumatically applied concrete or extensive caulking may be required to prevent or restrain surface leakage.

(3) Elastic rebound tends to occur in compaction shales when overburden is removed, whether by erosion or by construction excavation. This rebound commonly creates open joints in sandstones that are interbedded with the compaction shale, creating a highly permeable condition that may require an elaborate system of curtain grouting.

(4) Rapid erosional downcutting in steep-sided canyons and valleys sometimes leads to the development of deep, wide fracture systems and the subsequent separation and spreading of ridges on one or both sides of the valley. This phenomenon has been known to develop in

granitic rocks, adversely affecting the stability of engineering structures, but it more readily occurs in layered rocks such as limestones and sandstones, where pre-existing steeply dipping joints propagate downward and pull apart. The resulting rock blocks can be quite large and composed of competent material. However, they are often fully detached blocks bounded by wide fractures, many of which may be infilled with unconsolidated materials. Grouting of wide fractures surrounding fully detached blocks is unlikely to be an effective mitigation measure, and it is frequently necessary to remove the rock blocks by excavation until fractures are sufficiently tight for grouting. These features can easily be missed or misunderstood during exploration, particularly when investigation methods comprise only borings. They will, however, be readily apparent as excavation proceeds. When fully detached blocks are unexpectedly encountered on a naturally steep abutment, effective solutions can be problematic.

(5) Gravity slump features, which are a consequence of oversteepening of slopes in weak rock or soil masses as a result of erosion or excavation, sometimes require extensive grouting where conditions require that they be left in place in a dam foundation. They include so-called "gravity-slip faults," which occur along undercut joints or bedding planes. They also include landslides, which, if recognized before construction, are presumably avoided or removed during stripping of the foundation. However, landslides of significant size sometimes develop during the course of stripping operations for dams on weak rock foundations and must be left in place.

(6) Measurement of joint orientations (strike and dip) and frequencies is a critical step for appropriately designing grout hole alignment. Traditional surface geological methods such as mapping outcrops and detailed lines, provided that representative outcrops are located on or adjacent to the site and that wide variation in joint orientations is not expected, will provide the designer with this information. Stereonet analyses provide a convenient means of displaying the orientation of multiple joint sets. The orientation of the joints with respect to the grout curtain alignment must be considered. True dip is the measurement of joint inclination normal to the strike of the joint and is the value reported and/or displayed on the stereonet. Apparent dip, however, is the dip of the joint relative to the strike of the grout curtain. Apparent dip is always a shallower angle than true dip. In all likelihood, not all of the formations to be grouted will be exposed at the ground surface for mapping. Recent advances in down-hole imagery equipment and associated processing software now allow for mapping of joints in subsurface strata. Typically, these systems operate by recording high-resolution images of the borehole sidewall. Some post-processing software automatically detects likely joints and tags them for orientation calculation. The sidewall images allow operators to manually interpret and tag the joints. Regardless of the selection method, the strike and dip of the tagged joints are then calculated, considering the inclination of the borehole and any measured borehole deviations. The result is a comprehensive package of joint orientation data spanning the entire surveyed length of the boring. The use of these systems also aids in the field identification of geologic contacts in destructively drilled holes, and of the presence of joint infilling materials.

(7) Some extrusive igneous rocks present complex open networks that can be challenging. Contact zones between flows can be brecciated, and shrinkage during cooling produces highly interconnected open jointing with variable orientation. Quantities of grout accepted by the formation (grout takes) under these conditions can be high and hard to predict.

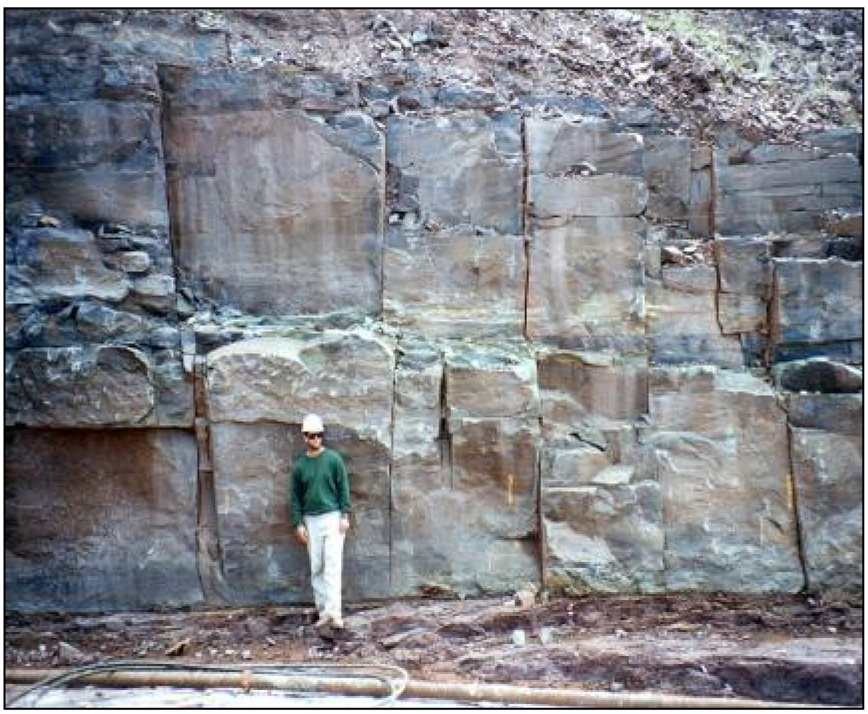
e. Grouting Considerations. Since many rock types have low primary permeability, but relatively high fracture and joint permeability, the importance of grouting the structural discontinuities is apparent. The type of structural feature (e.g., fault, fold, or joint) will dictate to a large part the type and extent of excavation treatment and the grouting methods. The spacing and nature of the fractures (e.g., open, weathered, or solutioned) influence the type of grout treatment selected, such as consolidation grouting and curtain grouting. The selection of a single-line or multiple-line curtain and the grout hole spacing are also affected. The orientation (dip and strike) of these features in relation to a structure influences the planned angle and direction of the grout holes and the drain holes. The spatial variations in the permeability of the fractures affects the depth of a grout curtain. The grout holes should intersect all the features, and each inclined or vertical feature should ideally be intersected by several holes at different depths. Faults may be gouge filled and impermeable, thereby forming a barrier, or they may be open and carry groundwater. Joints may be filled or open, may have weathered or non-weathered faces, and may intersect and be connected over a wide area. The condition of the joints will affect the drilling, cleaning, pressure testing, and grouting of the hole.

f. Engineering Grouting Programs in the Field. It is imperative that grouting projects be designed and executed to allow for changes based on field conditions. Advances in drilling and grouting technology in recent years make "engineered grouting" technically achievable, assuming that all phases of the work are performed, supervised, and reviewed by well-qualified personnel. However, accomplishing this result requires that the site's geologic conditions be thoroughly known and considered in preparing the site-specific grouting specifications. Additionally, the site's geological conditions must be continuously interpreted during the course of the work, and the grouting program must be modified on a timely basis whenever necessary to treat those conditions that differ from those originally anticipated.

g. Rock Structure and Rock Jointing Examples. Figures 4-6 to 4-16 show examples of a variety of rock structure and rock jointing types. These figures are included to allow visualization of the different orientations and spacings of grout holes that would be appropriate to accomplish effective grouting.

(Courtesy of Gannett Fleming)

Figure 4-6. Differential bed thicknesses, bed orientations, and joint spacings in sandstone, near Slate Run, PA.



(Courtesy of Gannett Fleming)

Figure 4-7. Thick sandstone unit with large, widely spaced vertical fractures terminating at mudstone units located above and below, Penn Forest Dam, PA.

(Courtesy of Gannett Fleming.)

Figure 4-8. Smooth, vertical fracture in massive sandstone with extensive continuity crossing a dam foundation, overlain by mudstone unit, Penn Forest Dam, PA.

(Courtesy of Gannett Fleming)

Figure 4-9. Rock outcrop adjacent to a dam abutment, Sierra Nevada Mountains, CA.

(Courtesy of Gannett Fleming)

Figure 4-10. Salt Springs Dam, CA.

(Courtesy of Gannett Fleming)

Figure 4-11. Sheeted and blocky jointing in Granite, CA.

(Courtesy of Gannett Fleming)

Figure 4-12. Abutment area of Olivenhain Dam, CA.

(Courtesy of Gannett Fleming)

Figure 4-13. Dolomite with a solution feature discontinuity at a quarry in PA.

(Courtesy of Gannett Fleming)

Figure 4-14. Weathered limestone exposure, Mississinewa Dam, IN.

(Courtesy of Gannett Fleming)

Figure 4-15. Limestone with solution and erosion features along the Wabash River, IN.

(Courtesy of Gannett Fleming)

Figure 4-16. Limestone with solution features, varied bed thickness, and vertical joints at a highway rock cut in south central Kentucky.

4-5. Rock Types.

a. General. The differing properties of various rock types, by nature of their origin, lithology, and structure, will influence the grouting conditions at a particular site. Thorough knowledge of the rock types at the site, and their geologic history, is therefore necessary for the design and treatment of the foundation. The geologic features affecting grouting may be grouped into two categories, primary and secondary features. Primary features include those physical parameters or characteristics that are related to the formational or depositional mechanism. Examples of primary features contributing to permeability and to a possible need for grouting include the intergranular interstices in alluvial deposits and in poorly cemented sandstones, lava tubes, cooling cracks in lava and shallow intrusive rocks (e.g., dikes and sills), and depositional contact zones. Secondary features are the result of geologic forces that have acted on a soil or rock mass subsequent to its deposition or formation. Examples of secondary features contributing to rock mass permeability include solution cavities, faults, shears, folds, and stress release joints. The exploration and grouting programs must be adapted to the site geologic conditions. An understanding of the site geologic conditions and the conditions of width, distribution, and interconnection of potential seepage or leakage paths should be considered an essential part of designing a grouting program that will have the optimum opportunity to identify and intersect potentially open geologic discontinuities and treat them effectively. An understanding of the site geologic conditions and their engineering implications also may be an essential part of a process in which it is decided that some remedial treatment other than, or in addition to, grouting may be appropriate. Different rocks with the same general fracture permeability and void characteristics can be loosely grouped together. As a broad generality, examples of some of the more common rock types, together with those general characteristics that could influence required foundation treatment, are listed below.

b. Crystalline Rocks. “Crystalline rock” is an inexact, but convenient term that identifies igneous and metamorphic as opposed to sedimentary rocks.

(1) Intrusive igneous rocks include granites, syenites, diorites, and gabbros. Some features commonly found in these rocks are sheet jointing, shear zones, dikes, and sills.

(2) Jointing in three directions is characteristic of intrusives. One set is usually nearly horizontal (sheet or uplift jointing), and the other two are near-vertical and generally normal to each other. The spacing of sheet joints near the surface is frequently close and tends to increase with depth.

(3) Grout take normally occurs in the joints and the fractures, and the volume depends on the size and continuity of the openings along the fractures. Certain metamorphic rocks such as gneisses react in a manner similar to that of the granites. Grout takes in schists and slates depend on the presence and characteristics of associated jointing or fine fracturing. Most quartzites are highly fractured and would readily accept grout. Marble is a crystalline rock, but should also be considered in the category of karstic formations since solution cavities should be anticipated.

c. Volcanics. Volcanics generally include the extrusive igneous rocks. Felsites, a group of very dense, fine-grained rocks, are extrusive and near-surface equivalents of granites, syenites, and other related crystalline rocks. In addition to granite-like jointing, they may exhibit columnar structure. Basalts are a group of very dense, dark, igneous rocks. The jointing may be platy or columnar. Basalts in many flows commonly exhibit columns with three to six sides. Pumice and scoria are often associated with basalts. Pyroclastics, such as agglomerates and tuffs, are materials formed by explosive volcanic activity and consist of fragments torn loose by such explosions, or they are formed from deposits of wind-borne ash. Large-scale engineering operations in pyroclastic rocks are generally difficult. Volcanics require extensive examination before engineering characteristics can be determined and will usually require special treatment. The presence of columnar jointing in lava flows tends to lower the strength of the mass as a whole, and extensive grouting can be expected. Permeability may be great in lava flows due to the extensive jointing that normally is present and also due to the presence of piped vesicles and gas cavities. These features may transmit copious flows of water. In some cases, however, the joints are tight and/or filled, and the rock mass may have a very low permeability. Each case must be evaluated individually to determine the need for and effectiveness of grouting.

(1) Volcanic and pyroclastic rocks can present substantial problems to the dam designer. They can be extremely complex in some areas, where the sequence of eruptions can include extremely fluid lavas, viscous lavas, solid ejecta of various sizes, and “glowing avalanches.” Lava tubes are commonly associated with the more fluid lavas, while the viscous lavas can produce highly porous and permeable clinker-like deposits. Both of these types of lava develop cooling cracks in various orientations, the most common and widest of which are perpendicular to the upper and lower surfaces. These cracks completely penetrate thin flows, but the centers of thick flows (such as are commonly found in plateau basalts) tend to be dense and impermeable. The most common, or widespread, type of solid airborne ejecta is pumice, which is the major

ingredient of air-fall tuff. Although highly porous, pumice is not necessarily highly permeable. However, due to its low density, it can be subject to piping if in the unweathered state. It decomposes to expansive clay, which may form an impervious barrier between flows. Glowing avalanches, which are extremely hot gaseous products of some types of explosive eruption, form welded tuffs. Having been formed from a hot gas, welded tuffs are even more likely than lavas to contain large, wide-open cooling cracks. However, if hot volcanic ejecta (especially hot gases) come in contact with snow-covered slopes, relatively impervious mudflow deposits are formed. It is not uncommon for these various types of volcanic and pyroclastic deposits to be interspersed with one another, creating challenges for geologic interpretation and engineering ingenuity. Moreover, due to the intermittent nature of volcanic activity, these deposits commonly are interspersed with alluvial (interflow) deposits.

(2) The various conditions described above can produce high permeability, sometimes to the point of bringing into question the technical and economic feasibility of constructing a water storage reservoir. Some reservoirs in basaltic terrain, such as Jerome Reservoir in Idaho, have failed to hold water. Therefore, siting studies for dams to be constructed in volcanic terrain should include geohydrologic investigations to verify that the reservoirs can be made to hold water. These investigations should include studies of the position and flow direction of the groundwater surface in the reservoir rims. Assuming that the conditions appear to be acceptable, extensive grouting to seal the dam abutments, foundations, and reservoir rims still may be required, but the likelihood that the grouting program will be both efficient and effective will be greatly enhanced by the knowledge gained from the exploratory program.

d. Soluble Rocks. Limestone, dolomite, gypsum, anhydrite, and halite are included in this group. The principal discontinuity in this rock group is solubility in varying degrees, which can ultimately cause high mass permeability, slump, collapse, and sinks, resulting in karst topography. Soluble rocks could be placed in two broad categories: those that are only slightly soluble, but that dissolve over geologic time, and those that have the potential to rapidly dissolve if exposed to reservoir conditions, including seepage flow. Limestone, dolomite, and calcareous dikes fall in the first category; anhydrite, gypsum, halite salt, and other evaporite minerals fall in the second category.

(1) Limestones and dolomites are the most widespread of the soluble sediments. These rocks may be vuggy and may display a wide range of permeability as a unit. Limestone and dolomite are generally jointed and usually exhibit two or three distinct sets of jointing. Solution features are frequently well developed along bedding planes, joints, and contacts with other rock types. Joints and cavities may be either filled or open, and the size may vary greatly. Depending on the extent of jointing and cavities, extensive treatment and grouting can be anticipated. The problems presented by cavernous limestones are well known. In general, the remedial treatment involves constructing a grout curtain or other form of cutoff to an underlying impervious formation or stratum. However, some limestone sites may not be underlain by impervious rock at a depth that can reasonably be reached by a grout curtain. Therefore, consideration may have to be given to whether acceptable results can be obtained by merely diverting the underflow to some selected depth beneath the dam foundation. It should be recognized that not all limestones are cavernous. Nonetheless, relatively minor solution activity along joint planes and bedding planes can lead to a need for an extensive grouting program. Moreover, the extent of fracturing

may dictate a need for extensive grouting, and the extent of weathering can make that grouting difficult to perform successfully.

(2) Anhydrite is pure calcium sulfate, whereas gypsum is the hydrated form. Both are soft and fairly soluble in water. Both may be jointed and have a varying number and size of solution cavities. The cavities often are filled with clay or other reworked material. Grout takes depend on the presence and characteristics of the joints and cavities. Gypsum and its parent mineral, anhydrite, are commonly associated with both shales and limestones. The failure of Quail Creek Dike in Utah pointed out that the presence of gypsum or anhydrite in dam foundations can present problems that can be difficult or impossible to mitigate by grouting, especially if the foundation rock is strongly jointed or fractured. Mosul Dam in Iraq is located on a gypsum/anhydrite-rich foundation and has been continuously grouted since first filling. The results of studies of the potential for dissolution of gypsum and anhydrite beds in dam foundations led one engineering organization to conclude that grouting can be effective in reducing seepage to a safe level in gypsum beds, but that a positive cutoff (such as plastic concrete) should be used in anhydrite. That conclusion was based on the results of calculations of solution rates for those two minerals, assuming fractures of various widths. The limiting fracture width for dissolution to occur in anhydrite was found to be 0.1 mm, which is about half the width of fissures that could be penetrated by grouts formulated with cements that were generally available at that time (ultrafine cements are capable of penetrating fissures 0.1 mm in width and, being pozzolanic, should be resistant to sulfate attack).

(3) Halite (rock salt) is soft and soluble in water. The extremely soluble halite is not found in outcrops, but may be found at depth. The principal engineering significance of halite is the effect its presence or proximity may have on the proposed project, such as solutioning and subsidence, in addition to its effects on groundwater.

(4) Clastic Sedimentary Rocks. Conglomerates, sandstones, siltstones, and shales are the principal types of clastic sedimentary rocks. The physical properties of sandstone, siltstone, and conglomerate depend on the degree and type of cementation. These coarser clastics may be tight and impermeable, or they may be sufficiently porous and permeable to need treatment. Jointing would be the main concern in the treatment of impermeable clastics. The finer clastics, such as claystones and shales, are made up of clay minerals, various oxides, silica, fine particles of ordinary minerals, and some amount of colloidal and organic materials. These clastics may contain a great amount of water. Two shale types are cementation shale and compaction shale. Compaction shales usually contain no joints capable of being grouted. Cemented shales are more resistant to change and have engineering properties superior to those of the compaction type. Cemented to slightly metamorphosed shales are sufficiently brittle to react to structural changes and develop joints similar to those in sandstone. Some pervious, poorly cemented sandstones may allow significant volumes of water to pass through their intergranular interstices, potentially contributing to excessive pore-water pressure. Whether this seepage can be mitigated by cement grouting is problematic. The greatest potential hazard presented by poorly cemented sandstones is internal erosion (piping). Thorough grouting of open foundation discontinuities is essential to prevent seepage flows from reaching potentially erosive velocities. However, the poor cementation might contribute to the joints being loosely filled with sand, and also to fractures and shear zones being made up principally of pipable loose sand. Chemical grouts may be applicable in such cases, but other control methods may be required.

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CHAPTER 5

Geohydrology and Flow Modeling

5-1. Introduction. Understanding of the site geohydrology, both qualitatively and quantitatively, is central and essential for grouting applications where the purpose of grouting is to alter flow rates, flow paths, flow velocities, or pressure distributions. Geohydrologic models, regardless of complexity, provide an invaluable tool for rationally evaluating design alternatives, optimizing grouting configurations for best technical value and cost efficiency, predicting performance of the completed work, and designing field verification programs to be used in execution of the work. Models can also be used to guide the sequence of how the work should be performed, which can be an important element for remedial work on existing dams.

Historically, at a significant number of U.S. dams that were grouted, the results of water tests for the hydraulic conductivity of the foundation materials were reported in units of ft^3/min. The flow volume was used as a guide for determining the need for grouting and the starting grout mixes. In reviewing grout records from previous construction, be aware that this is a flow estimate and not a hydraulic conductivity.

5-2. Hydraulic Conductivity Units.

a. Definitions.

(1) Intrinsic Permeability. Intrinsic permeability is a fundamental material property providing a measure of the ability of a material to transmit fluids. Intrinsic permeability values are independent of the type, density, and viscosity of the fluid. The intrinsic permeability is defined as:

$$k = Cd^2$$

where:

k = intrinsic permeability

C = dimensionless constant related to the configuration of the flow paths

d = average or effective pore diameter.

The units, therefore, for intrinsic permeability k are length squared (e.g., cm^2). Common units for intrinsic permeability include the Darcy (d) or millidarcy (mD), where 1 Darcy equals approximately 10^{-12} m^2. In most cases, the intrinsic permeability is determined by measuring hydraulic conductivity using Darcy's Law.

(2) Hydraulic Conductivity. The hydraulic conductivity, which is often simply referred to as "permeability," is represented by the symbol K. Hydraulic conductivity is defined as the discharge velocity of water flowing through a unit area under a unit hydraulic head. Unlike intrinsic permeability, which is a fundamental material property independent of the fluid, the hydraulic conductivity depends on the intrinsic permeability, the degree of saturation, the specific weight of water, and the dynamic viscosity of water. The units for hydraulic

conductivity are length/time (L/T), i.e., cm/s or ft/min. Hydraulic conductivity is related to the intrinsic permeability by the following equation:

$$K = k\,\gamma\,/\,\mu$$

where:

K	=	Hydraulic conductivity (units of L/T)
γ	=	Specific weight of water (units of $M/L^2\,T^2$) (i.e., N/m^3)
μ	=	Dynamic viscosity of water (units of M/LT) (i.e., kg/m × s).

Since the dynamic viscosity of water is temperature dependent, the value for K also depends on temperature. For nearly all normal types of engineering problems, dynamic viscosity and specific weight values are selected on the basis of water being at 50–60ºF.

b. Hydraulic Conductivity Units. A wide variety of hydraulic conductivity units are used in engineering and geohydrology practice, depending on common convention for various types of applications and also on preferences of individuals using the coefficient. Regardless of the units of preference, hydraulic conductivity can always be expressed directly in the form of L/T or, in some cases, units that can be reduced to the form of L/T. Common units in use include cm/s, ft/min, ft/day, and m/s. It is not uncommon for different units to be used for different purposes on the same problem. For example, a profile might be depicted in terms of hydraulic conductivity units of cm/s, while model calculations might be based on units of ft/min. The most important issue is that hydraulic conductivity units that are selected for use must be compatible with other units used in flow calculations. Less common and less recognizable units include gallons per day per square foot (gpd/ft^2), which can be reduced to ft/min or ft/day through simple conversions. For grouting on USACE projects, ft/min is the recommended standard unit for hydraulic conductivity.

c. Lugeon Unit. A special unit of hydraulic conductivity that is in widespread use in rock grouting is the Lugeon unit. The Lugeon test (named after Swiss geologist M. Lugeon) is a water injection test. One Lugeon (Lu) unit is equal to 1 L of water per minute injected into 1 m of borehole at an injection pressure of 10 bars. It is extremely rare to set up the test to duplicate those parameters in the field. In practice, the Lugeon value for a particular portion of the foundation is found by isolating a known length of borehole by packers, measuring the rate of water injection under a selected pressure, and then calculating the Lugeon value based on those parameters. For USACE projects, it is common to use a water testing pressure equal to the proposed grouting pressure. As with any test of this type, the water being injected will travel in any direction that is permitted by the intersected fractures. The test does not provide information about the size or spacing of fractures, since a single large fracture within the test interval could produce the same Lugeon value as numerous closely spaced, fine fractures. As originally applied, Lugeon values were not corrected for equipment head losses, and not necessarily corrected for the depth to static groundwater level. However, current practice includes both of those corrections for calculating the effective pressure applied during the test to improve the accuracy of the results. The dynamic head loss characteristics are, in fact, essential for determining whether water injection at any combination of pressure and flow is being limited by the fracture system or by the injection equipment. Lugeon values are normally reported as whole numbers only. An inherent advantage of determining the hydraulic conductivity in terms of the

Lugeon unit is that general guidelines exist that relate Lugeon values to both the need for grouting and the intensity of grouting required (Houlsby 1990).

d. Conversion Factors. Table 5-1 lists conversion factors for hydraulic conductivity units. The factor for converting between Lugeon units and hydraulic conductivity units is from Houlsby (1990). All conversions between Lugeon units and hydraulic conductivity units are approximate, a fact pointed out by Houlsby (1990) and other authors. Kutzner (1996) provides a discussion of the differences between the physical conditions for Lugeon tests performed in rock and hydraulic conductivity tests performed in porous media. However, the Lugeon/conductivity conversion factors listed in Table 5-1 have been generally accepted for use in practice within the grouting community with an understanding of their approximate nature.

Table 5-1. Hydraulic conductivity unit conversion factors.

Starting Unit	Desired Unit	Multiplication Factor
cm/s	ft/min	1.97
cm/s	ft/day	2834.6
cm/s	Lugeon	76923
ft/min	cm/s	0.508
ft/min	ft/day	1440
ft/min	Lugeon	39047
ft/day	cm/s	3.53×10^{-4}
ft/day	ft/min	6.94×10^{-4}
ft/day	Lugeon	27.1
Lugeon	cm/s	1.3×10^{-5}
Lugeon	ft/min	2.6×10^{-5}
Lugeon	ft/day	3.7×10^{-2}

e. Apparent Lugeon value. Special discussion is warranted of another unit that is now in common use in grouting and that has been called the Apparent Lugeon value. Hydraulic conductivity values are determined for steady-state flow conditions with water as the fluid. However, since the intrinsic permeability of the material is a constant regardless of fluid, when the fluid is changed to a grout (provided that it is a stable suspension with constant properties), it follows that during grouting an equivalent water hydraulic conductivity can be calculated if the following ratio of specific weights and dynamic viscosities is present:

$$\gamma_g \mu_w / \gamma_w \mu_g = 1$$

For normal field operations, Naudts (2003) proposed that an initial Apparent Lugeon value be calculated immediately after the start of grout injection based on:

Apparent Water Lugeon Value = Lugeon Value with Grout × Ratio of Marsh Funnel Viscosities or

$$\text{Apparent } Lu_w = Lu_g \times V_{\text{Marsh of Grout}} / V_{\text{Marsh of Water}}.$$

Naudts further proposed that an Apparent Lugeon value calculated in the field could only be valid when compared with a known Lugeon value determined with water. In other words, when the initial Apparent Lugeon value is calculated to be approximately equal to the actual Lugeon value determined by water testing, then it can be assumed that the grout is initially penetrating the fractures with the same ease as water. When the ratio of Apparent Lugeon value to actual Lugeon value is less than 1, then it indicates that grout is not penetrating all of the fractures and that the Apparent Lugeon value is not accurate. The practical and useful aspects of this are:

(1) When water Lugeon values and calculated Apparent Lugeon values are essentially identical, it is a very strong indicator that a proper starting mix for grout has been formulated and is being used.

(2) It is best practice to water pressure test each stage in all primary holes at a minimum. Depending on the purpose of the grouting program, when a large number of confirming data has been obtained under field conditions and the data show a consistent, nearly identical result for water pressure testing Lugeon values and calculated Apparent Lugeon values, the data can be used to support a reduction in the frequency of water pressure testing, and the initial permeability can be assumed to be equal to the initial Apparent Lugeon value.

(3) Since Apparent Lugeon values are calculated with both flow and pressure being applied, at any point in time the Apparent Lugeon value is constant. At any given moment, changing the pressure will increase the flow rate, but the Apparent Lugeon value will remain unchanged. This is a distinct advantage in monitoring progression toward refusal, and it is also highly valuable in monitoring dilation or uplift, since only a physical change of that type would suddenly alter the Apparent Lugeon value. It is also of high value in detecting premature refusal, since a dramatic decrease in the Apparent Lugeon value would indicate abrupt choking within the fractures.

(4) The Apparent Lugeon value should not be interpreted as indicating that a particular mass of rock has achieved a certain final permeability value. It is providing information about entrance of grout into the fractures in the immediate vicinity of the grout hole. It does not provide information about the extent of grout travel nor does it necessarily provide information on the completeness of filling. For example, a hole could be brought to rather rapid refusal with a thick grout, and it would show a very low final Apparent Lugeon value. However, there may have been very little penetration into the rock mass, and finer fractures may not have been grouted at all.

5-3. Geohydrologic Flow Systems.

a. General Nature of Flow Systems. In the most general case, flow systems within any physical region of interest can be extremely complex. There will be constant head boundaries subject to time fluctuation in levels, and the location and the values for constant head boundaries might differ for different geologic units. There can be flow both within soil materials, which are a porous media, and within rock materials, which can have both flow within the rock matrix, and fracture flow. There may be surficial recharge zones that periodically introduce water and alter the flow regime. There can be withdrawal points (e.g., wells) within the region of interest. There can be zones of confined, pressurized flow, zones with free water surfaces, and zones of partially saturated flow. Flow is often three dimensional. The various soil and rock media may not be homogeneous, and they may be subject to consolidation or compression under pressure changes. There may be temperature differentials, which can change the dynamic viscosity and density of water. Depending on flow velocities, flow can be either laminar or turbulent. Alteration of the flow regime due to a change in conditions also involves time dependency, which might or might not be an issue for particular types of problems. Simultaneous solution of all these factors would provide a comprehensive description of the complete flow system, including flow rates, directions, velocities, and their time dependencies. Fortunately, for most grouting applications, there are many simplifications that can provide quite adequate solutions and yet are relatively easy to solve.

b. Flow System Simplifications for Analysis of Grouting Applications. Normal simplifications that can be used for flow system analysis in the design of grouting include:

(1) The flow medium is homogeneous.

(2) Beneath the free water surface, the voids are completely full of water.

(3) The medium and the water are incompressible.

(4) Flow is laminar and Darcy's Law applies.

(5) Steady-state flow conditions exist.

(6) Steady-state flow is represented by the Laplace Equation. In two dimensions, the solution of the Laplace Equation can be most easily visualized as being represented by two families of curves that intersect at right angles to form a pattern of "square" figures known as a flow net. One set of lines are the flow lines or streamlines, and the other set represents equipotentials. The flow lines represent the directional paths of flow, and the equipotential lines are lines of equal energy or head.

c. Darcy's Law. Darcy's Law, empirically derived by Henri Darcy in 1856, is an equation that states that the amount of flow discharging through a given area is proportional to the cross-sectional area of flow, the hydraulic head gradient, and the hydraulic conductivity. Its most familiar form is:

$$Q = KiA$$

where:

Q = flow rate

K = hydraulic conductivity

i = flow gradient (head differential divided by length of flow path)

A = gross cross-sectional flow area including both voids and solids.

(1) Since flow rate Q = velocity $v \times$ cross-sectional area A, the equation can also be stated as $v = Ki$, in which v is defined as the Darcy discharge velocity, which differs from the actual seepage velocity, which is the true flow velocity within the pores in which flow is occurring. The actual pore velocity is obtained by dividing the Darcy discharge velocity by the effective porosity, which is the ratio of the actual volume of pore spaces through which water is flowing to the total volume. Usually the effective porosity is somewhat less than the total porosity.

(2) Darcy's Law is valid only for slow, viscous flow (laminar flow). Fortunately, most groundwater flow cases fall in this category. Experimentally, it has been shown that any groundwater flow with a Reynolds number of less than 1 is clearly laminar and that flow regimes with values of Reynolds numbers of up to 10 are generally suitable for application of Darcy's Law. The formula for the Reynolds number for porous media flow is:

$$Re = \rho v d_{30} / \mu$$

where:

Re	=	Reynolds number.
ρ	=	density of water.
v	=	Darcy velocity (not the true seepage velocity).
d_{30}	=	representative grain diameter of media (often taken as the 30% passing size from sieve analysis).
μ	=	dynamic viscosity of the water.

5-4. Flow in Saturated Porous Media.

a. Applicability of Darcy's Law. Darcy's Law, which originated from the study of water flow through vertical pipes filled with sand, is applicable for most soil types, and it can be applied directly in its original equation form when the geometry of the problem is sufficiently simple. For complex boundary conditions and/or complex configurations of differing hydraulic conductivity, other forms of modeling such as flow net analyses or numerical modeling are required. Laminar flow conditions necessary for application of Darcy's Law will normally be met by all soils with grain sizes smaller than clean gravels. When water is flowing in highly permeable media, such as clean coarse gravel, cobble, or rock-fill zones, the flow condition can vary from laminar to fully turbulent, depending on the hydraulic gradients and velocities. Significant turbulent effects have been found to occur at gradients less than 0.1 for aggregate with a nominal particle size of 9.5 mm (3/8 in.) or larger. As flow makes a transition into the semi-turbulent to turbulent regime, the increase in flow velocity and discharge, relative to an increase in hydraulic gradient, is no longer a linear relationship. Semi-turbulent and turbulent flows will result in velocity and flow rates that increase at a smaller rate than the gradient, and significant flow reduction may occur.

b. Corrections for Turbulent Flow Conditions. There are several methods for approximating the effects of turbulence in clean coarse gravels, cobble, and rock-fill materials. There are empirical formulas available for estimating flow velocities through various types of coarse materials. Another approach is to test coarse materials at the actual gradient that will occur in the prototype; this largely eliminates the errors from turbulence because the measured hydraulic conductivity, though not a true Darcy value, already includes the turbulence effects. A variation on this procedure is to perform hydraulic conductivity tests over a wide range of gradients ranging from laminar to turbulent and develop plots of “effective permeability” vs. gradient.

c. Hydraulic Conductivity for Porous Media. Provided that laminar flow conditions are present, the hydraulic conductivity for soil materials is a constant value. That value is a function of several interacting factors, including grain size distributions, particle arrangement, structure, and density. The hydraulic conductivity is extremely sensitive to the quantity, character, and distribution of the finest soil fractions. As a rule, hydraulic conductivity should be determined directly by laboratory and/or field tests, and design values should not be selected from individual properties such as grain size. General relationships that are available are of use in understanding the range of hydraulic conductivity that can be expected, but they should only be used as a general guide. Cedergren (1989), which is an excellent reference for understanding the nature of hydraulic conductivity, describes many types of test arrangements for determining its value, and provides typical ranges of hydraulic conductivities for a variety of soil materials. Table 5-2 lists some typical ranges of hydraulic conductivity for a variety of materials. It is the extremely broad range of hydraulic conductivity values that soils can exhibit that make it essential that careful testing be performed for selecting a design value.

Table 5-2. Typical ranges of hydraulic conductivities for soils.

Soil Type	Typical Range of Hydraulic Conductivity (cm/s)
Clean gravel	1 to 100
Clean sands, clean sand, and gravel mixtures	1×10^{-3} to 1
Fine sands; silts; mixtures of sand, silt, and clay; silty sand; clayey sand; sandy clay	1×10^{-7} to 1×10^{-3}

5-5. Flow in Saturated Fractured Media.

a. Nature of Fractured Media. A discrete mass of fractured media, which includes all rock materials and which might also include certain soil or soil-like materials containing discrete fissures, has two components: (1) matrix blocks and (2) fractures or fissures. The matrix blocks are typically characterized by a high porosity and low hydraulic conductivity. Conversely, the fractures or fissures have a very low porosity expressed as a percentage of the total block, but have extremely high hydraulic conductivity. As examples, the matrix portion of the discrete mass might have a porosity in the range of 2 to 30%, whereas the fracture portion of the mass might typically have a porosity of only 0.001 to 0.1% of the mass. Conversely, a non-fractured rock matrix might have a hydraulic conductivity in the range of 0 to 10^{-4} cm/s, while an individual fracture having a size of as small as 0.1 mm can have a permeability as high as 0.85 cm/s.

b. Dominance of Fracture Flow. To assist with a practical understanding of the magnitude of importance of flow through even very small open fissures or channels, Cedergren (1989) compared flow through a variety of soil types vs. the size of an equivalent open flow path. For example, the flow through 929 cm^2 (1 ft^2) of clean, 2.5- to 2.8-cm (1- to 1.5-in.) gravel is comparable to the flow that could be accommodated through a single, smooth, open pathway having a diameter of only 0.25 cm (0.1 in.). Similarly, flow through a unit area of coarse sand could be accommodated by a pathway with a diameter of only 0.25 mm (0.01 in.). Therefore, for the practical solution of most engineering applications, fracture flow is such an overwhelming component that flow through the matrix portion of the block can be neglected. Exceptions to this rule might be in environmental applications of grouting, where the long-term contaminant transport requires consideration of storage and release of contaminants from the matrix materials, and sometimes where rock is highly porous, as in certain sandstones.

c. Discrete Fracture Flow Theory.

(1) Parallel Plate Flow and the Cubic Law Equation. Though rarely applied directly in the design of grouting for engineering applications, it is still very useful to understand the implications of discrete fracture flow theory. The flow through a single fracture, when idealized as slow, viscous flow between smooth, parallel plates, results in this Cubic Law equation (EM 1110-2-1901):

$$Q = W\gamma b^3 i \,/\, 12\mu$$

where:

Q = rate of flow through a single fracture.

W = fracture access dimension as applicable for the rock block (i.e., vertical, horizontal, or diagonal dimension on the entrance face of rock block).

γ = unit weight of water.

b = aperture width.

i = flow gradient (head difference divided by length of flow path between parallel plates).

μ = dynamic viscosity of water.

(2) For the total flow through a rock block containing N identical fractures, the total flow rate is given by:

$$Q = NW\gamma b^3 i \,/\, 12\mu.$$

(3) Tests have shown that the Cubic Law is valid provided the Reynolds Number is in the range of 1 to 10, with the Reynolds number calculated as:

$$Re = V D_h \,/\, \upsilon$$

where:

Re = Reynolds number.

V = Average velocity.

D_h = Hydraulic diameter = $4A/W_P$ (A = cross-sectional area of flow, W_P = wetted perimeter).

υ = Kinematic viscosity.

(4) Figure 5-1 illustrates the implications of the Cubic Law idealization, and shows the unit discharge per foot of fracture exposure for a single ungrouted fracture in a rock block under a flow gradient of 5, which is in the range of gradients that might exist after construction of a grout curtain. For the range of values shown, the Reynolds number is approximately less than 10.

(5) If, for example, a 10-ft × 10-ft block of rock contained vertical fractures with an aperture size of approximately 1.9 mm and a fracture spacing of 2 ft, the total discharge through that single block of rock under a gradient of 5 would be:

Q = 100 gpm per foot of fracture exposure × 10 ft of exposure per fracture × 5 fractures
= 5,000 gpm

(6) To further put this into practical perspective, comparable flows through other fracture sizes would approximately match those listed in Table 5-3.

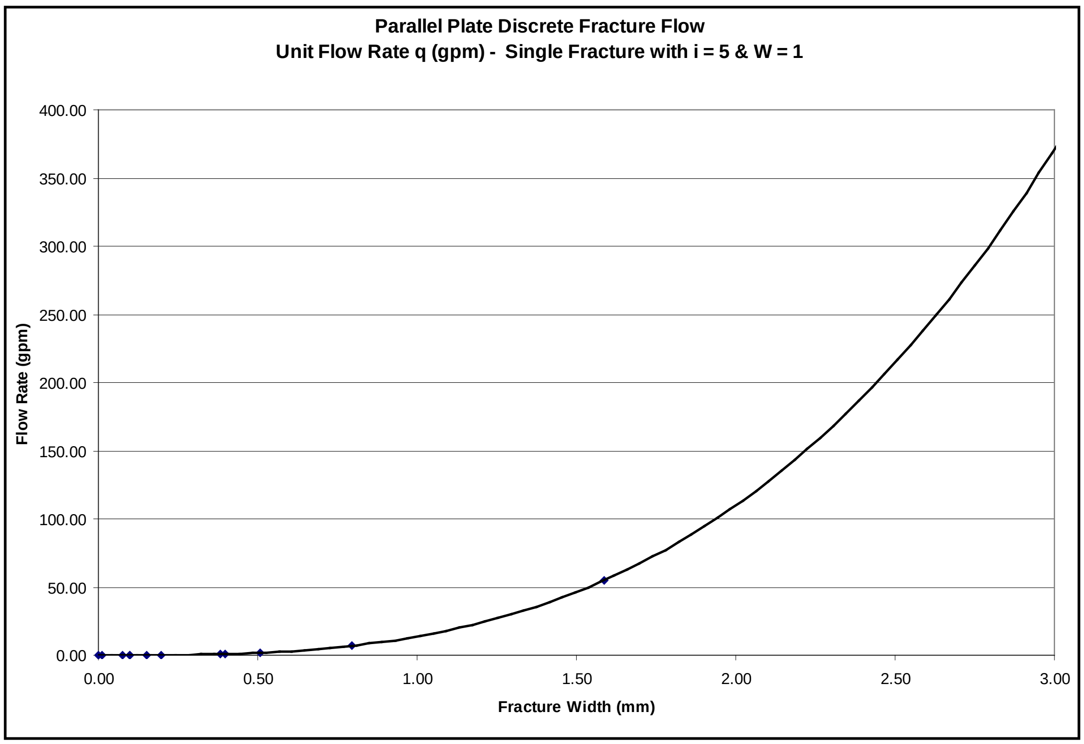


Figure 5-1. Parallel plate model of discrete fracture flow. Unit flow rate per foot of single fracture under gradient i = 5.

Table 5-3. Discrete fracture flow rates.

Fracture Aperture Size (mm)	Comparable Common Object Dimension	Total Flow for Five Fractures in 10 × 10 Rock Block Under i = 5
0.1	Diameter of human hair or flyash particle	Q = 0.01 gpm/ft × 10 ft × 5 fractures = 0.5 gpm
0.4	Common utility knife blade	Q = 0.85 gpm/ft × 10 ft × 5 fractures = 42.5 gpm
0.75	Credit card thickness	Q = 6 gpm/ft × 10 ft × 5 fractures = 300.0 gpm

(7) Implications of Discrete Fracture Flow Theory. The implications of discrete fracture flow theory are numerous and far reaching:

(a) The total flow capacity of even very small fractures is enormous and, in general, has not been fully understood and appreciated in grouting design and field execution.

(b) In the event that individual fractures are not penetrated by grout holes at sufficient intervals to allow overlapping penetration of grout, the unfilled portions of the fractures will tend to behave according to discrete fracture theory. High gradients, which induce high-flow rates through unfilled fractures, can be created by programs that are only partially effective.

(c) The use of excessively thick or viscous grouts, while applicable for very large fractures, may prevent access to the fine fracture systems. For that reason, grouting should always begin with a very thin mix to first induce grout to flow into fine fractures.

(d) Refusal criteria should be extremely low to allow filling of fine fractures.

(8) Direct Application of Discrete Fracture Flow Theory. Direct application of discrete fracture theory to the design of grouting in rock is possible in theory, but it is generally not practical due to numerous deviations from the assumptions in the basic equations. Fractures are, in general, not parallel, uniform, plane, smooth, regularly spaced, or uninterrupted. Additionally, flow may be either laminar or turbulent or some combination of the two. If full and complete information were available for all of the fracture systems present on a site, the system could be modeled as a network of interconnected features similar to a pipe network. However, that often is not practical for most normal engineering applications, and other simpler, more workable solutions are used. Discrete fracture flow theory can, however, be highly useful in analyses of flows through very simple features such as joints or fractures in concrete structures.

d. Equivalent Porous Media Flow Model for Fractured and Fissured Materials. A significant simplification for modeling fractured and fissured materials is to use an equivalent porous media flow model. The advantages of using equivalent porous media models for fractured and fissured materials include: (1) simplicity, (2) compatibility with modeling required for soils that are often present and that may materially impact the solution, and (3) direct use of field-collected hydraulic conductivity data. Provided the scale of the volume being modeled relative to the spacing of fractures is reasonable, it provides a satisfactory modeling solution.

(1) Scale. The scale issue is normally not a problem for typical projects involving hydraulic barriers, such as dams. The fracture networks are usually of sufficient density that, on the project scale, the rock can be idealized as a porous medium. While the model might have significant error in any specific, small slice of the foundation, it tends to be adequate on a bulk basis.

(2) Bulk Hydraulic Conductivity Data. Hydraulic conductivity data for fractured and fissured materials can be obtained through a variety of field tests. For grouting applications, the field method of choice is normally through the use of short-interval (e.g., 3 m, or 10 ft) double packer tests. When the holes are oriented to maximize the number of intersections with the various sets of fractures in the rock, the test results provide a large number of bulk permeability values that actually take into account both the permeability of the matrix materials and the equivalent permeability of the fracture systems. The in-place measurements of the equivalent permeability of the fracture systems, which are normally by far the dominant source of hydraulic conductivity, empirically take into account all of the unknowns in the fracture system, such as fracture continuity, individual aperture size, and many other variables. Further, if tests are performed under proposed service heads, the data can at least partially account for the effects of turbulent flow behavior.

(3) Pre-Grouting vs. Post-Grouting Conditions. Porous media models may tend to produce greater error when analyzing pre-grouting conditions than for post-grouting conditions. In the post-grouting conditions, flow behavior is greatly controlled by the properties of the grouted zone, which should have hydraulic conductivities several orders of magnitude lower than non-grouted portions of the mass. When a three orders-of-magnitude difference in relative permeability has been achieved, the zones upstream of the grouted zone tend to act as an infinite source and the zones downstream from the grouted mass tend to behave as an infinite drain. Assuming that the design and execution of the grouting have been such that no fractures or portions of fractures have simply been missed and that all fractures have been grouted to absolute refusal, the small residual permeability in the grouted fractures will be due to relatively uniformly distributed small discontinuities. Therefore, the post-grouting conditions should quite closely behave as a porous media obeying Darcy’s Law. If the design and execution of the grouting has resulted in not grouting certain fractures or portions of fractures, those types of defects in the grouting will tend to behave in accordance with discrete fracture flow theory and may result in leakage in great quantities.

5-6. Application of Seepage Flow Models in Grouting Program Design.

a. Purposes of Flow Models. Flow models greatly increase the level of understanding of seepage flow rates, flow paths, and pressures far beyond the intuitive level, and they can provide a logical basis for: (1) establishing the need for grouting, (2) determining the physical extent, depth, and intensity of grouting required, (3) comparatively evaluating alternate designs, and (4) assessing and communicating the technical and economic value of grouting to be performed. For all of these reasons, seepage flow models are essential components of grouting program design. Models are also vital tools for establishing critical locations for piezometers and other instrumentation for the project and for providing predictions of instrument readings both during and after successful completion of grouting.

b. Complexity of Flow Models. Seepage flow models are available in all ranges of complexity: (1) closed-form analytical solutions (equations) for specific, simplified conditions, (2) 2-D flow net models that can be either very simplified or quite detailed, (3) 1-D, 2-D or 3-D computer models based on finite element or finite difference solutions of flow equations, and (4) models to simulate flow in fractured rock. In addition to modeling seepage, many of the models can also be used to model contaminant transport to various degrees of sophistication. A recent count indicates that more than 75 models, or variations of models, are currently in existence. All of these models are useful, and the challenge is to select the complexity of the model that is consistent with the quality and amount of data available, the project phase, and the project size and needs. Multiple modeling techniques are often employed in different phases of a project, and nearly every project should include simple analytic, 1-D models or 2-D flow net models as the starting point. These models are easy to develop and can be quite accurate. Because these types of models are manually developed, they have often been found to provide a more sound fundamental understanding of seepage flows under the scenarios of interest. Additionally, these models are often the means of reasonably checking the results of more complex models. In grouting applications, complex computer models are not automatically favored over simpler models for multiple reasons: (1) they can be time consuming and costly to develop, (2) they can provide a false sense of accuracy that is not consistent with the quality of the input data, (3) highly complex models can be extremely difficult to review for fundamental correctness, and errors can be difficult to detect, and (4) highly complex modeling may require highly experienced specialists who may not be readily available for the project. Conversely, complex computer models, applied in the proper circumstances, can provide levels of sophistication that are of high value. Chapter 5 of EM 1110-2-1421 provides general guidance on important questions to consider when selecting a model to be used. Table 5-4 summarizes typical applications and/or situations suitable for the various types of flow models and provides comments related to their use. More detailed discussion and examples of the various types of models are contained later in this chapter.

Table 5-4. Typical uses of flow models in grouting design applications.

Model Type	Typical Applications	Comments
Analytic Models (Equations)	• These can be used at any level provided the site conditions closely match the conditions required by the model. • These are often used in Reconnaissance Phase or as a first-level analysis in feasibility phase provided that site simplifications required for use of model are reasonable and fundamentally correct.	• Provides an exact solution for a highly specific set of conditions. • Can be used to quickly evaluate seepage conditions and control alternatives on a comparative basis. • Higher level models normally used to refine the solutions before concluding feasibility phase.

Table 5-4. (Continued).

2D Flownet (Manual)	• It is suitable for simple site conditions in reconnaissance, feasibility, and/or PED phases. • It is suitable for moderately complex site conditions in reconnaissance and feasibility phases provided site simplifications required are reasonable and fundamentally correct. • It is suitable for rough checking numerical models prepared for moderately complex to complex site conditions. • Performing large number of analyses using this model is problematic.	• Must account for end effects (i.e., flows around ends of hydraulic structures) via curvilinear cross sections or via use of modified effective structure length. • Can be used in field to assist with rapid assessment of observed behavior. • Application becomes progressively more difficult as number of permeability zones increases, particularly when the permeability change from zone to zone is incremental rather than abrupt and substantial. • Difficult to facilitate sensitivity analyses and/or large number of alternatives.
2D Numerical (Computer)	• It is suitable for simple to complex site conditions in feasibility phase. • It is commonly required for completion of PED phase. • Performing large numbers of analyses using this model is easily facilitated. • Modeling results from this model can be imported into other programs (i.e., slope stability).	• Reconnaissance phase may not have adequate data to support use of 2D numerical model. • Results require careful review and checking. • Graphical output is professional in appearance. • This model must account for end effects (i.e., flows around ends of hydraulic structures) via curvilinear cross sections or via use of modified effective structure length.
3D Numerical	• This model is suitable for complex site conditions. • End effects are properly modeled rather than approximated. • 3-D flows related to topographic and/or spatial geologic variations are properly modeled. • Use is often limited to large, complex projects due to modeling effort and data required to obtain value from model. • This model is normally applied in the PED phase.	• This model provides most realistic modeling of flow behavior where large amounts of quality data are available. • Substantial effort and high degree of expertise are necessary for proper application of this model. • Pre- and post-processors are required (i.e., Groundwater Modeling System [GMS]) to use this model. • Results may be difficult to check and model may be cumbersome to use due to complexity.

Table 5-4. (Continued).

Discrete Fracture System Numerical	• This model is not commonly used in conventional grouting design applications. • This model is available for use in highly critical or specialized applications (e.g., critical environmental applications).	• Data requirements are substantial and may be difficult to obtain. • A limited number of modeling specialists with sufficient experience are available. • There is a high potential for increased use of this model due to available technology for down-hole fracture mapping.

c. Accuracy of Flow Models. As a minimum, flow models are useful as a guide; at their best, they are a reliable quantitative design tool. The primary factors affecting seepage flow model accuracy are: (1) the quality and level of detail of geologic information, (2) the quality and amount of valid geohydrologic data (hydraulic conductivities, boundary conditions, intermediate piezometric levels, and availability of multiple sets of calibration data), (3) the amount of simplification used in the model, and (4) the ability to execute grouting in the field to a verifiable end result. This last point is critical in that not only is the accuracy of the model highly dependent on achieving a verifiable end result in the field, the actual post-construction performance also depends on it. This value, once selected as the basis for design, must be achieved. If the combined effects of all results of all these factors do not instill confidence that at least an order-of-magnitude solution will be the outcome, the model has relatively limited value, and remedies are required for deficient items and/or a much higher level of intuitive judgment guided by the model results must be used. When all these factors are highly adequate and favorable, the resulting accuracy of the flow model may be significantly better by an order of magnitude.

5-7. Pre-grouting Hydraulic Conductivity Values for Flow Models.

a. Pre-Grouting Values. Hydraulic conductivity values for non-grouted materials are determined by direct testing. Values for soils are usually based on a combination of laboratory and field testing, whereas values for rock are determined solely by field test methods. It is important that testing programs be structured to provide a large number of test values and values representing small volumes of materials. Typically, for design purposes, packer tests with 3-m (10-ft) test intervals should be used so that reasonable resolution of the spatial variations in hydraulic conductivity is possible. With longer test intervals, important variations in conductivity can be masked by averaging effects.

b. Mean Values. The necessary end product and goal of hydraulic conductivity testing is to be able to divide the problem space into regions of similar hydraulic conductivity and to assign specific values to those regions. Given that hydraulic conductivity is fundamentally an order-of-magnitude property, in general, there is little or no value in attempting to define separate regions with less than an order-of-magnitude difference in value. In determining the mean value to assign to a region or zone, the data are often considered to have a log-normal distribution, and the geometric mean is used. The geometric mean of a data set is always smaller than or equal to the data set’s arithmetic mean (the two means are identical only if all values within the data set are equal). The geometric mean of a data set is calculated as the n^{th} root of the product of all the members of the data set, where n is the number of values in the set:

$$\text{Geometric Mean} = (a_1 \times a_2 \times \dots a_n)^{1/n}.$$

Alternatively, design values are sometimes established by using either an arithmetic or geometric mean after deleting strongly outlying values where there is a known, non-typical geologic cause of the outlying value.

5-8. Hydraulic Conductivity Values for Grouted Zones.

a. General. In contrast to establishing design values for non-grouted zones, which is an exercise in determining values for in-situ properties created by the natural geologic conditions, the design hydraulic conductivity values for grouted zones are assigned values that are known with certainty to be achievable through the application of grout and subsequently verifiable as part of those field operations. As simple as that concept is, selecting design values that are achievable for the grouted zone requires the utmost consideration and care.

b. Factors Affecting Hydraulic Conductivity of Grouted Zones. The known end performance for grouted zones has ranged from results that were so high that the grouting as planned and executed was essentially useless to values that were as low as 0.1 Lugeon under favorable geologic conditions combined with best practice in every aspect of design and execution. There have been project sites that were declared to be ungroutable based on the results of one grouting contract at the site, and then those same sites were successfully grouted under a later contract. Such extreme results are possible because of the large number of factors that influence the end results and end performance. Houlsby (1990) provides a summary and discussion of this multitude of factors in some detail. A partial listing of those factors includes:

(1) Geologic conditions.

(2) Drilling equipment and technique.

(3) Grout materials.

(4) Grout mixes.

(5) Mixing equipment.

(6) Hole washing.

(7) Grout pumping equipment.

(8) System setup.

(9) Valves and fittings.

(10) Bleed characteristics of grout.

(11) Grouting technique.

(12) Hole spacing.

(13) Hole depth.

(14) Hole sequencing.

(15) Hole staging.

(16) Pressures.

(17) Contractor experience.

(18) Refusal criteria.

(19) Contract incentives and disincentives.

(20) Inspection.

(21) Monitoring and control technology.

(22) QC testing.

(23) Effectiveness of analyses during execution.

(24) Verification programs.

c. Guidelines for Selecting Hydraulic Conductivity of Grouted Zones.

(1) Guidelines for Achieving Permeability Reductions. The impact of grouting on the behavior and performance of a hydraulic barrier has a great deal to do with two factors: the absolute value of hydraulic conductivity of the grouted zone and the relative permeability of the grouted zone. Either of these factors might control decisions on the value of hydraulic conductivity required to accomplish the design intent. The absolute value will control the rate of seepage through the completed work, whereas the relative value (i.e., as it compares to ungrouted zones) will control the pressure distributions that can be achieved and the relative benefit in flow reduction that can be derived from the grouting. Houlsby (1990) provides some broad-based characterizations of the general ease of accomplishing both relative and absolute reductions in permeability. The following paragraphs describe Houlsby's general framework.

(a) Readily Groutable Materials. Readily groutable materials are materials having an initial permeability greater than 100 Lugeons, which is greater than approximately 1.3×10^{-3} cm/s. These materials will, in general, have well-connected fracture networks, generally contain larger fractures, and/or have a high frequency of fractures. Accordingly, these materials are generally favorable for both injection and travel of grout. For these materials, Houlsby suggests that reductions of one to three orders of magnitude are possible, depending on the impact of factors listed above. Obviously, better technique in those factors creates better results.

Treatment of these materials is normally found to be required for adequate performance in most applications. Grouting will significantly alter flow rates and pressure distributions.

(b) Marginally Groutable Materials. Marginally groutable materials are materials having an initial permeability in the range of 10 Lugeons, which is approximately 1.3×10^{-4} cm/s. To put this value into perspective, it is in the range of the hydraulic conductivity for very fine sands and slightly silty sands. Houlsby (1990) suggests that grouting of these materials can produce a reduction in permeability of one to two orders of magnitude, again depending on the impact of factors listed above. Treatment of these materials might or might not be required, depending on the specific project performance requirements. Effective grouting may be beneficial in reducing the flows, but will be less effective in altering pressure distributions due to a smaller change in relative permeability.

(c) Barely Groutable Materials. Barely groutable materials are materials having an initial permeability in the range of 1 Lugeon, which is approximately 1.3×10^{-5} cm/s. Houlsby (1990) notes that a number of special measures are required for achieving permeability reductions in these materials. Special measures can include the use of materials such as silica fume, finely ground cements, and/or chemical grouts to achieve penetration of very fine fractures. Even though treatment of these materials may not be indicated based on the bulk initial permeability, it is not at all uncommon to grout them with a single line of holes to eliminate high-conductivity anomalies that may not have been disclosed in the exploration and characterization. Intensive grouting of these materials may be required in special circumstances, such as in environmental control applications or where the value of water is extremely high.

(2) Lower Bound Values. A second guideline that can be considered in choosing a design value is the lower bound value that can be achieved by grouting. It has been shown that rock can be grouted with balanced stable cement suspension formulations to mean values in the range of 0.1 Lugeon, which is less than approximately 1.3×10^{-6} cm/s. Grouting to this level requires an intensive program, special grout mixes, electronic monitoring and control technology, intensive quality control of mixes, and a high level of care and control in every operation.

(3) Philosophy for Selecting Design Values within the Range of Possibilities. Houlsby (1990) clearly indicates that grouting results can vary by orders of magnitude, depending on the quality of the design and execution of the grouting program. Wilson and Dreese (2003) reached similar conclusions after reviewing project results performed with varying degrees of care and sophistication. The incremental cost of performing work in accordance with best practice in every phase of the work is negligible compared to the cost of executing a poorly conceived and poorly executed project. Therefore, using less than best practice in every aspect cannot be endorsed in either a general way or as a cost savings measure, given the potential for unpredictable or unsatisfactory performance. If the approach of best practice is carefully adhered to for every factor of the program, then design values for normal applications can be selected at the lower end of the achievable ranges (e.g., in the range of 5 Lugeons or less).

5-9. Guidelines for Determining Number of Grout Lines for Seepage Control.

a. General. Hydraulic barriers have been constructed with as few as one line of grout holes and with as many as seven lines. The number of lines has a significant impact on project costs and schedule and therefore is an important factor. The selection of the number of lines to be used for a specific project application requires consideration of many factors. In the absence of performing a full-scale test section to logically and comprehensively assess and verify the number of lines required to achieve the design performance requirements, the decision on the number of lines must rely on a combination of modeling, technical assessments based on the geologic conditions at the site, risk evaluation, long-term performance considerations, and other elements.

b. Benefits of Multiple Grout Lines for Seepage Control. The benefits of multiple-line grout barriers are as follow:

(1) Grouted Zone Thickness. All other factors being equal, multiple grout lines will produce a thicker grouted zone. Flow gradients across the grouted zone will be smaller in direct proportion to the thickness, which is of high value, particularly in the vicinity of soil/rock interface areas. Rates of seepage will be reduced in approximately direct proportion to the thickness of the grouted zone (i.e., doubling the thickness halves the seepage).

(2) Grouted Zone Permeability. Multiple-line grout curtains generally have lower hydraulic conductivities. Whereas a reasonable target for hydraulic conductivity of a grouted zone for a high-quality, single-line curtain might be 10 Lugeons, a multiple-line program of the same quality might reasonably be expected to produce a grouted zone hydraulic conductivity in the range of 1.0 Lugeon. The reduction in permeability can result from several factors:

(a) The first-pass grouting of a rock mass (i.e., the first line) will tend to treat the larger fractures at the expense of the smaller fractures. When a stage consists of both fine and large fractures, it will display a large grout take that will result in thickening of the grout mix, which will then limit penetration into the fine fractures. Grouting of a rock mass with a second line will primarily be a process of grouting the finer fractures.

(b) It is not uncommon that the strike and dip of sub-vertical fractures will vary across a site. A single line of inclined holes can in some areas therefore result in hole orientations that are nearly parallel to some fractures, rather than intersecting them. In such cases, some fractures can be missed entirely. A second line, which is often oriented in an opposing inclined direction, will generally increase the chance of intersecting those fractures. Best practice is to orient the hole inclination to intersect the highest percentage of joint sets based on the site characterization.

(c) Where grout takes are very high, it may be necessary to limit the volume of grout injection in stages to prevent inordinate travel distances that do not contribute to the value of the grouting. However, this can also result in ineffective grouting of the portion of the rock mass of interest. In these cases, it may be necessary to use upstream and downstream lines as confinement lines, after which a third interior line is used for intensive grouting to refusal.

(3) Compensation for Quality Issues. Although compromises on quality in any regard are to be avoided in every aspect of the work, the reality is that they have occurred in the past and will occur in the future. If project circumstances are such that premium quality is not ensured, multiple-line curtains can help overcome quality deficits. In such cases, the end result, however, may be a multiple-line curtain with the performance of a very high-quality single line.

(4) Long-term Durability. All other factors being equal, thicker grouted zones will perform in a satisfactory manner for a longer period of time. There is always some residual flow through a grouted zone, and it is this residual flow that gradually attacks the integrity of the zone. The lower permeability of multiple lines results in much less residual flow, which decreases the attack on the zone.

(5) Confinement of Soil Materials in Fractures. There are situations where it is not possible to remove all soil-infilled fracture zones by excavation, so they remain in place at the time of grouting. These materials, in general, cannot be effectively removed by washing the grout holes. These soil-filled fractures are of considerable concern for long-term performance; if the soil filling is kept in place, performance will be fine, but if the materials are gradually eroded from the fractures, large amounts of leakage can develop. Multiple lines increase the probability that the soil materials can be confined in place, they reduce access of water to the infilled fractures through the clean fracture network, and they reduce gradients across the soil-infilled fractures. Additionally, it is possible to repeatedly hydrofracture soil infillings with grout to the point of inducing refusal within those fractures under pressures comparable to service heads, thereby creating "confinement cells" within those fractures. Multiple penetrations of those fractures through the use of multiple grout lines is beneficial; however, joint filling is probably the greatest source of uncertainty in the long-term durability of a grout job.

c. Guidance Based on Modeling. Flow modeling should always be part of the basis for selecting the number of grout lines required. Even if not a great deal is known about grouting per se, flow modeling provides a basis for evaluating the required permeability and width of the grouted zone based on certain project performance requirements. For example, it may be necessary to limit residual flows to a certain value, induce certain post-grouting pressure distributions or gradients, and/or match the performance of the grouted barrier to another element of the project (i.e., matching the performance of the grouted zone to an overlying cutoff wall).

d. Typical Applications for Various Numbers of Lines. Table 5-5 lists the typical numbers of grout lines commonly used in certain applications, and some of the considerations involved in decision making.

5-10. Effective Width of Grouted Zones and Spacing of Grout Lines. The effective width of grouted zones is the width within which it is either known or reasonably assumed that the hydraulic conductivity is equal to or less than the required value at all points. Grout travel distances obviously vary as a function of aperture size and orientation, grout mix, grout pressure, and time duration of grout application. A range of 5 to 10 ft between lines is reasonable for most site conditions. However, with grout containment lines installed for a cutoff wall, this may be increased because of concern that irretrievable drill tooling may interfere with the subsequent cutoff construction.

Table 5-5. Considerations for selection of number of grout lines for seepage control.

Number of Grout Lines	Typical Applications/Considerations
Single line	• Confirmation grouting in critical applications (i.e., dam foundation in which available data suggest that grouting may not be required, but where performance is critical and isolated fractures might not have been identified in exploration). • Dewatering or other temporary applications, particularly where moderate performance is acceptable. • Highly favorable foundation conditions (i.e., strike and dip of fractures creates favorable interval of penetration by a single line of grout holes, clean rock fracture systems, and well-fractured rock masses with high interconnection of fracture networks). • Grouting under new concrete and embankment dams with favorable foundation conditions for grouting, where moderate seepage performance is acceptable, and access is readily available for later remedial work if needed.
Two lines	• Foundation situations with fracture orientation variability such that a single line of inclined holes may miss fractures. • Foundation situations with fracture orientations such that the intersection interval of fractures with the final hole spacing on a single line is large (>10 ft). • Grouting under concrete and earth dams where a higher level of seepage performance is required and/or where foundation conditions are less favorable for grouting and/or when later remedial work would be difficult or costly. • Pre-grouting for slurry cutoff walls used in remediating existing dams. • Extensive use of verification holes within grouted mass is required.
Three lines	• Some earth dam foundations, particularly where the foundation is karst limestone, within a certain depth below the embankment/rock interface, which is simultaneously critical for performance and the most difficult to grout; and also important for limiting gradients in lower portion of core of dam. • Moderate-performance environmental containment grouting solutions (i.e., to create a barrier in rock with a flux compatibility similar to the soil cutoff wall). • Most limestone applications where long-term barrier is desired.
More than three lines	• Particularly adverse foundation conditions or extreme performance requirements due to consequences of non-performance. • Critical applications where there are many soil-filled fractures. • Critical environmental applications.
Other variations	• Site-specific features that require non-standard grouting patterns.

5-11. Constant Head Boundaries and Known Intermediate Piezometric Levels for Flow Models.

a. Constant Head Boundaries. A critical first step in developing a flow model is to define the constant head boundaries for the region being modeled. Constant head boundaries are, in essence, the location of either an infinite source of water flowing into the mass being modeled or the locations of constant head discharge points. The location of the constant head boundary can effectively change based on the relationship of relative permeabilities.

(1) Reservoir Levels. The reservoir level for a dam is always one source of a constant head boundary. On the upstream side of the dam, the constant head boundary will normally extend along the face of the dam and along the bottom of the reservoir. However, if the dam has an extremely pervious zone along its upstream face, the constant head boundary is effectively located within the dam along the interface between the pervious zone and a substantially less pervious zone (i.e., two to three orders of magnitude less permeable).

(2) River Levels. The river level on the downstream side of a dam is commonly selected as the downstream constant head boundary. However, selecting that as a valid boundary requires geologic configurations that allow free discharge to the river. If a geologic unit of high permeability is effectively confined beneath the river at the toe of the dam, the free discharge boundary for that unit may be located elsewhere.

(3) Springs. Springs often act as constant head boundaries. However, it should be carefully considered as to whether they are functioning as a general constant head boundary for the problem or as a local constant head boundary for only certain geologic units.

(4) Relief Wells. Properly designed relief wells will create a constant head boundary when upstream heads are sufficient to cause well discharge. At heads less than the level at which discharge occurs, the constant head boundary will be at a natural discharge location farther downstream.

b. Known Intermediate Piezometric Levels. Known intermediate piezometric levels within the region being modeled are of very high value for calibrating and validating the flow model, particularly when multiple sets of steady-state readings are available that were obtained under different sets of constant head boundary conditions. Intermediate piezometric levels must be carefully validated before they are used in a flow model. In validation, the particulars of each instrument installation must be examined carefully to determine what, in fact, that instrument might actually be measuring. When an instrument has been carefully constructed, is located within a single geologic unit, and is properly isolated, it approaches being a point measurement and produces valid values for that particular point. If the instrument has been poorly constructed, is located across multiple geologic units, or is not properly isolated from the other units, it may be highly uncertain what is actually being measured and highly erroneous flow models are possible. Instruments commonly referred to as “monitoring wells” are particularly suspect, in that they often cross multiple geologic units. The water levels that will be observed in such units are typically controlled by one of the geologic units at the expense of the others. Depending on the geometric configuration of the units and their relative permeabilities, erroneous interpretations could be either too high or too low.

5-12. Analytical Models.

a. General. Analytical models provide an exact mathematical solution of a groundwater flow problem in the form of an equation applicable for a specific, greatly simplified set of conditions. These simplifications normally include reducing flow to steady-state flow in one dimension, using a simple uniform media geometry to approximate the flow zone as a homogeneous and isotropic unit, and also using uniform flow properties and simple boundary conditions. Examples of analytical models include the Theiss or Neumann methods for wells and Darcy's equation for confined saturated flow. Because of the inherent simplifications required, analytical models cannot account for temporal or spatial variations. However, when field conditions can be reasonably represented in a simplified manner consistent with the equations, analytical models provide very quick answers based on a few basic parameters. Therefore, they continue to be of substantial value, particularly for rough estimation. Darcy's equation, $Q = KiA$, is directly applicable for one-dimensional flows. Conditions that must be present for valid use are that the flow is confined, laminar flow. Figure 5-2 shows a classic configuration for one-dimensional flow. An infinite supply of water is provided at one end of a confined sample, and free overflow conditions exist at the exit end. Flow through the sample is horizontal, and all head loss occurs linearly along the length of the sample.

(1) Note that, if the block of material shown in Figure 5-2 represented the foundation beneath a concrete dam or the block of material beneath the core of an earth dam, this calculation would significantly overestimate the seepage for a pre-grouting condition because substantial effective flow lengths in the entrance and exit regions are ignored. Depending on the geometry of the structure and the foundation, the error can be significant. However, the estimate could be improved by simply estimating the effective block length based on an extension for some distance into those regions. For example, for the relatively extreme geometry shown in Figure 5-2, the entrance and exit region effects would clearly be significant, and it is obvious just by examination that the effective length involved in flow would be significantly more than the basic block length. Simply by eye, it might be appropriate in this example to double or triple the basic block length to account for those conditions, which would produce a rough flow estimate in the range of 9.7/3 = 3.2 gpm per ft of width to 9.7/2 = 4.9 gpm per ft. Although this is extremely crude, it is also extremely easy to apply, and when very rough estimates are needed instantly, it can be effective.

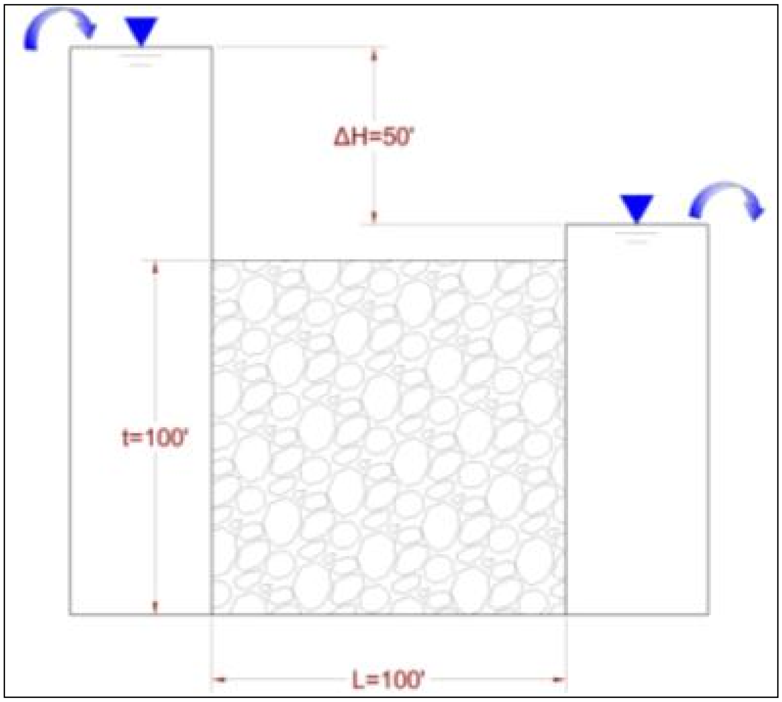
(2) If a fully penetrating hydraulic barrier of extremely low permeability (i.e., a grout curtain with Lugeon ~1) with a length of 20 ft was inserted in the center of the 1,000-Lugeon zone, the differences in permeability between the base material and the barrier would be so great that the modified flow regime would be essentially controlled by the barrier zone alone and could be estimated with very little error by only considering flow through the barrier. Figure 5-3 illustrates this scenario and the accompanying calculations for a post-grouted condition.

(3) An extremely low barrier permeability relative to permeable material permeability (1: 1000) permits the use of Darcy's equation with minimal error.

(4) Note that this type of simplified analytical approximation is only valid if there are extreme differences in permeability between the base material and the barrier zone (i.e., 1,000 or more). When these conditions exist, however, the accuracy of the analytical model is quite good;

ignoring the remaining portions of the block and the entrance and exit flow regions has little effect on accuracy. As indicated in the calculations, compared to the pre-grouting condition, the gradient increased by a factor of five, and all head loss occurs across the barrier zone.

(5) Despite the complexities that exist in natural geologic settings, a surprising number of conditions can be approximated by this type of analytical model with appropriate simplifications.



Flow calculation for porous material with permeability of 1000 Lugeons (1.3×10^{-2} cm/s):

L = 100 ft, Head Differential H = 50 ft, Thickness t = 100 ft;

Unit width d = 1 ft, and Permeability = 1,000 Lugeons = 0.026 ft/min

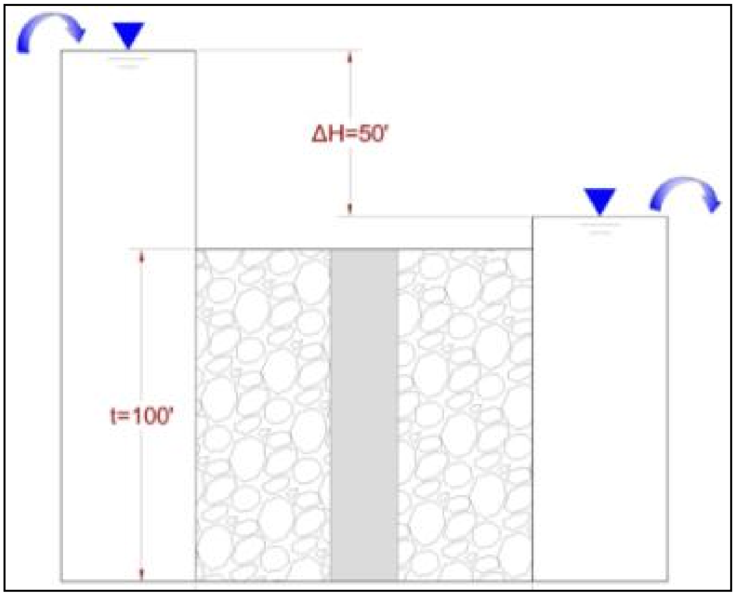
Unit Width Flow Area: $A = t \times d = 100 \text{ ft} \times 1 \text{ ft} = 100 \text{ ft}^2$

Hydraulic Gradient: $i = H / L = 50 \text{ ft} / 100 \text{ ft} = 0.5$

Flow Rate in ft^3/min : $Q = KiA = 0.026 \text{ ft/min} \times 0.5 \times 100 \text{ ft}^2 = 1.30 \text{ ft}^3\text{/min}$

Flow Rate in gal/min = $1.30 \text{ ft}^3\text{/min} \times 7.48 \text{ gal/ft}^3 = 9.7$ gpm per foot of width

Figure 5-2. Calculation of seepage when conditions are suitable for direct application of Darcy's equation.



Post-grouting flow estimate:

L = 20 ft.

Head Differential H = 50 ft, Thickness t = 100 ft.

Unit width d = 1 ft, and Permeability = 1 Lugeon = 0.000026 ft/min.

Unit Width Flow Area: $A = t \times d = 100\text{ ft} \times 1\text{ ft} = 100\text{ ft}^2$.

Hydraulic Gradient: $i = H / L = 50\text{ ft} / 20\text{ ft} = 2.5$.

Flow Rate in ft^3/min: $Q = KiA = 0.000026\text{ ft/min} \times 2.5 \times 100\text{ ft}^2 = 0.0065\text{ ft}^3\text{/min}$.

Flow Rate in gal/min: $0.0065\text{ ft}^3\text{/min} \times 7.48\text{ gal/ft}^3 = 0.05$ gpm per foot of width.

Figure 5-3. Simplified estimate of seepage through a fully penetrating grouted barrier.

5-13. Flow Net Seepage Flow Models.

a. General. A flow net is a two-dimensional graph composed of two families of curves: flow lines and equipotential lines. Flow lines indicate the path of the flow of water, and equipotential lines are lines of equal head, which means that a piezometer placed at any location along a particular equipotential line would register the same reading. In a homogeneous, isotropic medium, the flow lines and equipotential lines intersect orthogonally and therefore form “squares.” Flow nets provide a solution to the Laplace Equation and are applicable to modeling the flow of two-dimensional cross sections of porous media for both confined and unconfined flow problems. Flow net solutions are applicable when the following conditions simultaneously exist: (1) flow is steady-state flow, (2) the medium is composed of saturated, homogenous regions, and (3) flow is laminar and Darcy’s Law is valid. Most commonly, flow nets are constructed on the basis of isotropic conditions (i.e., vertical and horizontal permeabilities are identical). However, when it is known that anisotropic conditions exist within a particular zone, the flow net method can be extended through the use of transformed sections.

b. Requirements for Flow Nets. Any flow net that is drawn must meet the following basic requirements:

(1) Flow lines and equipotential lines must intersect at right angles to form "squares." The size of the "squares" normally varies, depending on their location within the flow zone. Because both sets of lines may be curvilinear, the requirement for "squares" is met when the average width of a "square" is equal to the average length of the "square."

(2) Specific entrance, discharge, and flow line deflection requirements at the boundaries of materials (shown in Figure 5-4) having differing permeabilities must be met.

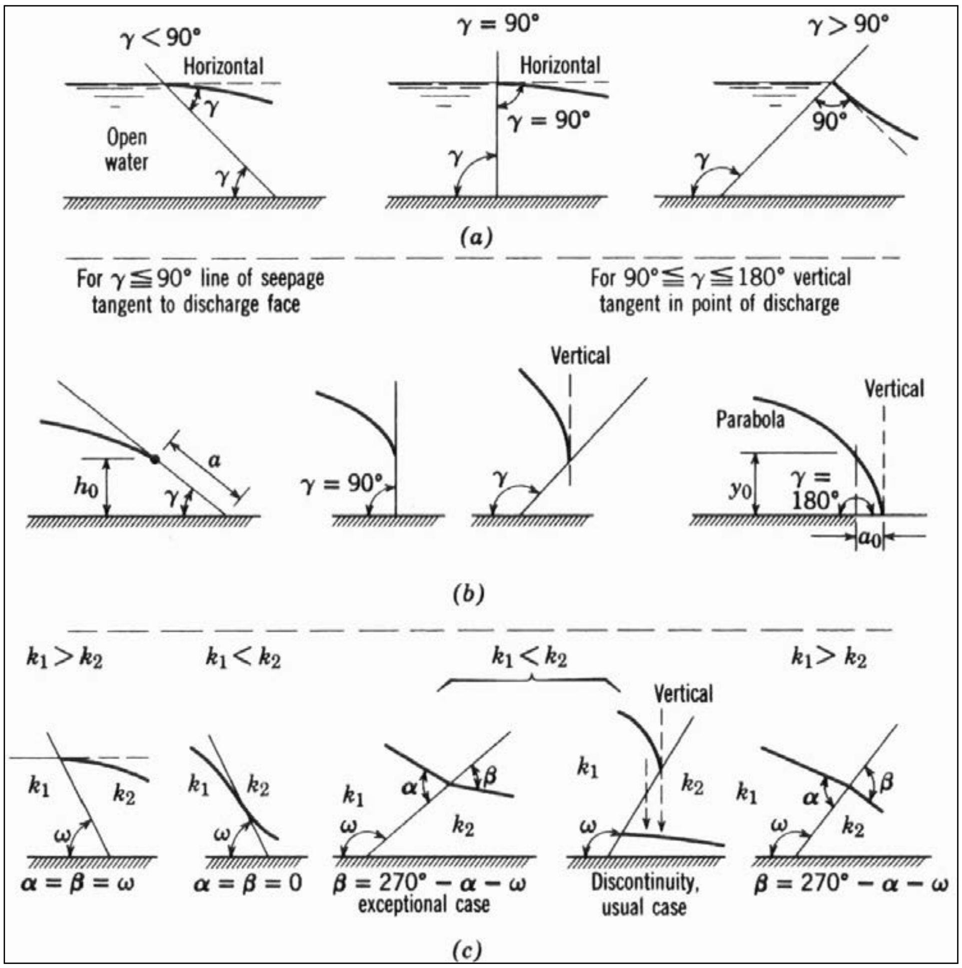


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Figure 5-4. Entrance, discharge, and transfer conditions of line of seepage: (a) conditions for point of entrance of line of seepage, (b) conditions for point of discharge of line of seepage, (c) deflection of line of seepage at boundary between soils of different permeability.

(3) When flow passes from one material into a material with differing permeability, not only are the flow lines deflected, but the "squares" become "rectangles" in the second material, with a dimensional ratio proportional to the permeability ratio of the two materials (Figure 5-5). The angle of deflection (measured from the boundary surface) of the flow lines and the side ratio

for the rectangles follow the relationships provided below. Note that the first equation will result in no-flow line deflection if the flow line has a perpendicular intersection with the boundary:

$$\tan \beta / \tan \alpha = k_1 / k_2$$

$$c / d = k_2 / k_1$$

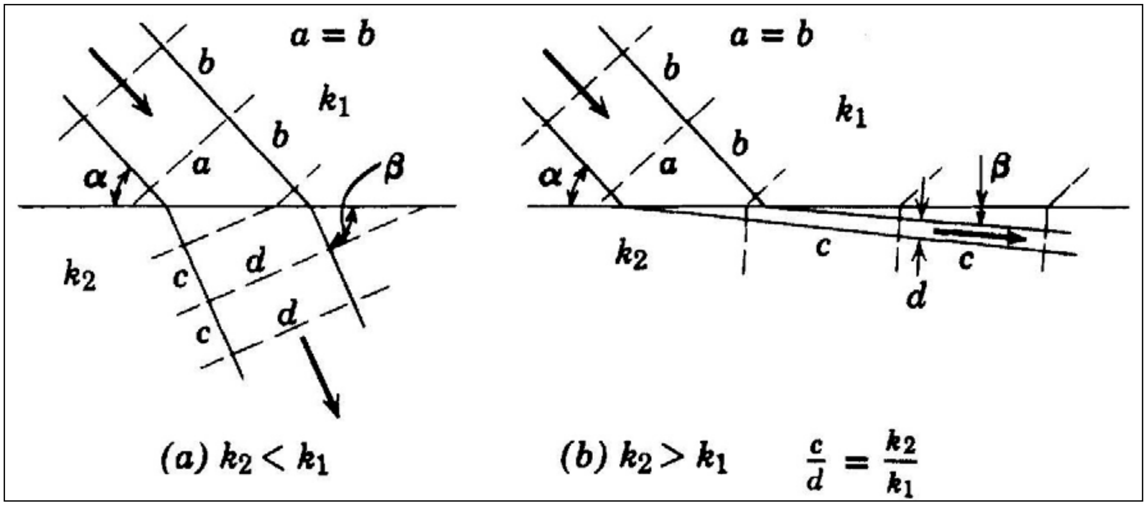

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Figure 5-5. Transfer conditions at boundaries between soils of different permeabilities.

(4) Equipotential lines must be distributed so that head losses between any pair of lines are equal.

(5) The same quantity of seepage flows between any pair of flow lines.

c. Basic Steps for Constructing Flow Nets. Construction of flow nets in isotropic materials involves the following steps:

(1) Draw the cross section to true scale, including the structure, material zones, and controlling water level boundary condition locations (i.e., pool level, tailwater level, drain discharge elevation controls).

(2) Identify the top and bottom flow lines based on the flow boundaries.

(3) Roughly sketch a few intermediate flow lines between the boundary flow lines. These lines will transition in shape between the top and bottom flow lines. Sketch equipotential lines by adhering to right angle intersections with the flow lines to create “squares.”

(4) After the initial sketching, adjust the flow net by successive trials as needed to reach a correct solution.

d. Other Guidance. The following tips will simplify flow net construction and help achieve a reasonably good first-pass solution, which can be further refined if desired:

(1) Research available references to locate flow nets for similar configurations and use them as general guides in drawing the flow net. This will help avoid making fundamental mistakes in the flow net.

(2) In most cases, five to 10 flow lines are sufficient. Using a large number of flow lines does not substantially increase accuracy and makes the sketching process far more difficult.

(3) When permeability differences in materials equal or exceed 1,000, the materials interact with each other as if one is infinitely permeable or impermeable with respect to the other. For example, when a core and cutoff in an earth dam is 1,000 or more times less permeable than the adjacent shell or foundation materials, the core or cutoff zone can be assumed to be impermeable for the purpose of solving the problem of flow bypassing the cutoff. Conversely, if the same core and cutoff ties into an "impermeable" zone beneath the dam, the permeable zones upstream of the cutoff will develop full reservoir head and permeable zones downstream of the cutoff will be at tailwater level. This essentially shifts the boundary conditions to the upstream and downstream sides of the cutoff, and flow will be controlled only by flow through the cutoff and through the "impermeable" zone beneath it.

(4) When permeability differences in materials are on the order of 100, the materials still generally behave as if they were infinitely permeable or impermeable with respect to each other, which still allows simplifications of the problems to be reasonably made. However, at this range of permeability differences, simplification errors start to occur that might not make a difference in preliminary estimates, but that might not be acceptable for final design.

(5) When permeability differences in materials are on the order of 10, the differences in relative permeability are such that it is normally necessary to consider flow and head loss through all the materials. Simplifications of the problem into infinitely permeable and fully impermeable zones can introduce significant errors. For flow lines passing from one material into the other, the flow line deflection rules apply. Additionally, the flow net will be composed of "squares" in one material and "rectangles" in the other material, with the rectangles having side ratios of 10:1. Solutions can be simplified by sketching in the flow lines with the deflections at material boundaries and drawing "squares" for the entire problem. Subsequently, the squares in the less permeable material can be subdivided into rectangles containing 10 equipotential losses to produce the proper proportions.

e. Flow Calculations for Flow Nets. When the requirements listed above are satisfied, seepage through a unit cross section can be calculated as:

$$q = K \times n_f / n_d \times H$$

where:

q	=	flow rate for a cross section having a unit thickness
K	=	hydraulic conductivity
n_f	=	number of flow lines
n_d	=	number of equipotential drops
H	=	total head differential.

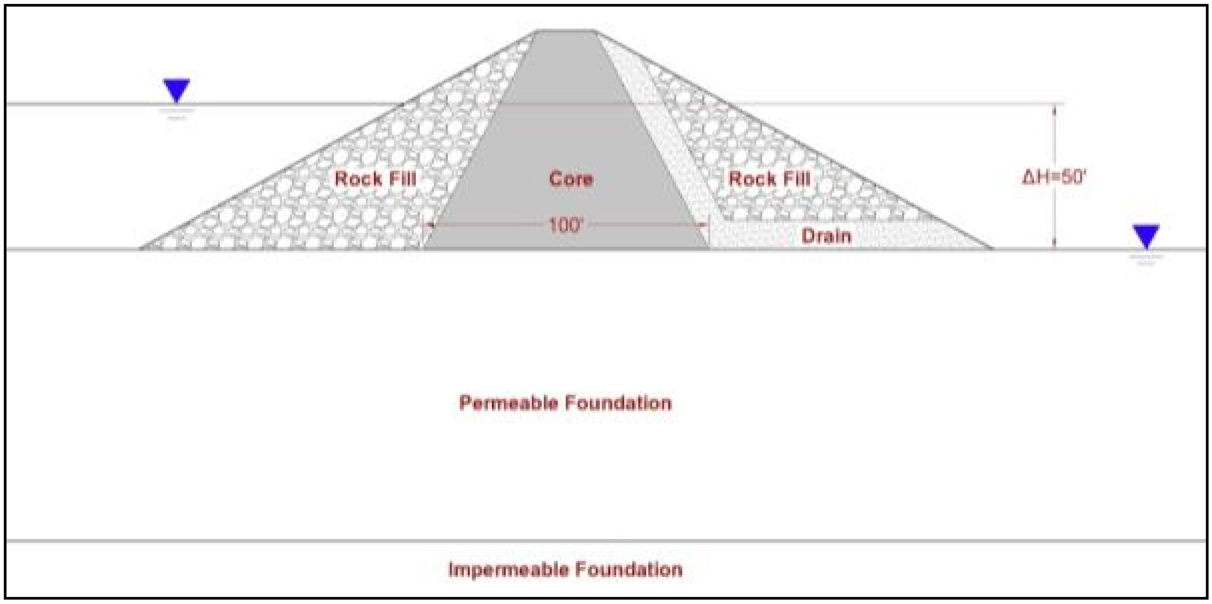
The total flow Q_t over a reach of length L represented by the cross section is calculated as $Q_t = q \times L$. When the region of interest contains multiple materials of differing permeability resulting in a flow net composed of regions that have been drawn as "squares" in one material and as "rectangles" in other materials, the flow calculation is based on the permeability for the material in which "squares" were drawn.

f. Advantages of Flow Net Models. Flow net models have endured as an extremely useful modeling tool due to their advantages: (1) they are highly versatile and can be used to solve quite complex problems, (2) remarkably adequate and workable solutions can often be obtained with very little effort through appropriate simplifications, (3) the learning curve for drawing proper flow nets is quite manageable, (4) they can be constructed by the persons most knowledgeable with the physical conditions of the site and do not require expertise in numerical modeling methods and software, (5) they are often used to provide a reality check on the results of numerical models, and (6) they promote a sound fundamental understanding of the problems, solutions, and critical aspects related to flow control on a particular site. One of the best references on flow net construction is Cedergren's text entitled *Seepage, Drainage, and Flow Nets* (1989).

g. Flow Net Examples. Figures 5-6 through 5-14 show examples of flow net construction and flow calculations for assessing pre-grouting and post-grouting conditions for an earth dam on a simple permeable foundation. Note that the solutions would essentially be the same for a concrete dam provided that the base width of the concrete section was the same dimension as the core contact area. The examples have been selected to illustrate: (1) general flow net construction and flow calculation procedures, (2) the importance that grouting achieve a very significant change in the relative permeability values for it to be effective, (3) and the amount of error introduced by simplification in various scenarios.

(1) Highly Permeable Foundation (Lu = 1,000). Figures 5-6 through 5-9 illustrate basic flow net construction and calculations for a grouted barrier in a simple geologic setting. These figures apply to a case where the differences in permeability of the various materials are sufficiently large that simplifications are applicable without introducing significant error.

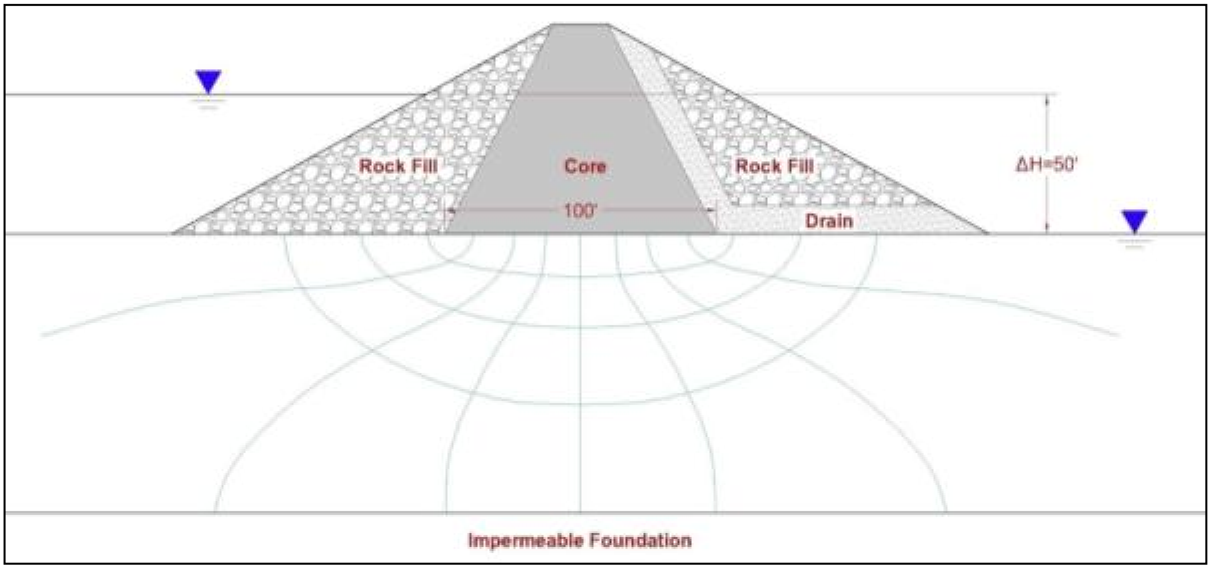
(a) Figure 5-6 shows a section with a highly permeable foundation (Lu = 1,000). The rock-fill zones are far more permeable than the foundation and therefore can be treated as infinitely permeable relative to other materials. Conversely, the permeability of the core material is infinitely impermeable in comparison to all other materials. The drain is highly permeable, and it can be designed to not impede entrance of flow. Therefore, since it can accept all of the flow that travels to it, the drain will also behave as infinitely permeable. Below the 30-m (100-ft) zone of permeable foundation material, it is assumed that tight shale with a permeability of 1 Lugeon exists, which therefore makes the shale effectively impermeable.



Strata	Permeability	Relative Permeability
Core material	2×10^{-6} ft/min	Impermeable
Rock fill	1,000 ft/min	Infinitely permeable
Drain system	2 ft/min	Infinitely permeable
Permeable foundation	2.6×10^{-2} ft/min (1,000 Lu)	Permeable
Impermeable foundation	2.6×10^{-5} ft/min (1 Lu)	Impermeable

Figure 5-6. Example of a section with a highly permeable foundation.

(b) Figure 5-7 shows the flow net solution for no treatment of the foundation and the simplifications described above. As shown in Figure 5-7, the seepage calculation indicates a flow rate of 4.9 gpm per foot of dam if no barrier is constructed. Note that, even if the permeability of the foundation were different from the 1,000 Lugeon value (i.e., 100 Lugeon instead), the flow net would remain identical and only the flow calculation would change. (The calculated flow would be 0.49 gpm for 100-Lugeon material.)



k = 0.026 ft/min (1,000 Lu)

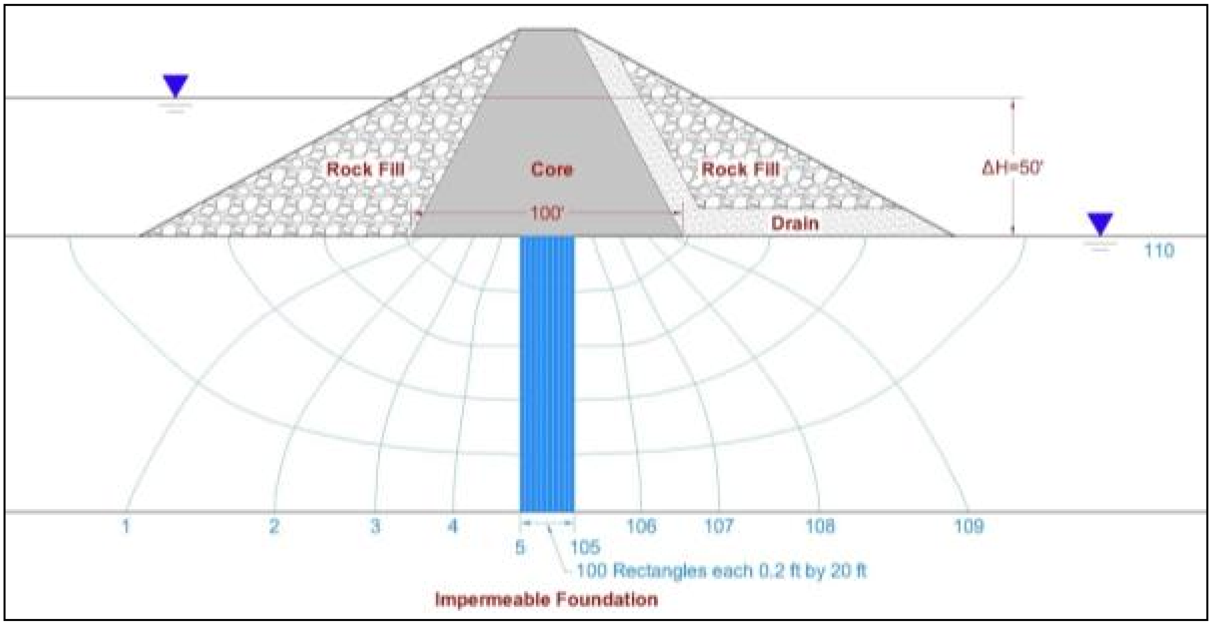
$n_f = 4$ $n_d = 8$ $H = 50$ ft

$$q = k \times (n_f/n_d) \times H$$

$$q = 0.026 \times 4/8 \times 50 = 0.65 \text{ ft}^3/\text{min} = 4.9 \text{ gpm per ft}$$

Figure 5-7. Example of flow net construction and calculations for a section with no treatment of the foundation.

(c) Figure 5-8 shows a flow net for the foundation with a fully penetrating grouted barrier having a width of 6 m (20 ft) and a grouted permeability of 10 Lugeons. As the relative permeability ratio of the permeable foundation material to the grouted barrier is 1,000/10 = 100, it suggests that the problem could be solved either by a flow net that accounts for flow through all materials or by further simplification with the recognition that some error would be introduced. Figure 5-8 shows the non-simplified solution, in which flow through all materials is properly considered. Note that “squares” are drawn in the permeable zone and that rectangles having a height-to-width ratio of 100 have been drawn in the barrier zone. (Each rectangle is 6 m [20 ft] high and 6 cm [0.2 ft] wide, thereby creating 100 equipotential lines within the barrier.) As the “squares” were used in the permeable foundation portion of the flow net, the flow calculation is based on its permeability rather than on the barrier’s permeability. The total number of head drops is 110, and 100 of those drops occur within the barrier. The calculated post-grouting seepage shown in Figure 5-8 is 0.44 gpm per foot of dam, which is only 9% of the pre-grouting seepage, but which is still a very high value in terms of absolute flow rates.



20-ft-wide grouted barrier (full depth) with k_g = 10 Lu

k_G/k_B = 10/1000 = 1/100 (rectangle side ratio)

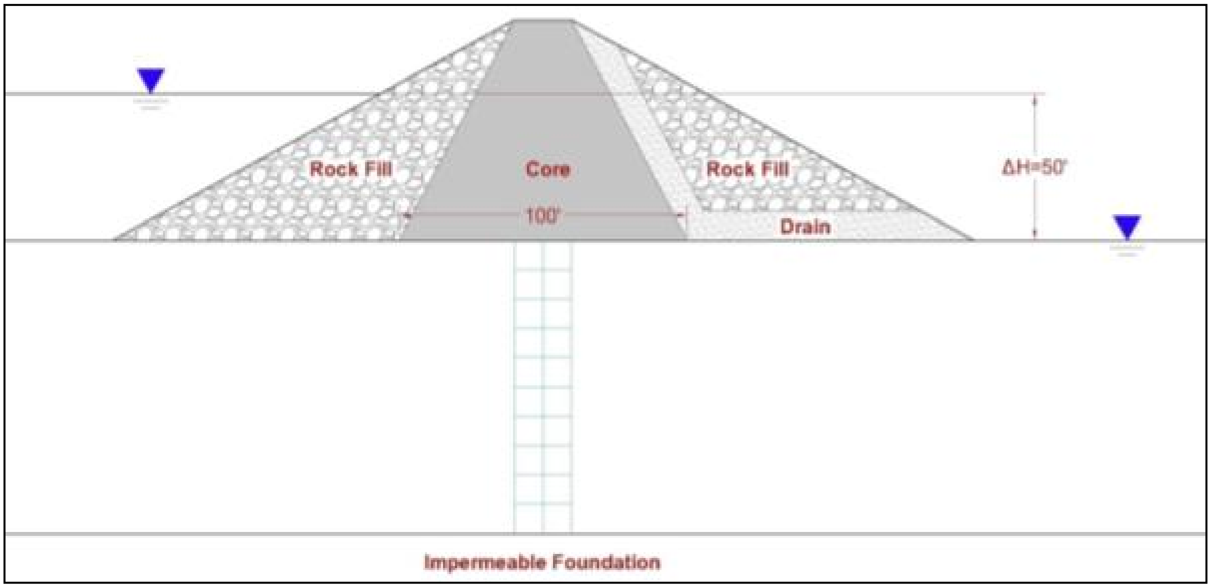
$n_f = 5 \quad n_d = 110 \quad H = 50$ ft

$q = k \times (n_f/n_d) \times H$

$q = 0.026 \times 5/110 \times 50 = 0.059$ ft^3/min = 0.44 gpm per ft

Figure 5-8. Example of flow net construction and calculations for a section with a foundation with a fully penetrating grouted barrier.

(d) Figure 5-9 shows two simplified solutions to the same scenario. In the simplifications, the permeable foundation is now treated as being infinitely permeable with respect to the barrier zone. Therefore, only the barrier zone is considered in the analyses. As shown in Figure 5-9, the simplified solution in this case can be solved either using $Q = KiA$ for the barrier or by drawing a small flow net just for the barrier. Both solutions yield the same result, with a calculated post-grouting seepage of 0.49 gpm per foot of dam. Compared to the “exact” solution used in Figure 5-8, the error resulting from this simplification is an overestimation of post-grouting seepage by 0.05 / 0.44 = 11%.



20-ft-wide grouted barrier (full depth) with k_g = 10 Lu

Simplified solutions:

$$Q = k \times i \times A = 0.00026 \times 50/20 \times (100 \times 1) = 0.065 \text{ ft}^3\text{/min} = 0.49 \text{ gpm per ft}$$

OR

$$n_f = 10 \quad n_d = 2 \quad H = 50 \text{ ft}$$

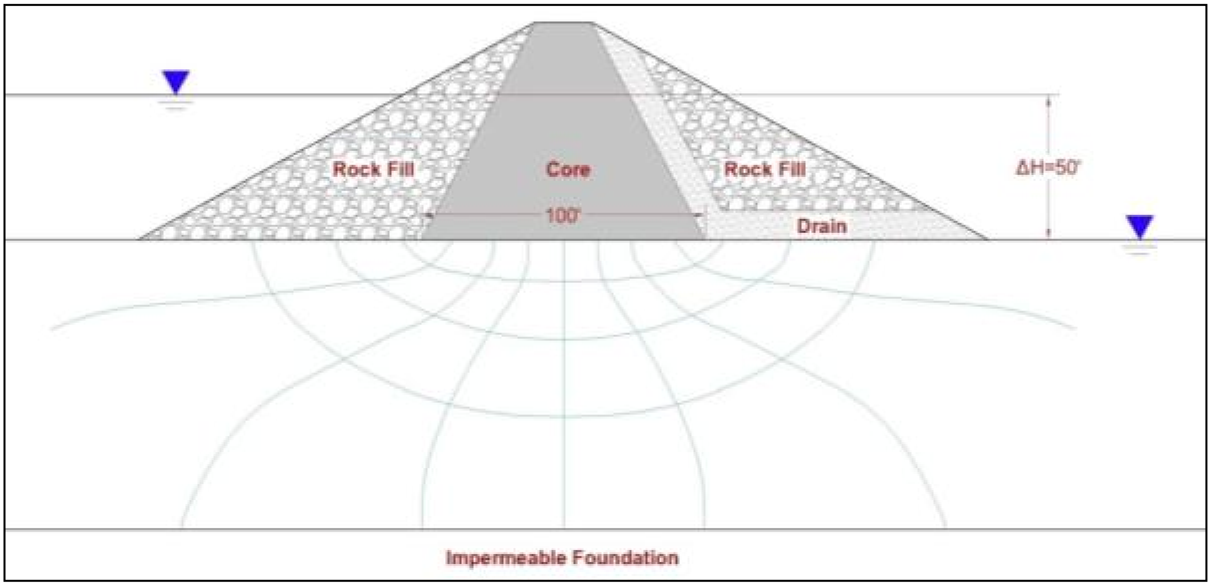
$$q = k \times (n_f/n_d) \times H$$

$$q = 0.00026 \times 10/2 \times 50 = 0.065 \text{ ft}^3\text{/min} = 0.49 \text{ gpm per ft}$$

Figure 5-9. Two simplified solutions for the same scenario as in Fig. 5-8.

(2) Moderately High-Permeability Foundation (Lu = 100). Figures 5-10 through 5-14 illustrate the effects of achieving various permeabilities for a grouted barrier in a foundation with moderately high permeability. Each figure shows a fully penetrating barrier of the same width. These figures, along with their associated post-grouting seepage calculations, illuminate the importance of achieving a significant change in relative permeability between the foundation material and the barrier zone.

(a) Figure 5-10 shows the flow net solution and flow calculations for a 100-Lugeon foundation with no barrier. Note that the net configuration is identical to the flow net for the 1000-Lugeon foundation. The calculated seepage for the site without a grouted barrier is 0.49 gpm per foot of dam, which in nearly every situation would be considered an excessive rate.



k = 0.0026 ft/min (100 Lu)

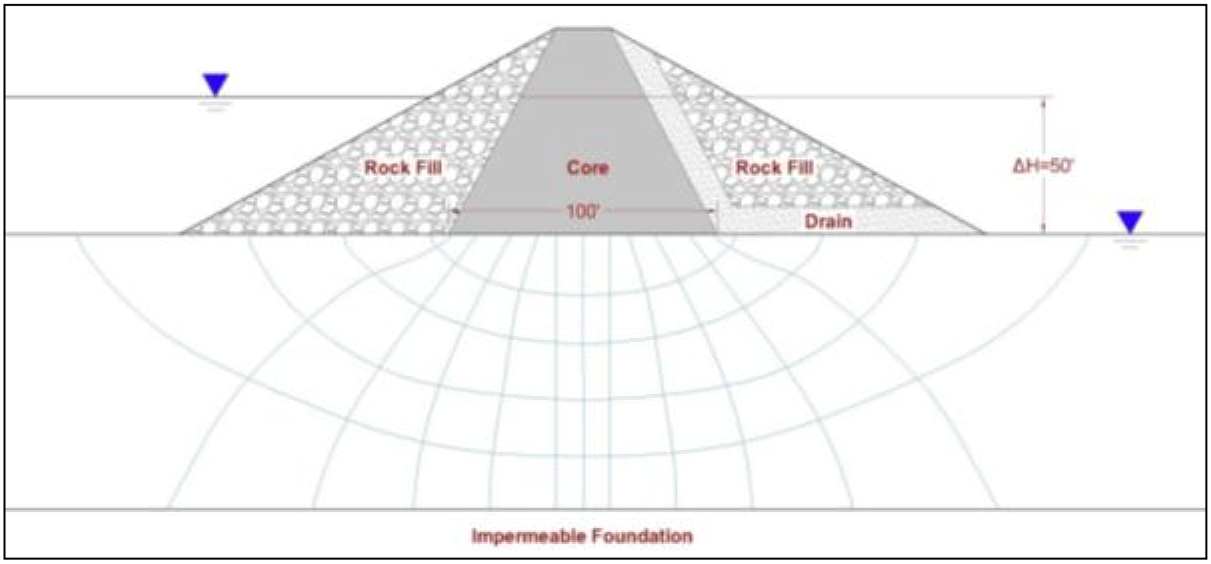
$n_f = 4 \quad n_d = 8 \quad H = 50$ ft

$$q = k \times (n_f/n_d) \times H$$

$$q = 0.0026 \times 4/8 \times 50 = 0.065 \text{ ft}^3\text{/min} = 0.49 \text{ gpm per ft}$$

Figure 5-10. Flow net solution and flow calculations for a 100-Lu foundation with no barrier.

(b) Figure 5-11 shows the results for a very poorly constructed barrier that achieved an end result of only 50 Lugeons within the grouted zone. This type of result is readily achievable if poor practice is used in design and/or construction. Using incorrect hole orientations, incorrect hole spacings, incorrect grout mixes, and excessive stage lengths, and not grouting to refusal are just a few of the causes that could produce a result of this nature. The calculated seepage reduction for creating a barrier with a relative permeability ratio of 100/50 = 2 is only 16%.



100-Lu foundation and 50-Lu barrier

$n_f = 5 \quad n_d = 12 \quad H = 50$ ft

$q = k \times (n_f/n_d) \times H$

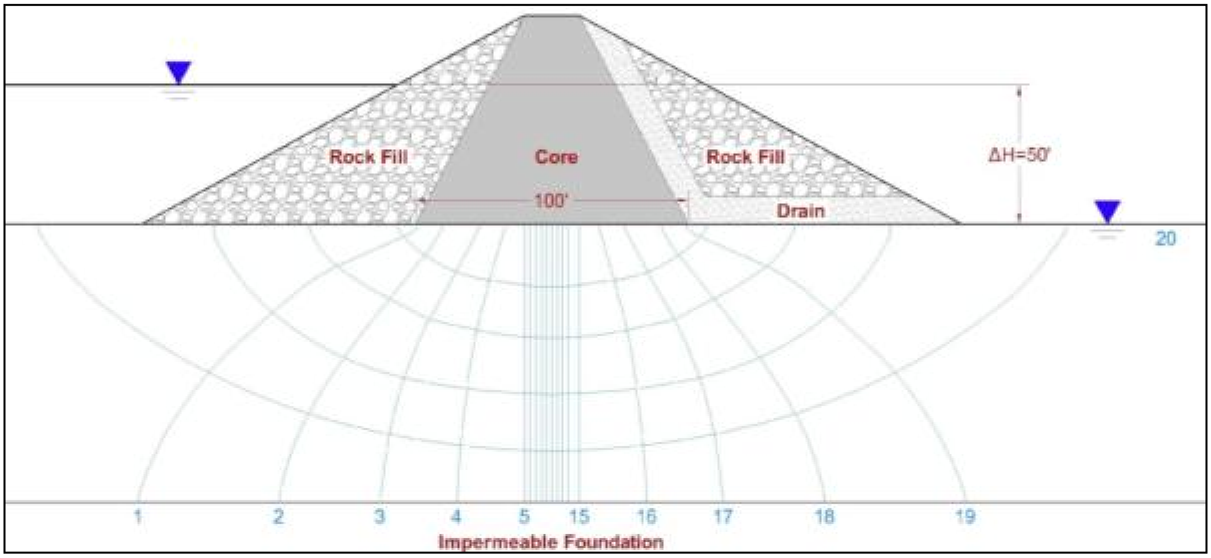
$q = 0.0026 \times 5/12 \times 50 = 0.054$ ft^3/min = 0.41 gpm per ft

Pre-grouting seepage = 0.49 gpm per ft

Seepage reduction = 0.08 gpm per ft = 0.08/0.49 = 16% reduction

Figure 5-11. Flow net solution and flow calculations for a very poorly constructed barrier that achieved an end result of only 50 Lu within the grouted zone.

(c) Figure 5-12 illustrates the results for grouting a barrier to 10 Lugeons within the 100-Lugeon foundation material. Achieving a grouted barrier permeability of 10 Lugeons, particularly with a multiple-line curtain required to create a 6-m- (20-ft-) wide grouted zone, would also indicate low-quality results due to deficiencies in design or construction. Furthermore, achieving a difference in relative permeability of only one order of magnitude results in very low performance efficiency of the barrier relative to the cost and effort expended. The calculated seepage reduction for the barrier is only 51%.



100-Lu foundation and 10-Lu barrier

$n_f = 5$ $n_d = 20$ $H = 50$ ft

$$q = k \times (n_f/n_d) \times H$$

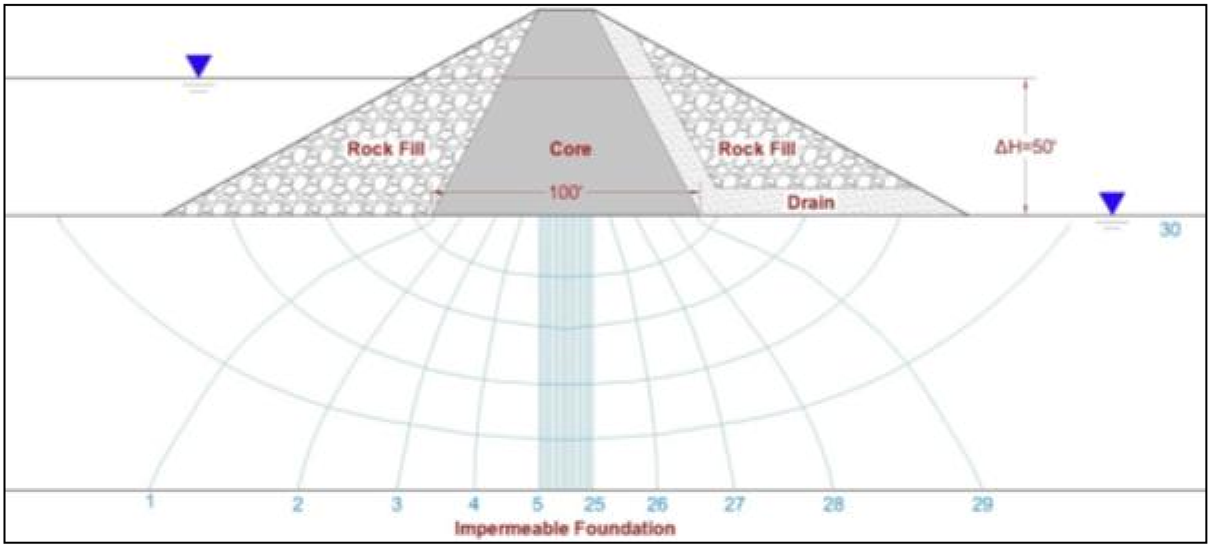
$$q = 0.0026 \times 5/20 \times 50 = 0.033 \text{ ft}^3/\text{min} = 0.24 \text{ gpm per ft}$$

Pre-grouting seepage = 0.49 gpm per ft

Seepage reduction = 0.25 gpm per ft = 0.25/0.49 = 51% reduction

Figure 5-12. Flow net solution and flow calculations for grouting a barrier to 10 Lu within the 100-Lu foundation material.

(d) Figure 5-13 shows results for grouting a 6-m- (20-ft-) wide barrier to 5 Lugeons for a 100-Lugeon foundation. However, it certainly is not a superior result compared to the cost of the effort expended. Results in this range for a multiple-line curtain could, among other things, indicate improper grout mixes or using a refusal criterion other than absolute refusal. The calculated seepage reduction is 67%, and the residual seepage through the barrier is 0.16 gpm per foot of dam. This residual seepage would result in 16 gpm for each 30 m (100 ft) of the dam, which is quite high for the 15-m (50-ft) head that is being applied.



100-Lu foundation and 5-Lu barrier

$n_f = 5$ $n_d = 30$ $H = 50$ ft

$q = k \times (n_f/n_d) \times H$

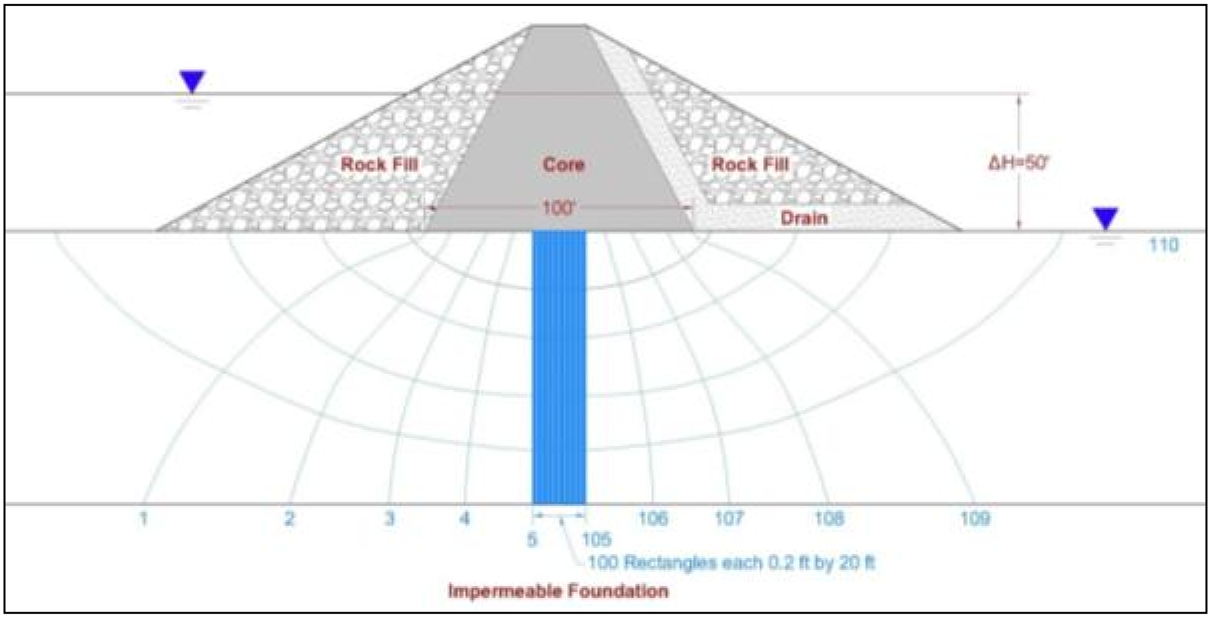
$q = 0.0026 \times 5/30 \times 50 = 0.022$ ft^3/min = 0.16 gpm per ft

Pre-grouting seepage = 0.49 gpm per ft

Seepage reduction = 0.33 gpm per ft = 0.33/0.49 = 67% reduction

Figure 5-13. Flow net solution and flow calculations for grouting a 6-m-wide (20-ft-wide) barrier to 5 Lu.

(e) Figure 5-14 illustrates the outcome for grouting the barrier zone to 1 Lugeon, which is a two-order-of-magnitude lower permeability and represents high-quality, superior results. This type of result is achievable for a multiple-line grout curtain, provided best practice is used in every aspect of design and construction. The seepage reduction for the barrier, when constructed in a 100-Lugeon foundation material, is 92%, and the residual seepage is only 0.04 gpm per foot of dam.



100-Lu foundation and 1-Lu barrier

$n_f = 5 \quad n_d = 110 \quad H = 50$ ft

$q = k \times (n_f/n_d) \times H$

$q = 0.0026 \times 5/110 \times 50 = 0.006$ ft^3/min = 0.044 gpm per ft

Pre-grouting seepage = 0.49 gpm per ft

Seepage reduction = 0.45 gpm per ft = 0.45/0.49 = 92% reduction for 80% penetration

Figure 5-14. Flow net solution and flow calculations for grouting a 20-ft-wide barrier to 1 Lu.

5-14. Numerical Seepage Flow Models.

a. Model Capabilities and Applications. Numerical seepage models can solve the more complex equations describing groundwater flow and solute transport. Although one-dimensional models exist, the numerical models are generally used to approximate multi-dimensional flows, contaminant transport, and chemical reactions.

b. Discretized and Non-Discretized Methods. Most numerical solutions for seepage fall into the general category of discretized methods. These methods include both finite difference and finite element solutions. These methods solve the groundwater flow equation by breaking the problem area into many small elements (i.e., squares, triangles, blocks, etc.) and solving the flow equation for each element. The individual elements are linked together using conservation of mass across the boundaries between the elements. This results in a solution that approximates the groundwater flow equation. Additionally, there are non-gridded methods available such as the Analytical Element Method and the Boundary Element Method, where the majority of the domain is mesh free. These latter methods, however, have seen little use outside of academic and research environments.

5-15. Department of Defense GMS.

a. General. Pre-processors are available to facilitate grid creation, interactive assignment of boundary conditions, and assignment of hydraulic conductivity properties to the geologic units. Similarly, post-processors allow numerical output to be presented as contour maps, raster plots, flow path plots, or line graphs. At their best, they provide a comprehensive graphical environment for numerical modeling tools to assist with site characterization, model conceptualization, mesh and grid generation, and geostatistics; and they also provide sophisticated tools for graphic visualization. Pre- and post-processing software can be specific to one type of modeling software, or it may be configured to work with multiple modeling software packages. GMS deserves special mention. GMS was developed by the Department of Defense in partnership with the Department of Energy, the Environmental Protection Agency, the Nuclear Regulatory Commission, and 20 academic partners. It offers the most sophisticated groundwater modeling environment available today. GMS is available at no cost to employees of the Department of Defense, EPA, DOE, and the National Research Council (NRC) and to on-site contractors to these agencies. It is also commercially available through Environmental Modeling Systems Inc. Training and support on GMS are available from the U.S. Army Groundwater Modeling Technical Center at the Engineer Research and Development Center (ERDC), Vicksburg, MS, or via commercial sources. Some of the features and highlights of GMS include:

(1) It incorporates a suite of graphical environment tools for every phase of groundwater simulation, including site characterization, model development, post-processing, calibration, and visualization.

(2) GMS is compatible with the Windows-based operating system.

(3) It supports a wide variety of finite difference and finite element models in two and three dimensions. Supported models include MODFLOW 2000, MODPATH, MT3DMS/RT35, SEAM35, ART35, UTCHEM, FEMWATER, PEST, UCODE, MODAEM, and SEEP2D.

(4) It allows models to be built using digital maps and elevation models. The Geographic Information System (GIS) Module includes direct linkage to ArcGIS and almost any format of GIS data.

(5) It allows borehole and cross-section data to be used in model development.

(6) Graphical representations of the site model include three-dimensional views with contouring and shading to enhance visualization (Figure 5-15). Models can be displayed in plan view or rotated interactively, and cross sections can be cut at any location.

(7) It provides automatic discretization of the site model.

(8) It provides model calibration tools.

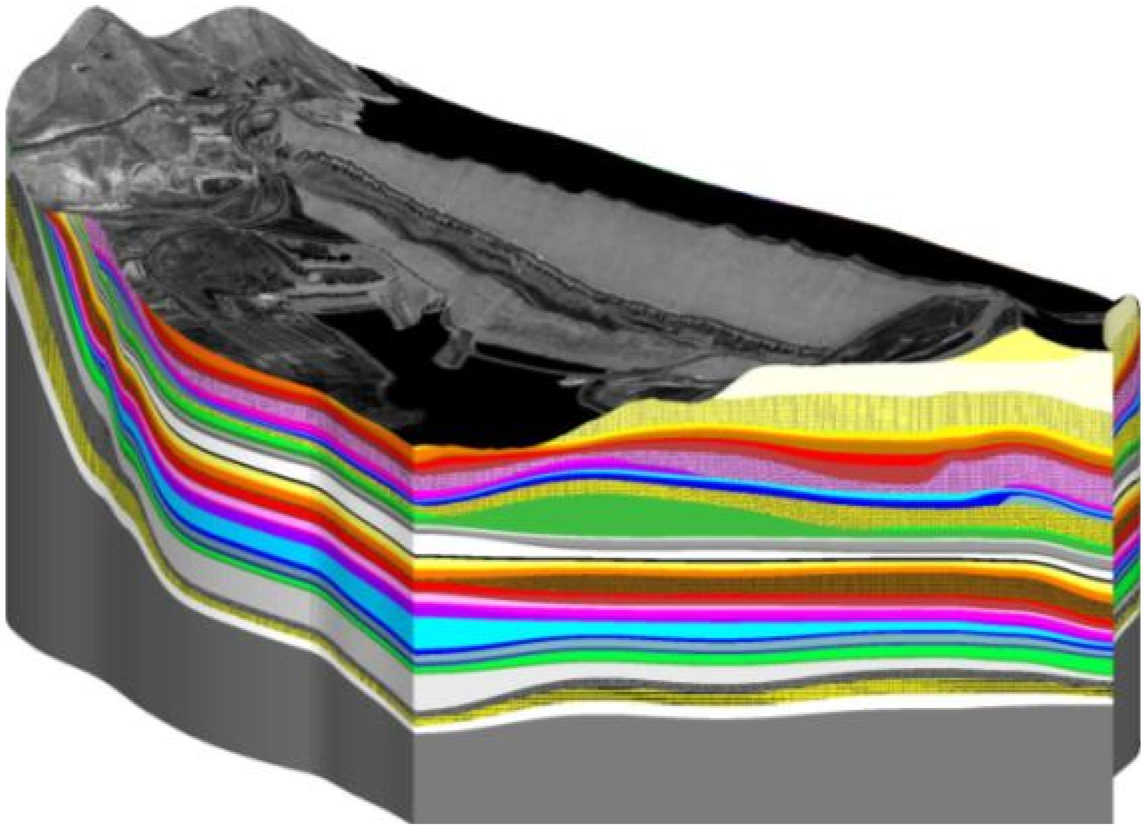
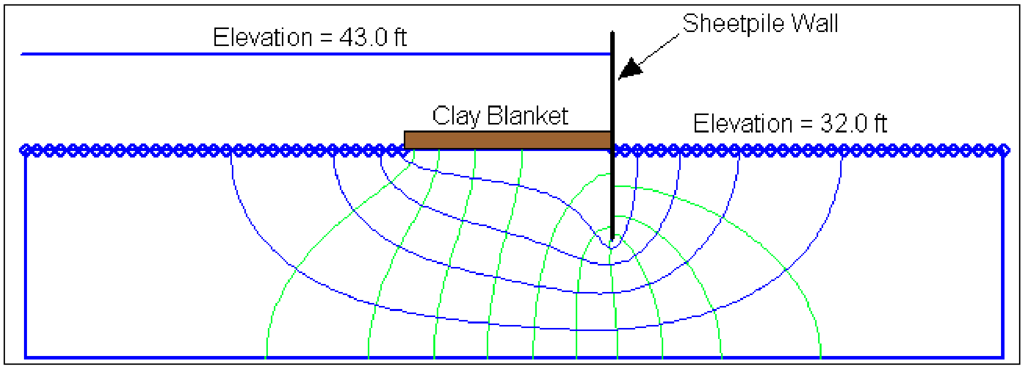


Figure 5-15. Three-dimensional flow model of an earth dam developed by ERDC with GMS.

5-16. Two-Dimensional Numerical Seepage Flow Models Commonly Used in Grouting.

a. Applicability. A majority of the numerical seepage models used for designing grouting are two-dimensional cross-section models that have been developed specifically for these types of problems. These models are relatively simple to use, and their output is easily understandable. Spatial variations of geometry, geologic conditions, or hydraulic conductivity along the length of the structure are modeled through the use of multiple cross sections. The two most common applications are SEEP 2D and SEEP/W.

(1) SEEP 2D is a finite element program designed to compute two-dimensional, steady-state seepage on profile models such as for dams, cut slopes, levees, canals, and foundation excavations (Figure 5-16). This model code was initially developed at WES in the late 1980s. The model can be applied to confined, partially confined, or unconfined flows. SEEP 2D provides the location of the phreatic surface in unconfined or partially confined flow problems. The model can be applied to problems involving complex geometries and materials that are nonhomogeneous and anisotropic. The output is similar to the results of a flow net, in which flow lines and equipotential lines are plotted. A SEEP 2D Interface Module provides a user-friendly graphical interface to the flow model. SEEP 2D is a model supported by GMS, which can provide options for displaying the results.

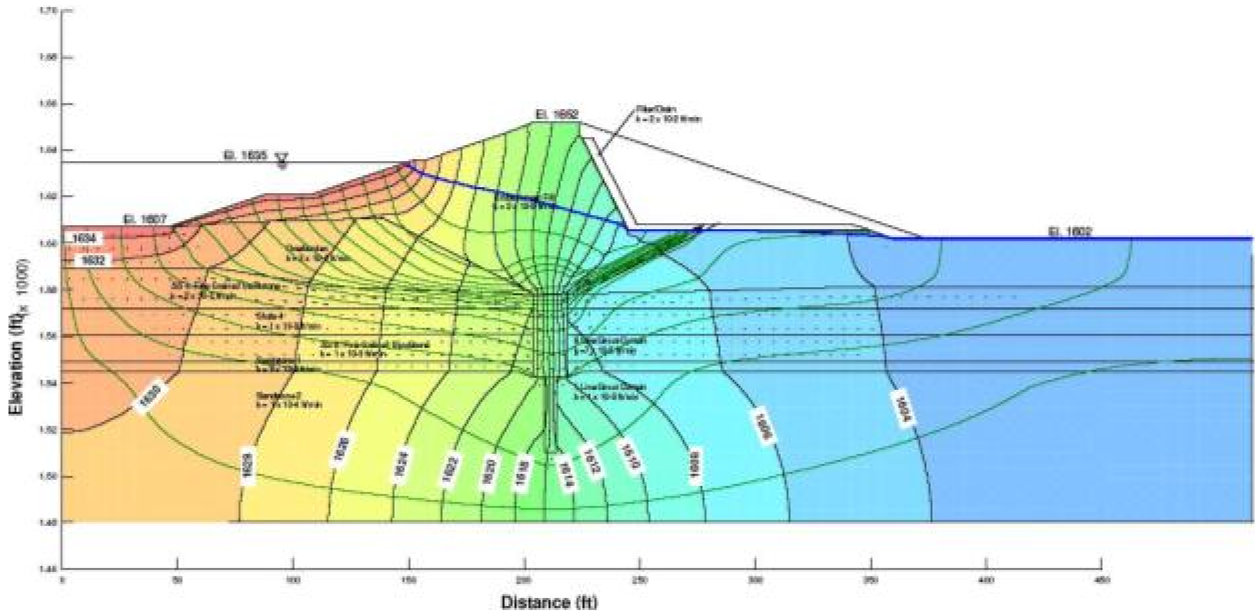


Courtesy of Scientific Software Group.

Figure 5-16. Seep2D sample output.

(2) SEEP/W is a finite element program available from GEO-SLOPE International Ltd. SEEP/W was also developed for two-dimensional seepage on profile models for dams, cut slopes, levees, canals, and foundation excavations (Figure 5-17). It can model both saturated and unsaturated flow, and, in addition to modeling steady-state conditions, it can solve sophisticated saturated-unsaturated, time-dependent problems such as time-dependent migration of wetting fronts, dissipation of excess pore pressures, and infiltration processes. While not supported by GMS, SEEP/W is highly popular modeling software for a number of reasons: (1) it has a very good graphical interface for creating the model, (2) it provides a wide variety of output product displays with interactive query capabilities, and (3) the results are exportable to applications such as Excel and Word. SEEP/W is also available as a component of GeoStudio, which is also available from GEO-SLOPE International. GeoStudio is composed of seven application products including: (1) SLOPE/W for slope stability analysis, (2) SEEP/W for seepage, (3) SIGMA/W for stress and deformation analysis, (4) QUAKE/W for dynamic earthquake analysis, (5) TEMP/W for thermal analysis, (6) CTRAN/W for contaminant transport, and (7) VADOSE/W for vadose zone and soil cover analysis. These products have a common look and feel that promotes easier development of proficiency. Also, the various applications share one model definition, and the results from one application can be used in other applications as needed. For example, pore-water pressures computed by SEEP/W, SIGMA/W, or QUAKE/W can be used in a SLOPE/W stability analysis; SEEP/W pore-water pressures can be used in a SIGMA/W consolidation analysis; and excess pore pressures computed by QUAKE/W can be dissipated over time in a SEEP/W transient seepage analysis.

b. End Effects. The models described above work perfectly well across most of the length of a structure, but treating end effects is often an issue (i.e., residual flow occurring beyond the termination point of a grout curtain). A simple method of approximating the end effects is to draw a planimetric flow net for the most permeable units and then use the curvilinear flow lines from that planimetric flow net as the cross-section lines for creating additional profiles for two-dimensional analysis.



(Courtesy of Gannett Fleming.)

Figure 5-17. Seep/W sample output.

5-17. <u>Three-Dimensional Numerical Seepage Flow Models Used in Grouting</u>.

a. Applicability. Three-dimensional numerical models are used frequently and extensively in steady and unsteady groundwater flow modeling of aquifers, contaminant transport, and similar applications. External stresses that influence flow, such as flow to wells, areal recharge, evapotranspiration, flow to drains, flow through river beds, and many other factors, can be included in the model. Hydraulic conductivities that vary spatially or are anisotropic can be accommodated. On a comparative basis, three-dimensional modeling is more complex, time consuming, and expensive to perform than two-dimensional models, and a high level of modeling expertise is normally required. This expertise is readily available through the ERDC Hydrogeology and Groundwater Modeling Team. While the use of three-dimensional models in routine grouting applications has been relatively rare, that use is increasing rapidly. Further, for certain grouting projects, three-dimensional numerical seepage modeling might be fully appropriate and necessary. Examples where three-dimensional modeling might be considered in grouting projects include:

(1) Situations where the impact of grouting on the local or regional groundwater flow regime may be of interest.

(2) Grouting for environmental applications (i.e., to modify contaminant transport).

(3) Dams with geologic and/or geometric conditions so complex such that it is obvious that a two-dimensional representation and modeling are not applicable or adequate.

(4) Problems where the visual output capability of three-dimensional models is of high value in facilitating communication and understanding of site conditions and flow behavior with other disciplines, other agencies, or the public.

(5) Large and/or long-term dam remediation projects, where the model is continuously refined to incorporate new data and used to display changes in flow pathways with time.

(6) Other time-dependent problems such as dissolution rates or large variations in seasonal recharge.

b. MODFLOW. MODFLOW is a well known, extensively applied, three-dimensional finite difference groundwater flow model (see http://water.usgs.gov/ogw/modflow/). Originally published in 1984, it was developed by the U.S. Geological Survey (USGS) as a modular simulation tool for modeling groundwater flow. The modular structure allows it to be easily modified to adapt to particular applications. Many commercial products that enhance the application are available to provide graphical user interfaces for input data and to provide pre- and post-processing of user data. Many other models have been developed to work with MODFLOW input and output, which allows the use of linked models to simulate multiple hydrologic solutions such as contaminant transport models, surface and groundwater models, and chemical reaction models. MODFLOW simulates steady and unsteady flow in irregular-shaped flow systems in which aquifers can be confined, unconfined, or a combination thereof. Flow from external stresses such as wells, areal recharge, evapotranspiration, flow to drains, and flow through riverbeds can be simulated. Hydraulic conductivities can differ spatially and be anisotropic, and the storage coefficient can be heterogeneous. The flow region is subdivided into blocks in which the properties are assumed to be uniform. In plan view, the blocks are made from a grid of mutually perpendicular lines that can be variably spaced. Model layers can vary in thickness. MODFLOW is supported by GMS and also by a number of other commercial graphical interfaces. Because it uses a finite difference solution, MODFLOW is computationally efficient. Further, its block structure readily facilitates changes in the model. MODFLOW can, however, have stability problems or produce non-representative flow field results in unconfined flow situations when a cell goes dry during the computational process.

c. FEMWATER. FEMWATER is a three-dimensional finite element model for use in saturated and unsaturated zones (see http://www.aquaveo.com/software/gms-femwater). It can also simulate density-driven flows (i.e., salinity intrusion or density-dependent contaminants). FEMWATER is supported by GMS, and solutions can be displayed using realistic three-dimensional plots and animation sequences.

d. FEFLOW. FEFLOW, developed privately by WASY in Germany and in widespread use throughout the world, provides both two-dimensional and three-dimensional finite element modeling of flows, contaminant transport, and heat transport in saturated and unsaturated zones (see http://www.feflow.info/). FEFLOW is not supported by GMS, but it has its own pre-processing and post-processing packages. Information can be read in directly from ArcInfo, ArcView, or MapInfo GIS data sources. Scanned photos and maps can be displayed as background images, thereby permitting digitizing into the model. FEFLOW is available from several commercial sources.

e. Seep3D. Seep3D, available from GEO-SLOPE International, is intended to provide three-dimensional solutions for comparatively simple civil engineering problems that still require a three-dimensional analysis, such as excavation dewatering, finger drains in dams, and dewatering wells (see http://geostudio-seep3d.software.informer.com/). Seep3D is not intended for modeling large or complex systems. Seep3D model formulation is based on geometric building blocks that are combined to create the full geometric model. The finite element mesh pattern is inherent to each region in the domain, and as the blocks are connected together, the mesh is built. The model will solve steady and non-steady-state flow problems.

f. Discrete Fracture Flow Models. The FracMan software suite, distributed and supported through Golder Associates, is a unique, integrated set of tools for discrete fracture network analysis (see http://www.fracman.com/home/software). FracMan includes modules for data analysis, three-dimensional structural modeling and visualization, finite element flow solutions, equivalent pipe network flow solutions, and contaminant transport modeling, as well as numerous other modules.

5-18. Effects of Grouting Defects on Performance of Grouted Barriers.

a. Impacts of Defects. The performance requirements for a grouted barrier are project specific. However, as a feature that is designed to produce a certain performance, the constructed barrier must conform to the geometric configuration and permeability used as the basis for design. Defects in the grouting, as a result of inadequate design or construction implementation, can seriously affect the performance of the barrier, sometimes to the point of rendering the work highly ineffective. Significant verification programs, combined with analysis of data acquired during construction, are performed as the last step in the construction phase after it appears that the barrier conforms with the design performance requirements. With an extensive verification program, defects of most types will be detected. In the absence of extensive verification, the performance may not be known until the project is in operation. This section describes the general nature of defects that can occur and their impacts on performance.

b. Partially Penetrating Barriers. Partially penetrating grouted barriers are grouted zones that do not fully penetrate the permeable foundation zones and/or that are terminated in a zone with a marginally lower relative permeability (i.e., a termination zone that is not at least 100 times less permeable than the permeable foundation zone). Partially penetrating barriers are sometimes intended to be partially penetrating by original design; if that is the intended basis of design and the expected performance is fully understood, it certainly can be a valid approach to achieve goals in certain geologic settings. They will alter flow paths and pressure distributions, but they are notoriously inefficient in seepage reduction. Partially penetrating barriers become defects when their performance behavior is not fully understood or anticipated in advance. Figure 5-18 shows the adverse impact of a partially penetrating earthen cutoff on seepage reduction. For this particular example, a 50% penetration of the permeable zone reduces seepage rates by only 25%, and a 90% penetration reduces seepage by only about 65%. This is the fundamental nature of partially penetrating barriers.

(1) Figure 5-19 provides an additional specific example using the same cross section and properties as were used in the prior flow net examples for a fully penetrating cutoff. For a 6-m-wide (20-ft-wide), 1-Lugeon barrier penetrating to a depth of 80% of the 100-Lugeon permeable foundation material, the seepage reduction provided by the barrier is less than 57% (ignoring flow through the 1-Lugeon barrier). This result is similar to the results indicated in Figure 5-18.

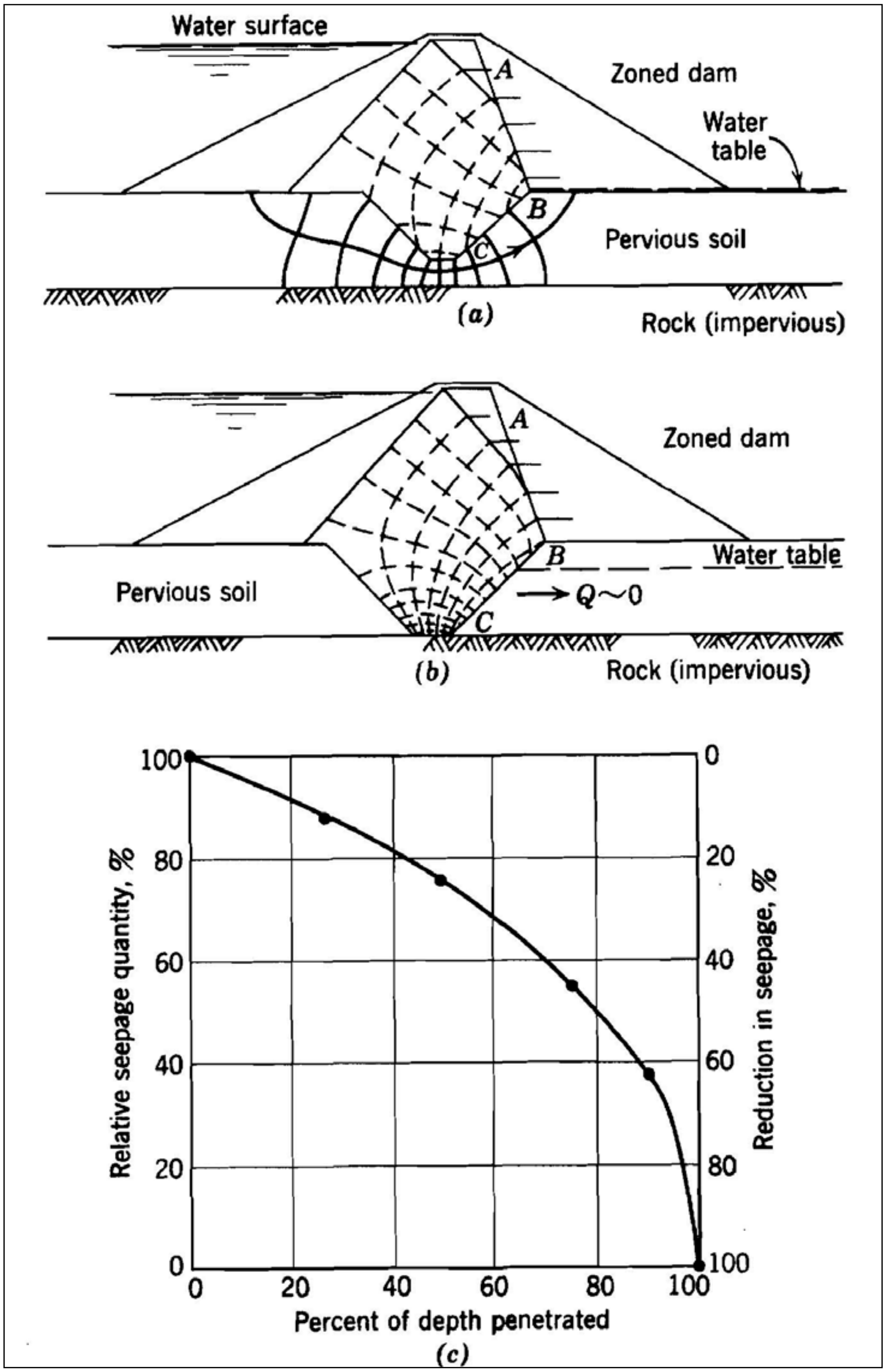


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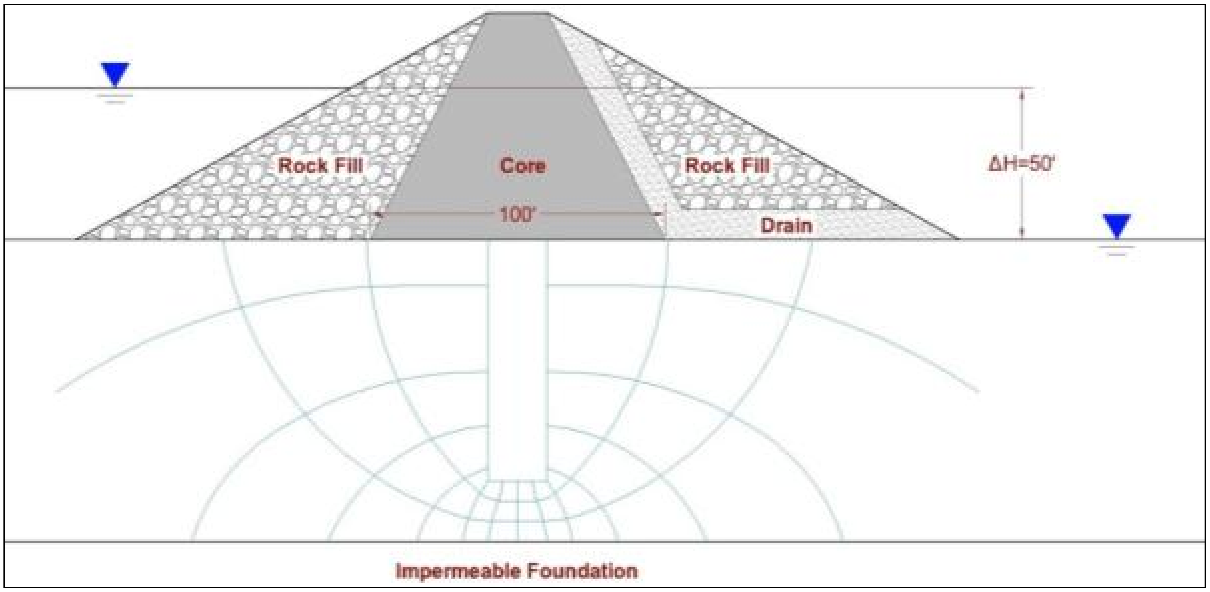
Figure 5-18. Study of partially penetrating cutoffs: (a) cross section and flow net for a partial cutoff, (b) complete cutoff (minute flow through dam), (c) relationship between depth of cutoff and seepage quantity. Note that suitable filters must be provided to prevent piping of soil at faces A-B-C in (a) and (b).



100-Lu foundation and partially penetrating barrier

$n_f = 3 \quad n_d = 14 \quad H = 50$ ft

$q = k \times (n_f/n_d) \times H$

$q = 0.0026 \times 3/14 \times 50 = 0.028$ ft^3/min = 0.21 gpm per ft

Pre-grouting seepage = 0.49 gpm per ft

Seepage reduction = 0.28 gpm per ft = 0.28/0.49 = 57% reduction for 80% penetration

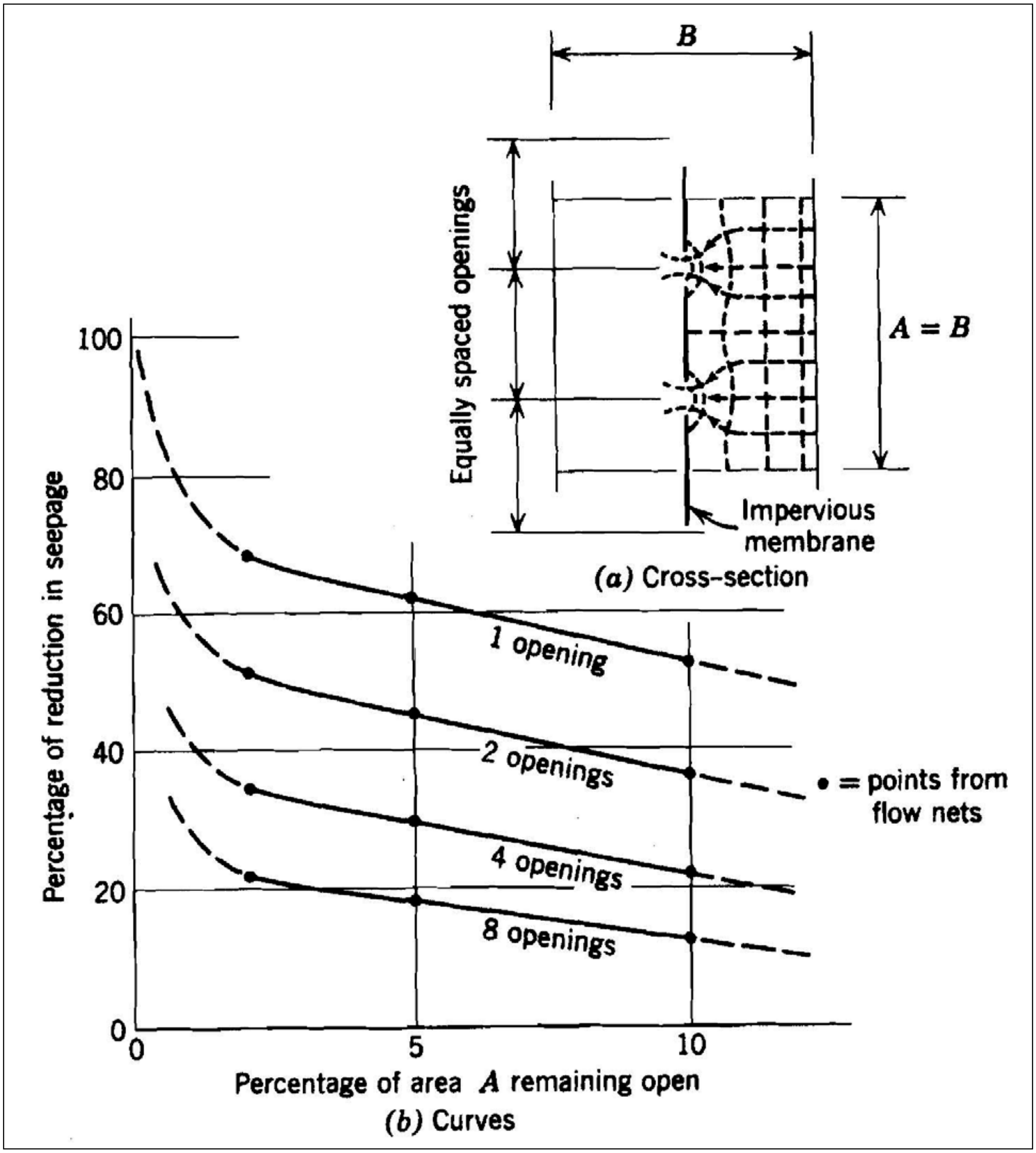
Figure 5-19. Flow net solution and flow calculations for grouting a 6-m-wide (20-ft-wide), partially penetrating barrier to 1 Lu.

(2) A special case of partially penetrating barriers can occur in the abutment areas of dams. Frequently, the geologic conditions in the abutments will differ substantially from the valley portion of the foundation in that elevation changes may result in dealing with different geologic units or with substantially differing fracture and permeability characteristics. In these areas, great attention must be given to providing the proper barrier penetration. Design must also properly address the seepage that will occur around the end of the grouted barrier.

c. Totally Ungrouted Fractures. Totally ungrouted fractures can occur when hole spacings and/or hole orientations are such that fractures are simply never penetrated by the pattern. The most classic example is using vertical holes in a geologic setting where one of the major fracture systems has a nearly vertical orientation. In that scenario, the only way to have grout enter those fractures is through connecting fracture systems, which are often much tighter than the vertical fractures and may be brought to refusal without injecting any significant amount of grout into the vertical system. The potential effects of having a number of totally ungrouted fractures are not quantifiable, but they have the potential to render a grouting program virtually useless. If the ungrouted fractures are continuous with direct access to the full applied head, then they will tend to behave in accordance with flow between parallel plates, and their flow capacity can be enormous (see Figure 5-1).

d. Locally Ungrouted Zones. Even if all fractures have been properly penetrated by the grout holes, it is also possible to have localized zones of ungrouted fractures within a barrier. This might occur in a particular geologic unit, such as a zone where the fracture pattern differs substantially from other regions; it could arise from inconsistent and inadequate construction quality control; or it could result from periodic construction events that severely compromise quality on a localized basis. Localized zones of ungrouted fractures can be visualized as a barrier with a certain percentage of its area having openings in it. As with totally ungrouted fractures, the effects on the performance of the barrier can be surprisingly extreme. Figure 5-20 illustrates that the impacts on performance are related to both the open area percentage and the distribution of the open area (i.e., the total open area consisting of a few openings vs. many openings). As shown in Figure 5-20, if only 5% of a barrier is composed of distributed open area, the barrier efficiency can easily be reduced to less than 20% of the effectiveness of a defect-free barrier.

e. Partially Grouted Fractures. Another type of defect is partially grouted fractures. In this scenario, all of the fracture systems have been penetrated, but for one reason or another the fractures are not 100% filled with grout. In general, this type of defect can be visualized as a well-distributed network of small defects, some of which may be interconnected. A typical reason for partially grouted fractures is termination of grouting at less than an appropriate closure criterion, or infilling in fractures. In the absence of other factors, this type of defect tends to result in a uniform, but substantially higher overall permeability than a grouted zone in which all the fractures were completely filled. Site experience should be used to determine the need for absolute refusal. In some cases, costs to reach absolute refusal may outweigh the additional benefits of grouting beyond a lower refusal criterion. The purpose of the grouting program and the geologic conditions should be considered when selecting an appropriate closure criterion.



From Seepage, Drainage, and Flow Nets by H.R. Cedergren (1989). This material is reproduced with permission of John Wiley & Sons, Inc.

Figure 5-20. Flow net study of imperfect cutoffs: (a) cross section, (b) curves.

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CHAPTER 6

Site Investigations

6-1. General.

a. Information Required. Sufficient investigations are required to assess the suitability of grouting, to prepare flow models and the design, to determine the technical procedures and requirements for drilling and grouting, and to estimate the costs. Information should be available on: (1) the nature and characteristics of unconsolidated materials, (2) the geologic structure, stratigraphy, and engineering properties of rock types, (3) the orientation, attitude, and spacing of fracture systems, (4) the fracture condition characteristics including the existence or absence of infilling, (5) the infill material type, (6) the boundaries of differing physical zones within the rock mass, (7) the boundaries of zones with differing permeability, (8) the locations of special features such as faults, highly broken zones, and solution features, and (9) the position of the water table.

b. Application of Information. Site investigation information is the basis for design and for establishing the specific requirements for: (1) hole orientation, (2) hole depth, (3) selection of upstage vs. downstage methods as the anticipated method(s) of operation, (4) stage length, (5) number of grout lines, (6) initial spacing for primary holes, (7) minimum number of holes, (8) and probable final spacing of holes. These items can only be rationally established based on an understanding of the site geology, on the physical characteristics of the rock and its fracture system, and on the project goals and requirements. If the exploration and testing program does not produce an accurate assessment of the site conditions, the basic elements of the drilling and grouting design may prove to be ineffective and may result in major changes during construction. In addition to its use for grouting design, site information is also the basis for determining the extent of required excavations, groundwater control requirements, foundation preparation and treatment requirements, and site access design.

6-2. Related Documents.

a. EM 1110-1-1802, *Geophysical Exploration for Engineering and Environmental Investigations*.

b. EM 1110-1-1804, *Geotechnical Investigations.*

c. EM 1110-1-1901, *Seepage Analysis and Control for Dams.*

d. EM 1110-2-1906, *Laboratory Soils Testing*.

e. ER 1110-1-1807, *Procedures for Drilling in Earth Embankments*.

f. *Rock Testing Handbook.*

6-3. Planning Considerations.

a. Staged Investigations. It is quite common for investigation programs to occur in staged sequences through an initial planning process. However, they can also result from a series of project steps occurring over a period of time that might or might not have been directly related to grouting issues. Depending on the nature of the project, site investigations can occur at multiple points in the project: (1) when there is a need to evaluate potentially problematic conditions, (2) during the Reconnaissance Phase, (3) during the feasibility phase, (4) during the PED phase, (5) during the construction phase, or (6) as part of long-term monitoring in existing projects. Grouting test sections, which are a special type of investigation, are normally performed in the PED or construction phases. The type and level of investigations that are appropriate for each phase differ, and it is essential that investigation data being acquired in each phase are appropriate for each stage of the project. One of the most common mistakes in grouting has been inappropriate staging of investigations, frequently resulting in unsatisfactory designs, deficient construction documents, erroneous cost estimates, unsatisfactory end performance, and/or large contract modifications. For those reasons, it is essential that at whatever point in the project grouting is first considered, the first step must be a comprehensive assessment of all existing site information and proper planning of the scope and timing of subsequent investigation phases.

b. Angle Holes. Extensive use should be made of angle holes to give the maximum information possible concerning high-angle fractures and other geologic features that would not likely be intersected by vertical holes. In many grouting applications, the majority of the holes should be inclined. Typically, holes are inclined 15–30 degrees from vertical. Inclinations larger than 30 degrees are possible, but can be problematic for many types of equipment. The inclination capability of the planned equipment must be considered. Drilling equipment and procedures that are capable of providing the highest quality information on the nature of discontinuities should be used.

c. Rock Outcrops. Where rock grouting is to be performed, examination and mapping of rock outcrops is valuable for assessing joint and fracture orientation and spacing, fracture continuity, and fracture conditions. Direct observation of outcrops, with visualization of drilling and grouting operations to be performed, can be vital for effective design and construction. Where geologic units of interest do not outcrop directly at the site, outcrops of the units should still be located and examined, even if they are remote from the site. With due consideration of the probable spatial uniformity or variability of the geologic units, remote outcrops are sometimes as valuable as on-site outcrops.

d. Direct Exposure of Geologic Features Using Test Excavations. It is not uncommon that geologic features that might significantly impact the design and/or execution of the project are not readily visible for examination nor is their nature fully decipherable from normal site investigation methods. In such cases, a highly effective technique that can be included in one of the site investigation phases is using large test excavations to allow direct examination of the features of interest. There is no prescribed limit on the size of test excavations; they are simply made as large as necessary to acquire the information. In some projects, they have been as large as several hundred feet in length and to depths of greater than 50 ft.

6-4. Site Investigation Equipment.

a. General. Each hole that is drilled should be designed to provide the maximum possible pertinent information. Therefore, a variety of investigative methods may be required (Table 6-1). The most common types of equipment used for subsurface site investigations are sampling equipment for collecting soil materials and diamond rotary drills for obtaining rock cores. However, it is becoming increasingly common to use more sophisticated methods, including down-hole video, geophysical logging instruments, seismic and electrical resistivity surveys, calyx holes for in-situ foundation inspection, borehole caliper surveys, and borehole deviation surveys. It has been found that using down-hole methods in conjunction with conventional methods can enhance the understanding of the site, improve interpretation of pressure test and grouting data, and guide grouting operations.

Table 6-1. General guide to the selection of exploration methods.

Characteristics To Be Determined	Exploration Method
Soil characterization (disturbed)	Standard penetration test Rotary sonic drilling Drive sampling methods Auger sampling Test pits/trenches Cone penetrometer test
Soil characterization (undisturbed)	Shelby tubes Piston samplers Test pits
Soil permeability	Falling head test Constant head test Laboratory permeability testing Infiltration test
Bedrock characterization	Rotary coring Down-hole acoustic and optical geophysics Surface geophysics
Bedrock fracture spacing, aperture, and orientation	Detailed line surveys Outcrop mapping Down-hole acoustic and optical geophysics Oriented core sampling

Table 6-1. (Continued).

Extent of voided areas in bedrock	Rotary coring Destructive drilling methods Surface and down-hole geophysics
Groundwater flow	Down-hole flowmeters Various geophysical methods
Groundwater depth	Piezometer and well data Water levels in borings Various geophysical methods

b. Investigation of Overburden Materials. Equipment for sampling overburden materials includes all normal methods, such as 2-in. split spoons, 3-in. split spoons, Shelby tubes, piston samplers, and bulk samplers. Both the size and the type of sampling equipment are determined by the particle size and density or the consistency of the materials being sampled. Denison-type sampling tools can be used to sample mixed soil and weathered rock materials. Sonic drilling equipment can be advantageous when it is desired to provide continuous overburden profiles and when disturbance of samples and modification of moisture content is not an issue. Other equipment, such as cone penetrometers or vane shear equipment, may be useful in certain situations.

c. Investigation of Rock Materials. Sonic drilling, while capable of penetrating and recovering rock materials, generally destroys the fabric and structure of the rock and is only recommended for detecting the location of the rock surface or in some cases for sampling highly weathered rock where rotary coring will produce very poor recoveries. For virtually all other applications, investigation of rock materials is usually accomplished with rock coring. In most cases, coring equipment consists of triple-tube coring systems with a split inner barrel, which provides maximum protection of the core and allows the core to be examined and logged in its least disturbed condition before it is removed from the inner barrel. The cost premium for triple-tube coring is marginal compared to the total cost of drilling, and its use is recommended because of the quality of the cores. Many types of down-hole logging systems are available for use, and in most cases, the systems offer multiple instrument systems. System capabilities that can be obtained as necessary include optical or acoustic televiewers, borehole deviation survey equipment, gamma loggers, neutron probes, resistivity probes, conductivity probes, flow meters, caliper probes, and others. In addition to information obtained from an individual boring, multiple borings are sometimes used simultaneously for cross-hole seismic exploration and imaging of features of interest. Figures 6-1 through 6-7 show examples of some of the equipment and results.

(Courtesy of Advanced Construction Techniques)

Figure 6-1. Sonic drilling samples from Mississinewa Dam, IN. From top to bottom: alluvial soils (top two bags), highly weathered rock, and moderately weathered rock.

(Courtesy of Gannett Fleming)

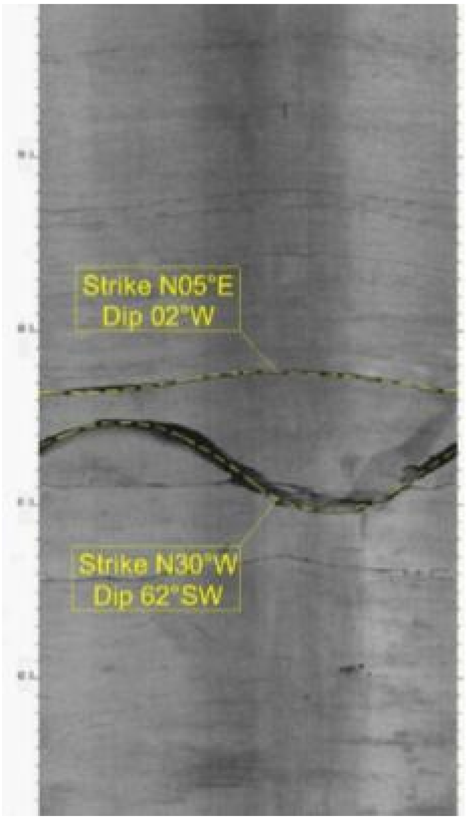
Figure 6-2. Core from a triple-tube split inner barrel from Mississinewa Dam, IN, showing excellent recovery and the intact structure of soil seams and rock materials from a highly weathered rock zone.

(Courtesy of Gannett Fleming)

Figure 6-3. Triple-tube core in slightly weathered rock at Mississinewa Dam, IN.

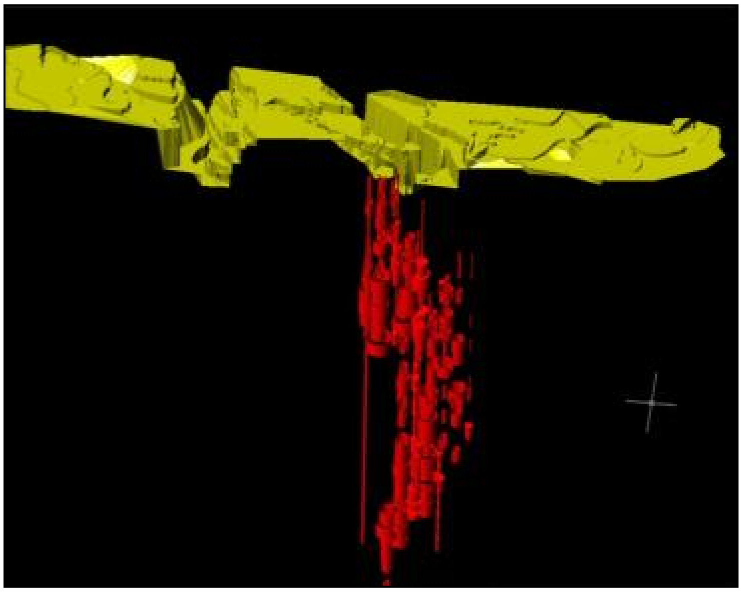
(Courtesy of Advanced Construction Techniques)

Figure 6-4. Borehole logging system for imaging the borehole sidewall and fractures at McCook Reservoir grouting test section, Chicago, IL.



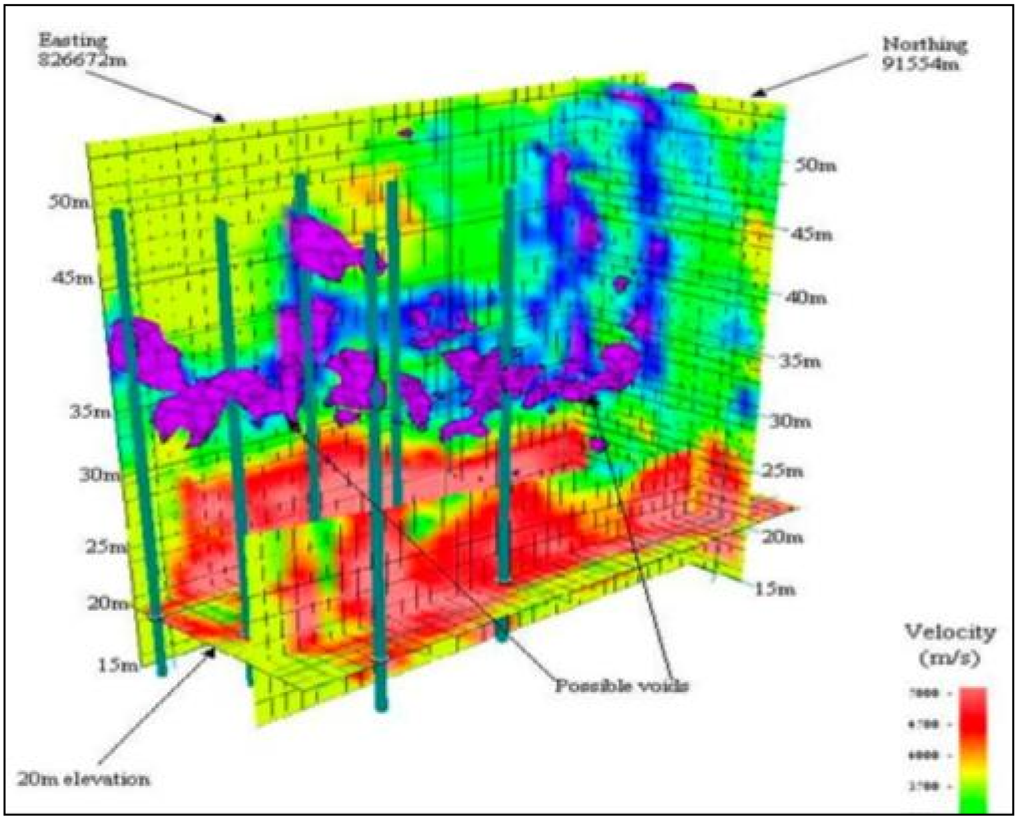
(Courtesy of Advanced Construction Techniques)

Figure 6-5. Digital image of a borehole sidewall and fractures taken with a down-hole logging system at McCook Reservoir Grouting Test Section, Chicago, IL. Post-processing allows determination of the fracture strike and dip.



(Courtesy of Gannett Fleming)

Figure 6-6. Use of exploration data and 3-D computer-aided drafting and design (CADD) to illustrate a karst feature beneath a foundation at Clearwater Dam, MO.



(Courtesy of Gannett Fleming)

Figure 6-7. Seismic tomography image of probable voids in karst in southeast PA.

6-5. Test Drilling and Test Grouting Programs.

a. General. Test drilling and/or test grouting programs are often used in the PED or construction phase to resolve critical technical issues before completing the design or going forward with full production grouting. The scope and content of the test programs are project specific, and they should be carefully designed to resolve specific issues or questions for the proposed drilling and grouting program. The test programs can range in scope from very limited activities necessary to evaluate one or two aspects of the design or construction process to comprehensive, full-scale programs that evaluate virtually every aspect in advance of the construction phase. Test sections can be configured to address issues related to equipment, procedures and methodologies, materials, performance, and/or estimating quantities. Whether one or more segments of test section are required depends on the complexity of the geologic conditions. When necessary, a test section segment should be used in each substantially different geologic environment that will be encountered.

b. Limited Test Sections.

(1) Unlike comprehensive, full test sections that provide evaluation of every aspect of the program, limited test sections are used to resolve issues for only certain elements of the work. Limited test sections are normally performed in the PED phase because of their smaller scope. The number of borings to be used in a limited test section will vary according to purpose, but

rarely will less than 10–20 be found to suffice. Normally, the holes will be located on a single line to provide maximum linear coverage. Limited test sections generally will not provide confirmation or verification of anticipated performance. Examples of limited test sections have included: (1) programs comprising only drilling programs with no water pressure testing or grouting (other than gravity grouting to backfill holes or stabilize zones for drilling) to evaluate suitability of drilling equipment, drilling methodology, drill hole deviation, and drilling production rates; and to identify and evaluate problematic drilling zones in the foundation such as unstable materials, water loss locations, and artesian conditions, (2) drilling with water pressure testing, but without grouting other than gravity grouting to assist with evaluating upstage vs. downstage methodology requirements, obtaining more data on foundation permeability, assisting with planning of appropriate stage lengths, verifying the suitability of termination zones such as at the proposed bottom of the barrier, determining the acceptable primary hole spacing as necessary to prevent connections, and evaluating safe grouting pressures, and (3) drilling with water pressure testing and pressure grouting to evaluate the entire drilling and grouting process for individual holes, which includes evaluating the suitability of proposed grout mixes, obtaining data on grout takes and grouting time to refusal, and evaluating the effectiveness of grout travel and barrier closure on a limited segment of a single line.

(2) Even extensive limited test sections do not test the entire design, nor are they large enough to alter the flow regime sufficiently to induce substantial changes in piezometric levels from which meaningful conclusions might be drawn. Therefore, limited test sections will not be able to provide comprehensive information on the overall performance of a grouting program or to project quantities, particularly when multiple-line programs are being used in the final design.

c. Full-Scale Test Sections. On larger projects, it is common to perform full-scale test sections to fully establish suitable drilling and grouting protocols and allow accurate prediction of both quantities and performance. Full-scale test sections for multiple-line grouting programs often form the basis for establishing the number of lines needed to accomplish the desired performance, and the depth of those lines. As such, they can be extremely valuable for optimizing the design and minimizing overall program costs. Through the use of extensive water-pressure-tested verification holes, both the geometry and the residual permeability of the grouted zone can be confirmed.

(1) A full-scale test section is typically on the order of 5–10% of the final project size or a minimum of 200 ft in length. If sufficient time is available, full-scale test sections can be performed in the PED phase so that all of the lessons learned, and an accurate estimate of quantities, can be translated into the technical requirements of the contract documents. The recent McCook Reservoir project (Chicago District) used this approach with a very substantial test section in advance of the main construction contract. Test sections performed in the PED phase normally: (1) eliminate nearly all uncertainties in the project before full construction, (2) allow comprehensive and thoughtful evaluation of test section data, (3) result in sound designs and performance expectations, (4) provide the basis for sound project documents, and (5) provide well documented and valuable information for preparing competitive proposals for the full construction phase. Potential disadvantages of test sections performed in the PED phase can include: (1) substantially longer lead time for construction, (2) the need for preparation, advertising, and award of multiple contracts, (3) higher costs in certain project elements due to inefficiencies related to partial site preparation and multiple contractor mobilizations,

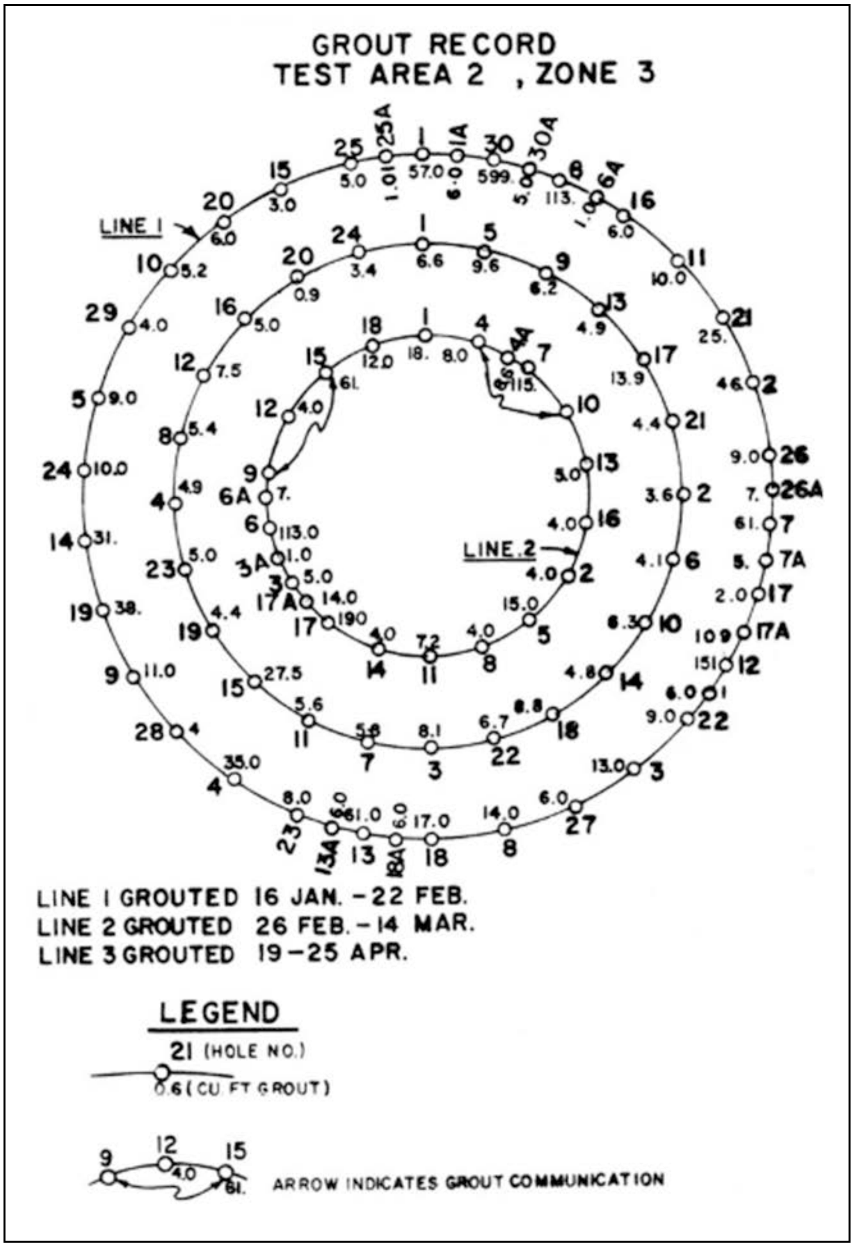
(4) potential loss of benefit of direct experience gained by the test section contractor, and (5) potential performance differences that might result from using somewhat different equipment or materials.

(2) Another alternative is to designate a portion of the project as a test section as the first element of a full construction program. This alternative still offers the very significant advantages of allowing refinement or modification of the drilling and grouting protocols at the earliest possible time, and of providing a greatly improved estimate of costs that will be incurred to complete the program. This approach was used at Mississinewa Dam for the slurry wall pre-grouting program. This approach can offer substantial schedule advantages due to the single contract approach; the potential to begin certain full production elements concurrent with completion of the test section; and a more rapid process of evaluation, assessment, and implementation of results. Additionally, there can be cost advantages resulting from single contractor efficiency and technical advantages in that all of the same equipment, materials, personnel, and procedures used in the test section are carried directly into the production work. If substantial changes in protocols or procedures are found to be necessary based on the test section data, highly effective partnering and negotiations will be required.

(3) A contractual variation known as "early contractor involvement" that can be considered when a test section is part of the full construction effort is to clearly specify a decision point for USACE at the completion of the test section and/or other specified elements of work. The decision point can be structured to allow USACE to either continue with the full program or to terminate the contract (without the need to show specific cause) at that point on the basis that termination is in the best overall interest of USACE. A specified time frame for this evaluation and decision is required. This type of contractual structure provides a safety valve for potential events that might occur, such as: (1) the test section disclosing conditions that differ so markedly from those anticipated that the proposed plan and requirements are not appropriate, (2) unsatisfactory contractor performance, or (3) the inability to successfully use the partnering process and negotiations to accommodate required changes for the full production program.

(4) Regardless of the contractual variations that might be used, it must be recognized in planning, scheduling, budgeting, and contracting that the restricted length of a test section, and the experimental nature of work within it, has a significant impact on productivity and costs. Premiums for full-scale test section work can easily be 50–100% higher than production drilling and grouting, depending on the size and nature of the program. Accordingly, both the test section units and the associated unit costs should be separate items for measurement and payment.

d. Circle-Grouted Test Sections. A special type of full-scale test section is one with a circular configuration surrounding a test well. This type of full-scale test section will provide all of the information described above and, in addition to providing the permeability reduction for each hole series based on the pressure testing data, will also allow an evaluation of grouting effectiveness by running pump tests before and after in the observation well. Despite their advantages, circle-grouted test sections are relatively rare in practice for several reasons: (1) normally, it is desired to incorporate test sections into the alignment of the project program so that the test section expenditure directly contributes to the finished work, (2) frequently, the working space required for a circle-grouted test section may not be compatible with the space available on site, and (3) the pumping well and associated piezometer installations can be damaged by the grouting and may need to be replaced to complete the evaluation program. Circle-grouted test sections have been used by the Jacksonville District at Cerrillos Dam. The information pertaining to this test is located in Design Memorandum No. 4, *Cerrillos Site Geology*, Volume 2 of 5, Appendix C – "Test Excavation and Test Grouting" (September 1983). A version of a circle-grouted test section was also used at the South Florida Storm Water Treatment Area that used different grout types, including portland cement, microfine cement, and chemical grout. Figure 6-8 shows some results of the circle-grouted test section at Cerrillos Dam.



(Courtesy of USACE Jacksonville District)

Figure 6-8. Grout results from a circle-grouted test section, Cerrillos Dam, Puerto Rico.

e. When a circle-grouted test section is deemed to be the proper approach, the following guidelines are suggested:

(1) A test should be made with a radius of about 25 ft, depending on the rock properties. The grout holes drilled around the circumference of the circle should be spaced as planned for the final grout curtain. The holes should be drilled and grouted according to the split-spacing procedures normally followed in grouting.

(2) At least two lines of piezometers should be installed along lines radiating from the test well, which should be drilled at the center of the circle. It is normally of most benefit for a dam project to align the piezometer lines essentially parallel to the anticipated lines of flow from the impounded lake. Good locations for piezometers are one inside the circle, one in the grout curtain, and two outside the circle on each line. Pressure testing data will indicate good zones or elevations in the bedrock to monitor. Multiple tip installations may be required.

(3) The well depth should be somewhat less than the depth of the grout holes. Pump tests should be made before and after the grout is placed. The differences in the permeability between the two tests are a reflection of the effectiveness of the grouting.

6-6. Supplemental Investigations During Construction.

a. Exploration During the Construction Phase. In advance of production drilling and grouting operations, it is common to supplement prior exploration data with additional core borings and water pressure testing. The purposes are to provide confirmation and/or refinement to the geologic knowledge base and the design and/or to explore in more detail specific conditions known or suspected to exist in advance of starting production operations. Exploratory equipment and procedures are also sometimes, but less commonly, used in the grouting verification program.

(1) When possible, it is preferable to perform supplemental explorations in advance of production drilling and grouting because, in general, it requires different equipment and proceeds at a different pace. Equipment for exploration is specialized, it may require special operators, and its use will adversely affect the efficient flow of production drilling and grouting operations. When this type of advance, limited-duration sequencing is acceptable for the project, the contract documents can be structured along conventional unit price items such as per-foot items.

(2) If it is deemed necessary to have the on-site capacity to perform exploration at any time in the program, the contract documents must reflect that requirement, both in the technical specifications and in the measurement and payment sections. When the nature of the project is such that the need for the equipment might arise at any time and the contractor must have the equipment available at any time requested, the contract documents should be structured to provide both a monthly rate for having the equipment available on-site, and per-foot measurement and payment items for its use.

(3) Even after supplemental exploration is complete, every hole drilled and tested as part of the grouting program provides additional information that should be used to improve the understanding of geologic conditions, to identify zones and conditions that are of special concern

and/or may require special treatment, and to verify the efficacy of the grouting program with respect to meeting the design requirements.

b. Variable Factors. Even with thorough investigation programs and well-conceived designs, there are variables that often require refinement throughout the course of the grouting program or specific decisions that must be made by the contracting officer. Specifically, items such as washing duration, grouting pressure, refusal criteria, starting mixes, and combination of stages for grouting typically evolve as the program progresses and as the results are analyzed. Other items, such as final hole spacing, final hole depths, and special treatment zones require specific decisions by USACE based on an analysis of the results with respect to the performance requirements.

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CHAPTER 7

Grout Materials, Mixes, and Their Properties

7-1. Types of Grout.

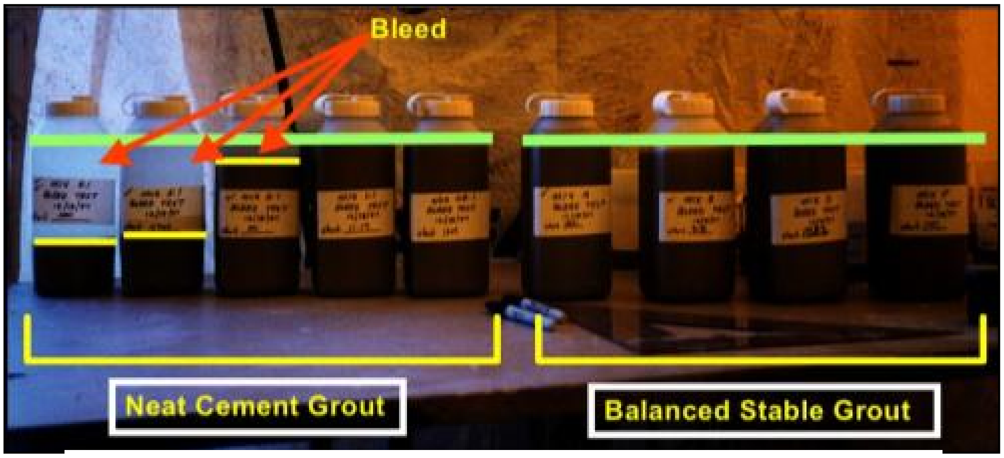
a. General. This chapter provides an overview of the commonly used grouting materials and mixes and provides guidance on material selection for a grouting program.

b. Cementitious Grouts. Cementitious grouts are the most commonly used materials for grouting and can be categorized based on their mobility. For a grouting program to be cost effective, the grout must have sufficient mobility to fill the discontinuities intended for treatment without being so mobile that the grout will flow significantly outside the treatment zone. High-Mobility Grouts (HMGs) behave as a fluid and can be mixed, circulated, and injected with relative ease using normal grout mixing and pumping equipment. HMGs range from pourable to a thick consistency that is just barely able to be mixed and pumped with normal equipment. Low-Mobility Grouts (LMGs) are of a mortar-like consistency and exhibit both plasticity (they stay together when deformed) and internal friction. LMGs expand as a non-permeating bulb of plastic material to either fill open voids or displace soil materials. HMGs are commonly used for permeation grouting of coarse soils and fractured rock, while LMGs are typically used for soil densification (compaction grout) and void filling.

(1) High-Mobility Grouts.

(a) Unstable Suspension Grouts. Until recently, most grouts used for permeation grouting have been unstable suspensions, which, in the absence of continuous agitation, will separate into two distinct phases (water and a very thick suspension). A commonly used definition of unstable grout is that it exhibits more than 5% bleed or sedimentation when placed in a graduated cylinder. At all locations except within the agitator itself, the properties of unstable suspension grouts are in a process of change throughout the grouting process.

(b) Balanced Stable Suspension Grouts. Stable grouts, which began to be used in the 1990s, do not separate into distinct phases in the absence of agitation and do not undergo significant property changes until they begin to take a set. Numerous additives are available to modify the flow and set characteristics (rheology) of cementitious grouts. The impacts to cementitious grout are sufficiently well understood to permit the grouting specialist to obtain the desired properties of a cement-based suspension grout. As implied by the name, balanced stable grouts contain additives to create a stable grout (significantly reduced or zero bleed potential) with the desired rheologic properties that remain nearly constant during injection. Additionally, if silica fume is used in the balanced stable mix formulation, this highly reactive pozzolan interacts with calcium hydroxide liberated during cement hydration and results in property improvement. In concrete, it has been found to appreciably reduce permeability and improve resistance to chemical attack. Figure 7-1 shows some of the differences between unstable suspension grouts and balanced stable grouts.



	Neat Cement Grout Mix (designation is w/c by volume)					Balanced Stable Grout Mix			
Designation	4:1	3:1	2:1	1:1	0.8:1	A (mf)	B	D	F
W/C by Weight	2.7	2.0	1.3	0.7	0.5	1.3	2.0	1.6	1.2
Bentonite (% bwc)	N/A	N/A	N/A	N/A	N/A	0	3.7	3.7	4.1
Welan Gum (% bwc)	N/A	N/A	N/A	N/A	N/A	0.06	0.11	0.11	0.11
Super P. (% bwc)	N/A	N/A	N/A	N/A	N/A	1.2	1.8	1.8	1.8
Marsh (sec.)	31	31	34	41	53	36	34	40	48
Specific Gravity	1.22	1.29	1.40	1.67	1.77	1.41	1.29	1.35	1.43
Pressure Filtration Coeff.	0.16	0.14	0.12	0.09	0.08	0.04		0.04	0.03

(Courtesy of Gannett Fleming)

Figure 7-1. Bleed testing of neat cement and balanced stable grouts with ingredient ratios and physical properties noted for mixes used at Penn Forest Dam, PA. These results represent grout bleed in the static state condition. Note: "bwc" denotes "by weight of cement."

(c) Bulk Fillers in HMGs. Houlsby (1990), Warner (2004), and Weaver and Bruce (2007) all discuss the extensive array of fillers that have been used in HMG formulations. The primary use of bulk fillers is for filling known, large voids to an extent that permits final grouting to be performed with normal HMG mixes. One of the most commonly specified bulk fillers is sand, which can be added to the HMG mix in limited proportions. Sand might be added in a special circumstance when extremely large quantities of the thickest consistency HMG have been used without apparent gain in pressure buildup and when it is simultaneously desired to grout to complete refusal without interruption. In this situation, the HMG might be formulated with sand to be a balanced stable suspension, but have a fluid consistency higher than the thickest HMG. The sand content will increase the viscosity and cohesion and contain larger particle sizes. Therefore, it will reduce the penetration distance into fractures and also limit the size of fracture that can be penetrated by the grout. The sand grain size and content must be limited such that the sand itself stays in total suspension within the carrying capacity of the base HMG grout mixture to which it is added. If the sand does not remain completely distributed in suspension, separation and segregation of the sand can create zones of clean, non-cemented, or poorly cemented sand within the treated area. The practical difficulties of using a mix of this type are considerable: special equipment may be required for mixing; alternate pumping systems may be required because the sand is abrasive materials and because higher pumping pressures are needed (if grouting is to continue uninterrupted when the mix is changed from the thickest HMG, the grout must still be forced down the same diameter grout line that has already been placed into the hole); and a special header system may be required to allow uninterrupted changeover to the supplemental mixing and pumping equipment, which normally must be located at the hole location to minimize line length and to allow direct injection without circulation.

(2) Low-Mobility Grouts. LMGs are grouts that behave as plastic materials. These grouts are not pourable and typically do not behave as a fluid. They have high internal friction due to the high concentration of solids in the mix. The best known application of LMG is for compaction grouting, where the grout is injected into a soil for displacement and/or densification, resulting in a higher modulus and higher strength. Other common uses of LMG are void filling in karst terrain and mine works. LMGs are sometimes used to provide containment barriers within a large feature, which then allows the feature to be thoroughly grouted to completion with HMGs. Figure 7-2 illustrates the typical consistency of LMG.

(Courtesy of Dr. Donald Bruce)

Figure 7-2. Low-mobility grout.

c. Non-Cementitious Grouts. There are times when the desired impact of a grouting program requires the use of materials other than cement. Applications such as structural grouting in soil and control of strongly flowing water commonly lead the grouting specialist to chemical or solution grouts. The following paragraphs describe commonly used non-cementitious grouts.

(1) Chemical Grouts. Often identified as simply chemical grouts, these grouts are more correctly termed “colloidal,” “chemical solution,” or “solution” grouts. Commonly used solution grouts are sodium silicate, urethane, acrylate, and acrylamide. Sodium silicate has commonly been used for structural or water control applications. Urethanes are the material of choice for control of flowing water in structures. Acrylates and acrylamides are highly penetrable in all mediums and form a gel when reacted. These materials are highly effective for water control because of their ability to set almost instantly within variable, but controllable periods of time. However, there are disadvantages, including shrinking and swelling during wetting and drying cycles, and some real and some perceived environmental impacts. Chemical grouts are beyond the scope of this manual. For complete coverage, refer to EM 1110-1-3500, *Chemical Grouting*, *Chemical Grouting and Soil Stabilization* (Karol 2003), and *Practical Handbook of Grouting* (Warner 2004).

(2) Asphalt Grouts. Asphalt grouts or hot bitumen are used in special circumstances to stop rapidly moving water, as in, for example, a large leak into a quarry from a nearby stream or river (Figure 7-3). Asphalt is a solid at room temperature and must be heated to above 275 °F (135 °C) to create a flowable, viscous liquid. The hot bitumen is pumped into the flowing water, where it cools rapidly and thickens, forming a low-strength plug. After the flowing water is stopped or slowed, cementitious grouts are usually employed to increase the permanence of the application. As an alternative, hot bitumen and cement grouts can be co-injected. Co-injection can be performed in the same hole, where mixing occurs at the bottom of the injection pipe, or the two materials can be simultaneously injected into two holes. This method is often more successful than the two-stage injection. When the hot bitumen is co-injected with cement, flash setting of the cement occurs due to the high temperatures. The use of two holes to accomplish co-injection is common and eliminates some practical problems that occur with co-injection in a single hole. At USBR Upper Stillwater Dam, hot bitumen and cement were pre-mixed before pumping, resulting in a higher specific gravity that prevented the mixture from floating when injected below water. Hot bitumen projects require special care due to the temperature of the material being injected, and they require containment of any material that spills on the ground. Although asphalts have been used for decades, it is only in the last decade or so that new concepts and delivery methods have made them reliable and durable.

(Courtesy of Dr. Donald Bruce)

Figure 7-3. Hot bitumen and HMG from co-injection that escaped to the surface from deep karst.

(3) Clay Grouts. Clay grouts are inexpensive grouts created from a suspension of clay minerals and cement. Two parts clay are typically combined with one part cement and water, resulting in a grout that forms a weak solid. Although clay grouts have been reported to have been used successfully in the former Soviet Union (Kipko and Williams 1993) when injected under high pressures, concerns about long-term durability are common, and clay grouts are rarely used in North America. They have been used with temporary success for stream

restoration after long wall mining. Long-term successful use in North America is not documented, and clay grout is not recommended for common applications.

7-2. Process for Selecting Grouting Materials.

a. General. Figure 7-4 shows schematics of various types of grouting applications. The selection of suitable grouting materials requires the answers to several questions:

(1) What medium is being grouted (soil, rock, concrete, or combination, e.g., karst)?

(2) What is the purpose of the grouting (strengthening or modulus reduction, permeability reduction, or water control)?

(3) What grouting methods can be used to achieve the purpose?

(4) How critical is the grout performance?

(5) How permanent must the grouting be?

(6) What is the rate of flow, if any, that must be stopped?

(7) What are the environmental considerations?

(8) What are the cost considerations?

b. Selection of Primary Grouting Material. After the questions above are addressed, candidate grouting methods and associated grouting materials can easily be identified. The primary grouting method and material should be selected to treat the majority of the anticipated conditions. If the desired improvements or results can be achieved with more than one material, the final selection should consider long-term durability and cost. The cost should not be based solely on the material cost, but on the total project cost for using that material.

c. Selection of Secondary Grouting Material. More than one grouting method or type of grout might be appropriate or necessary to achieve the goals established for a project. For example, a grouting program in karst terrain might be developed using HMG as the primary grouting material to permeate the rock fractures. However, LMG might also be provided to treat open or soil-filled solution features. High-flow conditions might require the use of asphalt grouts. A second example is a dam foundation grouting project. If the performance criterion of the completed curtain requires a low permeability (less than 1×10^{-5} cm/sec), then achieving this goal might require a combination of portland cement and ultrafine cement. A typical project with a multiple-line curtain might require that the outside lines be grouted with an HMG mix formulated with a standard portland cement and that the interior line be grouted with an HMG formulated with a microfine cement.

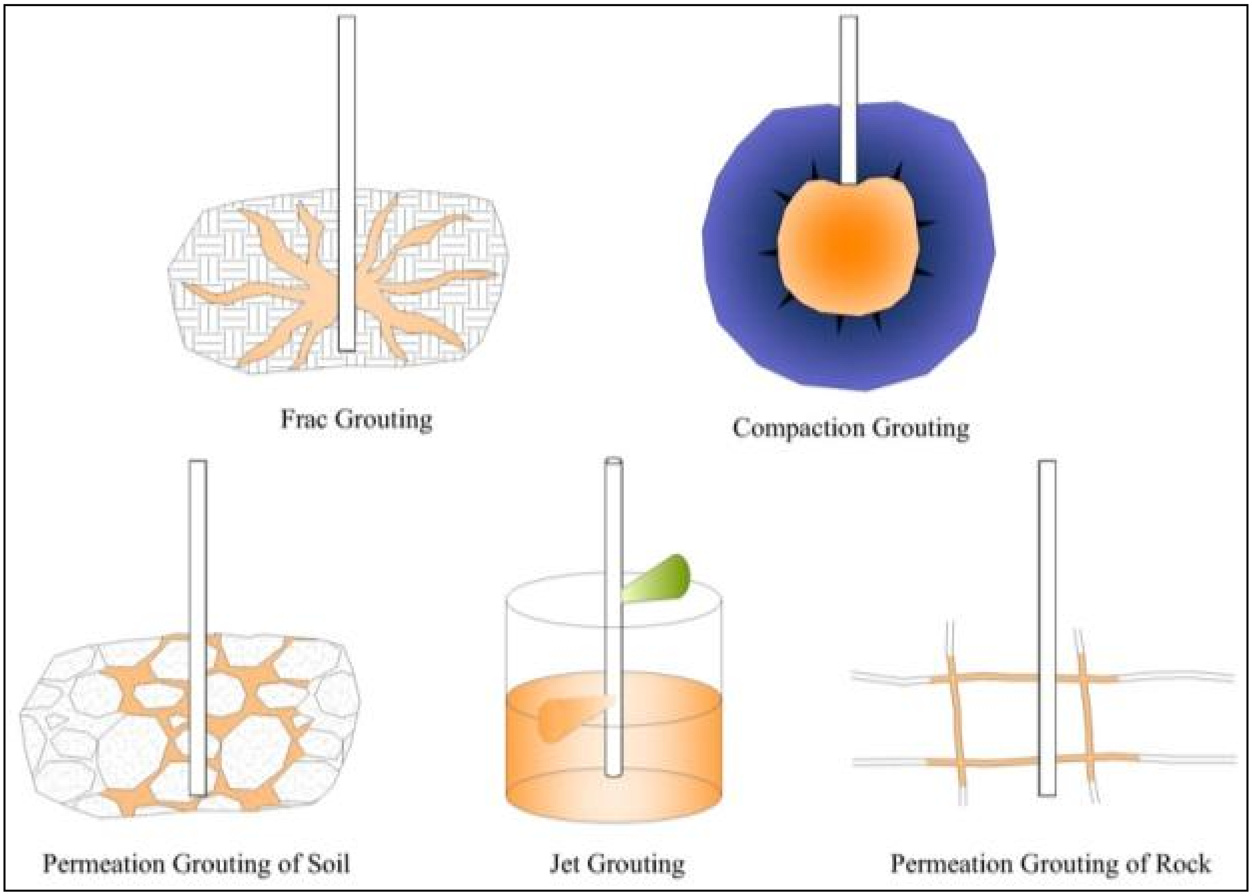


Figure 7-4. Schematic of grouting methods.

d. Guidelines for Common Grouting Applications.

(1) Rock Fracture Grouting. Applications involving grouting of rock fractures to cut off or reduce fluid flow are the most common types of permeation grouting. Permeation grouting is also used for mechanical applications where the purpose of the grouting is to strengthen a rock or soil mass or reduce the potential future deformations. Littlejohn (2003) defines permeation grouting as the introduction of grout into ground without disturbing the ground structure. Rock grouting requires that a hole be drilled that intersects existing fractures in the rock. The fluid grout is then pumped into a zone of rock under pressure. The expectation is that the pressurized grout enters the fracture at the intersection with the hole and fills the network of fractures connected to the intersected fracture in the proximity of the hole. Grout holes are typically drilled and grouted using the split-spacing method until areas of overlapping influence are created. Stable cementitious grouts are almost always the material of choice for routine rock grouting applications. Figure 7-5 provides guidance on selecting the appropriate grouting material for applications in fractured rock. On rare occasions, a solution grout might be used in a highly specialized or critical application where a near-zero residual permeability is required. Nearly all of the grouting between 1930 and 1990 was performed using unstable cement grouts, but due to concerns about bleed and variable rheology, the use of portland cement stable grouts has increased and is required on USACE projects. If the fracture size requiring filling is too

small for penetration with portland cement, ultrafine cements with a smaller average grain size are available.

(2) Soil Grouting. While only permeation grouting is used in rock, a variety of grouting methods and materials are commonly employed in soil grouting. The available grouting methods include permeation, compaction, mixing or replacement, and fracture grouting.

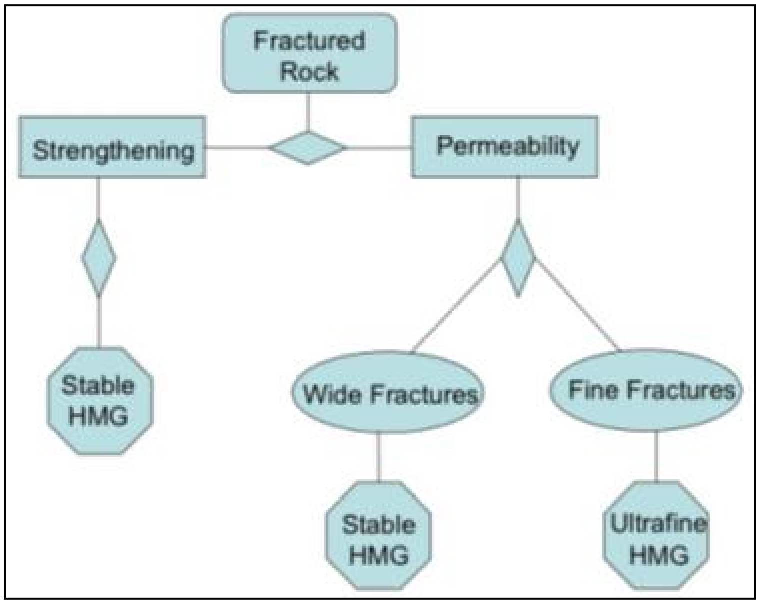
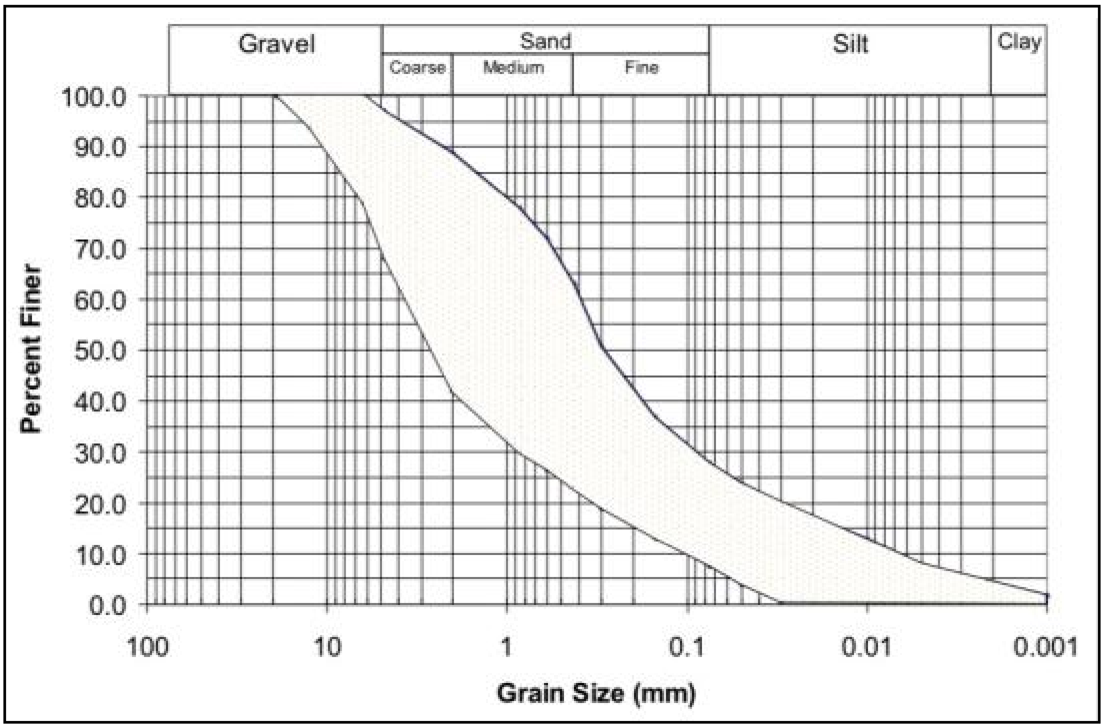


Figure 7-5. Grouting materials for use in fractured rock.

(a) Permeation Grouting. Due to cost considerations, permeation grouting of soil is limited to soils that can be permeated relatively rapidly. As a rule of thumb, permeation grouting can be considered possible for soils with less than 15% fines. This percentage is not exact and will vary, depending on the coefficient of uniformity of the soil, the plasticity of the fines, and the grouting material. When grouting of soils at the upper limits for fines content, permeation will be very slow. The type of grout used for permeating soils will vary, depending on the soil type and the application. Grouting of clean coarse sands and gravels can be accomplished with portland cement mixes. Permeation grouting of most natural sands requires ultrafine cements or a solution grout. The ultrafine cements have the advantage of higher achievable strength, lower cost, and greater durability for permanent applications. At times, strength and permanence are not crucial. For example, for soil tunnel applications, permeation grouting might be used to minimize water infiltration and increase the stand-up time of the soils. In this case, a moderate to low strength might be desirable so as not to impact the speed of the tunneling operation, so a sodium silicate grout might be a better choice.

(b) Compaction Grouting. Compaction grouting requires the use of an LMG. In compaction grouting, soil improvement is achieved by gradually injecting a growing mass of grout, which displaces the adjacent soils and results in densification. Soil improvement from compaction grouting is identical to applying static compaction to a fill. However, it also results in a pattern of embedded elements of substantially higher strength, which further modifies the behavior of the soil mass. The bearing capacity of the soil is increased, potential settlement is reduced, strength is increased, and permeability is reduced. Compaction grouting is the method of choice for repairing damage due to settlement and mitigating future damages by decreasing future settlement by densification. When used for compaction grouting, LMG consists of aggregate, cement, and minimal water. The requirements of the aggregate for compaction grout are well documented in various publications (Warner 1982, 2003, 2004). To be effective in densifying the soil without losing control and possibly causing damage by hydrofracture and uncontrolled heave, the grout mixture must stay together and act as a growing solid in the soil. To act in this manner, the grout must exhibit internal friction, and the rheology of the grout must be closely controlled. The data shown in Figure 7-6 indicate the recommended gradation band for the aggregate component. The gravel-size particles should be rounded, if available. If rounded aggregates are not available, the finer side of the envelope should be used (Warner 2004). The aggregate is commonly obtained from local borrow sources. Pre-blended aggregates can also be obtained from some aggregate producers. Cement content can vary from 0 to 12% by volume (Warner 2004). The water content is low, just enough to allow pumping. Many specifications require that the slump be less than 1 in.



(Courtesy of Warner 2004)

Figure 7-6. Recommended aggregate gradation for compaction grout.

(c) Jet Grouting. Mixing or partial replacement of soils can be achieved using deep mixing methods or jet grouting. Deep mixing uses mechanical means to mix the in-situ soils with grout. Shallow and deep mixing are beyond the scope of this manual. Deep mixing methods are well documented in Federal Highway Administration (FHWA) Publication No. FHWA-RD-99-138 (FHWA 1999). Jet grouting involves the injection of grout (often assisted by air and/or water jetting) under high pressure to mix the in-situ soil with cement to form "soilcrete." Three methods of jet grouting are commonly recognized: single tube, double tube, and triple tube. The number of tubes or rods is consistent with the number of fluids used in the process. Single tube uses only grout to excavate and mix the in-situ soils. Double tube incorporates a shroud of air around the grout to assist in the excavation process and to concentrate the grout jet. Triple tube uses water encapsulated in a shroud of air for excavation and injects grout separately near the bottom of the drill string to create the soilcrete. Depending on the jet grouting method selected and the withdrawal rate, jet grouting results can vary from minor mixing to almost complete replacement of the in-situ soils with grout. Jet grouting can be applied to any soil type. The achievable parameters of the soilcrete are greatly impacted by the soil type. Higher strengths and greater diameters can be achieved in clean sand or gravel. As the soil becomes finer and more plastic, the achievable strengths are reduced and the diameter of the soilcrete columns is reduced (assuming that the installation method is constant). Highly plastic clays are especially problematic, as they are difficult to break down and mix with the grout. Strengths over 2000 psi have been achieved in sands and gravels, while 300 psi might be the maximum achievable strength in a plastic clay. Large cobbles or boulders in the formation result in the high-pressure grout jet being deflected and can result in untreated soil areas or shadows behind the large-diameter obstruction. The grout material used in jet grouting is portland cement. The grout mix design is typically a neat cement grout prepared at water-to-cement ratios by weight ranging from 0.6 to 1.2.

(d) Hydrofracture Grouting. The premise of hydrofracture grouting, or “frac” grouting, as it is commonly called, is to inject grout under pressure, causing tensile failure of the soil, resulting in the injection of grout lenses within the soil. These pressurized lenses of grout densify the soil by plastic deformation in the vicinity of the lenses and also reinforce the soil mass due to the higher strength of the grout in comparison to the soil (soil strength is commonly measured in lb/ft^2 (psf) versus grout strength in psi, a difference of two orders of magnitude). In clay soils, there can also be a chemical reaction of the grout with the clay minerals (cation exchange) in the immediate vicinity of the lenses, which increases the strength. Stable grouts are used in frac grouting, so future deformations due to bleed water dissipation do not occur. In an underconsolidated or normally consolidated soil, the vertical effective stress is greater than the horizontal effective stress. In theory, for frac grouting in an underconsolidated or normally consolidated soil mass, the initial fracture that propagates is vertical. In subsequent re-injections at the same location, the stress state of the soil changes, and the angle of the fractures moves from vertical toward horizontal. The process continues until the fracture is horizontal and the available deformation within the soil mass has been maximized. At this point, heave or lifting of the mass is initiated and the injection is terminated. Under ideal circumstances, the result of the multiple injection process is a network of grout lenses in all directions, each of which has caused densification and which leaves in place a skeleton of stronger material that is more resistant to further settlement. The maximum degree of densification for both frac grouting and compaction grouting is controlled by surface heave, which occurs when the existing overburden pressure is exceeded by stresses created in the soil during the application. With frac grouting, the magnitude of the deformations is difficult to quantify, as the length of the induced fracture is unknown. While the benefit of the matrix of grout lenses is easy to envision, it is not quantifiable in engineering parameters. Therefore, although successful projects using frac grouting have been documented, no known procedures exist to design such a program. Another common use for frac grouting is compensation grouting, whereby frac grouting is initiated to compensate settlement. Compensation grouting is common in the tunnel industry to remediate settlement of structures induced by tunnel excavation in soils.

(3) Structure Grouting. Grouting in structures can take many forms and can be performed for a variety of purposes. Structural grouting is commonly performed to fill voids or cavities resulting from the original construction, such as honeycombed or segregated concrete. Grouting might also be performed to fill voids in masonry structures caused by incomplete mortar coverage, or to fill cracks and joints. The most recognized form of structure grouting is waterproofing of structures such as basements, concrete tanks, conduits, and dams. Other forms of structure grouting include contact grouting around buried structures, slabjacking, rock and soil anchors, post-tensioned tendons, conduit abandonment, and sliplining. Grouting is also a common element in the construction or rehabilitation of underground structures such as tunnels and shafts. Grouting materials used for structure grouting include cementitious grouts (portland and ultrafine), epoxies, urethanes, acrylates, and low-mobility grout. Figures 7-7 through 7-9 show examples of urethane grouting used to remedy permeable concrete resulting from improper consolidation.

Figure 7-7. Leaky grout cap caused by improper concrete consolidation.

Figure 7-8. Urethane grouting used to waterproof the grout cap.

Figure 7-9. Concrete cores before and after grouting with urethane grout.

Available literature covers structure grouting in detail. Warner (2004) dedicates several chapters to the grouting of structures. The most recent edition of the *Post Tensioning Institute Recommendations for Pre-stressed Rock and Soil Anchors* should be referred to for materials and procedures used in grouting soil and rock anchors. Henn (1996, 2003) provides guidelines for grouting underground structures and for backfilling and contact grouting of tunnels and shafts.

(4) Karst Grouting. Grouting in karst environments can be extremely challenging due to the wide variety of conditions that might be encountered. Fracture conditions can vary from clean, relatively tight rock fractures, to open solution-widened fractures, to completely soil-filled fractures. Depending on the degree of solutioning, cavities might also exist, varying in size from small openings to large caves. The cavities might be dry or wet and open or filled. Filling material can consist of collapsed roof rock, coarse outwash, fine-grained residual soils, or a combination of these. Flow regimes in karst can vary from nearly stagnant to high velocity. The multitude of conditions that might be encountered require that a grouting program in karst be planned to permit flexibility to react to differing conditions. Grouting materials used in karst include HMG, sanded grouts, LMG, pre-placed aggregate, flowable fill, conventional concrete, sodium silicate co-injected with HMG, and hot bitumen combined with HMG. In flowing water conditions, accelerators and anti-washout agents might be used in addition to standard admixtures. Plastic disks and geosynthetic fibers have also been added to grouts to assist in plugging large fractures. Grouting of the rock fractures and small cavities in karst are generally consistent with any other rock grouting program, except that some higher-cohesion grouts might be employed and that resting of stages might be performed. The difficulty increases as the size of voids increases and as the velocity of flowing water increases. Where voids are large, the best material to use is the cheapest material that can successfully be used to fill the voids effectively. After the voids are initially filled, they can subsequently be tightly grouted with HMG. The grouting specialist must consider the available drilling equipment and achievable hole sizes when considering materials. Three-inch-diameter holes are adequate for placing HMG and LMG materials. However, flowable fill and conventional concrete require significantly larger holes for successful delivery. Co-injection of sodium silicate or hot bitumen and cementitious grouts are reserved for high-volume flow conditions; their use requires an experienced contractor. These materials are typically employed only after other grouting methods and materials have been found or judged to be inadequate to treat the encountered conditions. Walz et al. (2003) described case histories of successful installation of grouted cutoffs in karst, and Bruce et al. (2001) documented grouting methods to deal with massive flows.

The USACE has many dams located in karst. Recent remediation projects in karst have been completed at Patoka Lake, Mississinewa Lake Dams in Indiana, Walter F. George Dam in Alabama, Clearwater Dam in Missouri, Wolf Creek Dam in Kentucky, and Center Hill Dam in Tennessee. Most of these projects involve composite cutoffs (Bruce and Dreese 2005), in which grouting in combination or in advance of positive concrete cutoffs have or will be employed. Figure 7-10 shows a rock core obtained from Mississinewa after grouting. Figure 7-11 illustrates various treatments for karst and large void grouting.

Figure 7-10. HMG-filled fracture. The grout is indicated in purple by phenolphthalein.

(5) Large Open Void Grouting. Large open voids requiring grouting include man-made openings in rock, such as abandoned mines and shafts. These are generally treated in similar fashion to voids in karst. Abandoned mines are often filled to reduce the potential for future subsidence or inhibit the flow of acid mine drainage. Where the existing mine is crossed by a highway or pipeline, the extent of the mine to be filled is often limited to the area in the vicinity of the surface infrastructure. This often requires that a bulkhead be created on either side of the zone to be filled or that the material used to fill the feature is simply placed to its angle of repose. Pre-placed aggregate followed by intrusion with HMG, flowable fill, conventional concrete, and LMG have all been employed for treatment of large underground voids. Complete filling with these methods is difficult near the roof of the opening. If complete filling is required, injection of HMG near the roof might be required.

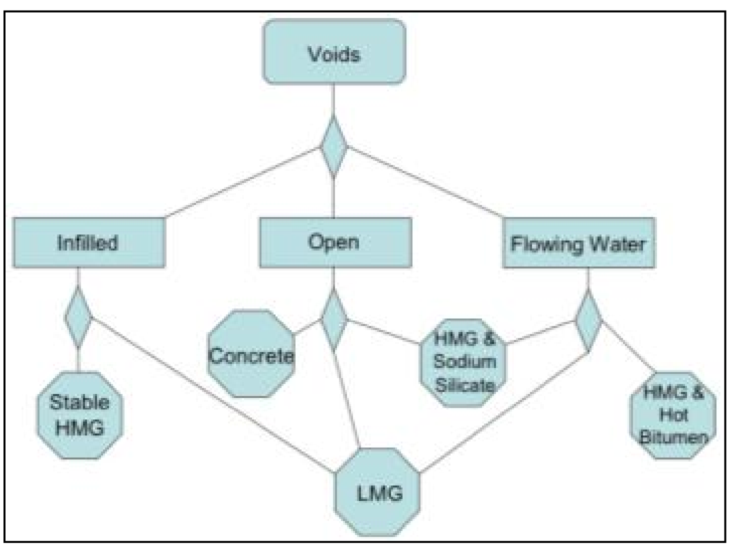


Figure 7-11. Recommended methods and materials for grouting of karst and large voids.

7-3. Basic Elements of Cement-Based Suspension Grouts.

a. Portland Cements. Portland cement is the most widely used material in grouting. ASTM C219 (2007), *Standard Terminology Relating to Hydraulic Cement*, defines portland cement as: “a hydraulic cement produced by pulverizing clinker, consisting essentially of crystalline hydraulic calcium silicates, and usually containing one or more of the following: water, calcium sulfate, up to 5% limestone, and processing additions.” The term “hydraulic” used in this definition indicates that the material reacts when combined with water. ASTM C150 (2012c) provides detailed specifications of the chemical requirements for each type of cement, along with physical requirements including the fineness, setting time, and strength. The fineness defined by ASTM is the Blaine fineness and is a measure of the specific surface area available for reaction. This method is based on air permeability and is expressed in units of cm^2/g or m^2/kg. ASTM does not specify any grain size distribution limitations or requirements. Ordinary portland cement typically has a Blaine fineness of 300–500 m^2/kg and a maximum particle size on the order of 45 microns. The Blaine fineness may be higher for Type III cement and lower for Type IV cement where the heat of hydration is controlled, but a higher Blaine fineness does not necessarily indicate a smaller maximum grain size. Detailed discussion of the hydration process can be easily found in available literature. The most important items to recognize are that the hydration process begins immediately upon the addition of water, and the hydration process results in the creation of calcium hydroxide, which is readily soluble and will adversely react with sulfates. Therefore, high concentrations of sulfates in the mix water or groundwater will cause deterioration of the hydrated cement and will have a negative impact on the grout performance. The five types of cement defined by ASTM are:

(1) Type I. This is the general purpose cement and is the most commonly available and used type of cement in the construction industry. Type I cement produces a high heat of hydration and is subject to deterioration from sulfate attack where moderate to severe sulfate concentrations are present.

(2) Type II and Type II (MH). Type II provides moderate sulfate resistance and is used when nominal resistance to sulfate is required. Type II (MH), or Type II moderate heat, is used when a lower heat of hydration is also required. For grouting, a lower heat of hydration might be considered in a hot environment to provide an extended set time.

(3) Type III. This is commonly identified as high early strength cement. Although not specified as such by ASTM, Type III cement is usually the finest ground of the readily available portland cements in terms of grain size distribution. For that reason, Type III cement should be specified for better penetration and quicker strength gain unless low heat of hydration or sulfate resistance is required.

(4) Type IV. Type IV provides lower heat of hydration than Type II (MH), which also results in a slower strength gain. Type IV is rarely used in grouting unless low heat of hydration is required due to climate or temperature-sensitive buried features. Similar properties can also be achieved using Type II (MH) cement with supplemental cementitious materials.

(5) Type V. This type provides higher resistance to sulfate attack than Type II. Type V cement is typically used only in severe sulfate environments.

b. Other Types of Cements. Other types of cement and cementitious materials that may be used include:

(1) Finely ground cementitious materials are used for achieving better penetration. These cementitious grouts are known as ultrafine or super-fine cements. The Blaine fineness for ultrafine cement is greater than 800 m^2/kg, and the maximum particle size is on the order of 10 microns.

(2) Supplemental cementitious materials (SCMs) may be used, depending on the required grout properties or the availability of portland cement. SCMs include natural pozzolans, fly ash, ground granulated blast furnace slag (slag), and silica fume. Physical and chemical standards for natural pozzolans and fly ash are provided in ASTM C618 (2012e), standards for slag are provided in ASTM C989 (2012f), and standards for silica fume are provided in ASTM C1240 (2012b). Since SCMs are waste by-products of different processes, use should be based on experience and on an understanding of the materials and their sources. Typically the Blaine fineness and the maximum particle size for fly ash and slag are similar to ordinary portland cement, but the properties may vary significantly among different sources. Silica fume is extremely fine, with a fineness of about 20,000 m^2/kg and a maximum particle size of 1 micron (PCA 2011).

(3) Blended hydraulic cements are portland cement that is pre-blended with pozzolans or slag. Physical and chemical standards for blended cements are provided in PCA (2011).

(4) Several other specialty cements include air-entrained cements, expansive cements, calcium aluminate cements, plastic cements, masonry cements, rapid-setting cements, and oil well cements (Warner 2004). These cements are not appropriate for the majority of grouting applications, but in some specific instances their properties may be desirable and their use should be further explored.

(5) Wherever grouting must be performed, it is recommended that soil and subsurface water conditions be assessed for the potential for alkali and sulfate attack. These conditions are prominent in the presence of or proximity to seawater or brackish water, or where sulfates or alkalis are present in the subsurface due to contamination or as a natural product of the environment. Type V cement is the most resistant to sulfate attack under these conditions.

c. Mixing Water. Water is a major component of any grout mix. The minimum percentage of water required for complete hydration of cement is 30% when expressed by weight (45% by volume) of cement. Greater quantities of water are used in grouting than in concrete, as the water is the carrier of the products in suspension during injection. The simplest manner to specify or control the quality of the mix water is to require the use of potable water. Potable water can be used in grout without any testing. Obtaining potable water on some grouting sites is not always convenient or economical (Figure 7-12). Where potable water is not readily available, groundwater or surface water can normally be used if proper controls are put in place. Groundwater or surface water testing is required to demonstrate that the water meets the requirements of ASTM C1602 (2012d), *Mixing Water Used in Production of Hydraulic Cement Concrete*. The samples prepared for testing to demonstrate compliance should be a typical grout rather than conventional concrete. Optional chemical requirements should be specified when appropriate, such as where sulfate exposure, alkali reactive aggregate, or reinforcing steel or anchors are present. Additionally, the specific gravity of the surface water should be tested regularly to document that the water quality has not changed; it should also be tested whenever a change in water quality is suspected.

7-4. Behavior of Cement-Based Suspension Grout.

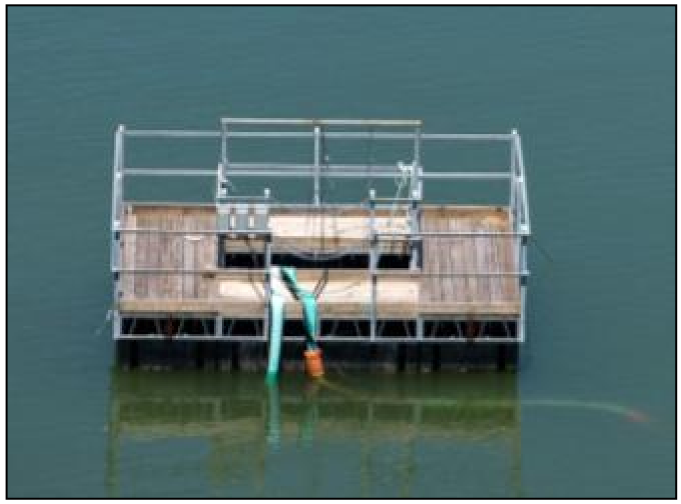
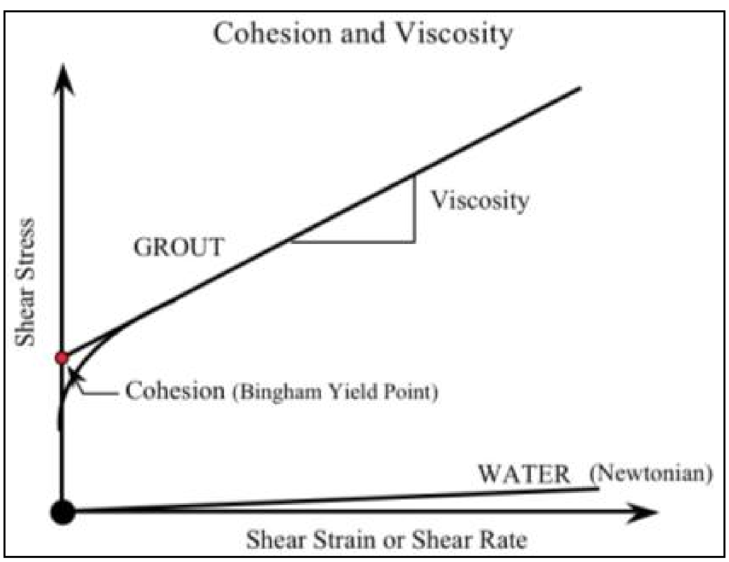

(Courtesy of Advanced Construction Techniques)

Figure 7-12. Floating water intake for a grouting project during a flood at Mississinewa Dam, IN.

a. General. It is widely accepted that cementitious suspensions behave as Bingham fluids. Some grouts also behave as visco-plastic fluids (Weaver 1991). Figure 7-13 illustrates the behavior of a Bingham fluid in comparison to water, which behaves as a Newtonian fluid (Figure 7-13). It is helpful to think of the shear stress (or Y-axis shown in Figure 7-13) as being a function of grouting pressure and the x-axis or shear rate as the flow rate of the grout. The most important aspect of a Bingham or visco-plastic fluid is that it has a yield point or cohesion that is equal to the minimum pressure required for the grout to move and a viscosity that is a measure of the incremental increase in resistance to flow (i.e., additional pressure required) for a change in the pumping rate. For some grouts, the viscosity does not remain constant as the flow rate increases. Grouts where the viscosity or slope increases with increasing flow rate are identified as exhibiting shear thickening. Shear thinning behavior refers to a grout for which the viscosity reduces with increasing flow rates. Grouts for which the viscosity changes with shear rate are identified as visco-plastic.



(After Houlsby 1990)

Figure 7-13. Bingham fluid behavior.

b. Yield Point. The yield point defines the pressure required to start grout flowing and the pressure at which grout stops moving (refusal) within a fracture. At any stress greater than the yield point, grout will continue to flow in the fracture. For a constant fracture width, the rate at which grout flows in the fracture is controlled by the pressure in excess of the yield point and the grout viscosity. As grout moves away from an injection hole, the pressure decreases due to head loss within the fracture. As the pressure experienced (or “felt”) by the grout farthest from the hole reduces, the flow rate continues to drop.

7-5. Desired Properties of Cement-Based Suspension Grout.

a. General. According to Wilson and Dreese (1998):

> The perfect grout for rock foundations of dams would have zero cohesion, since it would then penetrate all fractures in exactly the same way as the water that we are trying to impound. It would also have: (1) a low viscosity to permit fast penetration rates, (2) instant set at pre-defined, controllable time intervals, (3) zero shrinkage, (4) and a strength and durability similar to concrete. Obviously, that perfect grout does not exist. Solution or chemical grouts come closest to meeting the properties of this perfect grout. However, due to cost, durability, and/or environmental concerns, cement-based suspension grouts are normally the material of choice for dam foundation grouting.

The perfect grout cannot be developed using a cement-based grout because the resulting grout would be a suspension and not a solution. However, the achievable desired properties of a cement-based suspension grout are:

b. Zero bleed, so that the fractures or voids that are filled during the injection remain filled.

c. High resistance to pressure filtration, so that the water-to-solids ratio remains constant during the injection.

d. Water repellent, so that the grout suspension does not dissociate when injected into water.

e. Resistant to particle agglomeration due to electrostatic and chemical interactions, to deter the development of macro-flocs (increase in grain size) from the hydration process during the period of injection.

f. Cohesion values consistent with the desired penetration distance. A low cohesion is desirable to maximize penetration from a given hole and limit the total drilling footage required. In karst or very high permeability formations, a mix or mixes with a high cohesion might be desirable to keep the travel distance within the intended treatment zone.

g. Viscosity compatible with the pumping pressures and low enough that an economical injection rate is achieved.

h. Thixotropic, so that the grout is resistant to wash out after placement.

i. Well-graded grain size distribution of the cured grout (a well-graded structure of the cured grout reduces the matrix permeability and thus improves durability).

j. Long-term durability.

7-6. Unstable Suspension Grouts.

a. Introduction. The term "unstable suspension grout" is widely accepted to mean a grout with 5% or more bleed.

b. Neat Cement Grouts. The simplest and most basic recipe for grout is to add cement to water and mix them together to produce a neat cement grout. In North America, neat cement grouts have traditionally been defined on a volumetric basis.

c. Commonly Used Additives. The majority of past projects employing neat cement grouts did not incorporate any additives into the mix. However, in the 1980s, the U.S. Bureau of Reclamation began to take advantage of superplasticizers to lower the water-cement ratio. It was also fairly common at the time within the USACE to employ bentonite as an additive to neat cement grouts to reduce sedimentation. During the mix design process, careful attention should be given to the effects that additives like superplasticizers have on grout mixes. It is suggested that grout mix formulations be evaluated through laboratory testing.

d. Fluid Properties. This lack of stability is evident in the rapid occurrence of sedimentation or bleed when unstable cement grouts are not being continuously sheared, and in their low resistance to filtration (separation of water from the solids in suspension) when under pressure. Figure 7-1 shows the bleed that can occur for various water-to-cement (w:c) ratios for neat cement grouts and balanced stable grouts in the static condition. Houlsby (1990) also presents a diagram showing the percentage of bleed water from grout mixes with various water-to-cement ratios.

e. Setting of Grout. Set times for neat cement grouts depend on the type of cement and the water-to-cement ratio. Additionally, the set time may be impacted by the amount of water forced into adjacent fractures as a result of pressure filtration or possible water losses into the porous rock mass. Grouts with a water-to-cement ratio by weight of approximately 1.3:1 or less prepared with Type III cement generally set in less than 8 hrs.

f. Batching by Volume or by Weight. Volumetric batching uses the volume of material to quantify mixed materials, typically only water and cement. For example, a 2:1 mix (w:c ratio by volume) contains 2 ft^3 of water and 1 ft^3 of bulked dry cement. Conveniently, one 94-lb bag of cement equals 1 ft^3 of bulked dry material and 0.5 ft^3 of solid material (absolute volume). The standard specific gravity of cement is 3.15. Therefore, 94 lb of cement with a 3.15 specific gravity represents 0.5 ft^3 (94 lb divided by 3.15 times 62.4 lb/ft^3). As a result, a 2:1 mix by volume includes 2 ft^3 of water and 0.5 ft^3 of cement and will yield 2.5 ft^3 of grout. Batching by weight simply means that the weight of materials is considered instead of volume. For one 94-lb bag of cement, the required weight of water for a 2:1 mix (w:c ratio by weight) is 188 lb (94 lb times two). In this mix, the volume of water is 3.0 ft^3 (188 lb divided by 62.4 lb/ft^3) and the mix will yield 3.5 ft^3 of grout. The difference in the water-to-cement ratio given volumetric batching and weight batching is demonstrated by comparing these two 2:1 grout mixes. The water-to-cement ratio based on weight for the volume-batched grout is 125 lb of water (2 times 62.4 lb/ft^3) to 94 lb of cement and is 1.33. The water-to-cement ratio of the weight-batched grout is 2.0 (188 lb to 94 lb). It is important to identify whether grout mixes are based on volume or weight.

g. Grout Behavior in Rock Fractures. The relatively rapid occurrence of sedimentation and low resistance to pressure filtration in an unstable grout results in unpredictable behavior within rock fractures. This low resistance to pressure filtration results in the mix water separating from the grout when the flow velocity decreases and/or when placed under a moderate pressure. This results in changes in the grout rheology as the grouting application is in progress.

h. Water Separatin. As the water separates from the grout, it moves more rapidly through the fracture due to its significantly lower viscosity. This reduction in water content in the grout results in the remaining suspension becoming thicker as more and more water escapes. Figure 7-14 illustrates how unstable grouts are envisioned to behave when injected into a rock fracture. Some authors believe that this separation of the water from the suspension results in the grout acting first as a Newtonian fluid and then progressing to a Bingham fluid with internal friction (Gause and Bruce 1997, Chuaqui and Bruce 2003). An alternative explanation is that the behavior is different at any given time and place during an injection, depending on the aperture of the fractures and the distance from the point of injection. The behavior is likely Newtonian in finer fractures, where a filter cake forms at or near the borehole walls and only water is being injected; Bingham behavior likely occurs in the wider fractures until such distance is reached that the shear rate in the fracture decreases and the sedimentation process begins. Once the sedimentation process begins in the fracture, the grout quickly develops internal friction and refusal will develop rapidly. The effects of segregation, groundwater, bleed during the setting process and consolidation, and fracture geometry all make the final grout properties uncertain.

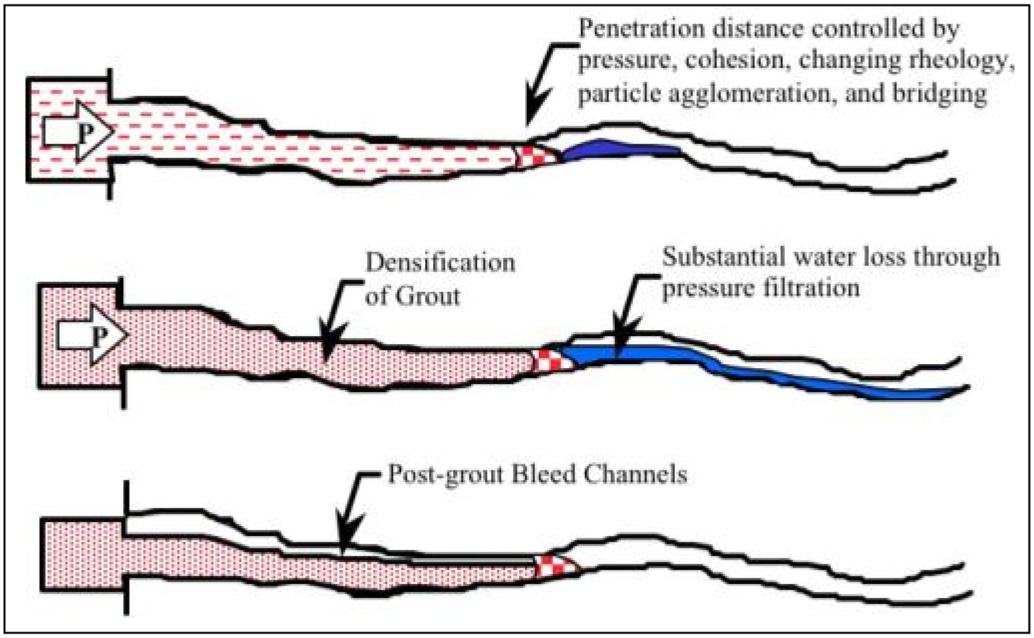


Figure 7-14. Grouting theory for unstable cement grouts.

7-7. Balanced Stable Grouts.

a. General. Admixtures can be used to improve bleed characteristics, cohesion, penetrability, durability, and workability characteristics and therefore should be considered in grouting projects requiring HMG. The number of admixtures and desired properties will vary, depending on the application. Experienced specialists within USACE and private industry, and published project data, are available to assist with preliminary mix proportioning, field mix testing, and troubleshooting.

b. Definitions. The term “stable grout” was previously defined as a grout mixture that exhibits less than 5% bleed. The term “balanced grout” refers to a grout mixture that is formulated to provide the desired rheological properties that also remain constant during the injection process. Balanced stable grouts are grouts that exhibit minimal bleed with desired properties and a rheology that remains constant throughout the injection process since the pressure filtration coefficient is low.

c. Commonly Used Additives and Admixtures. The use of additives in cement-based suspension grouts can improve the rheological properties of a grout. Each additive is used to improve one or more properties of the grout, although, unfortunately, the additive may improve one property while adversely affecting other properties. The grout mixture proportioning should be conducted by an individual experienced with the use of admixtures and proportioning to achieve the required properties. Table 7-1 lists common additives and admixtures, identifies their beneficial and adverse effects, and provides other comments pertinent to their use.

Table 7-1. Common grout additives. (After Wilson and Dreese 1998.)

Additive	Beneficial Effects	Adverse Effects	Other Comments
Fly ash Class C or Class F	• Improves grain size distribution of cured grout. • Inexpensive filler with pozzolanic properties. • Can be used as a replacement for some of the cement and reacts with the calcium hydroxide resulting from the cement hydration process. • Increases durability and resistance to pressure filtration.	• Increases cohesion and can be used to increase viscosity.	• Fly ash is a waste product and the properties vary depending on the source.
Bentonite	• Reduces bleed and increases resistance to pressure filtration. • Slight lubrication and penetrability benefits.	• Increases viscosity and cohesion. • Weakens grout.	• Should be added as pre-hydrated suspension.

Table 7-1. Continued.

Silica Fume	• Fine-grained powder that improves pressure filtration resistance and reduces bleed. • Improves water repellency and enhances penetrability. • Improves grain size distribution of cured grout.	• Increases viscosity and cohesion.	• Difficult to handle due to fineness.
Viscosity modifiers (diutan gum)*	• Makes the grout suspension more water repellant. • Provides resistance to pressure filtration. • Reduces bleed.	• Increases viscosity and cohesion.	• At higher doses, provides some thixotropy to the grout which is helpful for artesian conditions.
Dispersants or water reducers (superplasticizer, fluidifier)	• Overprints solid particles with a negative charge causing them to repel one another. • Reduces agglomeration of particles thereby reducing grain size by inhibiting the development of macro-flocs. • Reduces viscosity and cohesion.	• Depending on chemistry chosen, may accelerate or retard hydration process. This is not necessarily negative.	Dispersants have a distinct life span. Working life depends on dispersant chemistry chosen. •
*Welan gum previously used.			

(1) Many of the grout components identified will vary chemically from one source to another. The only components likely to be constant are the diutan gum (previously welan gum) and the superplasticizer. Due to the variability of the components, it is essential that a full-scale mix design program be carried out before production grouting. This mix testing should use all the actual components and actual mixing and proportioning equipment intended for use in the production program, including the mix water.

(2) It is recommended that the mix testing program be started with a “medium” mix and that the first mix be used to optimize the superplasticizer concentration. The superplasticizer concentration is optimized when an increase in concentration does not change the apparent viscosity of the grout. After the superplasticizer concentration is optimized, the thinnest mix should be formulated to establish the percentages of bentonite and viscosity modifier required to stabilize the mix and to provide adequate pressure filtration resistance. After the percentages of all the admixtures and additives have been determined, the water content is then systematically reduced to provide the range of apparent viscosities desired (Figure 7-15).

d. Mix Proportioning. Before the use of balanced stable grouts, mixes were often defined by the water-to-cement ratio based on volumetric batching. Additives initially derived for concrete are proportioned based on the weight of cement used. To avoid confusion for stable grouts, it is common to designate mixes by a letter or number. The letter or number uniquely identifies a mix based on physical properties such as the marsh funnel viscosity, compressive strength, pressure filtrate, or the specific gravity, and the additive dosage is defined based on the weight of cement used. The rheological properties of HMG depend not only on the additive dosage, but also on the chemistry of the bulk materials (cement, fly ash, silica fume, and water). Mix designs are typically unique, so an intensive mix design program is required for each project. In addition, whenever there is a change in bulk materials (i.e., cement from a new supplier or a new water source) on an ongoing project, the physical properties of the new mixes must be verified through testing, and the material proportions and additive concentrations may require modification to provide the desired properties. Typical percentages of each common grout component are:

(Courtesy of Gannett Fleming)

Figure 7-15. Testing for apparent viscosity and pressure filtration.

(1) Water-to-Cement Ratio (By Weight). The starting mix should depend on the geology and the objective of the grouting. The typical range is 3:1 (thinnest) to 0.6:1 (thickest) (Houlsby 1990, Weaver and Bruce 2007). Note that the weight of cement includes all cementitious materials including fly ash or silica fume.

(2) Fly Ash. Fly ash is designated as Class F or Class C based on the sum of the silicon, aluminum, and iron oxides. Typically, Class F fly ash has a lower calcium oxide (free lime) content (less than 10%), while Class C fly ash has a higher calcium oxide content that results in Class C being self-reactive to varying degrees. However, this is not part of the ASTM chemical requirements, and it should be recognized that any Class F fly ash meets the requirements of

Class C. Therefore, the class designation may be misleading if the chemical properties of the fly ash are not verified and understood. The benefits and detriments of the fly ash may depend on the fly ash source and should be checked if fly ash is to be used where heat of hydration, sulfate resistance, alkali-silica reaction, or expansion are critical to the grout performance. The typical dosage for Class F fly ash is 10–30% by weight of portland cement. Typically Class C fly ash should be less than 20% since Class C fly ash may be less beneficial.

(3) Silica Fume. The typical dosage is 5–10% by weight of portland cement.

(4) Bentonite. The typical dosage is 2–5% by weight of cementitious material. Bentonite is added as a pre-hydrated suspension, typically hydrated a minimum of 12 hrs before use (Weaver and Bruce 2007). When pre-hydrated bentonite is used, the amount of water in the pre-hydrated suspension must be considered in the batch calculations.

(5) Diutan Gum. The typical dosage is 0.1–0.2% by weight of cementitious material.

(6) Superplasticizer. The typical dosage is 1.5–3% by weight of cementitious material.

(7) When proportioning mixes, changing the concentration of only one or two ingredients from mix type to mix type is of great benefit, not only to the plant operator, but also to the inspector. This eases the burden of proportioning mixes during production and greatly simplifies recordkeeping. In many cases, only the volume of water added to the mix may need to be changed as mixes progress from thinnest to thickest. The thickest mixes may require additional viscosity modifiers to provide the desired viscosity at mixable water-to-cement ratios.

e. Sequence of Mixing. The typical sequence of mixing when preparing stable grouts with multiple admixtures is to add the water to the mixer followed by the pre-hydrated bentonite slurry. (The bentonite slurry contains a significant portion of the mix water.) After the water and bentonite slurry, the cementitious materials are added. This includes the cement and any pozzolanic additives such as fly ash or silica fume. Once the cementitious materials have been added and mixed, the water reducer or superplasticizer is added, followed by any viscosity modifiers. Any accelerators or retarders are added at the time recommended by the material supplier, but these are generally added last. For very thick mixes, the superplasticizer might be added to the water before the cement. While adding the superplasticizer to the water in advance of the cement does facilitate mixing, a higher dose (up to two times) of superplasticizer will be required if added before the cement.

f. Other Admixtures.

(1) Retarders. Retarders are commercially available from several manufacturers. Set times can be delayed from hours to days. Retarders are rarely used in typical grouting applications. For projects in hot climates and where extended permeation periods are desired in soil grouting, a retarder can be appropriate.

(2) Accelerators. Accelerators are not typically used in standard rock or soil grouting programs. However, there are times when an accelerator might be appropriate, including grouting in karst, tunnel grouting, and cold weather grouting. The most commonly used

accelerators are calcium chloride and sodium silicate, both of which can have negative impacts on long-term grout properties.

(3) Anti-Washout Agents. Viscosity modifiers such as diutan gum provide resistance to washout. Additional resistance to washout can be achieved using commercially available anti-washout agents. These admixtures can be employed if flowing water is encountered when using cementitious grouts. These products are cellulose based and may have compatibility issues with other admixtures. Consultation with the supplier is recommended before use, along with additional mix design testing.

(4) Foaming Agents. Foaming agents are used to lower grout densities. Grouts created using foam are often referred to as "foamed grouts," "aerated grouts," or "controlled low-strength material." The most common uses of foamed grouts are backfill of existing conduits for abandonment and annular space grouting when existing conduits are sliplined. The low density minimizes the applied load and reduces the buoyancy forces created when floating of a slip liner must be avoided. Any application where low to moderate strength can be tolerated and lightweight grout is advantageous is a candidate for foamed grouts. Foaming agents are readily available in dry and liquid forms. The dry admixtures are easier to use as they are simply added to the mixer as the final component. The wet foaming mixtures are less expensive, but require a foam generator. The dry mixes are applicable to batch-type mixing and placements, while the wet foaming agents are advantageous when large quantities are anticipated. Foamed grouts should not be used where the grout will be exposed to flowing water or the environment, as these grouts are typically less durable.

(5) Dyes/Pigments. Dyes have commonly been used in concrete, but have been rarely used on grouting jobs. Dyes can play an important role in differentiating grouts. For example, if a new grouting campaign is proposed on a previously grouted dam or levee, the new grout could be dyed to distinguish it from the old grout. Dyed grouts have been used at Wolf Creek Dam for this purpose. The standard ASTM C979, *Standard Specification for Pigments for Integrally Colored Concrete* (2010d), was established to ensure that dyes are cement-compatible; all dyes used in cement grouts should meet this standard. There are few disadvantages to the use of dyes in grout, but the designer should consider that dye must be removed from the wastewater stream. Dyes are available in both powder and liquid states.

g. Sanded Grouts. Sanded grouts may be employed for filling large voids or voids without any significant water flow. See paragraph 7-1 b. (1) (c), Bulk Fillers for HMG's for more discussion on the use of sanded grouts. If conditions encountered indicate that a grout with internal friction is required, then provisions should be made for the use of LMG. If inexpensive fillers are desired, then the fly ash content can often be increased.

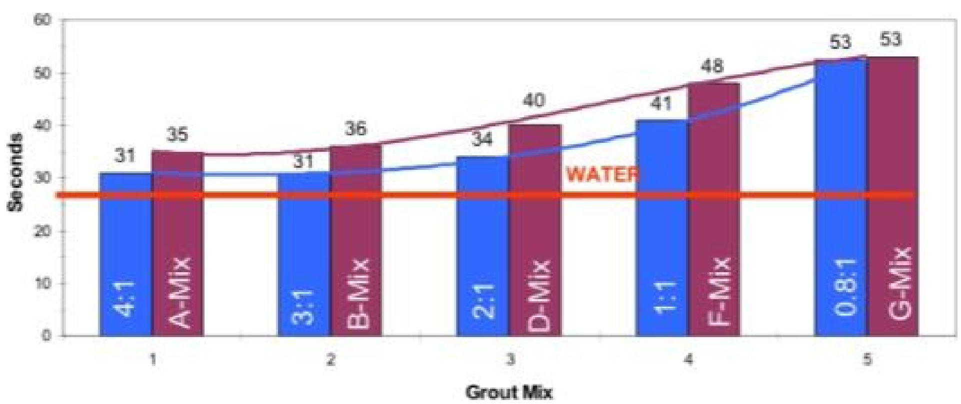
h. Fluid Properties. The fluid properties of balanced stable grouts can be varied widely depending on the application. For a typical grout curtain in fractured rock, a low cohesion is desirable, and a range of apparent viscosities as measured with a marsh funnel ranging from a low of 35–40 sec to a maximum of 60–70 sec would be common. A minimum 5-sec change in marsh funnel flow time should be provided, and a 10-sec change between mixes is common for thinner mixes. With balanced stable grouts, the various mixes are designed to achieve the desired viscosity and cohesion. Thin mixes have low viscosity and low cohesion, while thick mixes have higher viscosity and higher cohesion. The injected mix is progressively thickened in response to conditions observed in the stage, as with neat cement grouts. Table 7-2 lists example mix designs from Penn Forest Dam.

(1) Testing was conducted for the apparent viscosities (marsh funnel flow time) and cohesion values of the balanced stable grouts used for the installation of a grout curtain at Penn Forest Dam, a new dam constructed in sedimentary rock. A comparison between the properties of the Penn Forest Dam balanced stable mixes and those of neat cement grouts (water-to-cement ratios are by weight) was performed and are presented in Figures 7-16 and 7-17 (Wilson and Dreese 1998). The A-mix at Penn Forest Dam represents an ultrafine cement mix.

Table 7-2. Summary of mix designs for Penn Forest Dam, PA.

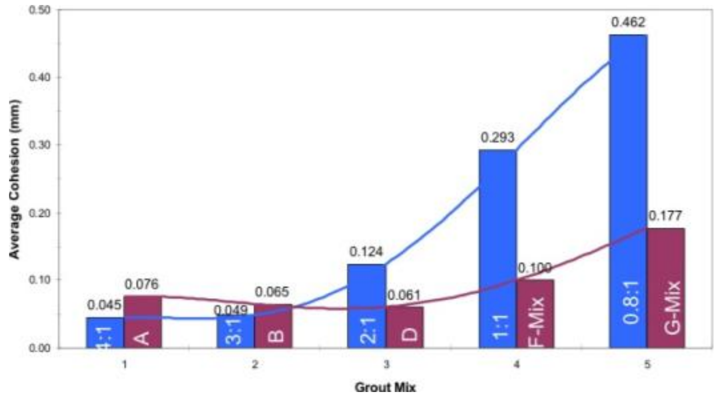
Item*	Grout Mix Design						
Mix number	A†	C	D	E	F	G	H
W/C by wt.	1.3	2.1	1.8	1.6	1.4	1.3	1.3
W/(C+F.A.) by wt.	N/A	1.8	1.6	1.45	1.2	1.1	1.1
Bentonite (% bwc)	0.0	3.7	3.7	4.1	4.1	4.1	4.1
Welan gum (% bwc)	0.06	0.11	0.11	0.11	0.11	0.11	0.11
Superplasticizer (% bwc)	1.2	1.8	1.8	1.8	1.8	1.4	1.1
Marsh (sec)	36	38	40	43	48	53	?
Specific gravity	1.41	1.32	1.35	1.38	1.43	1.47	1.47
Pressure filtration coefficient	0.035	0.009	0.039	0.016	0.025	0.024	

*Courtesy of Gannett Fleming.
†A-mix was an ultrafine cement mix. B-mix was not used for production.



(Courtesy of Gannett Fleming)

Figure 7-16. Marsh funnel viscosity values from the Penn Forest Dam mix testing program.



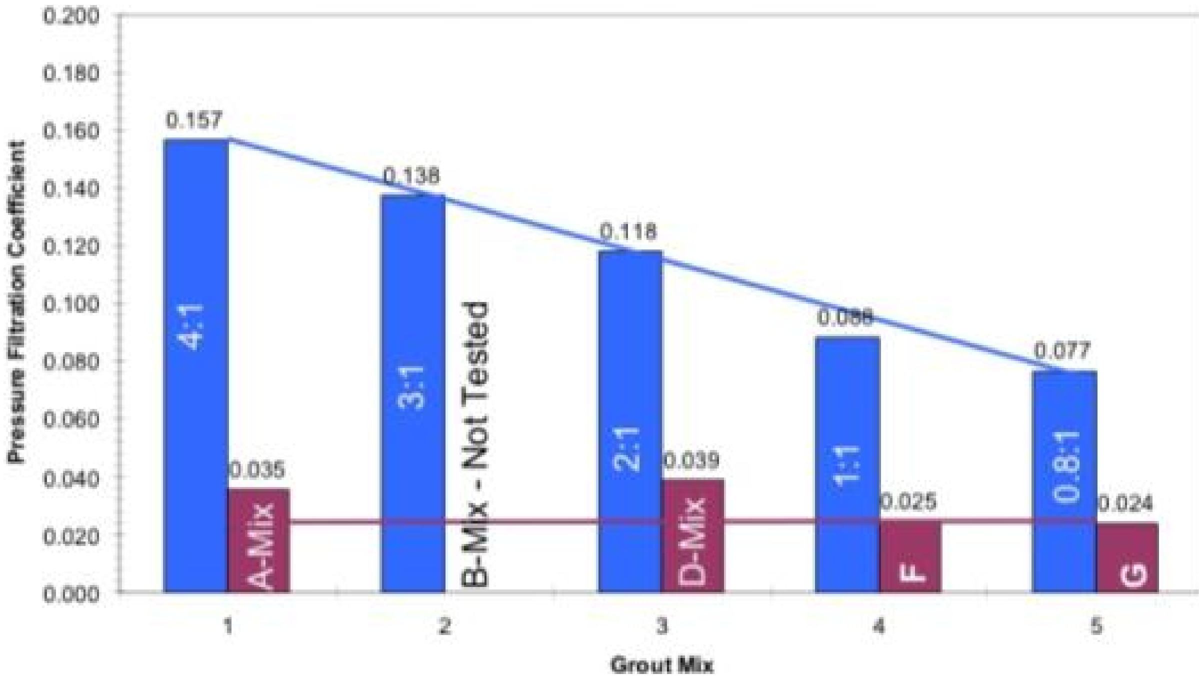
(Courtesy of Gannett Fleming)

Figure 7-17. Plate cohesion values from the Penn Forest Dam mix testing program.

(2) The other major fluid property to consider is the resistance of a grout to pressure filtration. Pressure filtration is measured by placing the grout in a filter press and applying pressure. The test, conducted in accordance with American Petroleum Institute Test Procedure API RP 13B-1, measures how easily water is removed or squeezed out of a grout. A high resistance to pressure filtration or a low-pressure filtration coefficient is desirable to ensure that the grout rheology is constant during the injection process. The pressure filtration coefficient K_{pf} is calculated as:

$$K_{pf} = \frac{\text{Filtrate Volume}}{\text{Initial Volume of Grout}(400\text{ mL}) \times (\text{Filtration Time})^{1/2}}$$

(3) Pressure filtration tests were also performed under the Penn Forest Dam testing program. Figure 7-18 shows pressure filtration coefficients of neat cement grouts and balanced stable grouts, side by side, for comparison. The standard test duration is 30 minutes. The neat cement grouts thinner than 1:1 blow air after less than 10 minutes, indicating that nearly all the mix water has been squeezed out of the grout. For most grouting projects, a pressure filtration coefficient of 0.05 or less should be targeted.



(Courtesy of Gannett Fleming)

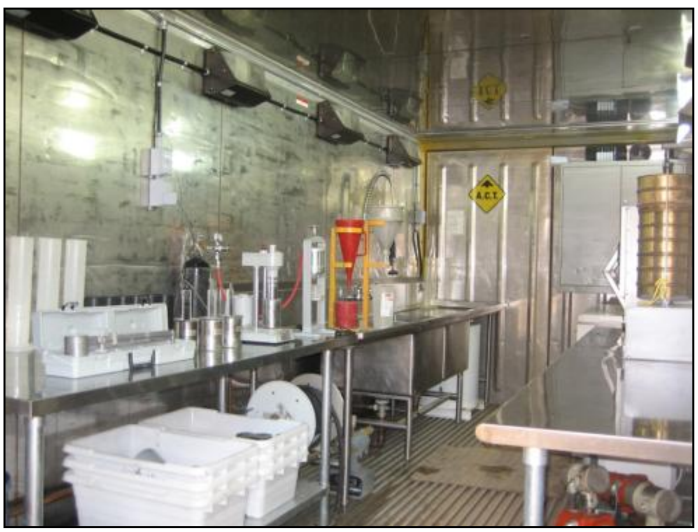
Figure 7-18. Pressure filtration coefficient values from the Penn Forest Dam mix testing program.

i. Setting of Grout and Hardened Grout Properties. Balanced stable grouts generally have a longer set time (whether initial gel or initial set) than neat cement grouts because the formulation of a stable grout with a fully dispersed structure delays the hydration process. Set times and strengths vary, depending on the water-to-cement ratio. A suite of mix designs for grouting fractured rock might have initial set times of 10–16 hrs and final set times of 12–20 hrs. The set time can be varied, depending on the purpose of the grouting program, by using admixtures such as retarders or accelerators. The 28-day unconfined compressive strength of typical balanced stable grouts varies from 750 to 1,500 psi. Due to the variables of the constituents in balanced stabilized grouts, the grout can be designed to be stronger or weaker than a neat cement grout by altering the mix proportioning.

j. Testing Grout Properties. Quality control testing is an important component of a grouting program. The type of testing performed must be appropriate for the application, and the testing must be frequent enough to ensure that the materials being used have been correctly formulated. Table 7-3 lists common tests used to evaluate the rheology of HMGs and the corresponding recommended test frequencies. Figure 7-19 shows a typical on-site laboratory for quality control testing.

Table 7-3. Quality control tests for HMG.

Test	Test Method	Equipment	Frequency
Apparent viscosity	API 13B-1	Marsh funnel	Once per mix per day
Specific gravity	API 13B-1	Mud balance	Once per mix per day
Bleed	ASTM C940 (2010b)	Graduated cylinder	Once per mix per week
Pressure filtration coefficient	API 13B-1	API filter press	Once per mix per week
Set time	ASTM C191-92 (2008)	Vicat needle	Mix testing program only
Cohesion and gel times		Viscometer	Mix testing program only
Strength	ASTM C942 (2010c)	Grout cubes	Mix testing program only
Apparent cohesion	Weaver (1991), p. 152	10-×10-cm steel plate	Mix testing program only



(Courtesy of Advanced Construction Techniques)

Figure 7-19. On-site quality control testing lab.

(1) The best method to ensure a quality grout product is to have the grout producer provide historical data analyses from past projects that indicate that the equipment, materials, and personnel are capable of producing grout to past contract specifications. As a check, statistical analysis by USACE personnel of the raw materials used for each batch will reveal the random (normal) variation to be expected in batching, for example, water volume, cement weight, and bentonite volume. This data analysis will produce the average and standard deviation, which can be used to predict the theoretical batch specific gravity and its range due to normal variation. Although 100%

inspection at the beginning of a project is labor and time intensive, it allows the baseline process capability of the grout production to be established and evaluated. A capable process warrants reduced inspection, but constant vigilance through the use of control charts to monitor the product. With control charts, trends can be readily identified and adjustments made as necessary, including increased inspection and testing to achieve a quality product. The goal of the process is to have the random variation of the grout performance metrics of viscosity, specific gravity, temperature, unconfined compressive strength, pressure filtration, and bleed consistently fall at least one standard deviation inside the upper and lower specification limits for each test.

(2) The marsh funnel and specific gravity tests can be performed rapidly and economically. In general, if these two tests show satisfactory results and thorough mix testing has been performed in advance, it can be assumed that the other parameters are in compliance.

(3) Typical test frequencies would be once per day per mix type for measuring the apparent viscosity with a marsh funnel and the grout density using a mud balance. Bleed and pressure filtration tests are typically performed once per week. Cohesion, gel time, set times, and compressive strengths are generally only tested once per project, depending on the grouting objective.

k. Grout Behavior in Rock Fractures. The fully dispersed structure of balanced stable grouts, combined with the significant resistance to pressure filtration, results in a grout mix that remains virtually unchanged during the course of the injection process. The superplasticizer delays the cement particles from coming together, resulting in a nearly constant grain separation within the grout over the course of the injection. The high resistance to pressure filtration assists in maintaining a nearly constant water-to-cement ratio.

(1) Figure 7-20 shows how balanced stable grouts are envisioned to behave when injected into a rock fracture. The consistent rheology of balanced stable grouts allows the injection process to be both mathematically modeled and analyzed. The flow of a Bingham fluid may be expressed as (De Paoli et al. 1992):

$$\tau = C + \eta_B \frac{dv}{dx} \cong \eta' \frac{dv}{dx}$$

where:

τ	=	shear stress.
C	=	grout cohesion or yield point.
η	=	dynamic viscosity.
η_B	=	plastic viscosity.
$\frac{dv}{dx}$	=	shear rate.

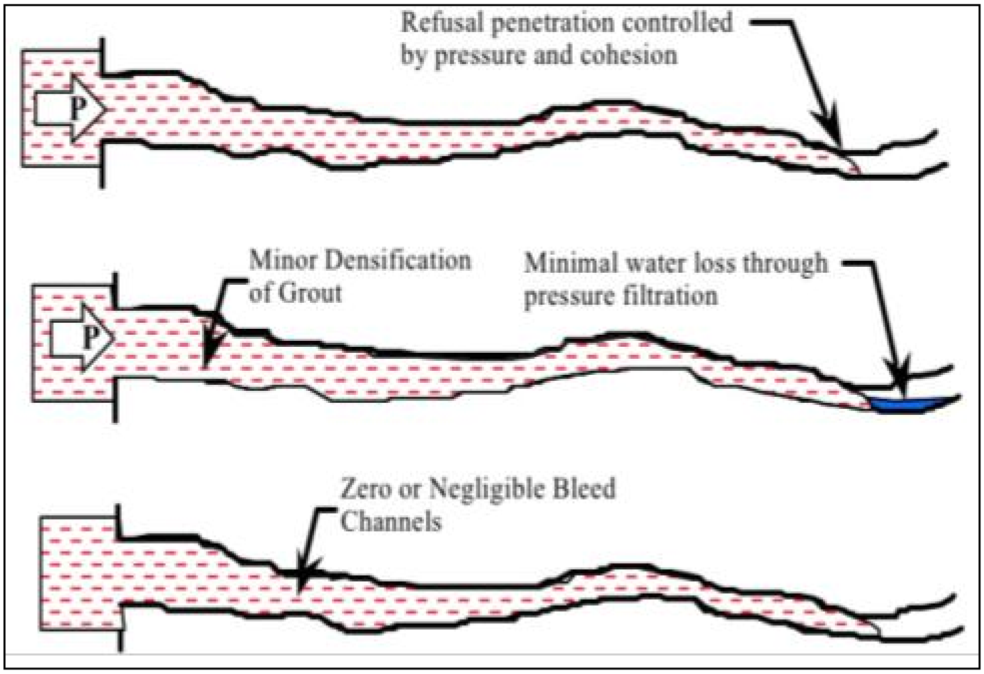


Figure 7-20. Grouting theory for balanced stable grouts.

(2) It is appropriate to consider the Y-axis of the Bingham model as pressure or resistance to flow and the x-axis as flow rate (Figure 7-13). Evaluation of the model then results in the following simplified explanation of the injection for a constant injection pressure in a fracture of uniform thickness: as a grout penetrates a fracture, the head losses increase and the pressure at the grout fringe decreases. As the pressure decreases, the flow rate decreases proportionally to the viscosity. This continues until the pressure at the grout fringe is equal to the cohesion, at which time refusal occurs and grout flow stops. Conclusions based on this simplified explanation are:

(a) Cohesion controls how far a grout will travel in a fracture of a given opening for a given injection pressure.

(b) For that same fracture aperture and pressure, the viscosity controls the flow rate and ultimately the time required to grout that fracture.

(c) Head losses will be greater in a finer fracture than in a wider fracture. Therefore, grout penetration distance will increase as the fracture aperture increases.

l. l. Penetrability. The Bingham flow model (Figure 7-13) shows that the penetration distance into a fracture depends on the cohesion and the applied pressure. The penetration distance also depends on the fracture aperture for two reasons. The first is that head loss is greater in narrower fractures. The second is that the larger grains of the grout suspension might clog the fracture. Lombardi (1985) developed the equations that relate the maximum radius of penetration, the maximum volume of injected grout, and the maximum total uplift force to the injection pressure, the grout cohesion, and the fracture aperture:

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

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

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

where:

p_{max} = applied pressure

t = half thickness or aperture of the fracture

C = grout cohesion or Bingham yield point

R_{max} = maximum radius of penetration

V_{max} = maximum volume of injected grout

F_{max} = maximum total uplift force.

Lombardi's equations for the maximum volume of grout and the uplift pressure are of limited practical use, as they assume uniform flow of grout radially in all directions, a uniform fracture aperture, and a horizontal fracture. The penetration distance also assumes a uniform fracture aperture and can generally not be used to predict the actual field penetration distance. The equation for grout penetration distance is still highly useful because it recognizes that the penetration distance is proportional to the injection pressure and the radius (half thickness) of the fracture and is inversely proportional to the grout cohesion. Therefore, a low cohesion is generally desirable for most grouting projects. However, in highly permeable or open formations, such as karst, grouts with a higher cohesion might be desirable to minimize the grout travel distance.

(1) Lombardi's equations ignore the grain size of the grout, and the implicit assumption is that the grout being used is appropriate for the formation being grouted. Penetrability into a rock fracture or soil medium depends on the grain size of the cement in suspension. Warner (2004) points out that the maximum grain size of the grout is important when grouting soil because the largest particles begin to clog the pore structure and lead to the rapid development of a filter cake. Warner states that, in permeating rock fractures, the maximum grain size is less important as long as the large grains are limited in number, well dispersed, and smaller than the void being penetrated. In either case, accurate determination of the range of pore diameters and fracture apertures is all but impossible. Therefore, an alternative method is needed to determine the suitability of a grout mix to the formation being grouted.

(2) Naudts (1995) proposed the Amenability Theory to satisfy this need. The Amenability Theory is based on the fact that the injection of grout is really just another permeability test using grout as the permeant, provided, of course, that the rheology of the grout does not change during the injection process (i.e., balanced stable grouts). If the conductivity of the formation with water is known, it can be compared to the conductivity with grout. The problem with this comparison is that one permeant is a Newtonian fluid with a very low viscosity, and the other is

a Bingham fluid with a yield point and a higher viscosity. To address this concern, Naudts defined the "Apparent Lugeon value," which is the standard equation for the Lugeon value when testing with water corrected by the ratio of the apparent viscosity of water to that of the grout. The Apparent Lugeon value must be determined early in the grout injection (in the first minute or two), but after flow and pressure have stabilized, the grout material quickly starts to develop larger head losses as the penetration distance increases. The amenability is then the ratio of the Apparent Lugeon value measured during grouting to the Lugeon value with water:

$$Lu_{app} = \frac{Q\ (L/min)}{Stage\ Length\ (m)} \times \frac{143\ psi}{P_{eff}} \times \frac{V_{Marsh\ of\ Grout}}{28\ sec}$$

$$Amenability = Lu_{app} / Lu_{water}$$

where:

Q = flow rate

$V_{Marsh\ of\ Grout}$ = time for 1 L of grout to pass through a Marsh Cone

P_{eff} = effective pressure

Lu_{app} = Apparent Lugeon value

Lu_{water} = Lugeon value with water.

(3) The amenability coefficient is an indication of the grout mixture's ability to permeate the fractures accessed by water during the water pressure test. If the coefficient is 100%, the grout is penetrating all the fractures accessed by water during the pressure testing. If the value is less than 100%, the difference is an indication that some of the fractures permeated with water are not accessible with the grout mix being used. If the amenability coefficient is low, it provides a clear indication that a change in grout mix should be considered. The definition of "low" depends on the application. If portland cements are being used, consideration should be given to changing to ultrafine cement. If ultrafine cements are being used, a change to solution grouts might be necessary.

(4) Other relevant grouting equations pertaining to penetrability and grain size have been published by various authors. These include the groutability ratio (GR) (Mitchell 1981) developed for fractured rock:

$$GR = \text{width of fracture} / (D_{95})\ \text{grout}$$

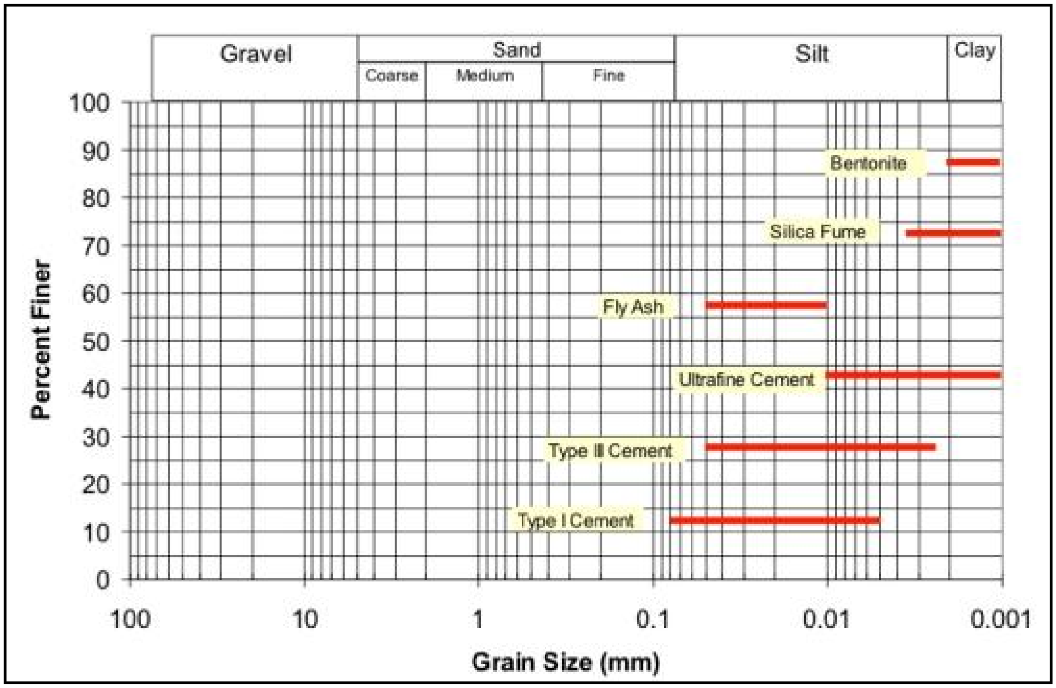
where:

(D_{95}) grout = the 95% size of the grout particles

GR > 5 indicates that grouting is consistently possible

GR < 2 indicates that grouting is not possible.

(5) The data shown in Figure 7-21 indicate the range of particle sizes that are typical of various grout materials. The advantage of using the ultrafine cement in fine fractures is obvious in light of the GR defined above and the significantly smaller grain size in comparison to the portland cements. Evaluation of the range of material sizes also indicates that fly ash should not be used with ultrafine cements, as the fly ash particles would be the largest and would control the penetrability.



(After Warner 2004)

Figure 7-21. Range of grain sizes of typical admixtures.

(6) Mitchell also provided the following equations for soil grouting:

$$N = D_{15}\text{soil} / D_{85}\text{ grout}$$

$$Nc = D_{10}\text{soil} / D_{95}\text{ grout}$$

where:

N and Nc	=	GRs for the soil.
D_{15}soil	=	15% size of the soil.
D_{85}grout	=	85% size of the grout particles.
D_{10}soil	=	10% size of the soil.
D_{95}grout	=	95% size of the grout.

N < 11 or Nc < 6 indicates that permeation is impossible.

N > 24 or Nc > 11 indicates that the soil is readily groutable.

(7) More recently, the wetability or surface tension between a grout and the medium being grouted has been identified as a property impacting penetrability. Chapter 4 of Warner (2004) includes an accessible discussion on the topic of wetability.

m. Balance Stable Mixes for Grouting Karstic Foundations. For projects in karstic foundations, a suite of grout mixes that cover a wide range of rheologic properties is usually required due to the joint and solution feature characteristics unique to this geology. For the tighter joints and fractures, the low-cohesion, low-apparent-viscosity mixes are still desirable for grouting fractures that have not been widened by solutioning. However, “thicker” mixes with higher apparent viscosities and high cohesion are desirable when solution-widened joints and cavities are encountered. The purpose of the grouting program plays an important role in defining the site-specific characteristics targeted and the mixes should be designed accordingly.

(1) USACE has numerous dam projects that have been founded on karstic foundations. Some of the projects have exhibited distress indicators signifying problematic performance. As a result, in some instances further investigation and analyses have led to remediation measures taken to address foundation defects. Cut off wall construction began at Mississinewa Dam, IN in 1999 and a pre-grouting program was deemed necessary in 2000 after the occurrence of slurry loss. Table 7-4 lists the grout mix designs for this program, and Table 7-5 lists their properties. In the year 2000, a grouting program that consisted of a three-line curtain was performed at Patoka Dam, IL after the development of sinkholes in the spillway in 1996. The program targeted a gap between the existing barrier cut off walls and grout curtains in the left abutment. Tables 7-6 and 7-7 list information on the grout mix designs and properties for the Patoka Dam, IL program. Wolf Creek Dam located in Kentucky was constructed on a mature karstic foundation that had developed muddy flows and sinkholes at the embankment toe in 1968. Seepage remained a problem even after the installation of grout curtains and a partial cut off wall between the years of 1968 and 1979. Further remedial measures were warranted with the construction of a second cutoff wall that extended from the right abutment to the concrete portion of the dam and was preceded with pre-grouting of the bedrock from January 2007 through September 2008. Table 7-8 lists information on the grout mix designs for Wolf Creek Dam, KY program. A sinkhole on the upstream face of the Clearwater Dam in Missouri was discovered in January 2003. Rehabilitation of the dam was performed in two phases; Phase 1 was started in 2006 and was designed as a grouting exploratory program to locate and treat large solution features and pretreat the foundation for cut off wall construction. Information gathered during Phase 1 identified an extensive epikarst and resulted in a more intensive grouting program, Phase 1b, to treat this zone and the underlying bedrock. Table 7-9 lists information on the grout mixes used in these two phases.

Table 7-4. Summary of grout mix designs for Mississinewa Dam, IN.

Constituent	Mix B	Mix C	Mix D	Mix E	Mix F
Water/Cement Ratio (BW)[1]	2.07	1.72	1.37	1.26	1.26
Water/Solid Ratio (BW)	1.4	1.1	0.9	0.8	0.8
Water (Liters)	115	85	85	70	70
Bentonite Slurry (Gallons)[2]	17	17	25	25	25
Portland Cement (lbs)	188	188	282	282	282
Fly ash Type ‘F’ (lbs.)	100	100	150	150	150
Welan Gum (lbs)	0.2	0.2	0.3	0.3	0.35
Super Plasticizer (ounces)	70	70	105	105	105
Yield (liters)	227	197	251	236	236
Yield (gallons)	60	52	66	62	62
[1](BW) means By Weight					
[2]Bentonite slurry is 8% weight of water; 1 gallon bentonite slurry contains 0.64 lb of bentonite and 3.64 liters of water.					

Table 7-5. Summary of grout mix properties for Mississinewa Dam, IN.

Grout Mix Properties		Mix B	Mix C	Mix D	Mix E	Mix F
Specific Gravity		1.38	1.44	1.53	1.57	1.57
Marsh Funnel (seconds)*		41	48	55		
Flow Cone (seconds)*					14	24
Cohesion (g/mm²)		0.180	0.348	0.490	0.630	0.630
	1 hr	0	0	0	0	0
Bleed	2 hrs	0	0	0	0	0
	3 hrs	0.5	0	0	0	0
Pressure Filtration Coefficient ($m^{\wedge-1/2}$)		0.045	0.042	0.04	0.036	0.031
Initial Gel Time (hrs:mins)		3:00	2:00	1:30	1:00	1:00
Final Gel Time (hrs:mins)		7:00	5:00	4:00	3:00	3:00
Initial Set Time (hrs:mins)		17:00	15:30	13:30	12:30	12:30
Final Set Time (hrs:mins)		22:30	19:00	15:00	13:30	13:30
Compressive Strength 7/9 days[1] (psi)		260	360	770	920	870

* Marsh Funnel versus Flow Cone depended on viscosity of mix because thicker mixes had infinite Marsh Funnel time, and thus flow cone results were used.

[1]The compressive strength for B and C mixes are 9 day results, and for D, E, & F mixes are 7 day results

Table 7-6. Summary of grout mix designs for Patoka Dam, IL.

Constituent	Mix A	Mix B	Mix C	Mix D	Mix E	Mix F	Mix G+	Mix G
Water/Cement Ratio (BW)[1]	2.07	1.60	1.37	1.13	1.01	0.96	0.78	0.66
Water/Solid Ratio (BW)	1.71	1.32	1.13	0.94	0.84	0.79	0.65	0.56
Water (Liters)	140	100	80	60	50	45	30	20
Bentonite Slurry (Gallons)[2]	10	10	10	10	10	10	10	10
Portland Cement (lbs)	2	2	2	2	2	2	2	2
Fly ash Type ‘F’ (lbs)	25	25	25	25	25	25	25	25
Silica Fume (lbs)	10	10	10	10	10	10	10	10
Welan Gum (lbs)	0.2	0.2	0.2	0.2	0.2	0.2	0	0
Super Plasticizer (ounces)	40	30	30	30	30	30	40	50
Yield (liters)	213	172	152	132	122	118	103	93

[1]By Weight
[2]Slurry is 8% by weight of water; 1 liter of slurry contain 0.17 lb of bentonite and 0.97 liters of water.
[3]94 lb bags.

Table 7-7. Summary of grout mix properties for Patoka Dam, IL.

Grout Mix Properties		Mix A	Mix B	Mix C	Mix D	Mix E	Mix F	Mix G+	Mix G
Specific Gravity		1.38	1.44	1.53	1.57	1.57	1.57	1.57	1.57
Viscosity (seconds)		34	40	50	55	65	120	120	>120
Bleed	1 hr	0.5	0.5	0.02	0.02	0	0	0	0
	2 hrs	1.5	1.0	0.5	0.5	0.3	0.3	0	0
	3 hr	3.0	2.0	1.0	1.0	0.5	0.5	0	0
Filtration Coef. ($min^{-1/2}$)		0.091	0.089	0.077	0.068	0.064	0.052	0.04	0.035
Initial Set Time (hrs:mins)		13:15	9:45	8:35	7:05	6:45	6:20	6:20	5:15
Final Set Time (hrs:mins)		17:45	12:25	10:05	7:55	7:20	7:00	6:50	5:40
Compressive Strength (psi)	3-day	160	195	240	375	475	750	810	915
	7-day	190	265	460	595	725	1005	1100	1175
	14-day	365	380	510	820	1060	1285	1410	1455
	28-day	420	535	735	1070	1490	2560	2395	2755

Table 7-8. Summary of grout mix designs for Wolf Creek Dam, KY.

Constituent	Mix A	Mix B	Mix C
Water (pounds)	110.2	70.5	48.5
Water (gallons)	13.2	8.5	5.8
Bentonite Slurry Concentration BWOW (%)	10.0%	10.0%	10.0%
Bentonite Slurry (gallons)	10.3	10.3	10.3
Dry Bentonite Specific Gravity	2.7	2.7	2.7
Bentonite Slurry Water (pounds)	82.9	82.9	82.9
Bentonite Slurry Water (gallons)	9.93	9.93	9.93
Dry Bentonite (pounds)	8.29	8.29	8.29
Dry Bentonite (gallons)	0.37	0.37	0.37
Portland Cement Specific Gravity	3.15	3.15	3.15
Portland Cement (pounds)	188	188	188
Portland Cement (gallons)	7.15	7.15	7.15
Super Plasticizer Specific Gravity	1.2	1.2	1.2
Super Plasticizer (pounds)	8.37	8.37	8.37
Super Plasticizer (ounces)	107	107	107
Super Plasticizer (gallons)	0.84	0.84	0.84
Water/Cement Ratio (by weight)	1.03	0.82	0.7
Water/Solids Ratio (by weight)	0.98	0.78	0.67
Liquids/Solids Ratio (by weight)	1.03	0.82	0.71
Water/Cement Ratio (by volume)	3.24	2.57	2.20
Water/Solids Ratio (by volume)	3.08	2.45	2.09
Liquids/Solids Ratio (by volume)	3.19	2.56	2.20
Theoretical Yield (gallons)	31.5	26.7	24.1
Theoretical Weight (pounds)	397.8	358.1	336.0
Theoretical Specific Gravity	1.51	1.60	1.67
Actual Specific Gravity	1.50±0.02	1.56±0.02	1.62±0.02
Marsh Funnel Viscosity (seconds)	50±2	65±4	80±6
Bleed at 2 hours (%)	0	0	0
Pressure Filtration Coefficient ($min^{1/2}$)	0.025	0.015	0.014
Initial Set Time (hours)	8:00	8:00	8:00
Final Set Time (hours)	13:00	13:00	13:00
28-Day Compressive Strength (psi)	1,490	1,590	1,830

Table 7-9. Summary of grout mix designs for Clearwater Dam, MO.

Constituent	Mix A	Mix B
Water (gallons)	21.13	18.49
Bentonite Slurry (gallons; 8% bw)	28	28
Portland Cement (lb)	376	376
Type F Fly Ash (lb)	50	50
Welan Gum (lb)	0.4	0.4
Superplasticizer (oz)	0.07	0.07
Theoretical Yield (gallons)	67	64
W:C Ratio	1.07	1.01
Specific Gravity	1.50	1.56
Marsh Funnel Time (s)	51	55

Grout Mix Designs - Clearwater Dam - Phase 1b

Constituent	Mix A	Mix B	Mix C	Mix D	Mix E	Mix F
Water (gallons)	26.42	21.13	15.85	13.21	10.57	8.45
Bentonite Slurry (gallons; 8% bw)	16	16	16	16	16	16
Portland Cement (lb)	198	198	198	198	198	198
Silica Fume (lb)	17	17	17	17	17	17
Welan Gum (lb)	0.22	0.22	0.22	0.22	0.22	0.22
Superplasticizer (oz)	109.9	109.9	109.9	109.9	109.9	109.9
Theoretical Yield (gallons)	51.46	46.18	40.89	38.25	35.61	33.50
W:C Ratio	1.62	1.42	1.21	1.11	1.01	0.93
Specific Gravity	1.36	1.41	1.46	1.49	1.53	1.56
Marsh Funnel time (s)	38-	44-	53-	62-	80-	>100
7-Day Compressive Strength (psi)	185	455	600	1,115	1,530	1,995
28-Day Compressive Strength (psi)	640	1,115	1,690	2,290	3,135	3,180

Note: Pressure filtration tests performed at the site indicated Pressure Filtration Coefficient ($min^{1/2}$) values were <0.02 on more than 99% of observations.

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CHAPTER 8

Overburden and Embankment Drilling Equipment

8-1. Overburden Drilling Equipment.

a. Introduction. Drilling holes for the purpose of injecting grout is a critical phase of a grouting program and one that can present numerous challenges. Often these challenges result from the materials that have to be penetrated, which can vary significantly across the project site in both the horizontal and vertical directions; to the terrain that the drilling equipment must traverse; to the environmental requirements of the drilling process; to the required accuracy and condition of the final grout hole. Chapters 8 and 9 of this manual present the major considerations that must be made in the selection of the equipment for drilling acceptable grout holes at a reasonable cost and in a reasonable time.

b. General Considerations in Selection of Method. Typically, the decision about the type of equipment to be used is left to the contractor. However, it is imperative that the project specifications provide the requirements for the grout hole alignment, accuracy, and special restrictions on equipment to protect the in-place conditions of the dam embankment or other structure and underlying strata. Each of these requirements can significantly impact the selection of drilling method. There are many types of drill rigs and many options in selecting drill tooling. Some are developed for high production and require road-like conditions for access. Other types of drill rigs have been developed to maximize mobility, while others have been designed for unusual applications such as drilling on steep slopes or in confined areas. The size of drilling equipment varies from very large track- or truck-mounted drills to small hand-held drills for use in confined areas. On large grouting projects, specialized drill rigs or attachments might be designed and fabricated for a specific application.

(1) Critical Versus Non-Critical Applications. Holes that are drilled through existing dam embankments or other earthen hydraulic structures are considered critical because of the potential for damage to the embankment materials. This damage generally results from excessive pressure of the flushing medium, which can cause significant scour or erosion of the boring sidewalls or can cause pneumatic or hydraulic fracturing of the embankment. Air, gas, water, mud, or any other drilling fluid shall not be used in these critical areas, specifically, the impervious core of the dam, and the core trench or foundation soil under the core. In addition, drilling fluids shall not be used in portions of dams where contamination of filters or drainage features is possible. Holes that are drilled outside the limits of the embankment core footprint are often considered non-critical when there is no potential for damage to the embankment due to the drilling process. A common example is drilling in coarse shell zones.

(2) Influence of Differing Material Types. Materials typically encountered during drilling for grout holes can vary from clay cores, random fills, and rock shells in dam embankments to boulders and running sands in natural deposits. Each of these zones or materials requires consideration of the equipment most suitable for penetration without damaging the material surrounding the hole. Characteristics of the different drilling methods in these various material types are presented later in this chapter.

(3) Sampling Requirements. Often samples are not required in the overburden since the primary purpose of drilling through the overburden is to gain access to the underlying rock that is to be grouted or to subsequently inject low-mobility or solution grouts into the overburden. In addition, the site investigation borings have normally provided sampling and characterization of the overburden materials in advance. However, if samples are required, this will influence the drilling method to be used. As discussed in the following paragraphs, it is not possible to obtain representative samples with some drilling methods since the material is changed significantly by the drilling process, which may pulverize the material. Other methods permit sampling of the return flush, which is highly disturbed. Some methods allow for retrieval of relatively intact bulk samples.

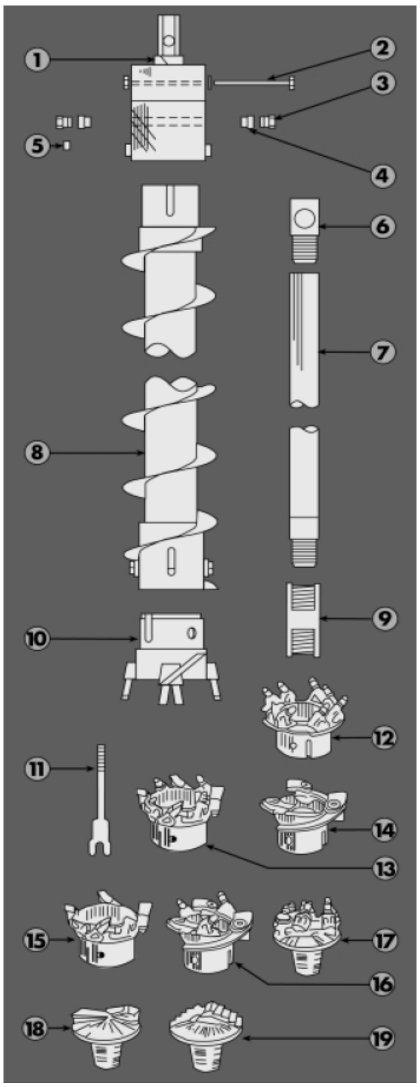
(4) Hole Size. Hole size is typically left to the contractor, who can make the decision based on available equipment and knowledge of the most efficient size for the materials to be encountered. The primary factor in selecting the hole size is that the hole must be sufficiently large to accommodate all of the subsequent drilling and grouting equipment and tools.

c. Special Requirements for Drilling in Embankment Dams. ER 1110-1-1807 provides guidance on drilling through earth embankments, with a specific prohibition against the use of air (including air with foam) or any other gas or water as the circulating medium. Exemptions may be made to this prohibition (subject to approval) only in special circumstances and when all other acceptable alternatives have been exhausted. These prohibitions are based on the primary concern of protecting the impervious section of embankments. High air pressures used in drilling have caused pneumatic fracturing of embankments (as evidenced by connections to other borings) and blow outs on embankment slopes. Hydraulic fracturing of embankments can also be easily induced when drilling with water. Drilling through coarse-grained zones of embankments, such as the upstream or downstream shells, is usually an appropriate instance to consider an exception to this regulation, depending on the specific locations. ER 1110-1-1807 is explicit with regards to drilling at earthen hydraulic structures. However, designers should consider the impacts of the drilling method if hydraulic fracturing could potentially damage other infrastructure.

8-2. Drilling Methods.

a. Introduction. Historically, there have been many methods of drilling through overburden. These methods all have advantages and disadvantages. The selection of the drilling method can be left to the contractor to take advantage of the contractor’s available equipment and thereby achieve a lower cost. However, methods or flushing mediums that are not acceptable should be clearly indicated in the contract documents. The method selected depends on a number of factors, including the type of material that must be penetrated, the need to eliminate disturbance to the overburden/embankment materials, the need to support the drilled hole, and the need to obtain samples. Additional factors may include overhead clearances for drilling in galleries and the ability to traverse and set up on steep terrain.

b. Hollow Stem Auger. This method consists of advancing an auger drill string through the material. The outer auger string is essentially a casing with helical flights welded to the outside (Figure 8-1). The bottom of the outer string is equipped with a cutting head. The inner drill string consists of a pilot bit that extends below the bottom of the augers, a plug near the bottom of the augers to prevent material from entering the inside of the augers, and drill rods extending to the drill head. Both the inner and outer strings are rotated during advancement. As the augers are rotated, the material is removed to the surface via the rotating action of the flights. In some cases, the inner drill string is not used during hole advancement, but this may result in cuttings advancing inside the outer string, producing erroneous sampling and incomplete evacuation of cuttings from the hole. The use of hollow stem augers without the inner string installed is not recommended. Auger methods in general are satisfactory for penetrating materials that do not contain boulders or other obstructions (e.g., see Figure 8-2). Nested cobbles can prove difficult to penetrate, and advancement into weathered and very soft rock is slow and may result in auger refusal at shallow penetration distances within these zones. Various types of cutter heads and pilot bits are available and tailored for penetrating particular materials and drilling conditions. During the drilling, the inner plug may be withdrawn for conventional sampling. Because the joints between the auger sections tend to be loose, maintaining the hole alignment can be difficult, particularly on non-vertical holes.



HOLLOW STEM AUGERS, KEYED DESIGN

1. Drive Cap
2. Nut & Bolt for Rod Cap Adapter
3. Lock Bolt
4. Replacement Bushing
5. Replacement Drive Key
6. Rod Cap Adapter
7. Drill Rod
8. Hollow Stem Auger
9. Pilot Bit Connector
10. Cutter Head - 5T Type
11. Knock-Out Wrench, Bullet Cutters
12. Cutter Head - Bullet Type
13. Cutter Head - Spade/Bullet Type
14. Cutter Head - Blade Type
15. Cutter Head - Spade Type
16. Cutter Head - Blade/Bullet Type
17. Pilot Bit - Bullet Type
18. Pilot Bit - Apex Type
19. Pilot Bit - Step Type

(Courtesy of Boart Longyear)

Figure 8-1. Components of a hollow stem auger system

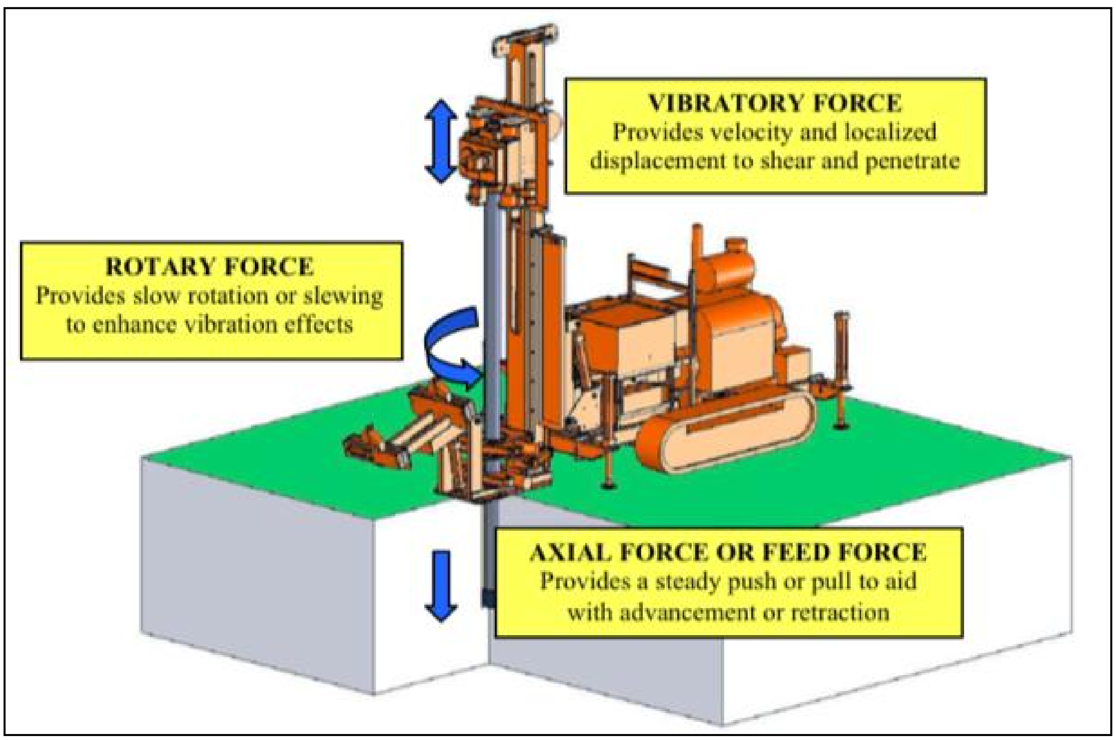
(Courtesy of Gannett Fleming)

Figure 8-2. Hollow stem auger system in use.

c. Driven or Reamed Casing with Cleanout. This method (referred to as the "combination method" by Weaver and Bruce [2007]) advances a hole by driving or rotating a casing. The bottom of the casing is equipped with a shoe appropriate for the materials to be penetrated. A hardened drive shoe is used for driven casing, and a shoe with carbide teeth, diamonds, or carbide buttons is used for rotated casing when cobbles and other obstructions are expected. The casing may also be equipped with a reaming bit near the bottom to enlarge the hole through hard materials to aid in advancing the casing. The casing with the appropriate shoe can be seated into bedrock. The hole sidewall is then well protected so that the inside of the casing can be cleaned with rotary wash drilling methods, solid stem augers, or flushing as permitted by the materials encountered. The casing is relatively rigid so that straight, properly aligned holes can be made. The equipment is typically mounted on a truck, track, or skid and can be relatively lightweight so that tight clearances and steep terrain can be accommodated. The advance of the casing can be interrupted at any time, and the casing can be cleaned to obtain conventional samples if required.

d. Slurry Supported Method. This method advances an uncased hole by rotary methods with the hole supported by a drilling mud or slurry. The slurries form a filter cake around the borehole, which is then supported by the pressure applied by the column of fluid against this impervious zone. This method is typically not appropriate for most grouting projects. For a soil grouting project, the slurry and developed filter cake negates future treatment from the hole by permeation. When rock is penetrated, the slurry can easily be lost. Additionally, it is not desirable to allow slurry to migrate into rock fractures intended to be grouted.

e. Sonic Drilling. This method generally consists of a dual casing that is advanced through the overburden and in some cases bedrock formations. The tooling is advanced by vibrations (typically 50–150 Hz) induced at the top of the casing by a hydraulically driven oscillator. The vibrations propagate down the drill string and are induced on the material to be penetrated. Simultaneous rotation and down forces are used to aid in advancing the hole (Figure 8-3). The vibration frequency can be controlled by the operator to maximize penetration rates. In certain conditions, the vibration frequency can produce resonance of the drill string, which can increase drilling penetration rates. Casings are commonly equipped with hardened shoes or shoes with carbide inserts that permit penetration through cobbles, boulders, and rock. Typical casing diameters range from 7.6 cm (3 in.) for the inner casing to 30.5 cm (12 in.) for the outer casing. The inner casing (commonly referred to as the "core barrel") is typically advanced ahead of the outer casing. Intervals of 10 or 20 ft are common in materials easily penetrated. In difficult conditions, shorter lengths may be used. In some cases, telescoping casing is used, particularly for deep holes or in difficult drilling conditions such as highly plastic clays. A nearly continuous representative core of the material penetrated can be retrieved from the inner casing (Figure 6-1). Some equipment permits the use of split inner barrels with clear plastic or stainless steel liners for sample preservation. The outer barrel remains in place during retrieval of the inner barrel, so the borehole remains fully cased at all times during drilling. The method can advance casing without the use of drilling fluids in most overburden formations, although small amounts of water are commonly used to accelerate the drilling process or control heaving sands. The larger rigs are typically truck mounted (Figure 8-4) and are not suitable for tight clearances or steep terrain. As with most other drilling methods, all-terrain or crawler-mounted small and large sonic rigs are available, as are small, skid-mounted and heliportable sonic rigs for steep terrain and difficult access. Sonically drilled holes to depths of 60–90 m are common in overburden, and deeper drilling depths are possible with this method.



(Courtesy of Boart Longyear)

Figure 8-3. Schematic of sonic drill operating principle.

(Courtesy of USACE Louisville District)

Figure 8-4. Sonic drill equipped with mast braced to permit angle drilling.

Sonic drills come in many sizes. For maximum efficiency, a large, truck-mounted sonic drill is used when possible, with a second support truck immediately behind the sonic drill to facilitate adding and removing tooling (Figure 8-5). In this configuration, sonic drilling requires a 21-m-long footprint to access a single hole. More mobile and accessible drills are now available, including small tracked rigs, as shown in Figure 8-6. A significant advantage of these systems is that little waste material is generated. The method explicitly meets the requirements of ER 1110-1-1807 if no water is used. According to the guidance provided in ER 1110-1-1807, minimal water may be used for two main reasons in sonic drilling: (1) to facilitate the penetration in very tough soil materials when the in-situ moisture content is not high enough to facilitate the shearing of the material, and (2) to control heave at the bottom of the borehole. Using water for sonic drilling is prohibited in an embankment dam in the impervious core and the cutoff trench, including foundation soil under the engineered embankment or near sensitive areas of the dam (homogeneous soil shells, adjacent to conduits, etc.). Using water for sonic drilling in other areas of the dam, levee, or appurtenances shall be avoided and shall only be used as a last resort to facilitate the penetration in very tough dry material. Users of this method are cautioned that the quantity of water permitted to be used and the timing for injecting or placing water in the hole must be specified. The amount of water used for any purpose shall be gravity fed to the collar of the borehole and shall be closely monitored to avoid raising the water level in the borehole to 15 ft above the phreatic water level in the embankment before drilling. The use of water during certain phases of sonic drilling can subject the embankment to significant water pressures. For example, injecting water during advancement of the outer override casing requires that the water travel up along the outside of the outer casing in direct contact with the embankment, and is prohibited.

(Courtesy of USACE Little Rock District)

Figure 8-5. Sonic drill with support truck.

(Courtesy of Gannett Fleming)

Figure 8-6. Track-mounted sonic rig.

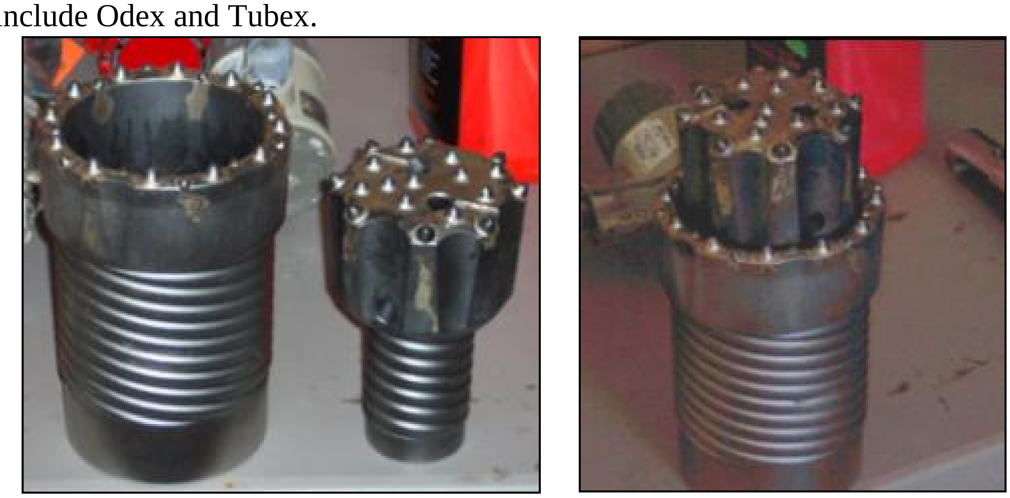
f. Single Tube Methods. These methods of drilling involve the advancement of a single casing by driving, rotation, washing, or jacking. Single tube methods are commonly employed in soil grouting projects where a drill string with a knock-off bit is provided. This is a common method of casing installation for compaction grouting projects. Jacking of steel, multiple-port sleeve pipes has also been used successfully for fracture grouting (Heenan et al. 2003). Single tube methods installed as "wash borings" with water flush are not appropriate in water-retaining embankments.

g. Duplex Methods. Several variations of duplex methods exist, all of which have the common characteristic of an inner drill string and an outer casing that are installed simultaneously. The flushing medium is injected through the drill string and returns to the ground surface within the annular space between the drill rods and the casing. The drill string is equipped with a suitable bit for the material to be drilled, and the bottom of the casing shoe can be equipped with carbide teeth or carbide button inserts. The bits used for drilling are as required for the formations to be penetrated and can be either rotary bits, such as a roller bit, or percussive bits (Figure 8-7). The inner drill string may consist of drill rods, where fluid flush evacuates cuttings to the surface, or drill rods with auger flights, where rotation of the auger and in some cases fluid flush evacuate cuttings. When drilling mud is used as the flushing fluid, approval will be required to use these methods in or near an earth embankment to satisfy ER 1110-1-1807. Typically the rigs for this method are medium to heavy in weight and are track or truck mounted (Figure 8-8). Numerous variations of the duplex drilling method have been developed. Some of the more common duplex drilling systems currently in use are:

(1) Rotary Duplex. This method advances and rotates both drill strings simultaneously.

(2) Rotary Percussive Concentric Duplex. This method is identical to the rotary duplex with the exception that one or both of the strings is percussed with a top-hole hammer.

(3) Rotary Percussive Eccentric Duplex. This method incorporates an eccentric bit that drills an oversized hole to simplify advancement of the outer casing. Trade names for this method include Odex and Tubex.



(Courtesy of Advanced Construction Techniques)

Figure 8-7. Example duplex casing shoe and drill bit.


(Courtesy of Moretrench)

Figure 8-8. Duplex drill installing relief wells at the toe of a dam.

(4) Double Head Duplex. This method can be the same as rotary duplex or rotary percussive concentric duplex with the exception that the casing and the drill string are rotated in opposite directions. The counter-rotation of the drill and casing minimizes the required torque of the machine and provides a straighter hole than when both rotations are in the same direction.

(5) Duplex with Auger. Another variation is the use of an auger string as the inner drill string (Figures 8-9 and 8-10). The auger can be used to remove the cuttings to the surface without the use of flush fluids. If no flush fluids are used, this method strictly satisfies all the requirements of ER 1110-1-1807 without having to resort to drilling mud. The system is also more protective than most other methods, even when water is used as the flushing fluid, since the majority of the flush water is directed up-hole and inside the outer casing, so the walls of the drill hole are not subjected to direct water pressure from the drilling operation. Plugging the hole in the bottom or face of the bit would increase the level of protection, as all fluids would be maintained within the outer casing. Should the use of this system be approved in an embankment dam as required by ER 1110-1-1807, all water discharge holes in the auger bit should discharge up-hole within the outer casing to prevent direct water pressure on and direct damage to the embankment material.

8-3. Casing of Holes.

a. General. Casing of holes through overburden is required if subsequent rock drilling and grouting is to be performed in the underlying bedrock. Casing eliminates erosion of the hole during rock drilling, washing, and grouting operations; eliminates the potential for holes to cave in the overburden; and aids in the alignment of subsequent rock drilling. Cased holes also allow the packer to be seated inside the casing for treatment of the soil/bedrock interface zone and the top several feet of rock, which is often very fractured. Cased holes are also appropriate in some solution grouting applications. In these applications, multiple-port sleeve pipe (MPSP) casing provides grouting ports at regular intervals along the casing (Bruce 1988). The ports are covered with a rubber gasket that expands under pressure and allows grout to flow from the inside of the casing to the sidewall of the hole and into the soil. When the injection pressure is removed, the gaskets contract, preventing the injected grout from intruding back into the casing. In essence

MPSP acts as a check valve, allowing for multiple injections in a particular zone. Figure 8-11 shows an example MPSP system.

(Courtesy of Advanced Construction Techniques)

Figure 8-9. Rotary duplex auger drill rig with dual hammers.

(Courtesy of Advanced Construction Techniques)

Figure 8-10. Duplex system equipped with a carbide casing shoe and an inner auger string with a carbide bit.

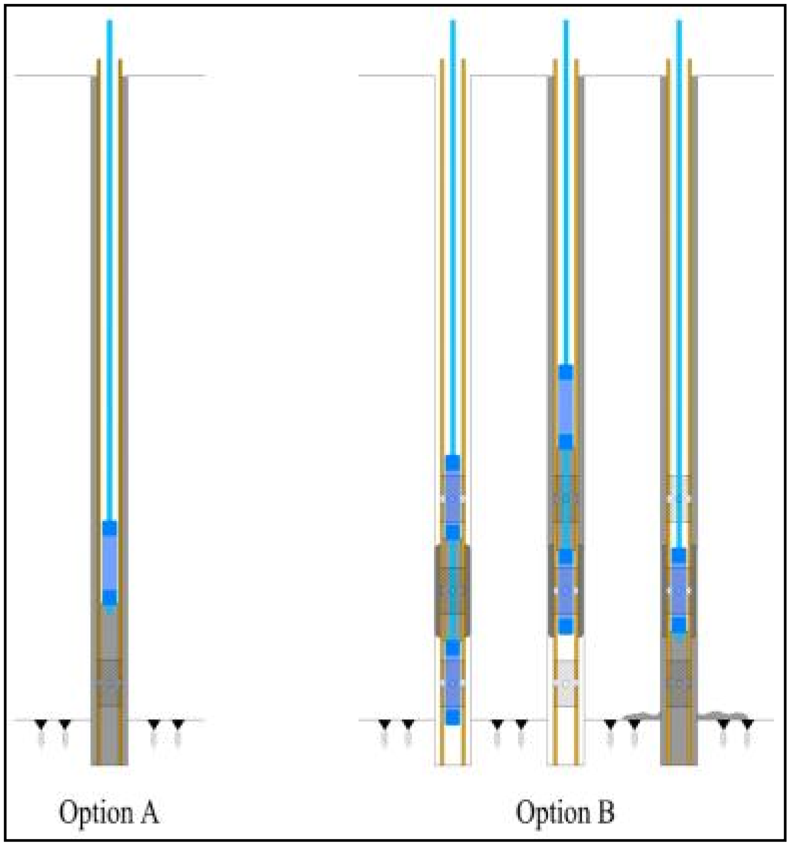
(Courtesy of Gannett Fleming.)

Figure 8-11. MPSP; the photo on the left shows a port beneath the gasket.

b. Casing Materials. The grouting contractor can determine the size of the casing to accommodate the tooling used for rock drilling and grouting. Obviously, more accurate holes can be drilled with larger-diameter drill strings, but they require larger casing and more grout backfill. The selection of casing materials is somewhat dependent on the method and depth of drilling, as some methods use casing as part of the drilling process. This casing can be left in place or later removed with a casing puller at the completion of grouting. It is often more economical to support the overburden bore hole with plastic or light-gauge steel casing. Depending on the size of the hole drilled through the overburden and the required casing size, this casing can be flush joint or coupled pipe. In soil permeation grouting applications where MPSP is used, plastic casing is the most common. Where LMG is to be performed in soil, it is most common to leave the steel drill casing in place and inject grout directly through the tooling. However, steel casing may be withdrawn at a controlled rate during placement of LMG, depending on the length of the zone to be treated.

c. Casing Sealing. Sealing of the casing within the embankment or overburden is necessary before drilling and grouting operations in underlying rock. Several alternatives exist for proper isolation and sealing of the casing. In some cases, installation of a temporary drilling casing seated a minimum distance into rock has been found to be acceptable. That minimum distance should be clearly specified in the contract documents. If a higher level of protection is required, such as for use in remediating a water-retaining structure, then one of the options described in the following paragraphs should be specified. When casing is sealed within an embankment, the procedures used shall comply with the requirements in ER 1110-1-1807.

(1) Option A. Figure 8-12 (left) shows this option. The bottom section of the casing must have at least one port for grout injection. After a hole has been drilled the required distance into the bedrock, the overburden casing is installed and grouted into place. The overburden casing is grouted by setting a packer near the bottom of the hole and injecting grout so that the grout exits the ports and travels up the annulus between the overburden casing and the drill casing or the wall of the hole. Grout is injected through the bottom of the casing and through the ports until grout is returned at the surface through the annulus. At this point, the drill casing can be extracted and the grout “topped off” as each section of temporary or drill casing is extracted.



(Courtesy of Gannett Fleming)
Option A – Grout the annulus to the surface through a single port in the casing.
Option B – Inflate a barrier bag, grout the annulus to the surface, and treat the interface.

Figure 8-12. Overburden casing sealing options in the non-critical sections of the dam.

(2) Option B. If specific testing and/or treatment of the soil-rock interface is required, then alternative details must be developed. MPSPs equipped with geotextile barrier bags or other barrier systems can be used to isolate the interface for treatment. Once the MPSP casing is seated into rock the desired distance, a barrier bag located a defined distance above the top of rock is inflated with grout. After the bag is inflated to isolate the interface, the straddle packer is moved to the next port above the barrier bag and grout is injected to fill the annular space. The ports below the barrier bag are then available for testing and treatment of the interface and top of rock. Alternatively, the barrier bag may be located at the embankment/ foundation interface. However, grouting of the interface would occur concurrently with grouting of the casing annulus. This type of system is somewhat more time consuming to use, but it results in a high level of confidence that the borehole annulus has been completely filled, and it allows subsequent treatment of the foundation interface and uppermost portions of bedrock.

(3) Completely filling the borehole annulus and positively isolating the embankment from the foundation are of paramount importance in remediating water-retaining structures. In many instances, damage to the embankment has occurred during the life of the structure due to the loss of materials into inadequately treated or untreated solution features or fractures in the foundation. Inadequate casing grouting methods could result in a hydraulic connection to the foundation and further loss of embankment material. Positive isolation of the embankment from the foundation

is therefore critical. Barrier bag systems are particularly attractive in these circumstances. Once the bag is inflated in the borehole, positive isolation is immediately achieved, and completely filling the borehole annulus is simply a matter of raising the straddle packer system and injecting through the next port. Inflation of the bag typically takes about 30 minutes after the hole is drilled. (If, instead, casing grout were used as the sole method of achieving isolation from the foundation, positive isolation would not be achieved until the casing grout sets.)

(4) If casing is being installed at a location where there is low confining stress, then it may be necessary to seal the casing by grouting in stages to prevent hydraulic fracture and significant loss of grout. This can be accomplished by drilling boreholes that provide enough clearance for the use of tremie grout tubes on the outside of the casing. The grout should be allowed to reach its initial set before the next stage can be done. Additives such as calcium chloride or other accelerators can be used to control set times. The bottom of the augers or casing should not be raised until the first grout stage is placed to its full height. Once it is placed, the augers or casing can be slowly raised to the top of the grout column. The grout column should be closely monitored and checked after each section of drill string is removed. Supplemental grout should be placed if minor grout losses occur. Once the augers or casing reaches the top of the grout column, the grout shall be allowed to reach initial set. Once initial set is reached, the next grout stage can be placed and the process repeated.

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CHAPTER 9

Rock Drilling, Washing, and Pressure Testing Equipment

9-1. Rock Drilling Equipment.

a. Introduction. This chapter provides information on the basic requirements associated with drilling grout holes in rock and discusses the various methods for drilling. The selection of the drilling method primarily depends on the characteristics of the rock to be drilled, the hole depth, access to the hole location, and the requirement to drill a properly aligned clean hole at a reasonable cost and in a reasonable time.

b. Basic Requirements.

(1) Hole Diameter. Typically, the diameters of grout holes are in the range of 2–5 in. with the preference of 3-in. diameter in rock. Any specific requirements for hole size should be clearly stated in the contract documents. Depending on the application, such as drilling through a concrete structure and into rock, a maximum hole size provision might also be appropriate to minimize damage to the structure, such as cutting reinforcement. The selection of the minimum or maximum hole diameter requirements, which should be site specific, is based on the hole depths, the equipment access requirements for the subsequent grouting operations, and the need for any special treatments, such as telescoping casing through unstable strata within the rock.

(2) Hole Orientation. Proper orientation of the drill hole begins with an accurate layout of the hole locations at the surface. Proper alignment is enhanced by providing a stable platform for the drill rig, an accurate setup of the rig, and monitoring of the continued correct orientation. This is accomplished by conventional survey equipment used to confirm the proper drill layout (Figure 9-1), electronic levels, or orientation equipment that can be attached to the drill mast. Electronic levels with accuracies of 0.1 degrees are readily available and should be used in the absence of other methods. Sophisticated orientation equipment can be fit to most drilling rigs and typically consists of a sensor unit and a remote display. The sensor unit is permanently mounted on the drill mast, and the remote display is mounted near the drill operator controls. These devices provide the ability to rapidly and accurately set up a drill properly for alignment. The sensor unit that is permanently mounted to the drill mast contains two digital inclinometers. The internal inclinometers measure both dip and dump angles simultaneously. During the setup process, mast angles are constantly being measured and displayed. This improves the driller's efficiency and accuracy during setup, and since the display is mounted near the drill controls, the operator is able to monitor the alignment of the mast during drilling of the hole.

(Courtesy of Gannett Fleming)

Figure 9-1. Use of conventional survey equipment to align a drill rig.

(3) Hole Deviation. Factors that affect hole deviation, typical deviations to be expected, and impacts of hole deviation are addressed in Section 15-6. b. Historically, deviation of holes has been determined by lowering a light to the bottom of the hole and observing the portion of the hole that is visible at various depths. This provides only an approximate measure of the hole deviation, and depending on the hole diameter, the light may go out of sight, even for holes that meet the alignment requirements. Another method consists of a camera with a pendulum and compass deployed down the hole. The instrument is stopped at regular intervals, and a photograph is taken of the pendulum and compass, indicating the direction and dip of the hole. More accurate measurement of the hole alignment can be obtained using more sophisticated techniques. Gyroscopes and inclinometers can be used to accurately measure hole alignments in nearly any set of conditions and at frequent intervals. Available borehole imaging systems, such as the Robertson Geologging System (see Section 6-4), integrate borehole deviation surveys with imaging of the hole sidewalls. The imaging of borehole walls and fractures combined with the measure deviation of the hole allows the strike and dip of the fractures to be accurately calculated. Systems that use magnetic north for orientation cannot be used inside metal casings.

(4) Hydraulic Applications. Grout holes that are drilled for the purpose of grouting rock must be drilled with water as the flushing medium. Drilling with air as the flushing medium should not be permitted if subsequent rock permeation grouting is to be conducted. Air can significantly damage the formation if the hole becomes plugged even for a short time. Air can also blow dust and particles of rock into the fractures and prevent grout penetration, particularly when there is a small amount of water in the hole, which can combine with the dust to form a paste that then coats the fractures. Use of air flush methods for other grouting applications such as void filling, compaction grouting, and structure-foundation stabilization might be acceptable, since the clogging of fractures is not critical.

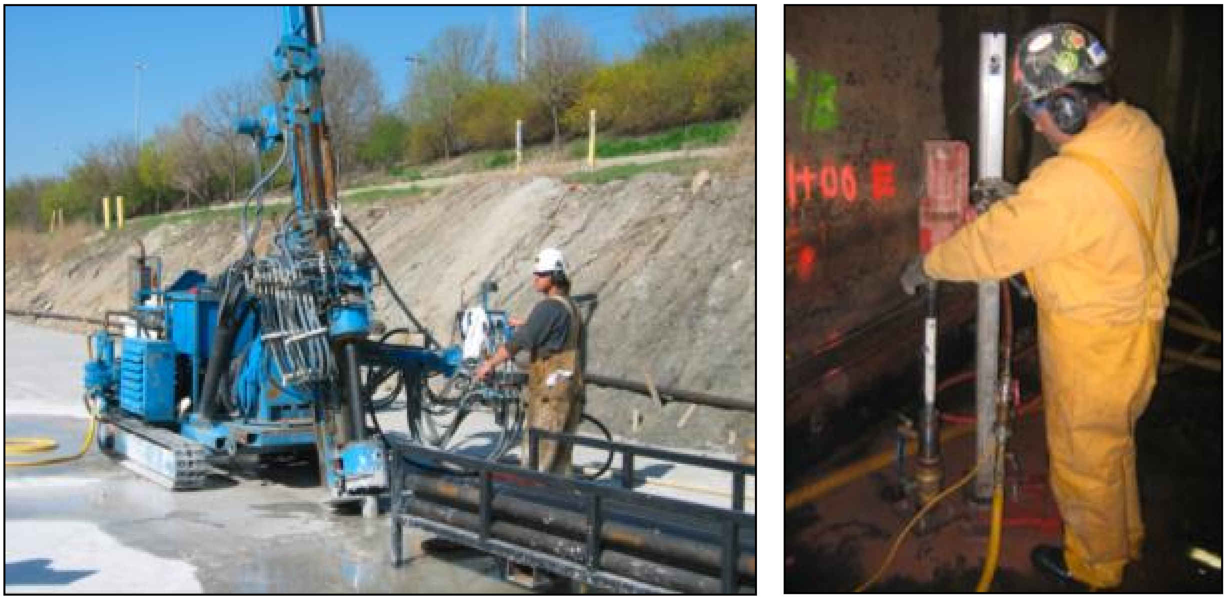
c. Selection of Drilling Method. There are two basic methods for drilling grout holes in rock: rotary drilling methods and rotary percussive drilling methods. Each of these basic methods has variations that may be used. Rotary methods can be subdivided into high rotational speed and low rotational speed. The rotary percussive methods can be subdivided into top-hole rotary percussion and down-hole rotary percussion.

(1) Rotary Methods. High-speed rotary drills apply limited torque and thrust and rely on the high-speed rotation of the bit to cut the rock. Low-speed rotary drills require large drills with high torque and thrust to destroy the rock fabric.

(a) High-Speed Rotary. The most common method of high-speed rotary drilling is diamond drilling. This method has long been used for drilling rock. In the past, it was considered to be the best method because it created smooth-walled holes that made setting packers easier, and it resulted in straighter holes. However, it has been shown repeatedly in various tests that other methods described in this section produce acceptable grout holes at a lower cost and generally in less time. An advantage of the diamond rotary method is that equipment is readily available in smaller and lighter rigs, and they can easily drill to the depths generally required for grouting. Diamond rotary holes can be drilled using destructive or nondestructive methods. If no sample is desired, plug bits can be used. With the advent of wire line drilling methods, plug bits are rarely used, as the area of rock requiring cutting is the total area of the hole. Coring is generally faster than plug bits and provides the advantage that the rock being grouted can be directly observed. Figures 9-2 through 9-4 show examples of high-speed rotary drills.

(Courtesy of Gannett Fleming)

Figure 9-2. Truck-mounted high-speed rotary drill rig.



(Courtesy of Advanced Construction Techniques.)

Figure 9-3. Track-mounted and hand-held high-speed rotary drills.

(Courtesy of Gannett Fleming.)

Figure 9-4. High-speed rotary drills for difficult access and confined spaces.

(b) Low-Speed Rotary. This drilling method employs large drills capable of installing larger-diameter holes to considerable depth. The large down-pressure and torque applied to the drill string and bit are used to penetrate the rock. The most common use of these drills is for water well drilling, where air is commonly used as the flushing medium. If water is used as the flushing medium, there is no reason that this drilling method would not be acceptable for drilling grout production holes. However, economics and drill rig accessibility issues generally lead to the selection of alternative methods.

(2) Rotary Percussive Methods. These drilling methods use a hammer to impart percussive energy to the bit while the drill head imparts slow rotation. For top-hole percussion, the hammer is mounted on the drill, and the energy is applied to the bit through the drill rods. For down-hole methods, the hammer is installed just above the bit and is activated by the drilling fluid while the drill head rotates the string.

(a) Top-Hole Percussion. The bottom of the drill string is equipped with a chisel or button bit. Flushing of the hole for this type of drilling can be air, water, or drilling mud. However, only water is permitted for grout hole drilling. Percussion drills can be truck mounted, but more often they are track mounted. Although these rigs can be relatively lightweight, such as the familiar "air-track drill," the tendency is toward heavier rigs with self-contained power packs that are more stable during the significant vibration that occurs during drilling. The significant advantage of the track-mounted rig is its maneuverability, especially on steep slopes. The masts for these rigs can also typically be tilted front to back and side to side so that holes can be drilled in nearly any direction. Since this method results in the drill string being percussed at the top, the energy must be transmitted through the drill rods and couplings to the bit, which results in significant loss of energy. Therefore, the achievable depth with this method is limited, and the hole orientation accuracy decreases with increasing hole depths and decreasing tool diameters. The smaller the diameter of the tooling, the greater the bending that occurs when impacted. A stiffener or larger-diameter lead or starter rod (Figure 9-5) should be used to increase drill hole accuracy. Depth limitations with top-hole percussion drills vary from drill to drill. An air-track drill (Figure 9-6) is challenged beyond depths of approximately 18 m. Deeper depths can be realized, but production rates drop rapidly with increasing depths. The drill in shown Figure 9-7 is limited to a depth of about 45 m. However, the last 15 m might take as long to drill as the first 30 m.

(Courtesy of Advanced Construction Techniques)

Figure 9-5. Top-hole percussion drill tooling. Bit is shown at top left followed by larger-diameter starter rod, followed by traditional drill rod.

(Courtesy of Gannett Fleming)

Figure 9-6. Air-track, top-hole percussion drill.

(Courtesy of Advanced Construction Techniques)

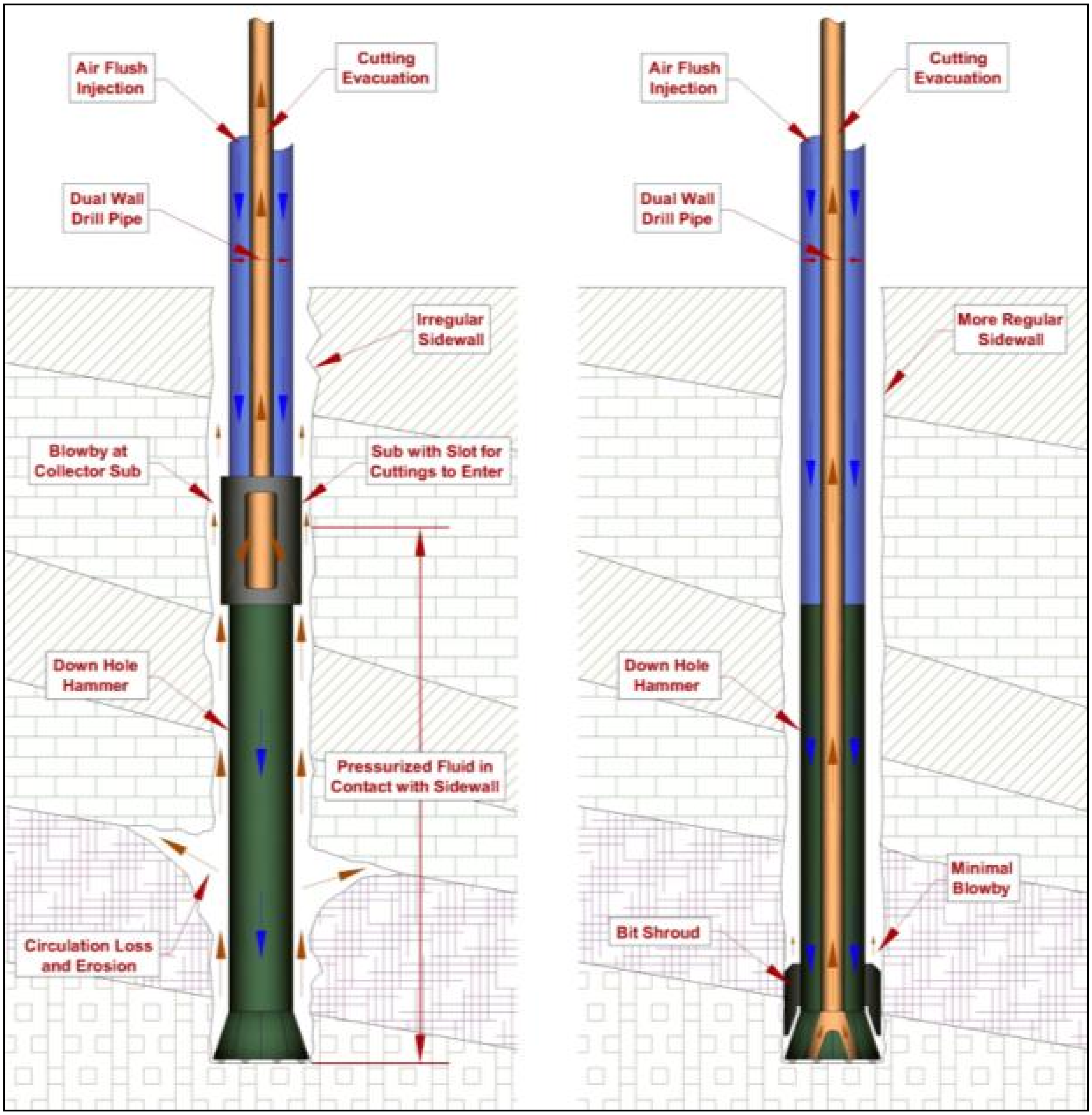
Figure 9-7. Self-contained, track-mounted, top-hole percussion drill with a rod handler.

(b) Down-the-Hole (DTH) Hammer Methods. DTH hammers significantly improve production and drill hole accuracy and are capable of drilling significantly deeper than top-hole percussion methods. Production is dramatically improved because the hammer is located at the end of the drill string at the bottom of the hole. This eliminates the need to transfer the hammer impact energy through the drill string, which results in 100% of the impact energy on the bit regardless of depth. Drill hole deviations are dramatically reduced for the same reason. There are currently four available technologies for DTH hammer design. These include:

- Direct Circulation (DC) air-driven DTH hammers use air flushing in contact with the sidewalls over the entire length of the drill hole.

- Reverse Circulation (RC) DTH hammers use dual-wall drill pipes (Figure 9-8). RC hammers use air or air with a water mist to drive the hammer and to flush the hole. The air or air/water mist flush for scouring the hole is returned to the ground surface through an inner rod, which helps keep the hole cleaner by protecting the hole from the cuttings and the flushing medium. Two RC hammer technologies are available:

- The first, more common system uses a sub or oversized adaptor at the top of the hammer to deflect the cuttings and flushing medium into the inner drill string and away from the hole. With this RC hammer technology, the cuttings and the flushing medium are in contact with the hole over the length of the hammer. This technology significantly reduces, but does not eliminate, hole contamination due to the cuttings and flushing in comparison to the DC hammers. Erosion in soft rock formations and irregular borehole sidewalls should be expected.



(After SDS Corporation)

Figure 9-8. Details of RC systems.

- The other technology for RC hammers uses a hammer body and bit with an inner open annulus that connects to the inner drill rod. A bit shroud directs the cuttings and flushing medium into the inner drill string right at the base of the hole. This technology reduces hole contamination to an even greater degree, as drill cuttings have no contact with the hole sidewall except in the event of "blow by" of the oversized bit. This occurs occasionally in softer rocks and broken zones within a formation.

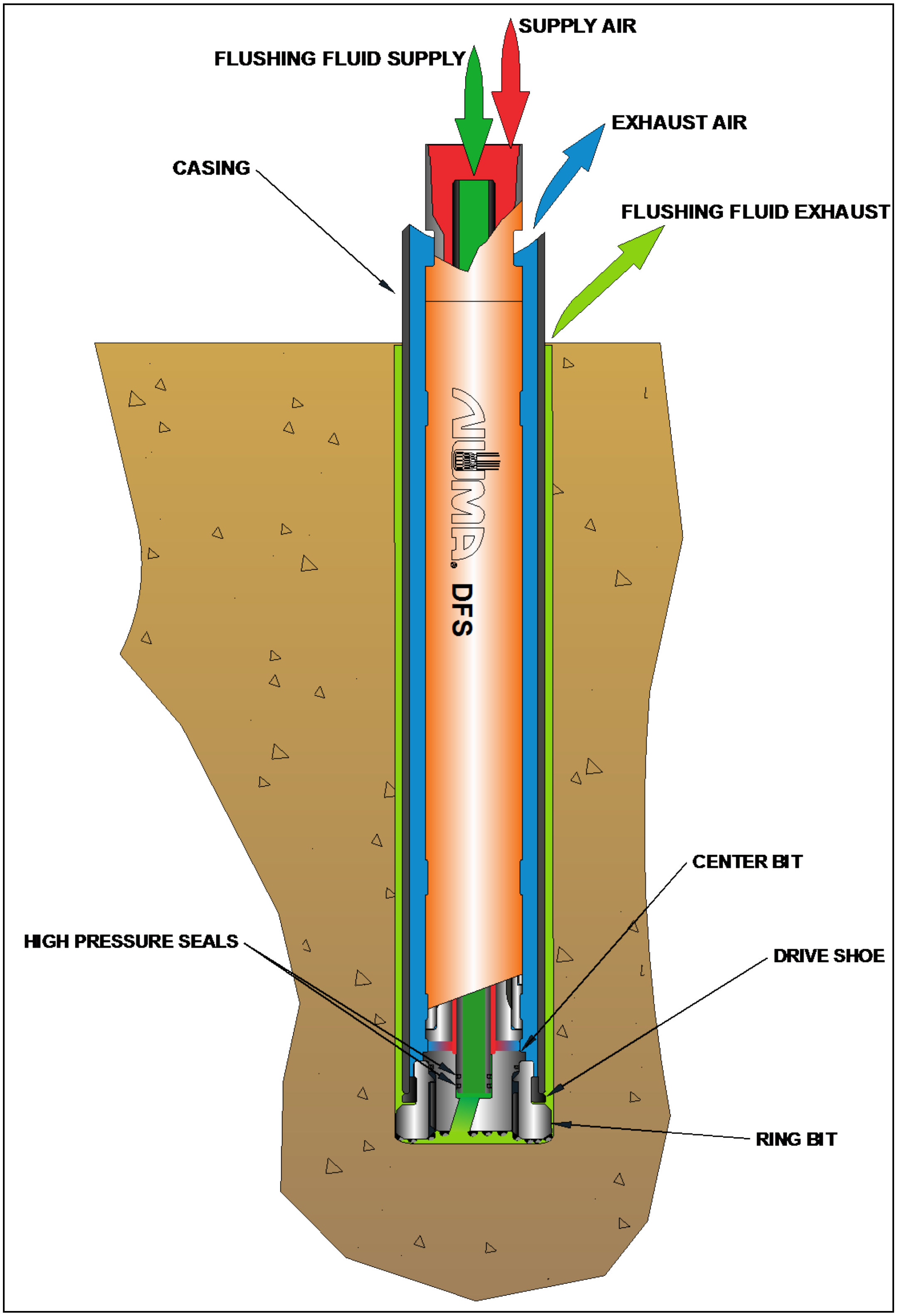
- Neither RC system is appropriate for use in drilling holes for grouting, unless the intention is to encounter a large void for a void-filling application. The likelihood of system clogging is dramatically increased with these systems and cannot be eliminated. When the system is clogged, the high-volume, high-pressure air used for hole flushing can cause significant damage to the geologic formation.

- Dual-Fluid Drilling Systems (DFS) are a recent addition in DTH technology and are available from Numa (Figure 9-9). The DFS consists of a hammer that is air activated, but incorporates a center tube through the hammer body that allows 100% water to be used as the flushing agent. Water is pumped down the center tube and out the bottom of the bit. The water cleans the face of the bit and forces the cuttings up the annular space between the drill string and the hole. This system virtually recreates the top-hole percussion drilling scenario described previously that has been widely accepted in the grouting industry. The only difference (which is a large advantage for deep holes) is that the hammer is now at the bottom of the hole. The air for driving the hammer is exhausted between the outer casing and the inner drill string back to the ground surface and is never in contact with the formation.

- Water DTH Hammers (WH) are available from Wassara through Cubex in North America (Figure 9-10). These hammers use high-pressure water to activate the hammer. Pump pressures of 1,000–3,000 psi and water flow rates above 70 gpm are used to drive the piston. After impacting the piston, the water goes through a dump/diverter valve and an energy dissipation tube. The energy dissipation tube carries the water to the bit, where it exits into the hole as the flushing medium. Pressures exiting the bit are similar to pressures in other rotary percussive top-drive systems. The design of the water hammer and the use of water rather than air for activation result in a percussion rate about twice as high as other down-hole hammers, which increases productivity. The maintenance on the hammers is relatively high, but this cost can be offset by productivity gains on large projects with deep holes.

d. Measurement While Drilling (MWD). MWD, also known as “Monitoring While Drilling” or Drilling Parameter Recording, is the recording of certain mechanical properties during drilling of a borehole to assist in interpreting subsurface conditions. The values of interest typically are penetration rate, torque, rotational speed, and thrust. These variables can be used to calculate a parameter known as specific energy, which is a measure of the energy required to penetrate a unit volume of material and is expressed in units of energy over volume. In some cases, water flush pressure and/or flow rate may be measured and correlated with zones of high permeability. MWD has become more common as drilling equipment has become more sophisticated. In many cases, equipment manufacturers monitor these same parameters to gauge equipment performance and troubleshoot problems. It is natural that these same parameters are of interest in quantifying the conditions encountered during drilling.

(1) In a few cases, MWD has been specified for use on USACE grouting projects. One project of interest is the Clearwater Dam Major Rehab project, where all rock drilling rigs on the site were equipped to record drilling parameters. In general, MWD is successful in identifying zones of fractured rock or poor rock quality. These zones often correlate well with zones of high permeability and high grout take.



(Courtesy of Numa)

Figure 9-9. Numa dual-fluid system schematic.

(Courtesy of Advanced Construction Techniques)

Figure 9-10. Water-powered down-hole hammer in action at McCook Reservoir. A large drill was used as depths exceeded 400 ft on this project.

(2) The necessity of using MWD methods on grouting projects should be evaluated during project design. It is recommended for projects where detailed delineation of foundation conditions or discontinuities is required. The premium paid for using drilling equipment capable of MWD is generally minimal. However, post-processing and review of the data by qualified persons are necessary for meaningful interpretation.

9-2. Hole Washing Equipment.

a. General. Holes must be thoroughly washed before grouting to maximize access of grout to fractures. At the completion of drilling the grout hole, the washing operation begins by raising and lowering the drilling tools several times a short distance and allowing the drill water to circulate. This removes the coarser cuttings that tend to settle at the bottom of percussion-drilled holes. However, drill water circulation alone is not adequate to properly clean the holes. A separate hole washing step is a standard requirement after drilling.

b. Equipment. Equipment used in the washing process includes washout bits, pumps, and hoses and pipes to inject the water. Typically hoses with a minimum diameter of 2.5 cm (1 in.) are used for injecting water. Hose is preferred to pipe since it is lighter and can be used to a greater depth because the washing process is often done by raising and lowering the wash equipment by hand. The insertion and retraction of the hose can also be done more quickly than pipe, which has to be added or removed in sections. A special washout bit is attached to the bottom of the hose. The diameter of the bit should be similar to that of the borehole, but small enough to permit rock fragments and cuttings to pass between the bit and the hole sidewall. The bit has closely spaced holes (approximately 3.2 mm [1/8 in.] in diameter) spaced around its perimeter to permit radial flow to be directed at the sidewall, in addition to the bottom discharge. Water pumps should be capable of supplying water at a minimum pressure of 100 psi and flow rates at a minimum of 30 gpm at the top of the hole. Depending on the formation being cleaned, the pressure may need to be limited to prevent damage to the rock. After a hole is completely washed, a plug should be installed in the hole collar to prevent debris from entering the hole. If the hole is in a depression, a standpipe can be grouted into the top of the hole and then plugged. Figures 9-11 and 9-12 show hole washing equipment.

(Courtesy of Gannett Fleming)

Figure 9-11. Lightweight washout bit and hose for manual applications.

(Courtesy of Advanced Construction Techniques)

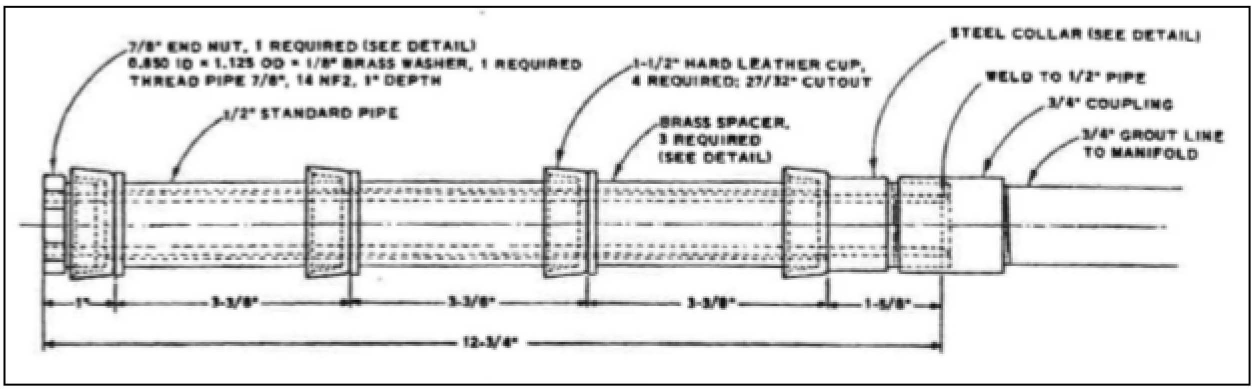
Figure 9-12. Track-mounted mechanical washing system for deep holes.

9-3. Pressure Testing Equipment.

a. General. Grout holes are pressure tested to provide information on the permeability of the formation to be grouted, to identify highly permeable zones, to help identify unstable zones that may require special grouting, and to evaluate the permeability reduction being achieved as grouting progresses. The equipment for water testing is generally the same as that used for grouting. Refer to Chapter 10 of this manual for a more detailed discussion of pumps, gauges, flow meters, and valves.

b. Packers. Packers are used to isolate a portion of a hole for the purpose of injecting water under pressure into the isolated area. Packers are essentially a length of pipe, smaller in diameter than the grout hole or casing, with expandable devices attached to the pipe to create a seal between the pipe and the grout hole. There are several varieties of packers, including friction, mechanical, and inflatable. The type of packer recommended depends on the expected uniformity of the hole size and whether the packer will be installed in the hole casing or directly in the grout hole.

(1) Friction packers can only be used where the packer is seated within a grout hole standpipe. It employs U-shaped cups or O-rings to create the seal (Figure 9-13). With the U-shaped cups, the pressure of the water acts on the inside of the U-shape to expand the cup against the casing. The O-ring packer creates the seal by a tight friction fit between the O-ring and the casing. Both of these can sustain high pressure, but are subject to high wear due to the tight fit and must be replaced frequently. Currently, these types of packers are generally used only with MPSP and have been mainly replaced by pneumatic packers.



(Courtesy of U.S. Bureau of Reclamation.)

Figure 9-13. Cup-type friction packer.

(2) Mechanical packers include a soft rubber or elastomeric sleeve that fits snugly over a threaded metal tube (Figure 9-14). When a threaded coupler is turned, the sleeve expands laterally against the sidewall. The amount of expansion depends on the sleeve material, but is typically about 1.2 times the original diameter. Mechanical packers have some limitations. Due to their short length and limited expansion capability, there is a possibility that grout will bypass the packer because roughness in the borehole has caused an inadequate seal or because grout has flowed through the fractures and around the packer. This can cause the packer to become stuck in the hole and can cause loss of control of the flow of grout. Mechanical packers are commonly

used when grouting the top stage through a grout cap if the length of an inflatable packer is greater than the thickness of the concrete. Another common application is installation at the top of adjacent grout holes when interconnections occur with the hole being grouted.

(Courtesy of Palm Equipment, Inc.)

Figure 9-14. Mechanical packers.

(3) Inflatable packers, also called pneumatic packers, consist of an expandable sleeve that fits over a tube attached to the grout pipe (Figure 9-15). The ends of the sleeve are sealed, and a fitting is incorporated so that air, water, or gas can be injected into the sleeve to cause it to expand. The sleeve is typically 0.9–1.2 m (3–4 ft) in length, but can be of any length. Longer packers provide additional bond length for successful seating and will resist higher pressures. The amount of pressure that the sleeve can sustain depends on the material, which is typically reinforced hose, but sleeves with allowable working pressures of 500 psi are readily available and are adequate for most grouting projects. Inflatable packers can be constructed with fixed ends or with one sliding end. Fixed-end packers have both ends of the sleeve sealed against the grout tube. As the sleeve is expanded, longitudinal stresses are introduced into the sleeve, which restricts the amount of radial expansion. Typically, the sleeve can expand about 1.3 times the initial diameter of the sleeve, with a maximum of about 1.5 times the diameter. Sliding-end packers can achieve a maximum expansion of about 1.9 times the initial sleeve diameter. However, sliding-end packers are more expensive and more prone to problems, especially with maintaining the O-ring seal and maintaining free movement of the sliding end.

(4) In downstage grouting, a single packer can be used. Upstage grouting normally requires a double packer system with a perforated delivery line between the packers to deliver water to the formation, as shown in Figure 9-16. It is important that the system performance characteristics be verified by pumping water and the grout mixtures being used through the

system at various pressures to measure the volume the system is capable of delivering at each pressure and to measure head losses for calculating effective pressures.

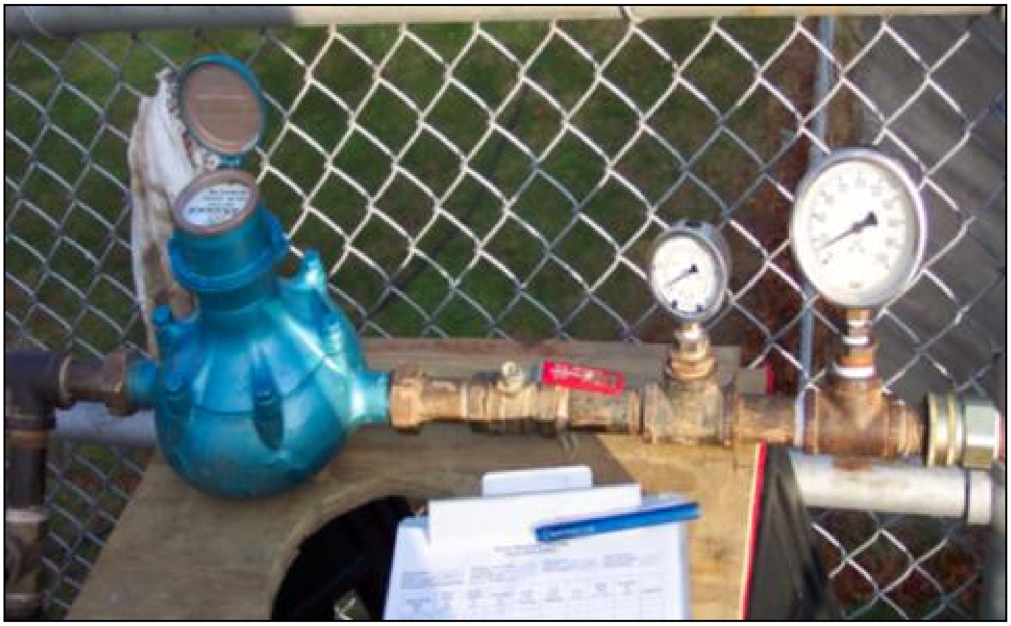
(Courtesy of Palm Equipment, Inc.)

Figure 9-15. Inflatable or pneumatic packer.

(Courtesy of L.G. Hetager Drilling, Inc., of Punxsutawney, PA.)

Figure 9-16. Assembled double packer system.

c. Mechanical and Electronic Pressure and Flow Measurement Systems. Pressures and flow rates during pressure testing can be monitored either with mechanical systems or electronic systems. During the design investigation phase, mechanical systems composed of pressure gauges and mechanical flow meters are sometimes used because the scope of the program does not warrant electronic systems. However, mechanical systems are rapidly disappearing even in these applications and are being replaced by pressure transducers and electronic flow meters. For pressure testing during the grouting phase, sufficient testing is usually performed to warrant the use of electronic pressure transducers and flow meters. Refer to Chapter 15, Paragraph 15-9. c. (1) for further discussion on in-stage direct pressure measurement. For projects that use automated grouting control systems, the pressure testing is also done with the same system, which requires the use of electronic controls for the pressure gauges and flow measurement. Chapter 12 of this manual gives additional information on electronic controls. Figures 9-17 and 9-18 show the measuring devices for a pressure testing system. Note that both systems have a tee just before the instruments. This tee and the associated valves are used to control the pressure applied to the hole by bypassing water back to the pump before the pressure is applied to the formation. This type of system is commonly referred to as a "return loop" or "circulation loop." All equipment should be calibrated, and the calibration should be routinely checked on-site with an economical system. Checking the calibration might vary from filling a vessel of a known volume for mechanical meters to keeping a "master header" to periodically check the calibration of electronic equipment. A master instrument system consists of a calibrated flow meter and pressure transducer pair that is not used in production and is held by Government officials for the sole purpose of checking the calibration of production equipment.



(Courtesy of L.G. Hetager Drilling, Inc., of Punxsutawney, PA)

Figure 9-17. Manually read mechanical water meter and pressure gauges for pressure testing.

(Courtesy of Gannett Fleming)

Figure 9-18. Electronic system for flow and pressure readings used in pressure testing.

CHAPTER 10

Grouting Equipment for HMGs

10-1. General. There are many equipment variations available for each component of a grouting system. Likewise, there are several ways to assemble the required components, ranging from self-contained trailer or skid-mounted systems (Figure 10-1) that can be located immediately adjacent to the grout hole, to individual components that are distributed across the project site with the grout being pumped 250 m or more from a central mixing plant (Figure 10-2). Because the number of holes to be grouted, the accessibility of the holes, the site topography, the geology, the climatic conditions, and the size of the project will have major impacts on the selection and layout of the equipment and system(s) used, it is best to give contractors the flexibility to determine the best configuration and layout for their plant. USACE's role regarding this aspect of the work should be to specify what minimum equipment capabilities are required, to designate the contractor's work limits, and to prepare USACE's construction cost estimate. These tasks require knowledge of the equipment, coupled with assumptions regarding a practical layout for this equipment. Adequate staging area should be provided, and the equipment capability needed to complete the job within the schedule must be estimated. Chapter 13 of this manual provides additional information regarding site development. This chapter describes the characteristics of components and systems.

(Courtesy of Gannett Fleming)

Figure 10-1. Mobile grout plant capable of being moved around a site near the injection point.

(Courtesy of Advanced Construction Techniques)

Figure 10-2. Site with a centralized mixing station.

10-2. Grout Mixers. The grout mixing equipment must be capable of thoroughly wetting all of the particles of cement and other admixtures to create a homogeneous suspension. In the past, paddle mixers have been used for mixing grout on some projects. These employ slowly rotating paddles mounted horizontally or vertically in a tank. Paddle mixers alone are not satisfactory for producing high-quality HMG and should only be used for large backfilling or void-filling projects where incomplete dispersion of the cement particles and interruptions of the grouting are of little consequence. Current technology for high-quality grouting with HMG employs high-shear mixers. These are sometimes referred to as "colloidal mixers," although this name is not accurate since colloidal particles are generally considered to be less than 5 μm in size, which is smaller than most ultrafine cement particles. High-shear mixers generally consist of a vertical conical tank with a rotor located at the base rotating at 1,500–2,000 rpm. This imparts a high shearing force to the grout as the material is forced through the rotor housing by the rotor. The rotor then re-circulates the grout to the mixing tank. The grout re-enters the tank tangentially, which produces a vortex that aids in the thorough cement wetting process. Figure 10-3 shows a schematic of a high-shear mixer; Figure 10-4 shows a photograph of a rotor; and Figure 10-5 shows the vortex that forms within the mixing tank. Mixing with high-shear mixers is extremely efficient and can be completed rapidly (~1 minute) after all components have been added to the mixer. The high shear imparted to the grout generates substantial heat, which accelerates set time. Therefore, mixing time should not be excessively long and must be controlled. These mixers are capable of mixing neat cement grout mixes with water-to-cement ratios as low as 0.5:1 by volume or about 0.35:1 by weight. A minimum mixer size or capacity of 225–280 L (8–10 ft^3) is acceptable and appropriate for most rock permeation grouting projects. A mixer of this size can easily handle "two-bag" or "three-bag" mixes, which are common for most grouting projects. Mixer capacities that are too small result in an overworked mixer operator and interruptions in the grout supply. Mixer capacities that are too large result in excessive minimum batch sizes that in turn result in extended time periods between mix changes, increased average age of the injected grout, and greater quantities of wasted grout. High-shear mixers on the order of 2 m^3 (70 ft^3) in size are available for projects requiring large volumes of grout, such as backfilling behind pre-cast tunnel segments or void filling. Paddle-assisted high-shear mixers provide even larger quantities, more than 5.6 m^3 (200 ft^3) per batch, and consist of a series of high-shear rotors surrounding a paddle mixer. Paddle-assisted mixers allow the rotors and paddle to operate independently, so that once the mix is batched, it can be agitated by the paddles until dispensed. Figure 10-6 shows a 500 L (17.7 ft^3) mixing plant.

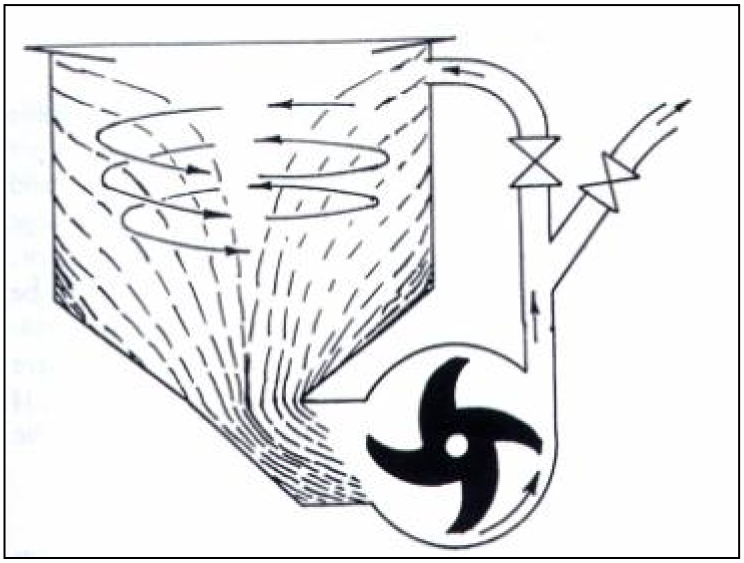


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Figure 10-3. Schematic of a high-shear mixer.

(Courtesy of Advanced Construction Techniques)

Figure 10-4. Rotor of a high-shear mixer with the mixer cover removed.

(Courtesy of Gannett Fleming)

Figure 10-5. Vortex inside a high-shear mixer.

(Courtesy of Gannett Fleming)

Figure 10-6. 500-L high-shear mixer (on left) with a 1,500-L agitator.

10-3. Agitators. The purpose of an agitator is to keep the grout mix in suspension after the initial mixing is complete. The mixer discharges to the agitator, which is a cylindrical tank with one or more paddles rotating at the speed necessary to keep the grout in suspension. The speed of rotation of the paddles is generally about 60 rpm. It is important to keep the speed just high enough to prevent settlement of the mix while slow enough that excessive heat is not generated, as the grout may remain in the agitator for an extended period of time. The agitator should have a capacity greater than that of the mixer to minimize the possibility of the agitator running dry. Tanks should be equipped such that they impart turbulent motion to the grout. The motor should be capable of turning the paddles at 100 rpm, and the tank should be equipped with baffles to prevent vortex formation. Specifications should require a minimum of four baffles and two paddles. One additional paddle should be required as near to the bottom of the tank as practicable to sweep the bottom of the tank. Figure 10-6 shows a typical agitator tank integrated on a platform with the high-shear mixer. Figure 10-7 shows an internal view of an agitator.

(Courtesy of Advanced Construction Technique.)

Figure 10-7. Inside view of an agitator tank with a bottom sweeper and other paddles.

10-4. Additive and Admixture Dispensers. Accurate measurement and dispensing of additives and admixtures is essential for proper mixing and quality consistency of multiple component grouts. While this is not a difficult issue, suitable measuring devices are required to measure and add precisely the correct amount of each additive or admixture. For manual batching of grouts, it is best that dry components be delivered in bags that are sized for each mix or that they be pre-blended with the cement, if possible. This cannot always be achieved, depending on the admixture being used and the concentration required. For fluid admixtures, admixture pumps are available that can be used to measure and add the correct volume of the additive directly into the mixer without the operator ever handling the fluid. Figure 10-8 shows an air-operated fluid admixture dispenser, and Figure 10-9 shows a gravity dispensing system.

(Courtesy of Advanced Construction Techniques)

Figure 10-8. Pneumatic additive dispensers.

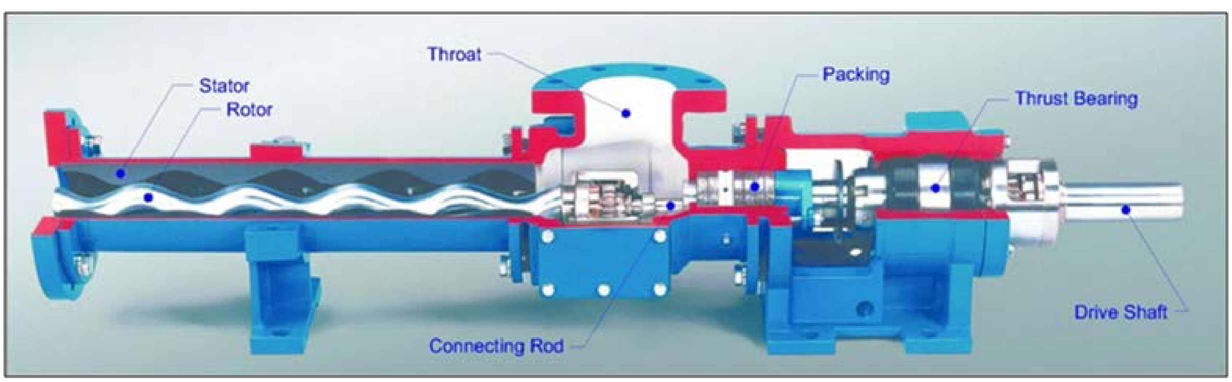
(Courtesy of Gannett Fleming)

Figure 10-9. Gravity-feed admixture dispenser tanks re-filled via admixture pumps, with a 15-L polymer tank on the left and a 200-L bentonite slurry tank on the right with sight glasses for dosing.

10-5. Pumps. There are numerous combinations of pumps, speed and pressure controls, and power systems available for this important part of the grout injection system. The type of grout to be injected, the importance of pressure control during the injection process, and the serviceability of the pump are important considerations in selecting the correct pump. Pumps should be sized appropriately to match the expected pressures and flow rates required for the project.

a. The two types of pumps used most often in grouting are progressive cavity and piston pumps. Progressive cavity pumps output a nearly constant pressure throughout the grouting process. Piston pumps output cyclic pressures with a significant pressure difference between the maximum and minimum pressures as the valves open and close during the piston cycles. The standard practice in North America is to require constant pressures to inject HMG for permeation grouting, which therefore requires the use of progressive cavity pumps. In contrast to the standard practice in North America, some grouters in Europe argue that cyclic pressures actually improve penetration. Houlsby (1990) dismisses this claim, and North American practice continues to require the use of constant pressures and a grout circulation line.

b. Progressive cavity or helical rotor pumps, also typically known by the trade names Mono or Moyno, consist of a steel, helical rotor that rotates inside a double-helix stator, which is made of a softer material such as wear-resistant rubber or nitrile. Figure 10-10 shows a cutaway view of a progressive cavity pump. Grout enters the pump throat from the agitator and is moved along in a screw-like motion with a positive seal between the stator and rotor. The rotor is chrome plated, which does wear and should be re-plated before exposure of the less-durable base metal. An advantage of the progressive cavity pump is the ease of servicing. Immediate access to the stator for replacement is accomplished by removing a few bolts. The life of the stator is highly dependent on the care that is given, including frequent cleaning. The maximum pressure developed with a progressive cavity pump depends on the number of pump stages. As a rule of thumb, each stage of a progressive cavity pump will generate approximately 85 psi. Higher pressures are achieved when pumping higher-viscosity fluids. Lower-capacity progressive cavity pumps can be staged in series to achieve higher pressures. The other advantage of this type of pump is the absence of valves within the pump, which tend to wear and are difficult to replace in other types of pumps. Many manufacturers report that their pumps are capable of pumping slurries containing solids (e.g., sand). However, experience has shown that the pump wears significantly faster than when pumping slurries absent of abrasive solids.



(Courtesy of Moyno, Inc.)

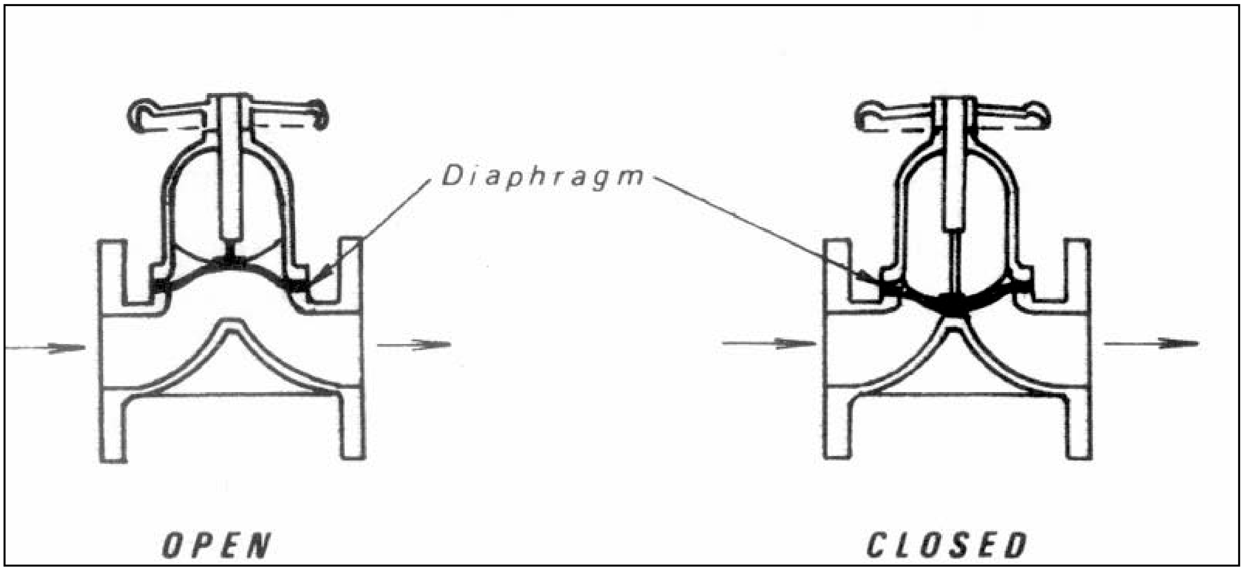
Figure 10-10. Cutaway section of a progressive cavity pump.

c. Piston pumps have a piston or ram reciprocating within a close-fitting cylinder. The pumps can be single acting or double acting. Single-acting piston pumps only pump material when the piston moves in one direction, typically forward. Double-acting piston pumps pump material when the piston moves in both the forward and backward directions. Grout is pulled into the cylinder from one side and simultaneously expelled from the other end. This requires a combination of valves at each end of the cylinder that are actuated by the grout pressure to control the flow. Newer models of piston pumps have two pistons with a reciprocating power source located between the pistons (Figure 10-11). The advantage of this arrangement is that the grout only comes in contact with the face of the piston. Still, parts that come in contact with the grout should be chrome plated to extend the life. Piston pumps can generate very high maximum pressures in excess of several thousand psi. Some pumps also contain multiple cylinders so that quantities of grout pumped can be quite high. Piston pumps are not generally accepted for pumping HMG in North America. They are widely used for chemical solution grouting, in jet grouting methods, and on drills as booster pumps to increase the water flush pressures.

d. Piston-type pumps are available in a wide variety of sizes and configurations. In these pumps, a piston is advanced inside a cylinder, normally using hydraulics, which generates very high output pressures. Figure 10-11 shows an example of a double-acting piston pump typically used for sanded HMG applications. The unit is powered by an electric motor, which powers the hydraulic system.

(Courtesy of Gannett Fleming)

Figure 10-11. Piston pump with a reciprocating power source.



(From Houlsby 1990; this material is reproduced with permission of John Wiley & Sons, Inc.)

Figure 10-12. Schematic of a diaphragm valve.

10-6. Power Sources. Power for mixers, agitators, and pumps can be provided by direct connection to an electric motor, internal combustion engine, or hydraulically or pneumatically operated motors. The rotational motion of high-shear mixers, agitators, and progressive cavity pumps makes them ideal for direct connection to motors and engines through belts or gear boxes. As discussed above, piston pumps typically use hydraulic rams. The type of power used is generally based on the contractor's preference. In confined spaces, electric and pneumatic systems are ideal due to air quality concerns.

10-7. Valves. A variety of valve types are used in grouting. Valves are used at multiple locations in the grout distribution system, including between the agitator and the pump, at the grout hole to control grout flow and pressure into the hole, on the return side of the circulation line, on the hole so the grout pressure on the hole can be maintained after grouting is completed, and at the hole so that the hole can be bled of grout or excessive pressure during grouting. Some of the valve locations require operation in various settings, while other locations only require the valve to be fully open or closed. Typical valves include diaphragm, ball, and plug cocks. Each type has characteristics that make it suitable for specific applications. Several important characteristics of valves include the ability to prevent grout from interfering with the closure mechanism, good flow through at various settings of the valve to minimize clogging of the valve, and permanently attached handles so that no time is lost in searching for a handle when the valve must be adjusted.

a. Diaphragm valves. This type of valve operates by pressing a diaphragm, via a screw mechanism, against a raised ridge in the flow path (Figure 10-12). The advantage of this valve is that fine adjustments can be made in the flow, which is important during times when precise control of the flow and pressure is required. They can be fitted with a lever or a wheel-type handle. The valves hold up well under service, with the main problems being rupture of the diaphragm and wear on the ridge. Another advantage is that access to the diaphragm for inspection or replacement is simply performed by removing a few bolts that hold the diaphragm in place. Diaphragm valves should be specified as a requirement on the grout return line and at other locations where required to accurately control grout injection pressure (Figure 10-13). Diaphragm valves are provided on both the down-hole line and the return line. In general, the valve on the down-hole line is fully open and only the return line valve is used to control the pressure. Closing the return line valve decreases the flow back to the agitator and increases the pressure down the hole. Opening the return valve lowers the injection pressure. In some circumstances where the pressure in the circulation line is too high and cannot be controlled solely by the return line valve, the diaphragm valve leading down the hole must also be used to limit the injection pressure. Using the down-hole valve to control the injection pressure should not be a standard practice and should only be used as a last resort. The disadvantage of diaphragm valves is the time required to go from fully open to fully closed. Therefore, a ball or plug cock valve is generally placed just downstream of the diaphragm valve on the line of fittings leading to the hole. Diaphragm valves are typically limited in pressure to approximately 300 psi due to the flexible internal diaphragm. When higher pressure ratings are required, more expensive valves designed specifically for flow control must be obtained.

(Courtesy of Advanced Construction Techniques)

Figure 10-13. Header operator using a diaphragm valve on a return line to control injection pressure.

b. Ball Valves. This type of valve consists of a sphere with a hole through it, with the sphere able to rotate in its seat (Figure 10-14). The hole in the sphere should be the same diameter as the grout line so that there is no obstruction to flow when the valve is fully open. The ball is controlled via a handle and can be rotated 90 degrees from fully open to fully closed. Ball valves can become clogged when operated in a partially open position because of the restricted flow. The ball and seat are also subject to higher wear when operated partially open. These valves should only be used in applications where they are to be either fully open or fully closed. Ball valves should never be used to control the grout injection pressure.

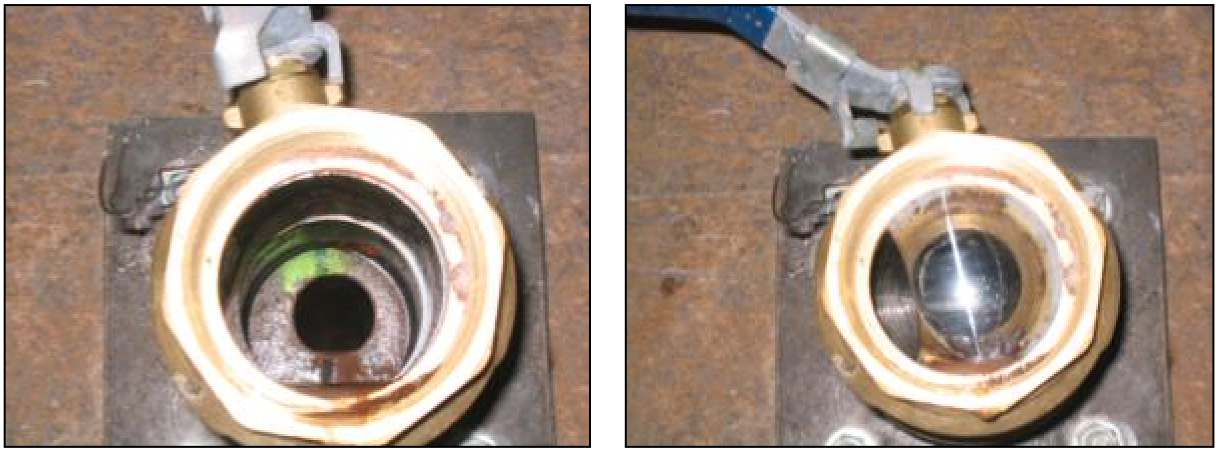


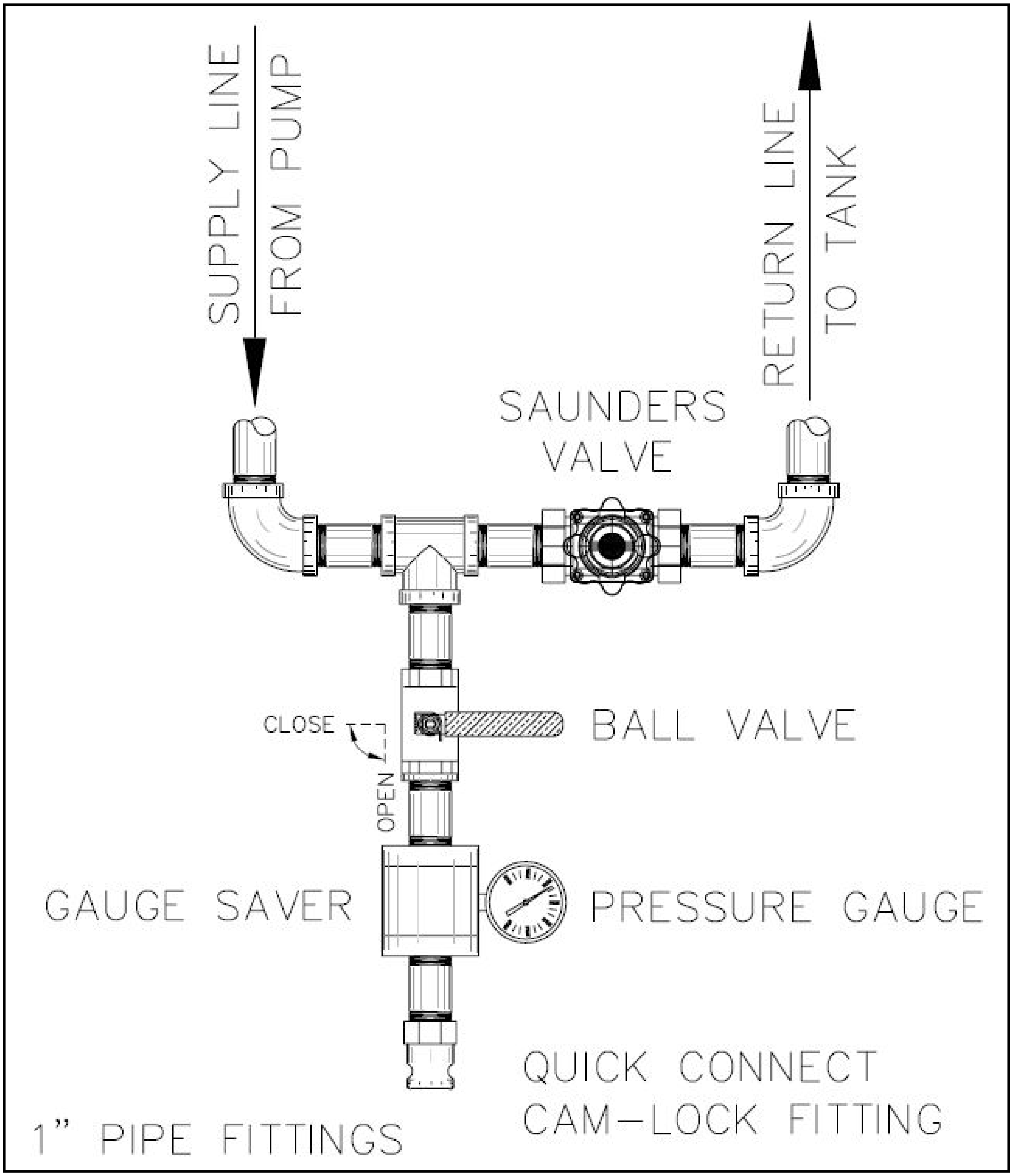
Figure 10-14. Ball valve fully open (left) and partially open (right).

c. Plug Cock Valves. These valves consist of a tapered plug in a seat with a hole through the plug. The plug is turned with a lever handle to adjust the flow. The seal of the tapered plug within the seat can be tightened to permit high grout pressures. However, this comes at the cost of increasing difficulty in turning the valve. They are subject to wear of the plug opening and clogging when operated in the partially open position and should therefore only be used in locations where a fully opened or closed position is required. Plug cock valves should never be used to control the grout injection pressure.

10-8. Grout Headers.

a. Grout headers include an arrangement of valves and pressure gauges used to control the quantity of grout that flows to the hole, control the pressure of the grout applied to the hole, bleed fluid off the hole during grouting, and seal the hole after grouting is completed. Figures 10-15 and 10-16 show typical arrangements of the fittings on common grout headers. The header consists of a pressure gauge or pressure transducer to monitor the injection pressure, a flow meter if electronic monitoring is being performed, a diaphragm valve on the return line to control the injection pressure, a diaphragm valve and a ball or plug cock valve on the injection line, and sometimes a blow-off or bleed line with a ball or plug cock valve (Figure 10-17).

b. The size of the piping and fittings should be compatible with the anticipated injection rates. Normally 2.5 cm (1 in.) diameter is adequate for mixers in the 225- to 280-L (8- to 10-ft^3) size. All fittings, piping, and valves should be similar to the diameter of the grout injection pipe or hose. Smaller fittings tend to block more easily, and larger fittings permit a reduction in the grout velocity and subsequent blocking and settling of cement particles.

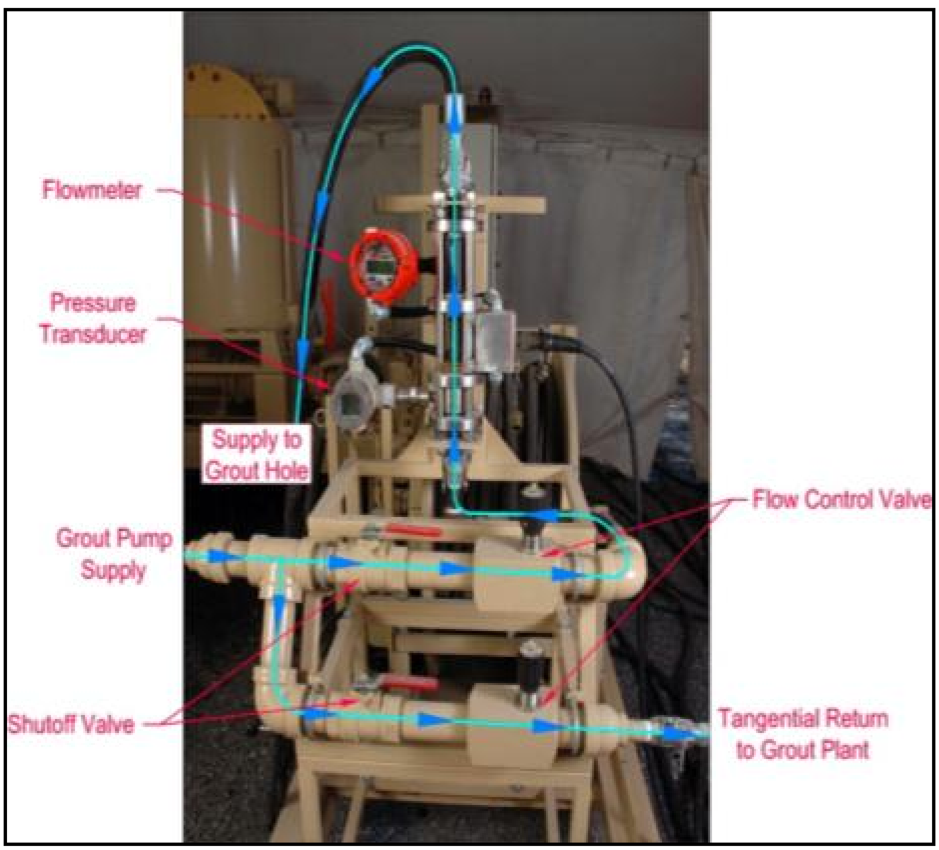


(Courtesy of Nicholson Construction Company)

Figure 10-15. Typical Grout Header.

(Courtesy of Gannett Fleming)

Figure 10-16. Grout header for manual pressure observation and control during HMG injection.



(Courtesy of Gannett Fleming)

Figure 10-17. Header system for electronic monitoring of flow and pressure for HMG injection.

10-9. Circulation Lines and Equipment Arrangement.

a. Circulation Loop System. Figure 10-18 shows a typical arrangement of the circulation lines and equipment for a circulation loop system. The grout is pumped from the agitator to the header. Grout not needed to satisfy the demands of the hole is returned to the agitator tank. Additional headers can be inserted in the loop if multiple hole grouting is permitted or for treating connections between holes. The use of a return line allows excess pressure in the circulation line to bypass the hole, eliminates wasted grout when a hole is completed, and maintains the velocity of the grout at a high rate in the circulation line during times of low flow at the grout hole.

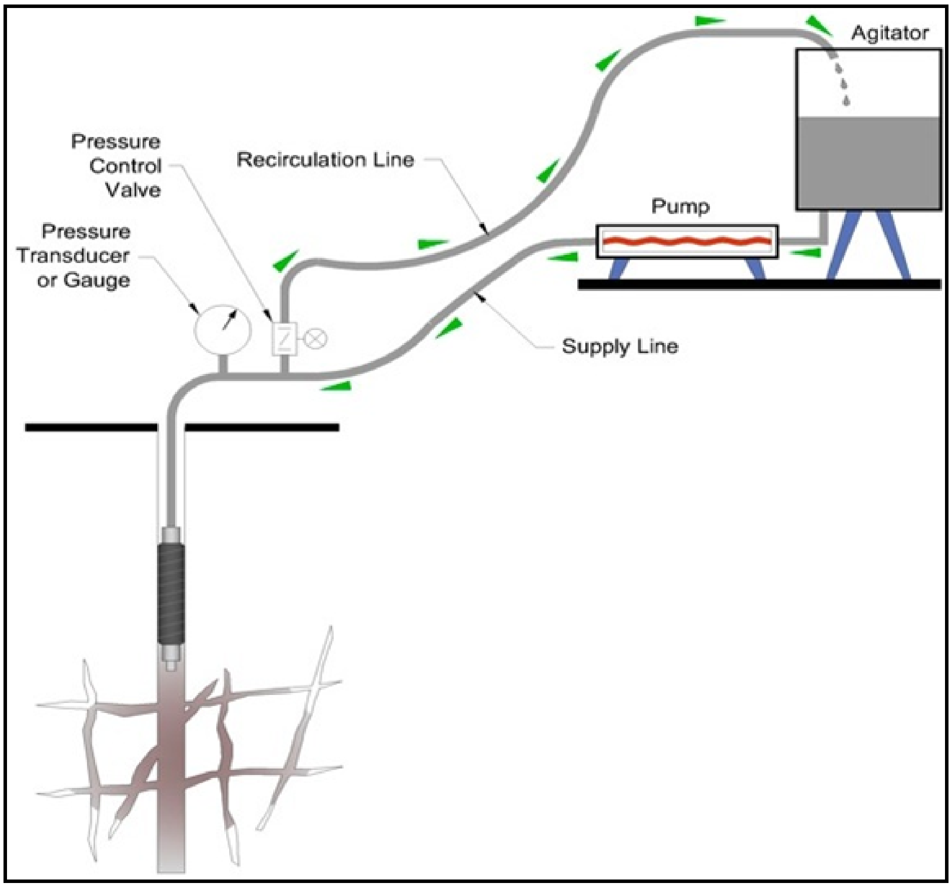


Figure 10-18. Grout circulation system with return. (Flow meter not shown; see Fig. 10-17 for configuration.)

b. Direct Injection System.

(1) Figure 10-19 shows a schematic of a direct injection system. The direct system involves pumping grout from the agitator to the hole, with no return line. The direct system has no mechanism to control the pressure down the hole other than closing the down-hole valve. At times of low flow the grout in the supply line basically stops moving and is no longer being sheared. For thixotropic grouts, this can result in premature refusal of the stage or false refusal within the grout distribution system. The direct system is commonly used by some grouters in

Europe who use piston pumps with cyclic injection pressures. Its use is not normally permitted in North American work.

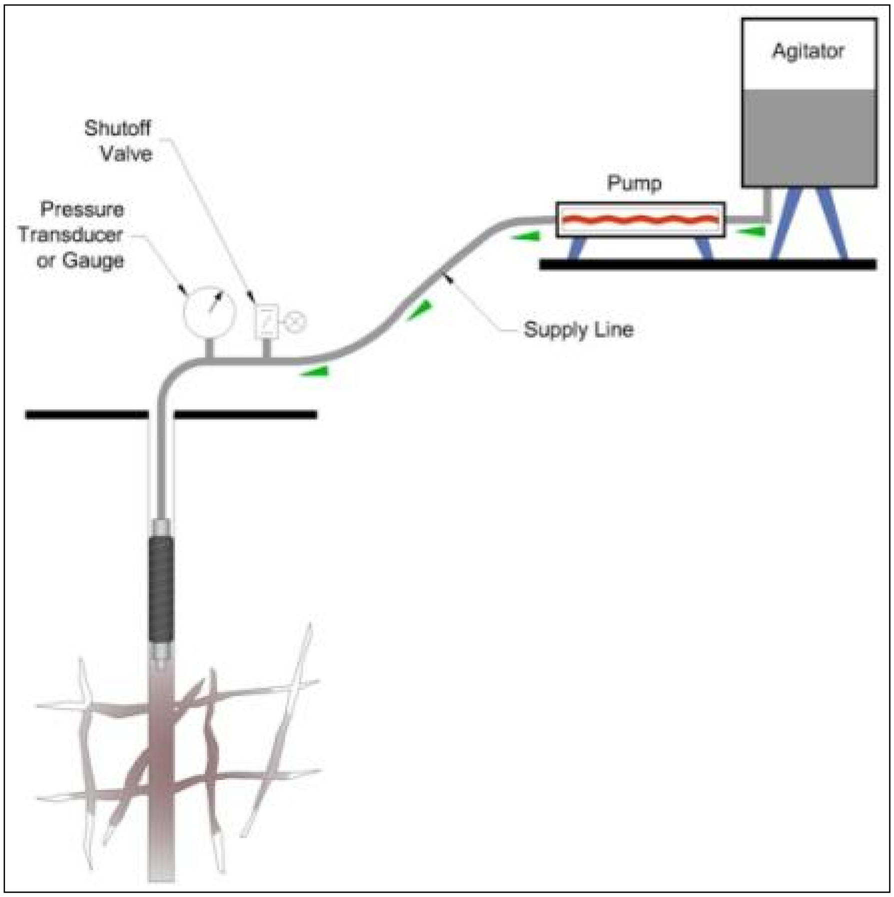
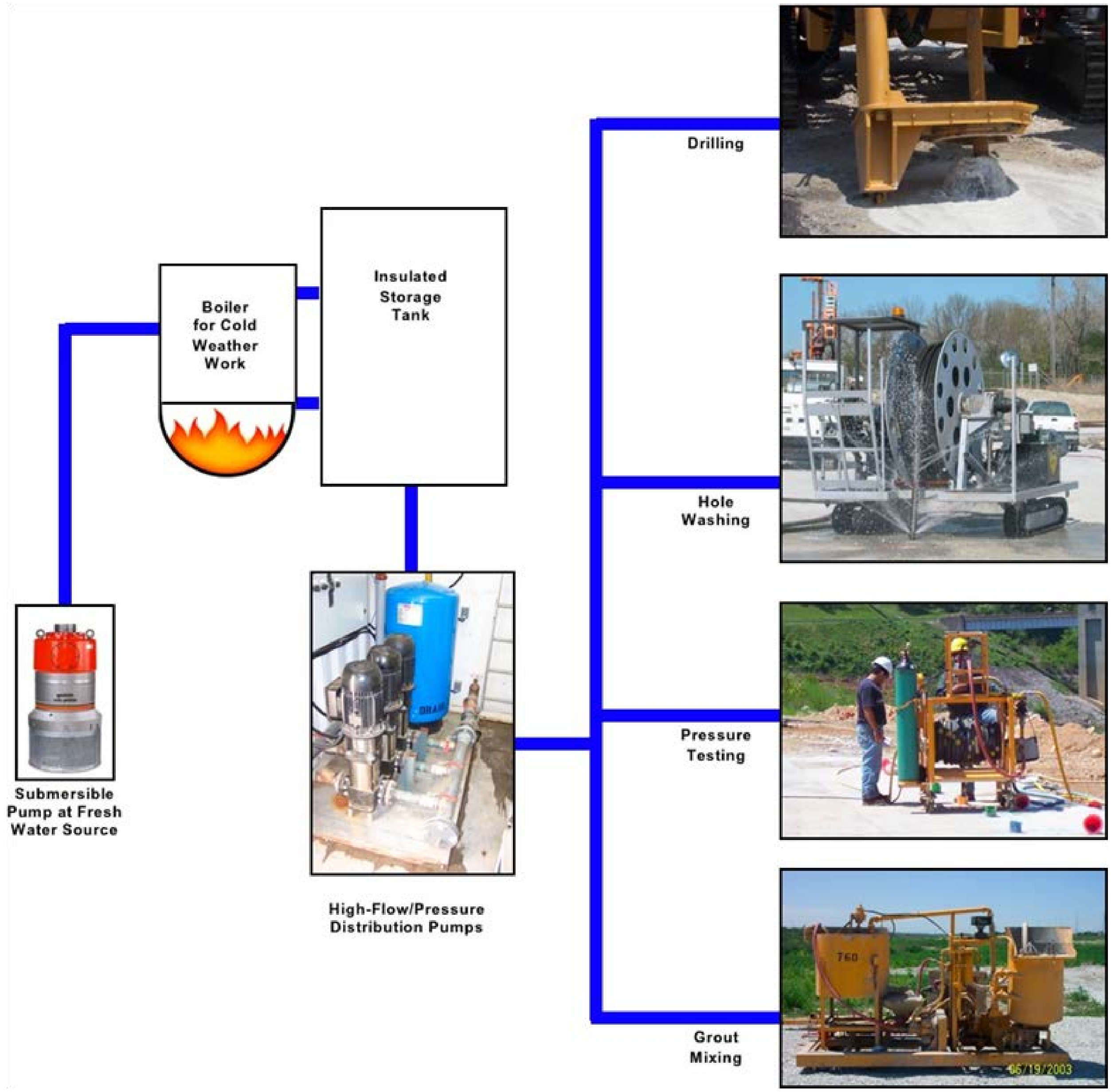


Figure 10-19. Direct injection grout supply system with no return.
(Flow meter not shown; see Fig. 10-17 for configuration.)

(2) Hose is almost always used for HMG circulation lines. Hoses have the convenience of being lighter and more easily moved around the site. The inside diameter should be 1.9 to 2.5 cm (¾ to 1 in.) for mixers in the 225- to 280- L (8- to 10-ft^3) range. Smaller lines tend to clog due to their size, whereas larger ones tend to clog because of lower velocities. The smaller diameter also minimizes wasting of grout during the required flushing of the lines. The important consideration is to keep the grout flowing at a reasonable velocity. In large void-filling applications, direct injection with larger-diameter hose may be appropriate because it is unlikely that material will remain in the lines at low velocities for significant periods of time. Connections between sections of hose should be quick-connect unions or couplings to minimize time in disassembling the lines for flushing and length changes. Lines should be flushed at least once per shift to minimize the buildup of grout in the line.

10-10. Water Distribution System. Every element of grouting requires water (Figure 10-20). A properly planned distribution system is essential for efficient operations. The system should extend across the site and have tees and associated valves strategically located. It should also be located where it does not limit equipment maneuverability. A water heater or boiler might be required if cold weather grouting is anticipated. Insulation of the water distribution line between the pump and the batch plant should be considered for both hot and cold weather.



(Courtesy of Advanced Construction Techniques)

Figure 10-20. Typical water distribution system with field demands.

CHAPTER 11

Grouting Equipment for Other Types of Grouts

11-1. Other Types of Grouting. This section provides information on equipment associated with non-HMG grouting applications. Other USACE Engineer Manuals offer complete and detailed discussions on some of these topics and should be referenced by the practitioner. In some cases, particularly with bitumen, the application is particularly specialized and only used on the rarest of occasions.

11-2. Chemical Grouting. Chemical grouts, or more appropriately solution grouts, are covered in great detail in EM 1110-1-3500, *Chemical Grouting*. Additional resources include *Chemical Grouting and Soil Stabilization* (Karol 2003) and *Practical Handbook of Grouting* (Warner 2004).

11-3. Low-Mobility and Compaction Grouting.

a. General. Low-mobility and compaction grouting, while two distinctly different applications, involve mixing, delivering, and injecting similar, if not the same, materials. LMG is common in void-filling and karst treatment applications, while compaction grouting is a ground improvement, displacement method typically for densification of soil or foundation support. See Chapter 27 of this manual for additional information on compaction grouting. The *Practical Handbook of Grouting* by James Warner is a valuable reference for low-mobility and compaction grouting that provides significant additional information on the subject.

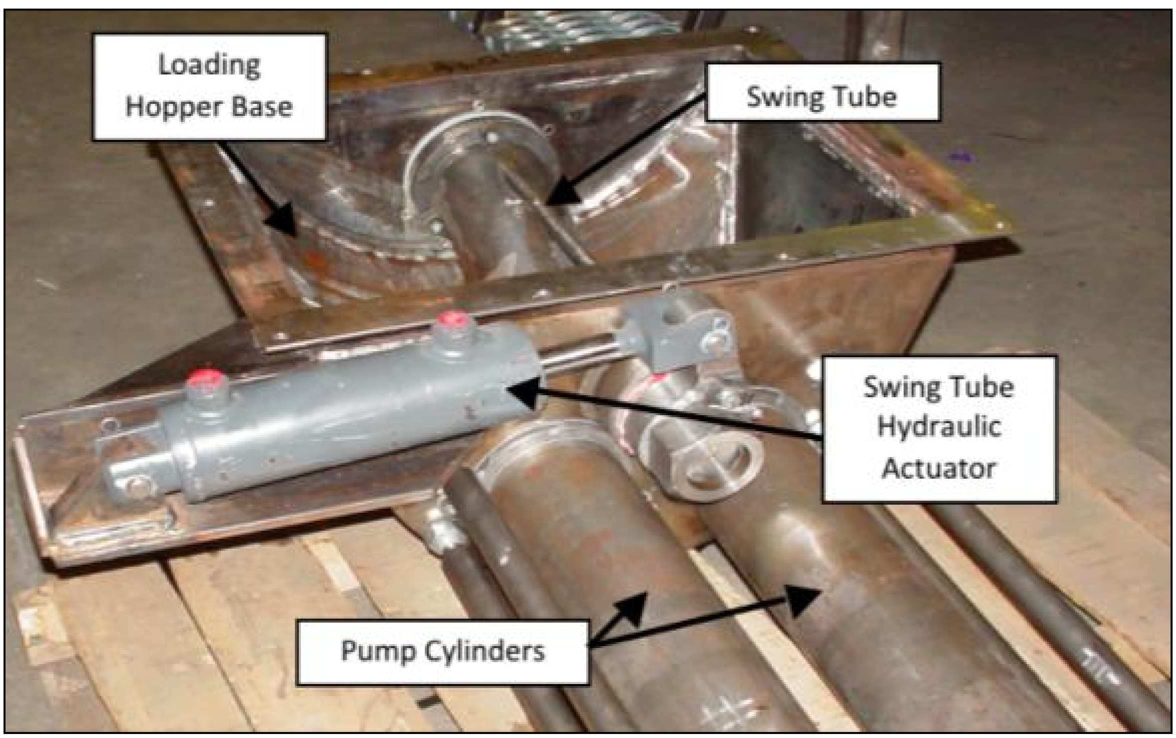
b. Drilling. Drilling methods for holes vary based on the application. For void filling and treatment of karst solution features, nearly any drilling method can be used as long as it successfully encounters the discontinuity and subsequently allows injection either through the drill tooling or through casing installed after the hole is drilled. In compaction grouting applications at shallow depths, driven casing is typically installed to the desired depth and then cleaned out from the surface. Alternatively, sacrificial knock-off points may be affixed to the end of the casing and driven to the required depth with the casing. Removing the points simply entails pulling the casing upwards slightly and then tapping out the plug with a rod or weight. Deeper holes may necessitate the use of drilled casings. The selection of the drilling method is typically an economic consideration in low-mobility and compaction grouting.

c. Mixing. Mixing of low-mobility and compaction grout is commonly performed by one of two methods. Centrally mixed material can be purchased from a local concrete supplier and delivered to the site in ready-to-inject form. Mix-on-site methods include self-contained batching trucks (essentially portable concrete plants on wheels) that deliver dry material to the site and mix on site as needed, and smaller, self-contained mixing/pumping units that use bulk material sources and/or stockpiles on site. Self-contained mixing/pumping plants consist of a mixing hopper in which materials are loaded, mixed, and then fed to the loading hopper, where mixed materials enter the pump. If centrally mixed or batch truck materials are used, they are simply delivered directly to the loading hopper via the chute on the truck. Centrally mixed and portable batch truck methods are appropriate for large void-filling operations or when significant quantities are otherwise required.

d. Pumping. LMG generally has low to moderate slump with a high concentration of fine aggregates. The pressures required to pump this material necessitate the use of piston pumps. In nearly all cases, these are single-acting piston pumps (Figures 11-1 and 11-2). In most circumstances the pump has two cylinders, allowing one cylinder to fill while the other is discharging, thus minimizing the time between pump strokes. A swing tube rotates between the cylinders receiving the pressurized grout. On some systems a rotating paddle is located above the swing tube to assist with loading the cylinders during the reverse stroke. High-pressure hose or steel pipe conveys the pressurized grout to the hole. When stiff materials are pumped in this manner, it is likely that the cylinders will not be completely full of grout before the swing tube closes and the piston moves forward. Some percentage of the cylinder volume will then likely consist of air pockets. Since the volume of the cylinder is known, injected volumes are commonly measured by counting pump strokes and assuming a full cylinder. Because of the potential air pockets, a correction factor may be required for some materials. A simple volumetric field test usually provides adequate accuracy. Due to the high pressures and the abrasive nature of the pumped materials, piston pumps typically require significant maintenance and frequent rebuilds.

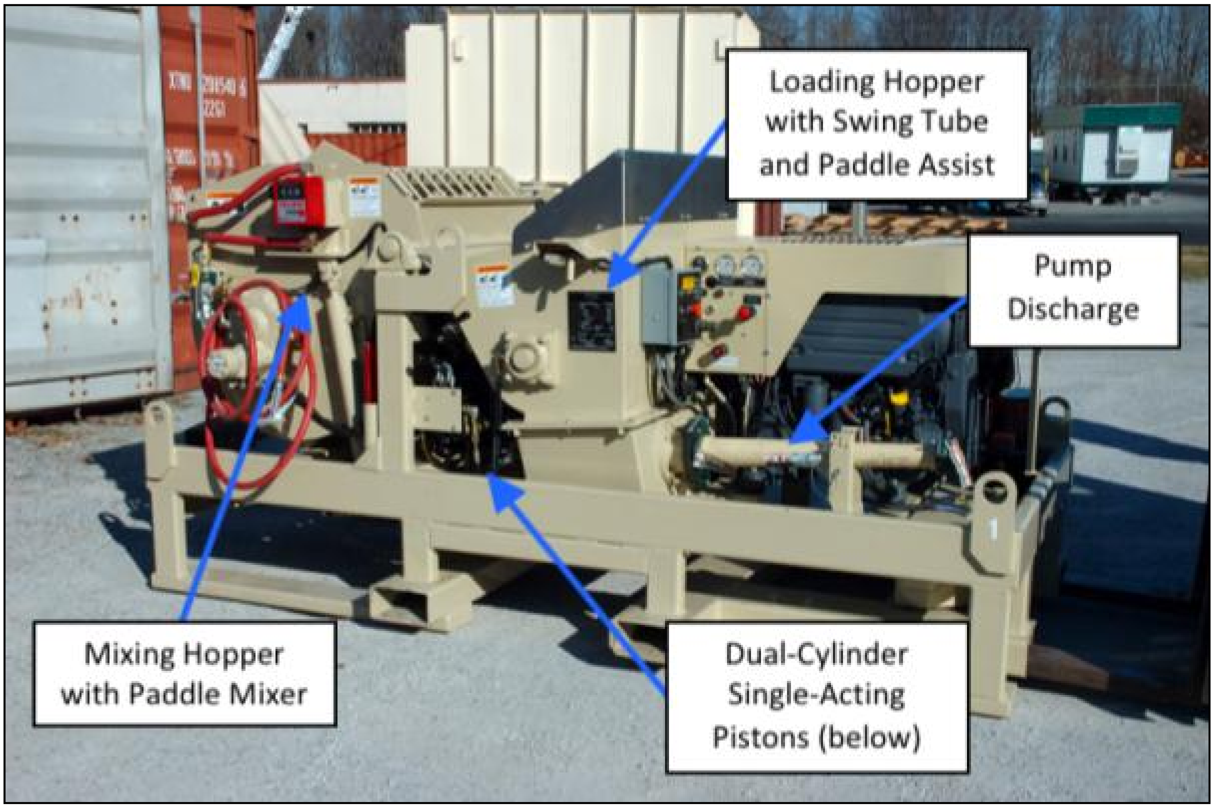
11-4. Bitumen Grouting.

a. General. Bitumen grouting is a specialized grouting technique normally reserved for massive inflows that cannot be remedied using other methods. Application of this method is therefore rare; however, several noteworthy projects where bitumen injection was successfully used are described by Bruce et al. (2001) and Naudts (2003). In some of these cases, the inflows to be stopped were on the order of several thousand liters per second.



(Courtesy of Gannett Fleming)

Figure 11-1. Single-acting, dual-cylinder piston pump during assembly.



(Courtesy of Gannett Fleming)

Figure 11-2. Portable, diesel-powered LMG plant.

b. Equipment. Bitumen injection requires the use of equipment that can accommodate injected material that is on the order of 392 °F (200 °C), and the rapid thickening of the material when it cools below this temperature. Therefore, the temperature of the injection equipment itself must be maintained high enough so that the material does not cool and thicken before injection, leading to obstructions in the delivery system. Heated bulk storage containers are required to maintain, and in some cases elevate, the temperature of the bitumen before injection. Conveyance systems must be heated and insulated (Figure 11-3). Given the high operating temperature, particular safety issues must be considered. In a common application, one or more holes of suitable diameter, typically on the order of 15 cm (6 in.) (Bruce et al. 2001), are drilled to intersect the zone where the inflow is occurring. Pipes are then inserted to the prescribed depth for subsequent bitumen injection. Before injection the conveyance system must be heated, which can be done by steam injection, injection of hot oil, or the use of a heat trace (Naudts 2003). After bitumen injection is initiated, the high temperature of the product will normally prevent rapid cooling of the casing and therefore prevent clogging. Monitoring and frequent observation of the injection pressure at multiple locations in the injection system will provide an indication if and where clogging is occurring in the pipe, in which case additional insulation, heat trace, and/or an increase in the temperature of the bitumen may be necessary.

c. Methods. Large inflows suitable for bitumen grouting are commonly confined to a particular zone or fracture within bedrock. Conditions such as limestone with solution features that are open or partially or totally filled with unconsolidated materials are candidates for such a condition to develop, given the appropriate hydraulic conditions. In some cases, these features may be prevalent throughout a site, but with only one or a few that are actively flowing. Treating only these few “bad actors” may simply change the groundwater regime such that the condition reappears through previously inactive discontinuities. An intensive drilling and investigative program is necessary to diagnose the exact location of the water-bearing zone and to identify other potential water-bearing zones that may become active as a result of plugging the currently active discontinuity (Figure 11-4). A combination of exploratory drilling, temperature surveys, and geophysical techniques may be necessary to develop a thorough understanding of the conditions. Once these zones are identified, treatment of the inactive discontinuities should be initiated while flow velocities are relatively low due to the active flowing condition elsewhere. This can typically be accomplished using HMG and LMG. The intent is to confine the flow to the active water-bearing zone while strengthening the inactive zone to prevent reappearance of the problem. Once confined, injection of bitumen to seal off the active water-bearing zone is initiated. It is common to simultaneously inject HMG and LMG upstream of the bitumen injection hole. When the HMG and LMG mixes with the bitumen, the high heat flash-sets the cement in the HMG and LMG. As bitumen cools, it contracts, which can open channels through which the seepage can re-emerge. It is therefore common to continue injection of HMG for some time after bitumen injection is terminated to fully fill any flow paths left due to bitumen contraction. The quantities of bitumen, HMG, and LMG consumed in an application of this type can be enormous.

d. Contractors and Consultants. Application of bitumen grouting is typically necessary only when a massive inflow has been encountered. In many cases, this is an unanticipated event that results in potentially catastrophic damage to infrastructure. Timely remediation is normally critical. The equipment required to adequately institute a bitumen grouting program is highly specialized and generally very large compared to the small sites that are typically available for treating isolated discontinuities. Experience among consultants is typically not vast because of the limited application of the method. Under these near-crisis circumstances, experienced consultants and contractors are required, as are the best available equipment and technology.

(Courtesy of Dr. Donald Bruce)

Figure 11-3. Bitumen injection system with insulated piping and instrumentation system.

(Courtesy of Dr. Donald Bruce)

Figure 11-4. Bitumen injection project with heated holding tanks and pumps (foreground), bitumen transport trucks, and injection system (background). Note the high concentration of equipment on this small site.

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CHAPTER 12

Monitoring and Control Equipment

12-1. Introduction.

a. General. The raw data collected during water pressure testing and grout injections are pressure and flow measurements as a function of time. Processing of these data on a real-time basis using other known parameters, such as hole depth, static elevation heads, depth to groundwater, system head loss, and specific gravity of injected fluids, is necessary to:

(1) Determine and control the effective pressures being applied to the stage

(2) Evaluate the nature of the stage's response to mix changes and applied pressures

(3) Verify that the required refusal criteria are reached

(4) Determine the cumulative volume of injection.

b. Data Compilation. The compilation of both the raw data and the calculated data provides the record of water pressure testing and/or grouting for the stage and is normally the basis for at least a portion of the measurement for payment activities.

c. Data Processing. The necessary real-time processing of the data required for controlling the injection can be performed either manually or semi-manually by individual inspectors at each hole, or it can be performed automatically by computer systems that are centrally located and capable of managing multiple headers simultaneously.

d. Records. After completion of the stage, the stage records are analyzed to evaluate the effectiveness of the grouting program. As with data collection, the post-processing analyses can be performed either manually or by computer systems designed specifically for this purpose. This chapter discusses the various types of raw data measurement systems currently available for use with HMG injection systems. Examples are provided to describe common methods of deploying these systems, and commonly associated equipment.

12-2. Water and Grout Injection Measurement Equipment.

a. General Recommendations.

(1) Table 12-1 lists a summary of the types and characteristics of available measurement equipment, including both systems that have historically been used and more advanced equipment now readily available and commonly used. Chapter 16 of this manual addresses the choice of equipment for a particular application, along with the real-time processing and post-processing equipment and methods that will greatly affect the accuracy of measurements, the frequency of readings that are possible, the accuracy and consistency of detecting refusal points, the amount of time each injection operation may require, the ability to detect uplift and discern other important behavior during the injection, the number of inspection personnel required, the effort needed for post-processing and generation of all final project records, and the project costs.

The time stamping capability of the automated systems, when used in conjunction with time stamped automated instrumentation, facilitates the evaluation of project response. For dam safety projects, this ability can be extremely valuable and necessary for managing construction risks.

Table 12-1. Summary of measurement equipment and accuracies.

Parameter*	Monitoring Equipment or Technique	Accuracy	Comments
Water and/or grout take (volumetric flow)	Agitator dipstick reading	< ±0.25 CF or 7 L per volume measured	Accuracy depends on diameter of the agitator and plant operator's attention to detail. Requires averaging total flow over test time to acquire rate.
	Agitator riser tube	Slightly better than dipstick	Allows easier confirmation of readings by inspector. Lessens operator attention factor. Requires averaging total flow over test time to acquire rate.
	Nutating disk water meter	±1.5% of rate when new	Significant loss of accuracy at low-flow rates. High wear rate of measurement components can require frequent meter rebuild/replacement.
	Magnetic flow meter	±5% for flow rates greater than 0.25 L/min. ±25% for lesser flow rates (depends on line size)	Zero reading is zero flow. Requires power supply at header or remote system with signal and power cables. Easy to check calibration on site.
Water/ grout gauge pressure	Dial-type mechanical gauges	Accurate to 1% of range. Visual observation required. Pressure spikes and fluctuations often missed or ignored in manual calculations	Various gauge ranges required. Gauge calibration should be checked weekly against master gauges. Weekly calibration requirement often missing or not enforced. Gauges routinely go out of calibration and require correction or replacement.
	Electronic pressure transducers	<0.5% of full scale	Calibration on site possible, but cumbersome. Requires power supply at header or remote power with cables.

*After Wilson and Dreese 1998.

(2) In general, the advantages of using the latest measurement system technologies (pressure transducers and electronic flow meters) are so great that their use is recommended for all projects, regardless of size and regardless of selection of the real-time processing and post-processing methodologies and systems, which might vary by project size and application. Proven electronic measurement equipment is readily available, and it has been found to be at least as durable as older mechanical systems, even in the harsh environment of grouting. This does not negate the need to frequently check production instruments with a master instrument and to keep an adequate supply of replacement instruments on hand in the field. All instruments supplied to the project should preferably be of the same make and model for each type of instrument to simplify maintenance and avoid range and configuration errors.

b. Mechanical Pressure Gauges. There are a number of important considerations in selecting gauges. The accuracy to which a gauge can be read is proportional to the diameter of the gauge. In practice, gauges should have a minimum face diameter of 4 in. to allow easy reading with acceptable accuracy. Although larger gauges are available, they are often less durable than 10-cm (4-in.) gauges and are not significantly more accurate. There may be a variety of pressures required during the grouting process, so gauges with a variety of pressure ranges must be on hand. Gauge accuracy is generally some percentage of the full scale; gauges should have a pressure range exceeding the pressure expected to be used in grouting, but only by a limited amount. If the maximum pressure recommended by the manufacturer is exceeded even for a very short duration, the gauge can be damaged. Pressure gauges should not read the grout column pressure directly, but rather should use a glycerin-filled annular hydraulic diaphragm (see Gauge Savers below). Gauge pressures should be indicated in pounds per square inch. Gauges should be provided new with calibration certificates, and the calibration should be verified on a weekly basis against a master gauge or at any time when malfunction is suspected. Gauges that show a difference of more than 2% from the master gauge should be discarded.

(1) Gauge Guards. Grouting is a difficult environment for gauges, so the use of guards to protect the gauges should be considered. Gauge guards are typically a metal strap, bar, or cage that extends around the gauge to provide some protection from accidental damage. Even heavy-duty gauges can be damaged by impact to the bezel. Gauges incorporating threaded process connections are often broken off at the threads. For this reason, it is best to select either larger-diameter threaded process connections or flanged process connections.

(2) Gauge Savers. Gauges are destroyed if grout is permitted to enter the gauge. There are several commercial gauge savers available. The use of short tubes filled with grease or U-shaped tubes filled with oil should not be permitted. The preferred design of a gauge saver is an annular hydraulic sensing diaphragm in the grout line with the hydraulic cavity between the grout and the gauge filled with glycerin or silicone oil. This type of gauge saver is available in several common configurations. The apparatus can typically be quickly disassembled from the grout line and flushed. The fluid contained in the gauge saver also acts as a pressure snubber and helps to minimize the detrimental effects that pressure spikes have on gauges.

c. Electronic Pressure Transducers. In automated systems of grout monitoring, electronic pressure transducers are the norm. These instruments function by generating electrical signals (typically with frequencies in the hertz or low milliamp range or low direct current voltage) as a linear function of their range. For example, a pressure transducer with a full scale of 1,000 psi and a 4- to 20-mA output will generate approximately 4 mA at 0 psi and approximately 20 mA at 1,000 psi, with intermediate readings calculated using the linear relationship. The instrument requires a power source, which can either be integral to the header unit or provided remotely via cable. Signals are typically transmitted to a centralized monitoring location via a signal cable or a wireless data system. The pressure transducers are mounted on the header instead of the gauge as the primary pressure indicator. Transducers should be provided with a readout display at the instrument location indicating the process pressure in psi. Backlit displays are available and useful during nighttime operations or for work in confined spaces.

d. Mechanical/Electronic Flow Meters. Flow meters can provide an accurate record of the flow to the hole. Flow meters are usually installed on the header with the transducer. An alternative system is to install flow meters on both the feed line and the return line. The difference between the readings from the two meters then represents the rate of grout injection. There are many varieties of flow meters, including those that operate by measuring differential pressure, positive displacement, velocity, and mass flow. Flow meters that operate by measuring material velocity include turbine, magnetic, and sonic devices. True mass-flow meters are also available. All of these types of flow meters are available with analog signal transmitters with linear outputs similar to those described above for the pressure transducer. Regardless of type, the readout of the flow meter is typically provided through an LCD display at the meter, and the data can be transmitted via cable or wireless to a central data acquisition location. Display units should be in gallons per minute, cubic feet per minute, or liters per minute, and total volume injected should be in gallons, cubic feet, or liters at the header. The electronic flow meters should indicate zero flow for no-flow situations, and the totalizer should be easily reset to zero. It is essential that electronic flow meters used in the work be known to have proven suitability for grouting.

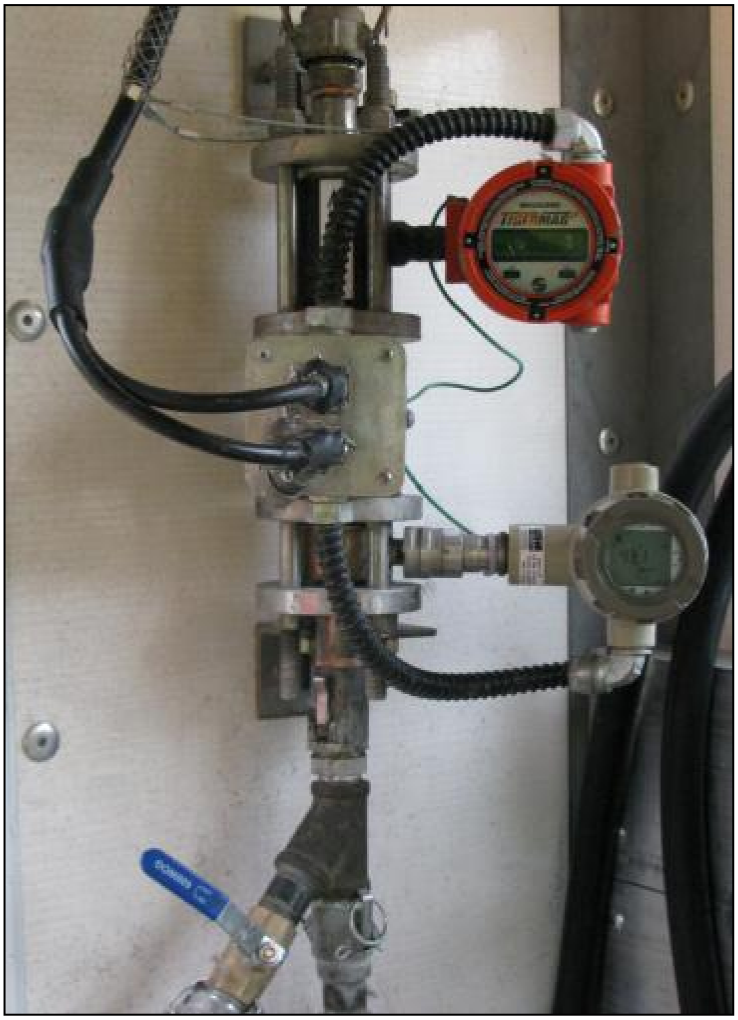
(1) Differential Pressure Flow Meters. Differential pressure meters measure the drop in pressure as the fluid passes through a metering orifice or venturi. The meter must be compatible with the properties of the fluid being measured. For example, water flows are often measured with thermoplastic insert venturis, but grout would quickly erode such an insert. Also, the differential-producing element must be sized appropriately for the desired flow and viscosity of the measured fluid. Differential pressure meters are typically not well suited to grouting, where there are often large viscosity variations in the grout mixes.

(2) Positive Displacement Flow Meters. Positive displacement meters use a rotating vane or disk to measure the grout in discrete units, which are counted. These are not well suited for grouts and can become inaccurate with buildup of grout on the vane or disk or with abrasion due to the cement and bentonite in the grout.

(3) Turbine Flow Meters. Turbine flow meters measure the velocity of the grout as it passes through a turbine. The rotational speed of the turbine is related to the flow rate. The turbine and stator components of these meters wear quickly when used with grouts, so they are not very suitable for measuring grout flow.

(4) Magnetic Flow Meters. Magnetic flow meters are the most common type of flow meter used to measure the volume of HMG injected in a hole (Figure 12-1). Magnetic flow meters are well suited to abrasive fluids such as grout because they minimally impede the fluid

stream when the flow sensing tube is approximately the same size as the grout delivery line. Magnetic flow meters apply a known magnetic field to the measurement area and measure the voltage induced across the fluid as it moves through the meter. The grout needs to have a high water content to provide the proper electrical conductance. HMG mixes typically have the necessary water content.



(Courtesy of Advanced Construction Techniques)

Figure 12-1. Magnetic flow meter and pressure transducer pair with LCD readouts.

(5) Ultrasonic Flow Meters. Several types of ultrasonic flow meters operate on either Doppler or time-of-transit principles. The Doppler-type ultrasonic flow meter uses transducers to transmit a beam of ultrasonic energy through the fluid to be measured. Particles or bubbles in the fluid reflect the beam to a receiver. The frequency shift of the beam is proportional to the velocity of the fluid and is calculated by the meter. The time-of-transit type of flow meter uses ultrasonic transducers that are similar to those of the Doppler meter, but are placed on opposite sides of the flow tube. This type of meter is not a good choice for grouting operations as it is sensitive to scattering and absorption by particles or bubbles in the flowing fluid. For either type of meter, once the measurement is taken, the value is transmitted as a linearly proportional signal.

(6) Coriolis Mass-Flow Meters. Mass-flow meters employ a U-shaped tube that is enclosed in a sensor. The tube is the same diameter as the grout delivery line and is vibrated at its natural frequency by a magnetic device. The U-tube vibrates at a varying frequency that depends on the mass of the grout. The rate of these vibrations is detected by magnetic sensors on opposite sides of the tube and converted to a voltage difference, which correlates with flow rate.

12-3. Electronic Instrument Accuracy.

a. Accuracy. With the growing prevalence of electronic measuring devices, it is important that grouting practitioners who intend to use them understand the definitions of measurement accuracy and how this affects their view of the actual grouting process. Accuracies for electronic instruments are most commonly specified in one of three ways: percentage of full scale, percentage of calibrated span, or percentage of actual reading.

(1) Full-Scale Accuracy. Full scale is the maximum reading a given instrument is designed to read. For example, a pressure transducer may be designed for a maximum process pressure of 1,000 psi, so its full scale is 1,000 psi. An accuracy of ±1% of full scale would mean that for any given pressure reading the accuracy is ±10 psi. For an indicated pressure of 700 psi, the actual process pressure may be between 690 and 710 psi. For an indicated pressure of 150 psi, the actual process pressure may be between140 and 160 psi.

(2) Calibrated Span Accuracy. Many instruments have the ability to span, or turn down the maximum reading from the rated full scale. In the above example, the pressure transducer had a full scale of 1,000 psi. If the span were turned down to 200 psi and the accuracy specified as 1% of the calibrated span, then at any given pressure reading the accuracy is ±2 psi. If the span were changed to 500 psi, the accuracy would be ±5 psi.

(3) Actual Reading Accuracy. Accuracy may be specified as a percentage of the actual reading. For the same transducer as above with the accuracy specified as 1% of the actual reading, at an indicated pressure of 700 psi, the actual process pressure would be ±7 psi. At an indicated pressure of 50 psi, the actual process pressure would be ±0.5 psi.

b. Pressure Transducers. Transducers are available in a wide range of pressures, from fractions of a psi to thousands of psi. The transducer's maximum operating pressure or full-scale pressure should be selected based on the pressures necessary to perform the work, but, as with gauges, this should not be exceeded by a significant amount as the accuracy will suffer. If, for instance, a pressure transducer with a full scale of 10,000 psi and a specified accuracy of 1% of full scale is used to monitor grout injections at an actual process pressure of 300 psi, the instrument would indicate that the pressure is between 200 and 400 psi and still be within manufacturer-specified tolerances. For most HMG applications at reasonably shallow depths, electronic pressure transducers with a maximum range of 5,000 psi provide adequate accuracy, even at operating pressures significantly lower than the full scale. This is possible because the typical accuracy specification for such pressure transducers is ±0.1% of the calibrated span or better.

c. Electronic Flow Meters. The accuracy of electronic flow meters is typically a function of the velocity of the fluid in the flow tube. Typically the accuracy for magnetic meters is specified with respect to a range of velocities, commonly 0.3–1 m/s (1–3 ft/s) and 1–10 m/s (3–33 ft/s). At velocities in excess of 10 m/s (33 ft/s), the instrument will be out of range, but it may continue to show a reading of the maximum rate. Because of the nature of their operation, magnetic flow meters have limited accuracy when velocity in the flow tube drops below approximately 0.3 ft/s (approximately 0.5 gpm in a 1-in. meter since the flow tube diameter is often slightly smaller than the nominal pipe diameter). However, when permeation grouting to absolute refusal or when flow rates are very low (such as 0.1 gpm), it is important that readings be relatively accurate to ensure that grouting is neither terminated prematurely nor continued at additional cost. Magnetic meters are available that will accurately measure very low-flow rates, but since the measurement principle is based on velocity in the flow tube, the diameters of these instruments are small (0.1–0.25 in.) and not practical for grouting applications since they are easily blocked and not capable of reading higher flows when necessary. As a rule of thumb, if HMG is to be injected, the recommended magnetic flow-meter diameter is 0.75–1 in. The accuracies are likely sufficient provided that the manufacturer's specified accuracy is on the order of 1% of full scale with flow velocities greater than 3 ft/s and the instrument is freshly calibrated. Many magnetic flow meters have accuracies significantly better than 1% of full scale within this flow range. This should provide reasonable accuracy at refusal and adequate capacity for measurement of high flows (above 50 gpm during pressure testing). If additional flow rate capacity is desired, a larger-diameter meter must be selected; the accuracy near refusal flow rates will suffer, but in instances such as massive grouting applications, this is not of concern. Since grouting is performed over a wide range of flows, it is important that the grouting practitioner understand when to suspect flow inaccuracies and take steps to minimize their impacts. Specifications should require freshly calibrated meters, and meter specifications should be provided before starting work. The maximum flow rate that the meter is capable of reading should be specified, and injections should be conducted below that rate to avoid anomalous readings. Calibration checks against a master, non-production meter should be specified and conducted regularly. Since this master meter is likely to be of the same diameter as the production units, this form of calibration check is satisfactory for low to high-flow rates. If verification of very low-flow rates (less than 0.5 gpm) is required, then it is recommended that this be performed on a volumetric basis by timing the volume displaced at some very low-flow rate. It is important to note that the accuracy of a flow meter depends on the piping configuration in which it is installed. Most flow meters, except for true mass-flow meters, sense a representative cross section of the flow in the flow tube. Turbulence due to piping configuration, mismatched gaskets, turns, and other configurations can cause unpredictable measurements in derived flow measurements. It is recommended the flow meter manufacturer's installation recommendations be followed when specifying or designing flow sensing installations and when evaluating the performance of meters in the field.

12-4. Automated Monitoring Systems. Chapter 16 of this manual discusses the levels of the technology currently available and the records produced by automated monitoring systems. All contractors capable of a sophisticated grouting program have proprietary automated monitoring systems that differ widely in their capabilities; however, it is possible to design one with off-the-shelf hardware and software with relative ease. Two key factors that make this possible are advances in computer monitoring systems, and GISs that facilitate the final display of the grouting data that has been organized into a usable database. Common elements of all systems include pressure transducers and flow meters, power and signal cables or wireless data transmission, analog-to-digital converters, computer hardware, and monitoring software (Figure 12-2). Computer and communication systems change rapidly, and prospective users of automated monitoring systems are best served by contacting contractors or engineers who provide these systems for information on features and performance.

(Courtesy of Gannett Fleming)

Figure 12-2. Centralized automated system for real-time processing and display for actively monitoring and controlling injections.

CHAPTER 13

Site Access and Site Preparation

13-1. Introduction. This chapter addresses aspects of various site preparation activities necessary to prepare a site for grouting, and common applications of concrete platforms and plinths as they apply to grouting operations.

13-2. Site Access and Site Preparation Equipment.

a. General. One aspect of grouting that has been given little attention, but yet is critical to successful execution of the grouting program is site access and preparation. Detailed consideration in the design phase must be given to these issues because the way in which they are approached will affect many aspects of the grouting. The designer must know the physical characteristics of the surface from which grouting will be conducted, the environmental conditions under which grouting will be performed, and the safety issues for both the contractor's personnel and Government personnel.

b. Soil Surfaces. Drilling and grouting operations that are planned to be conducted from soil surfaces always warrant substantial preparation. Experience has shown that special measures will not be implemented unless specifically required by the contract documents. Without the construction of substantial working platforms, soil subgrades inevitably become a quagmire, resulting in unsafe working conditions, uncontrolled environmental conditions, and substantial risk to the quality of the work. For that reason, contract documents should define the minimum requirements for the working platform, including its minimum width, thickness, and slope, and for its waste collection facilities. Contractors may, of course, propose something exceeding the minimum requirements based on their operational needs.

(1) Gravel Platforms. Gravel or sand and gravel working platforms are not recommended. They will provide stability for a brief initial period, but water penetrating into the soil subgrade and/or choking of the surface with waste materials will rapidly destabilize the platform. Maintaining positive surface drainage is problematic. Periodically scraping and replenishing gravel surfaces has been found to be ineffective.

(2) Figure 13-1 shows an example of a gravel platform after a short period of use. When initially constructed, the platform was clean, dry, and stable. Even with frequent maintenance, it quickly reverted to the condition shown.

c. Hard Surface Platforms. Working platforms should be constructed of concrete, soil-cement, roller-compacted concrete (RCC), or other hard, durable material. Hard surface platforms offer advantages; they provide safer working areas, positive drainage and control of water and waste products, easier setup of drilling and grouting equipment, improved operational efficiency, better alignment control, hole stability at the surface, a point of fixity for casing at the ground surface, better protection of holes during the various operations, and easy marking of hole identification numbers. Typically, the minimum width of the platform should be approximately 25 ft to allow equipment to pass at any location; however, if multiple grout lines are to be constructed from a single platform, a wider platform may be warranted. The minimum thickness of concrete or soil-cement should be selected based on the anticipated equipment loads, soil subgrade condition, and design life of the platform. Concrete should be placed directly on a soil subgrade compacted to at least 95% of standard dry density. A granular base should not be used to avoid having a raveling layer beneath the hard platform. The platform should have a minimum cross slope of 2% to provide positive drainage and facilitate easy removal of cuttings and waste materials. Water and waste product collection facilities such as an excavated ditch should be located along the length of the platform. Provisions must be made for frequent removal and proper disposal of waste products. Figure 13-2 plainly shows the benefits of a hard surface platform, at the same site (as shown in Figure 13-1), but at a later date with a hard platform in place. The cost of a hard surface platform is minor relative to the benefits provided.

(Courtesy of Gannett Fleming)

Figure 13-1. Gravel drilling and grouting platform after a short period of use.

(Courtesy of Gannett Fleming)

Figure 13-2. Hard surface platform after extensive drilling and grouting operations.

d. Rock Surfaces. Most grouting for new structures, and sometimes for remedial grouting, is performed from an excavated rock surface. Understanding the nature of the rock surface from which grouting will be performed is essential to developing appropriate designs and contract provisions.

(1) Rock Surface Characteristics.

(a) Groutability. Except in unusual circumstances, excavations should go to the depth at which rock will grout readily. Rock that will grout readily will have the following characteristics: (1) it will be sufficiently massive that the surface does not consist of easily dislodged, small rock fragments, (2) it will not unduly ravel with equipment operating on the surface, (3) it will support borings in an open condition without having them blocked by detached or partially detached rock fragments, (4) it will not be so fractured as to make management of grout surface leaks an untenable operation, (5) fractures will be open and sound and generally free of weathered materials or infilled materials except for the occasional seam, and (6) it will permit successful setting of packers. In general, experience has shown that when the RQD approaches 40% or more, the rock will grout more readily, assuming that weathering and infilling conditions are also satisfied. If it is determined that the rock surface and rock mass does not have the characteristics to grout more readily, numerous special and costly measures may be required. The long-term performance of the finished grouting may also be uncertain. Measures that have been used when there was a need to perform grouting in low-quality rock include: (1) very short stage lengths grouted with downstage techniques, (2) repeated applications of grout to stages, (3) operating with undesirably low pressures, (4) greatly extended washing operations, (5) extensive surface caulking, (6) redrilling of stages, and (7) many additional holes on very close spacing. An alternative to attempting to grout rock that does not grout readily is to construct a concrete cutoff wall through the zone.

(b) Durability. Consideration should be given to the durability of the rock surface. If the rock is subject to air and/or water slaking, it will be subject to deterioration throughout the course of the grouting. There are two potential courses of action for slaking conditions. One is to place a concrete grout cap over the location where grouting is to be performed immediately after a fresh surface is exposed and cleaned. The second option is to use standpipes grouted into the rock and allow the slaked materials to accumulate on the surface, which will reduce the depth of slaking that will occur. Repeated cleaning should be avoided to prevent increasing the depth of slaking.

(c) Surface Roughness. The roughness of the excavated rock surface is a critical consideration in successfully planning, specifying, and executing many aspects of grouting. The actual surface roughness, when found to differ substantially from representations contained in the contract documents, has been a source of many delays, cost overruns, safety issues, and claims. Roughness is controlled by the site geology, and it may differ substantially between valley and abutment areas. The primary geologic factors controlling roughness are structure, jointing, and rock hardness. Some generalizations regarding surface roughness are:

1. The roughness of hard rock surfaces is controlled by natural asperities created by intersecting fracture systems. Widely spaced fracture systems will create large asperities.

2. Soft rocks can be shaped by excavating equipment, and smoothing of the foundation surfaces is possible to a varying extent.

3. Interbedded hard and soft sedimentary rocks will create ledgy conditions on abutments. Also, depending on the attitude of the bedding, the attitude of excavation surfaces may be controlled by the bedding.

Figures 13-3 through 13-7 show examples of excavated foundation surfaces with varying degrees of roughness.

(Courtesy of Gannett Fleming)

Figure 13-3. Smooth foundation surface along sedimentary rock horizontally bedded in a valley area.

(Courtesy of Gannett Fleming)

Figure 13-4. Rough foundation surface along sedimentary rock bedding in an abutment area.

(Courtesy of Gannett Fleming)

Figure 13-5. Large-amplitude roughness resulting from a fracture system and deep-weathered seams in metamorphic rock.

(Courtesy of Gannett Fleming)

Figure 13-6. Moderate-amplitude roughness in metamorphic rock.

(Courtesy of Gannett Fleming)

Figure 13-7. Smooth valley and abutment surfaces in sedimentary rock sufficiently soft to permit significant smoothing by excavation.

(d) Methods to assess probable rock surface roughness include examination and mapping of rock outcrops and quantitatively understanding the rock structure and fracture systems from investigation data. The recommended and most direct method is to perform large test excavations in the design phase so that the actual surface can be mapped and photographed. If surfaces are sufficiently smooth to permit reasonable access, movement, and servicing of drilling and grouting equipment, special measures may not be required. However, if the roughness will jeopardize efficient and safe execution of the work or will likely lead to claims, the design and construction documents must specify special measures required to provide reasonable access and operation. Most commonly, the measures will be either the extensive use of backfill concrete or a thick, continuous grout cap along the entire alignment. Figure 13-8 shows a planned solution and Figure 13-9 shows the difficulties that can occur when a planned solution is not provided.

e. Surface Steepness.

(1) Realistic assessment is required regarding the steepness of rock foundations and whether steep slopes will be smooth, rough, or ledgy. For smooth, steep slopes, it may be possible for equipment to access the work areas by using anchor and winch systems and providing stair systems for workers. For rough slopes flatter than about 1 V on 2 H, unformed dental concrete and/or an unformed grout cap is a good solution. Although contractors have attempted to use unformed concrete on steeper slopes, this has been found to be unsatisfactory. To keep the concrete from moving downhill, contractors may use concrete that cannot be effectively consolidated, they may avoid attempting thorough consolidation, or they may use stay-in-place forms at frequent intervals. Each of these techniques has been found by experience to produce negative effects, such as a highly porous grout cap or a grout cap with many porous construction joints. Figure 13-10 shows an application where stay forms were attempted on a 1 V on 1.5 H slope with minimal consolidation of the concrete; the results were so poor that chemical grouting of the entire cap was necessary.

(Courtesy of Gannett Fleming)

Figure 13-8. Extensive use of dental concrete for grouting access on a rough foundation.

(Courtesy of Gannett Fleming)

Figure 13-9. Working conditions resulting from not using a planned access solution.

(2) Therefore, for rough slopes steeper than about 1 V on 2 H, it will be necessary to provide a top form for the concrete so that it can be properly consolidated without losing it downhill, or, as an alternative, the contractor may be required to use movable work platforms. Other less-desirable alternatives include scaffold systems or column-mounted stand drills. Unless a fully mechanized scaffolding system such as a movable platform is used, scaffolding for individual setups is slow, often dangerous, and potentially cannot provide sufficient support or workspace for heavy drills. Column-mounted stand drills attached to pre-installed standpipes often do not have sufficient power to drill holes of the desired hole diameters and the required depths, and they may not have sufficient rigidity to be able to adequately control hole deviation. Figures 13-11 to 13-13 show examples of access systems used on steep slopes.

(Courtesy of Gannett Fleming.)

Figure 13-10. Porous grout cap requiring chemical grouting.

13-3. <u>Grout Standpipes and Grout Caps</u>.

a. General. Grout standpipes and grout caps are used separately or in combination where required to properly conduct the work. The reasons for their use and their specific configuration will vary depending on the application.

b. Grout Standpipes. Grout standpipes are steel pipes that are embedded in either the rock surface or the grout cap. Standpipes are also sometimes referred to as a “collars,” “nipples,” “surface pipes,” or “connectors.”

(1) Functions. Standpipes can serve one or more functions, depending on need: (1) they can be used to provide a point of connection for the grout delivery lines and grout header, either by direct mechanical connection or by providing a length of pipe for inflating a pneumatic packer, (2) they can be used in weak or broken rock to facilitate setting of packers or to help control surface leaks, (3) they can facilitate proper re-alignment when multiple drill setups are required on a hole, and (4) they can serve as barriers against the entry of surface water and waste materials into the hole.

(Courtesy of USBR)

Figure 13-11. Ridges Basin Dam right abutment scaffolding for grouting operations.

(Courtesy of Advanced Construction Techniques)

Figure 13-12. Drilling rig operating on an inclined concrete platform.

(Courtesy of Advanced Construction Techniques)

Figure 13-13. Crane-supported drilling platform on a steep abutment.

(2) Usage. Standpipes are not always required. For example, they may not be necessary when the rock surface meets the definition of readily groutable rock, provided there is not a surface drainage issue. They may also not be necessary when a thick grout cap is used, since a drilled hole in concrete can provide the same functions. In general, they are required in highly fractured rock and at any location where surface drainage may contaminate the grout hole.

(3) Materials and Procedures. Standpipes should either be black or galvanized steel. Plastic standpipes are not workable. Standpipes are grouted or concreted into place in slightly oversize holes. Standpipes should have a minimum embedment of 2 ft and a minimum exposure of 0.5 ft. If mechanical fittings are used, they can either be directly threaded fittings or quick disconnects. After grouting is finished, standpipes should be cut off flush with the surface. Figure 13-14 shows standpipes being used in an appropriate situation.

(4) Specifying. Standpipes should be specified by material, minimum length, and minimum depth of embedment. Specifying diameter is typically not necessary, since the requirement is that the diameter must be suitable for introduction of all equipment that the contractor will be using; however, if rebar is present in a grout cap, a maximum diameter specification may be appropriate to minimize cutting of reinforcement. Measurement and payment is usually based on the number of standpipes installed. Even if the majority of work is not anticipated to require standpipes, provisions should be included in the contract documents to use them at selected locations where required.

c. Grout Caps. Grout caps are used in a variety of configurations for a variety of purposes. They are not necessarily required, but their usage is quite common. Grout caps can be used for the following functions: (1) to provide a positive cutoff in highly fractured surface rock, (2) to act as a barrier to surface leaks, (3) to set standpipes, (4) to replace standpipes in conjunction with pneumatic packers, (5) to provide an effective working platform on rough rock surfaces, (6) to provide effective environmental control, and (7) in special cases, to act as a connector between the grouted zone and other elements of the structure.

(Courtesy of Advanced Construction Techniques)

Figure 13-14. Standpipes embedded in rock for protecting holes in a wet excavation.

(1) Cutoff Caps. Grout caps are sometimes constructed to provide positive cutoffs in highly fractured, weak, soft, or seamy rock. By definition, these conditions do not correspond to "readily groutable rock." Without a cutoff cap in this type rock, it is difficult to set standpipes or packers in the rock, surface leaks may be nearly uncontrollable, the rock may not be able to withstand the pressures being applied without uplifting, and it may not be possible to successfully grout badly weathered or infilled joints. Given that near-surface rock is both the most difficult zone to grout effectively and often the most critical zone of the foundation, intentionally planning to reduce the amount of excavation and overcome the resultant problems by using a cutoff cap is not generally recommended, but rather a cutoff cap may be required when excavation to readily groutable rock is precluded. In such cases, the required depth of the cutoff cap may be considerable. Figure 13-15 shows conditions where a cutoff cap would be appropriate and beneficial.

(a) Shape and Dimensions. If the prime purpose of the cap is to serve as a cutoff, it should be approximately 3 ft wide for depths up to 10 ft. Wider caps may be used when depths exceed 10 ft. The sides of the cap should be nearly vertical to develop shear resistance against uplift. The minimum depth of the cap should be 3 ft. Trapezoidal, triangular, and shallow slab configurations are not acceptable because they will be uplifted from the foundation under minimal pressure.

(b) Excavation. Cutoff caps are excavated by a suitably sized backhoe. Explosives are generally not necessary; if the rock is sufficiently competent to require explosives, it probably does not require a cutoff cap. In intermediate materials, it may be possible to effectively use a rock trenching machine, which also has the advantage of being able to excavate a narrow trench in rock to depths up to or exceeding 10 ft. Hand-held tools, such as jackhammers or pick and shovel, may be required in conjunction with a backhoe, but given the unit cost and production rates, this is not practical except to assist with a few isolated problem areas.

(Courtesy of Gannett Fleming)

Figure 13-15. Heavily jointed rock suitable for excavating a cutoff cap.

(2) Surface Grout Caps. Surface grout caps are seeing wide usage as a multiple function platform. Objections to initial cost are being set aside in favor of the many benefits they can provide, both in quality and overall program costs. They are used on slaking rock surfaces to protect rock surfaces from deterioration, on rough rock surfaces to provide efficient access and increase production, and in almost any setting to provide better hole protection, facilitate drill setups, reduce surface leakage problems, provide better environmental control, allow higher grouting pressures in the upper zone of rock, and increase safety. Surface grout caps are thick, typically on the order of 18–24 in., and their width is selected based on constructability needs. This thickness of slab allows pneumatic packers or standpipes to be set entirely within the concrete slab, which then permits direct grout access to every fracture and to the interface of the slab and the top of the rock. To eliminate uplift concerns, the cap slabs are normally anchored to the foundation. Typically, anchors will consist of 10-ft-long No. 11 bars on 5-ft centers. Temperature steel or light reinforcement in slabs is common. Treatment of construction and contraction joints must also be considered and might vary depending on the application. Figures 13-16 to 13-18 show examples of surface grout caps.

(Courtesy of Gannett Fleming)

Figure 13-16. Inclined and stepped surface grout cap on a rough, ledgy abutment of a concrete dam foundation.

(Courtesy of Gannett Fleming)

Figure 13-17. Smooth-surface grout cap used on a steep, ledgy, and broken rock surface.

(Courtesy of Gannett Fleming)

Figure 13-18. Grout cap used on a smooth, slaking rock surface.

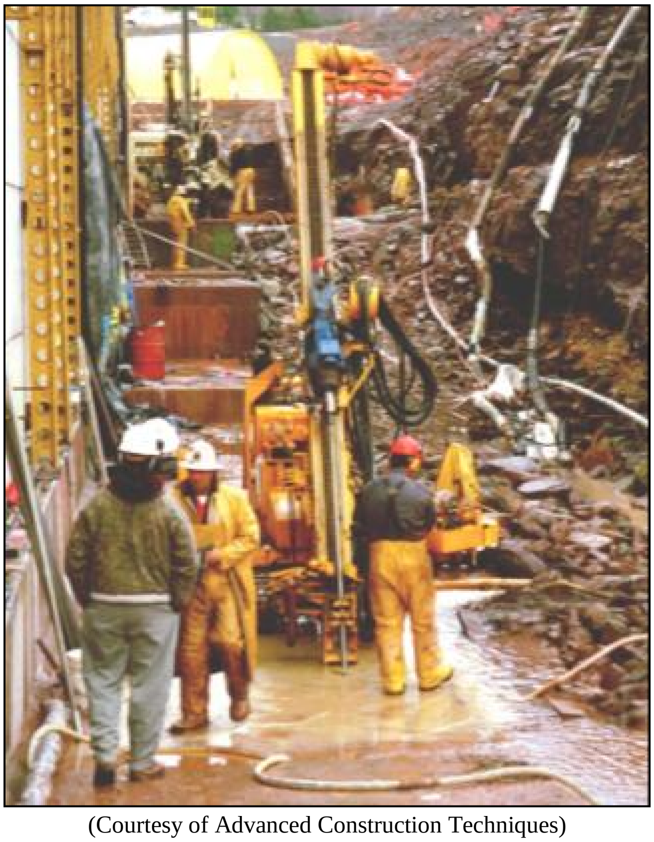
(3) Plinths. Plinths are a special type of surface grout cap. Plinths are usually constructed at the upstream toe of dams, either as part of new construction or as part of a remedial program. Plinths are physically integrated with and connected to other seepage control features in the project. In some cases, the plinth is the logical extension of a seepage barrier. An example is a rock-fill dam with a concrete or synthetic liner system, where the plinth acts as the connector between the facing system and the foundation. In other cases, the plinth might serve multiple purposes, e.g., it may provide a starter wall at the upstream toe for RCC dam facing systems, and it may function as a surface grout cap. An advantage of plinths, even when not required strictly for integration with other features, is that they often allow grouting operations to be removed from the critical path. With proper safety precautions, grouting can be performed simultaneously with other construction operations rather than being required to be performed separately in advance. Depending on the project, this can reduce construction time by as much as a year. Figures 13-19 through 13-21 show plinths that have been used on several projects.

13-4. Galleries.

a. General. Galleries are provided in most large concrete and RCC dams, and have occasionally been used in earth dam construction. Galleries provide access for inspection, a location for collecting seepage, a location for instrumentation cabling, and a means of access for grouting. Where grouting has been initially planned to be performed from a gallery, the reasons are usually to remove grouting from the critical path, to take advantage of the weight of the structure to use higher pressures without fear of uplift, and/or to ensure full sealing of the structure with the foundation surface. In reality, grouting from a gallery is both difficult and expensive due to space limitations; there are limitations on hole orientations that can be used, and it is a difficult working environment for the contractor. For those reasons, the preferred use is for remedial grouting activities. A plinth and properly oriented grout holes, along with electronic monitoring systems that can detect uplift, provide essentially the same benefits while at the same time allowing the use of large production equipment and unrestricted access. Figure 13-22 shows a gallery within an RCC dam.

(Courtesy of Gannett Fleming)

Figure 13-19. Plinth allowing grouting (on left) to be concurrent with RCC dam construction (on right).

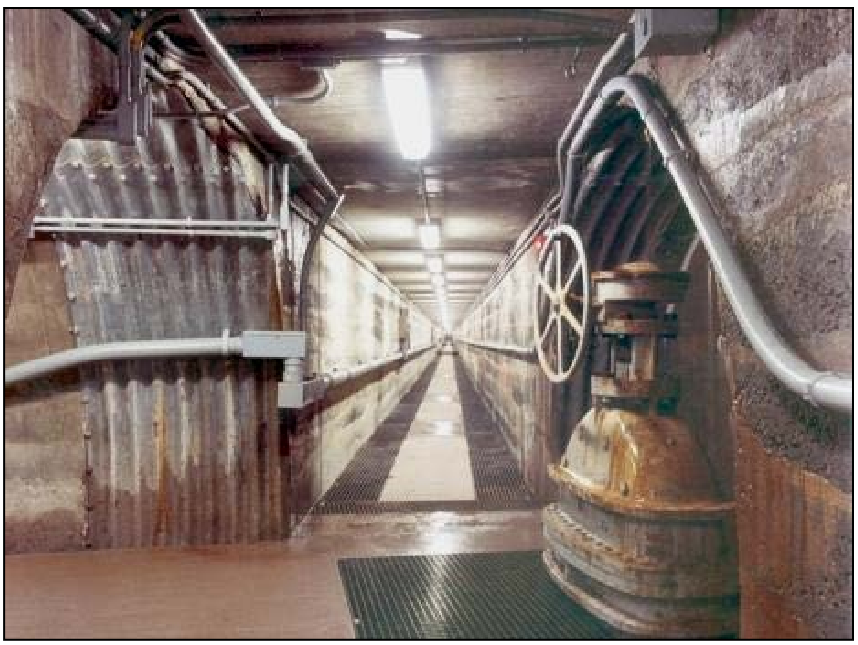


(Courtesy of Advanced Construction Techniques)

Figure 13-20. Drilling operations concurrent with RCC dam construction.

(Courtesy of Advanced Construction Techniques)

Figure 13-21. Final grouting operations being completed as an RCC dam nears completion.



(Courtesy of Gannett Fleming)

Figure 13-22. Entrance to a gallery in an RCC dam reserved for drainage and remedial grouting.

b. Drilling Equipment. Drilling equipment used for grouting in galleries is similar to that used for surface work. Typically it is electrically or pneumatically powered and configured for low overhead clearance and narrow widths. Drilling in galleries can be particularly slow due to the use of short drill rods (typically 5 ft or less) and the limited power of the equipment. Traditionally, rotary coring methods are used. Grout batching equipment is typically located outside of the gallery due to its size and environmental concerns with particulates. Batched grout is commonly delivered to the gallery through a circulation loop. A small agitator and grout pump receive the batched materials and convey them to a header unit located near the hole. Production rates for gallery grouting are on the order of 50% of similar surface work. Sequencing of activities is critical, since often the available space prohibits the passing of even the smallest equipment. Figures 13-23 and 13-24 show sample equipment and conditions.

(Courtesy of Advanced Construction Techniques)

Figure 13-23. Low-clearance pneumatic rotary drill.

(Courtesy of Advanced Construction Techniques)

Figure 13-24. Remedial grout injection. Note the crowded conditions in this 9-ft-wide space.

13-5. Site Infrastructure Systems.

a. General. In general, insufficient attention has been paid to the site systems that are required for both effectively controlling the work environment and protecting the natural environment. With proper site infrastructure systems, grouting has been performed at temperatures below –50 °F to above 130 °F. Grouting is often performed on either a double-shift basis or on a round-the-clock basis. Grouting is often performed in environmentally sensitive areas that require protection from contamination by materials and waste products used in the grouting process. Historically, specifications have contained minimal performance requirements related to these items, and it is common that sufficient funds are not allocated to properly address these items. Even in relatively temperate climatic situations, it is common for productivity to suffer, for quality to be adversely affected, and for worker safety to be compromised. If prescriptive specifications are used, the conditions that will be encountered and the project requirements must be carefully considered. While it is not necessary to specify the details of site systems that must be provided, the general nature and types of systems required should be listed. If the Best Value selection process is used, the contractor should propose in detail both the equipment and site layout of the equipment that will be used.

b. Environmental Control Systems. Unless specifically required, protective shelters for storing materials and mixing grout may not be provided. It is not uncommon that the only protection provided is a sheet of plastic over a pallet of cement. Even for the smallest projects under the most favorable conditions, effective shelters, even if makeshift, should be provided for equipment, materials, and operators at both the point of mixing and the injection header. On any sizeable project, particularly where grouting will be conducted from one or more central mixing sites, the shelters should be sufficient in size for all planned materials storage, operations, and quality control operations. Depending on the nature of the project, it may be required to provide shelters for the entire work area. Figures 13-25 through 13-29 show examples of environmental control systems.

(Courtesy of Gannett Fleming)

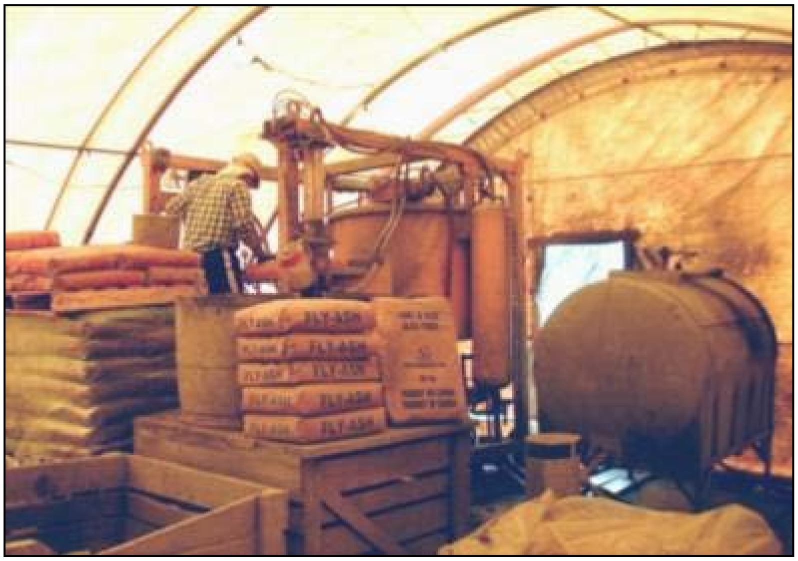
Figure 13-25. Minimal sheltering limited to plastic sheeting to cover bags of cement.

(Courtesy of Advanced Construction Techniques)

Figure 13-26. Field-constructed shelter over grout mixing station.

(Courtesy of Advanced Construction Techniques)

Figure 13-27. Fully enclosed and heated prefabricated shelter for materials storage and mixing.



(Courtesy of Advanced Construction Techniques)

Figure 13-28. Inside view of a fully enclosed, prefabricated shelter.

(Courtesy of Advanced Construction Techniques)

Figure 13-29. Custom mobile grout headers and injection control equipment.

c. Lighting. Only rarely can grouting projects be conducted without some night operations, and even more rarely is the lighting adequate for the work. Prescriptive specifications should be clear on the amount of light required at each area of operations, and on general site lighting requirements. Refer to EM 385-1-1 for all lighting requirements.

d. Access. Specifications should contain requirements for constructing stairways with railings at all ledges and on steep slopes. In general, slopes steeper than 1V on 2H should have stairs for access by both the contractor's workers and Government forces. If the number and length of stairways that will be required are uncertain, a simple method of covering this item is to require stairs to be constructed the full height of both abutments. Figure 13-30 shows typical examples of abutment stairway systems. Refer to EM 385-1-1 for all access requirements.


Figure 13-30. Typical access stairways on abutment areas, Folsom Dam (Sacramento District).

e. Environmental Protection Systems. Drilling and grouting operations use large quantities of water and generate considerable waste products. The environmental protection requirements for each project need to be carefully considered, and the specifications need to be tailored to meet those needs. In general, the contractor is normally required to exercise control over all water and waste products by using ditches, pumps, temporary collection sumps, and sedimentation ponds. It is not acceptable for water and waste products to be flowing uncontrolled through the work areas. Work performed over or adjacent to water bodies require particularly stringent specifications. Erosion and sediment control features may be required throughout the entire area of operations. Specifications should cover in detail both the method and location of waste disposal. Maintenance of the environmental protection systems should be covered in the contract documents. Figures 13-31 and 13-32 show project sites with adequate environmental controls.

(Courtesy of Gannett Fleming)

Figure 13-31. Hard surface platform with a cross slope leading to a collection trench.

(Courtesy of Advanced Construction Techniques)

Figure 13-32. Wash water and cuttings being directed to an environmental control system.

13-6. Hot and Cold Weather Operations.

a. General. Equipment for operating in hot and cold weather are special considerations. These conditions are not obstacles to performing quality grouting or even to productivity if proper site equipment is provided at the outset. When not dealt with properly, both quality and productivity decline dramatically.

b. High Temperatures. Even in temperate regions, it is not uncommon for the foundation surface to reach temperatures as high as 130 °F at the peak of the summer. The temperature of grout, measured at the agitator tank, is not permitted to exceed an upper limit, usually 90 °F. Temperatures beyond that point begin to have adverse effects. Set time is greatly accelerated, and grout can either begin setting in the delivery lines before injection, resulting in the injection of agglomerations of particles that will choke off fine fractures without penetration or that will limit penetration greatly; or grout in the agitator can hydrate without setting due to the mechanical breaking of bonds, such that poor quality, very weak grout may result after injection. Normally, contract documents will not specify the particular means the contractor is to use to control high temperatures, but will possibly list a group of potential remedies that may be required. Methods that have been found to be effective include: (1) providing shade shelters for materials and equipment, (2) using groundwater supplies rather than surface water supplies, (3) adding ice to water supply tanks, (4) restricting grouting to night operations, (5) using water chiller units, (6) providing a field-built water or ice bath through which grout delivery lines are run, (7) using reflective insulation, or (8) using liquid nitrogen cooling systems.

c. Winter Grouting. In general, winter grouting operations are more problematic than hot weather operations. The grout temperature must be maintained at or above 40 °F, and the surface rock must be maintained above 40 °F when the grout is injected and for 5 days thereafter.

(1) Rock Surface Temperature. The rock temperature must be above freezing at the time of grout injection and for a lengthy period thereafter to prevent freezing of the grout and the formation of voids after thawing of ice in fractures. Additionally, it must actually be maintained at the higher temperature of 40 °F for grout to set within a reasonable period of time. If winter grouting of exposed rock surfaces is required, the surfaces can be maintained at the required temperatures by: (1) covering the foundation with insulation blankets, with or without heating beneath the blankets as required, (2) enclosing and heating the entire workspace to maintain temperatures, or (3) using a surface grout cap with a thickness exceeding the depth of freezing.

(2) Grout Temperature. Grout can be maintained at 40 °F or higher by using one or more of the following methods: (1) protected, heated enclosures for materials storage and for mixing, agitating, and pumping equipment, (2) insulated tanks and lines, and (3) heated mixing water. Figures 13-33 through 13-36 show effective measures for project sites in which winter work was performed without interruption.

(Courtesy of Advanced Construction Techniques)

Figure 13-33. Winter drilling operations.

(Courtesy of Advanced Construction Techniques)

Figure 13-34. Water heating and pumping plant.

(Courtesy of Advanced Construction Techniques)

Figure 13-35. Winter-ready site with covered equipment storage, a fully enclosed and heated shelter for materials and mixing plant, and a water heating system.

(Courtesy of Gannett Fleming)

Figure 13-36. Winter shelter for core logging operations.

(3) Productivity Issues. The impact of cold weather operations on productivity varies greatly between contractors, depending on their available equipment and their experience. For well-equipped and experienced contractors, the impact on productivity may be minimal. Other contractors may have productivity decreases of 50% or more, and some contractors will simply not conduct winter operations at all. While some may argue that winter operations and productivity are strictly the contractor’s problem, in fact, projects with critical timelines may require winter work. On projects where winter work is known to be likely, the project will be better served by declaring that winter work will be required and specifying all of the winter measures that will be necessary. The Government will make the bidders aware of the schedule and conditions that are to be expected throughout the course of the contract.

13-7. Site Safety.

a. General. Grouting operations must be conducted in accordance with EM 385-1-1. Beyond the normal safety issues typical of general construction, the following topics are of particular concern on grouting projects.

b. Slips and Trips. The presence of grout and other grouting operation by-products on even well prepared work surfaces creates slip and trip hazards. Grout should be removed through the use of high-pressure water wash before setting. Hardened grout may be removed by chiseling. Grout should be removed from all traffic and pedestrian areas immediately upon spillage. Site safety in winter operations is a critical consideration. The combination of freezing weather with a process that fundamentally uses voluminous amounts of water increases the potential for slip and fall hazards, and for personal injury related to the handling of icy equipment. Leaking lines and anything less than complete care and control of discharge water are unacceptable. Efficient and effective snow removal is required to maintain safe access to both the site and all work areas.

c. Traffic, Signage, and Lighting. Grouting projects often involve numerous pieces of equipment traveling frequently across a confined job site. Control of traffic is critical to worker safety under these conditions. All ingress and egress routes and rights-of-way should be clearly delineated in the field. A site speed limit should be identified, posted, and observed. Lighting should be provided at regular intervals, particularly at intersections for night operations. Use of traffic lights, flagmen, and physical barriers may be necessary to safely facilitate the work when in close proximity to public thoroughfares. Local departments of transportation should be consulted for particular requirements.

d. Air Quality. Batching of grout, particularly when bagged cement or flyash is used, can present an inhalation hazard to the plant operator and nearby work force. Work performed inside galleries, tunnels, or other locations that may or may not be considered confined spaces may require ventilation. Methods to ensure worker safety under these conditions vary and should be identified by the contractor before starting work.

CHAPTER 14

Hydraulic Barrier Design

14-1. Introduction. Constructing hydraulic barriers is one of the principal applications of grouting. In every case, the purpose of a hydraulic barrier is to alter flow rates, flow paths, pressure distributions, and/or gradients to achieve specific and predictable results. The required geometric configuration and permeability of the barrier necessary to achieve the design performance objectives are project specific and can depend on a wide variety of factors and considerations. This chapter provides a brief discussion of a variety of applications and considerations that might be used as a basis for establishing the design performance requirements for grouted barriers and for some of the approaches that might be required to achieve the required performance.

14-2. New Earth Dams.

a. Flow Rate Reduction. Flow rate reduction beneath an earth dam or levee is a common application of hydraulic barriers. The more difficult issue is establishing an acceptable residual seepage rate. In some cases, it has been selected on the basis of what is generally considered to be visually acceptable amounts of seepage appearing in the drains or at the toe of the dam. While this is not a particularly rational approach, it obviously has a strong intuitive basis in that large amounts of seepage are, in general, not good for the long-term performance of an earth dam due to potential long-term degradation caused by flowing water. In general, flow modeling will show that achieving an acceptable result will require a grouted barrier with a very low residual permeability.

b. A second basis for establishing the acceptable flow rate is to compare the residual seepage rates with the entrance capacity of the drainage system. Most earth dams use both a chimney drain and a blanket drain system. Flow through the foundation must be able to enter the blanket drain system unimpeded and then be carried away under small upstream to downstream gradients to the designed exit points. Otherwise, excessive heads may occur beneath the drainage layer and redirect the flow paths to the toe of the dam without providing filtration and/or result in piezometric levels extending above the drainage system rather than being contained within the drain. Sand is the most common material used in the entrance layer of the drain, and therefore its permeability and flow capacity can be the limiting factor for acceptable residual seepage rates. It is critical that the actual properties of the drain be included in the flow model rather than making an assumption that it will be sufficiently pervious to provide drainage under all flow rates. When possible, it is highly recommended to have an expected drain capacity of 10–100 times that of the predicted residual seepage rate. The longitudinal capacity of the drain (i.e., upstream to downstream) can normally be enhanced by using coarser drain fill to provide the capacity required under small drainage system discharge gradients while still meeting the filter criteria.

c. Protection of the Core/Foundation Interface. The interface between the core of the dam and the rock foundation is one of the most critical zones in an earth dam, and multiple protection features are normally used to prevent potential erosion of core material at that location: (1) intensive rock surface treatment is normally used to eliminate interface flow channels in rock fractures that daylight on the surface, refer to Chapter 20, Paragraph 20-2 for a discussion on blanket grouting and Figure

20-1 for blanket grout hole layout, (2) core contact zones are often widened at that location to reduce the risk of hydraulic fracturing in the vicinity of the interface region, decrease the probability of the core extending over the length of exposed continuous fractures, and reduce flow velocities in any interface defects, and (3) drains are carefully configured in that region to capture and filter any flow that might result from minor construction defects.

d. Similarly, grouting of the interface zone must be given special attention. This zone is a critical location, but it is also the most difficult zone to grout. Effective grouting is impeded by the necessity of using low grouting pressures, by surface leaks that can detrimentally influence the quality of grouting, and by having packers in rock at the top of the hole that prevent access to near-surface fractures. Additionally, if the grouted barrier is constructed to a very low permeability relative to the foundation materials, it will shift the boundary conditions in the flow regime such that the full reservoir head will be applied at the upstream side of the barrier and can increase flow gradients in that region. Accordingly, it is common to use the following measures for the barrier in the interface zone.

(1) Expanded Width of the Grouted Zone. As noted above, a grouted barrier with very low relative permeability will shift the location of the full reservoir head to the upstream edge of the grouted zone. If the grouted zone is narrower than the core contact area, it functionally negates some of the intended benefits of a wider core contact area. A narrow barrier will induce high-head differentials over a short distance, and can increase both the erosive velocities in any foundation defects and the potential for hydrofracturing in the base of the core. For all of those reasons, it is common to widen the grouted zone in the top 10–20 ft of the foundation to match the dimension of the core contact area by constructing additional lines of grouting to that depth. These lines should be grouted to the same performance requirements as the deeper portions of the barrier.

(2) Thick, Reinforced, Anchored Grout Caps. Thick, reinforced, anchored grout caps can provide substantial benefits when grouting a highly fractured interface zone. As described elsewhere in the manual, this type of cap provides a relatively smooth surface for easy and safe access for drilling and grouting, and allows drainage and cuttings to be controlled, thereby decreasing the potential for contamination of drilled holes and allowing far better environmental controls. Thick caps can also substantially improve the quality of grouting in a highly fractured interface zone. In the absence of such a cap, surface leaks routinely occur when blanket grouting and grouting the upper stages of curtain holes on abutments. Grouting operations can be severely compromised by surface leaks, which can slow production and adversely affect quality. A common response to controlling surface leaks is to rapidly thicken the grout mix, which reduces the permeation of the grout and can result in ungrouted finer fractures. This practice is not recommended for this reason and other materials such as lead wool or oakum that are commonly used to plug surface leaks are preferred. Further, in a highly fractured foundation, applications of more than minimal pressures are difficult, and packers placed in rock can obscure the access to fracture systems. The use of a thick, reinforced, anchored cap allows the packer to be seated in concrete, permits the use of higher pressures, and eliminates many of the surface leaks.

(3) Pressure Distribution Control. It is relatively rare that the grouted barrier for an earth dam would be designed with the primary goal of creating a particular pressure distribution. However, the pressure distribution that will be created by the grouting program must be considered and accommodated in the design of other features.

(4) Partially Penetrating Cutoffs. Generally, fully penetrating grouted cutoffs should be used and should terminate in zones that are 100 times less permeable than the permeable zone of interest. Such cutoffs are highly effective in reducing seepage rates, and seepage bypassing the cutoff (i.e., flowing under the cutoff) tends to be relatively low. While the flow and pressure behavior of fully penetrating cutoffs that meet this condition must always be carefully examined in the flow model, the residual seepage, flow paths, and exit gradients must be accommodated in the overall design, but will normally not be found to be a controlling design issue.

(5) In reality, however, despite their extremely low seepage reduction efficiency, as discussed in Chapter 5 of this manual, partially penetrating cutoffs (i.e., those that terminate within the permeable zone or in a zone with only a slightly lower permeability) are sometimes required. As flow models will show, their behavior is quite different from a fully penetrating cutoff, and the basis for design can be quite different. A low-permeability, partially penetrating barrier may have high rates of residual seepage beneath the barrier, and depending on its depth it can materially affect the flow path directions, the length of the flow paths, the flow path exit locations, and the exit gradients. Each of these issues must be carefully examined to determine if the resulting behavior is manageable and acceptable in the design.

14-3. <u>New Concrete Dams</u>.

a. Flow Rate Reduction. Concrete dams often result in a non-erodible dam material in contact with a non-erodible foundation material. In such situations, high rates of seepage might be acceptable from a dam safety standpoint. In practice, however, it is common to establish an acceptable residual rate even if the technical basis for the amount may be subjective.

b. Foundation Pressure Control. It is common in concrete dams to include both a grouted barrier and drilled foundation drains on 5- to 10-ft centers immediately downstream from the grouted zone (the drains must be designed to properly intercept residual seepage paths and must have sufficient capacity to prevent pressure buildup). While the grouted barrier reduces the rate of seepage, it also works in conjunction with the drilled drain system to reduce foundation uplift pressures, which improves the stability of the dam. Functionally, the two systems work together to increase the difference in relative permeability between the barrier and the foundation materials. The grouted barrier decreases the foundation permeability in one portion of the foundation, and the drilled drain holes increase the effective permeability in an immediately adjacent region. This relationship of the two systems results in increased efficiency in limiting uplift pressures. EM 1110-2-2200, *Gravity Dam Design*, provides the criteria for evaluating uplift.

(1) Critical Applications. Some dams, mostly large ones, have stability designs based on achieving a specific, reduced foundation pressure distribution. This design approach can result in a more slender cross section and can result in considerable savings in construction, particularly in wide valley settings. When a specific pressure reduction is absolutely required for adequate stability safety factors, achieving this pressure distribution and maintaining it over the life of the dam becomes critical. Accordingly, it substantially elevates the importance of an effective combination of foundation grouting and drainage, along with extensive verification of results. Additionally, because the pressure reduction is required to be permanent, the following elements are also normally required as part of the programs, which sometimes are found to be infeasible or which involve long-term costs that may offset the initial savings:

(a) Extensive, Long-Term Instrumentation Program. When permanent pressure reductions are required, an instrumentation program of sufficient intensity is needed to monitor the pressures at many locations. Multiple instrumented cross sections are required, and the number of instruments at each section must be sufficient to clearly define the piezometric conditions beneath the structure. The monitoring program must be carefully designed, and sufficient qualified staff must be available over the life of the structure to obtain and analyze the piezometric data. Long-term maintenance and periodic replacement of the instrumentation system will also be required.

(b) Access for Remedial Grouting and Drainage System Observation and Maintenance. Including a gallery near the base of the dam is almost always a requirement when the long-term proper functioning of the grouting and drainage system is critical to stability. Both the grouted barrier and the drainage system should be accessible from the gallery. Strictly scheduled inspections, observations, drain discharge measurements, and drain testing programs in the gallery, combined with piezometric data, are necessary to assess and verify long-term behavior and performance. The gallery also provides access for periodic remedial grouting and for restoration of the drainage system performance by cleaning holes, redrilling drain holes, or drilling additional drain holes. Note that remedial grouting, if required, can severely compromise the drainage system.

(c) Factors of Safety for Stability. Even when all of the above elements are in place, it is common for the design to be based, in part, on maintaining a factor of safety greater than 1.0 under extreme loading conditions and with the grouted barrier/drainage system not functioning (i.e., full uplift conditions). This approach helps ensure that a poorly functioning portion of the system cannot create an imminent dam safety issue.

(2) Non-Critical Applications. Because the useful life of dams is extremely long and often indeterminate, comprehensive long-term compliance and proper execution of monitoring and maintenance in accordance with the requirements for critical applications for periods of 100 years or more may not be practically feasible. Accordingly, it has become increasingly common to provide a grouted barrier/drain hole system, but to not rely on its performance as a long-term critical design requirement. In these applications, the dam is designed for adequate factors of safety for stability on the basis of experiencing full uplift pressure distributions, and the grouting is primarily provided to reduce seepage rates. Galleries are often included, but the drainage system, instead of being a critical element, becomes more of a tool for monitoring the long-term performance of the grouting in the various reaches of the foundation. Improvements in stability that result from incidental uplift reductions in a properly functioning system are a favorable byproduct of the system rather than a critical long-term performance requirement. This non-critical application approach is often determined to be in the best overall interests of the project, even if the initial cost savings resulting from an engineered pressure reduction appear to be favorable.

14-4. Remedial Grouting at Dams. Remedial grouting at dams, both earth and concrete, involves the same general considerations as grouting for new dams, but it can involve more complex issues because the reason for remedial grouting is often related to one or more types of fundamental defects in design or construction. Geologic investigations, determinations of existing piezometric conditions, analyses, and flow models must be sufficient to thoroughly define material properties and reproduce the current flow regimes under various head conditions.

Subsequently, the desired performance behavior must be established, and then the site must be carefully evaluated to determine if the performance can, in fact, be achieved by grouting. It is not uncommon for a site to have zones that will be extremely difficult to treat; that will require special procedures, materials, and/or techniques for safe and/or effective treatment; or that simply cannot be treated effectively by grouting. Problems are often greatly compounded when pool levels cannot be substantially reduced during the treatment program. It obviously involves risk when numerous holes are to be drilled into a problematic structure and foundation that are carrying flow or are under pressure. Care, proper planning, and technical restrictions must be observed to prevent damage to embankment dams when drilling through them, to carefully control all grouting operations in the embankment/foundation interface zones, to prevent connecting zones that are currently not connected and that might create hazardous conditions if connected, and to provide proper means for penetrating and treating high-flow or high-pressure zones. When treatment by grouting is feasible, consideration should also be given to whether grouting in one region might cause adverse flow concentrations or pressure changes in another region of the structure or its foundation.

14-5. Grouting in Connection with Cutoff Walls at Dams.

a. General. Cutoff walls have been used for remedial treatment of seepage in problematic embankments and foundations. Slurry is a fluid used to support the earth sidewalls of an open excavation during cutoff wall construction. It is a mixture of water and clay (bentonite or a mineral clay) or polymer in a colloidal suspension that is only slightly more viscous than water. The stability of a slurry-filled trench relies on keeping the slurry level sufficiently high so that it exerts a positive pressure on the walls of the trench and on the formation of an extremely low permeability filter cake on the trench walls to act as a membrane upon which the pressure can act. When the excavations penetrate rock and encounter open fractures of sufficient size the slurry can flow away freely. The critical fracture size for slurry loss in rock is not known; however, if large enough, slurry losses may be uncontrollable, causing the trench to become unstable and collapse. When cutoff walls extend into rock, grouting of the rock may be required if there is a concern for loss of the support slurry during excavation of the cutoff wall. Trench collapse when constructing cutoff walls is unacceptable. These collapses can endanger personnel; cause equipment loss; damage an embankment section, which must then be explored, assessed, and repaired; and create immediate and severe dam safety hazards. The slurry used should be engineered according to the site-specific characteristics of the soils to be excavated. The use of grouting as either a systematic and deliberate pre-treatment method or as part of a combination solution that uses cutoff wall construction and grouting, each to its best advantage as dictated by the foundation conditions, is becoming more common. The following paragraphs describe the various approaches.

b. Grouting Operations as Exploration and Partial Pre-treatment for Cutoff Walls. When the geologic conditions are largely unknown or when the final cutoff wall depth has not yet been established, grouting can be used as an exploration tool, and as a way to provide at least a partial pre-treatment of rock fractures. In this scenario, one line of closely spaced grout holes (i.e., 5-ft centers) is meticulously drilled, while water testing and grouting records are obtained. Holes should be grouted to refusal so that the maximum future benefit of the drilling and grouting is obtained. The data and the results are used to make a final determination of cutoff wall depth, and the records either are used to determine the required pre-grouting on a second line of grouting on the opposite side of the cutoff trench, or are furnished to the slurry wall contractor for use in planning additional grouting and cutoff excavation requirements. The grouting lines should be located outside the limits of the cutoff wall excavation so that significant obstructions will not be created along the alignment due to the inevitable periodic loss of drilling and grouting tooling in the holes. The lines should be located as close as possible to the wall alignment without interfering with wall construction.

c. Grouting in Advance of Cutoff Construction. When the required depth of the cutoff is established, advance grouting can be used as a planned, comprehensive, and systematic pre-treatment method. At Mississinewa Dam (Louisville District), where it was found that the initial cutoff wall test sections could not be completed without uncontrolled slurry loss and trench collapse, it was decided to pretreat the entire foundation to a prescribed Lugeon value. A simplified analysis indicated that, if the zone in which the cutoff wall was to be constructed consistently had a final water permeability of less than or equal to 10 Lugeons, then the slurry losses would be manageable. A minimum grouted zone thickness extending 5 ft beyond the limits of the proposed trench excavation was found to be suitable for this residual permeability. During construction of the pre-grouted zone, the grouting refusal requirements were tuned by experimentation and experience, which allowed grouting to less than absolute refusal on each stage. After completion of the systematic grouting, the entire cutoff wall was constructed without further incident.

d. Combined Hydraulic Barriers. Historically, a choice has generally been made to use either a cutoff wall or grouting for remedial treatment. That concept is changing, as single projects now often use both methods, each to its best advantage.

(1) Cutoff walls are excellent engineering solutions for treating zones that are not readily groutable. Examples of good locations for application for cutoff walls include: within the embankment itself, in embankment/rock interface areas, in foundation zones containing mixed soil and rock materials or many soil-filled joints, and in areas of highly developed karst with soil-filled features. The disadvantages of cutoff walls include their very high cost; the risk of trench collapse; issues related to the quality and watertightness of panel overlaps; and construction tolerance and alignment issues at depths exceeding 200 ft. The methods for monitoring, measuring, and analyzing the overlap of cutoff wall elements and their positions in space have greatly improved with the technological advances made in this area of construction.

(2) Cutoff walls are not necessary in zones in which the rock fractures are clean and readily groutable. These zones can be grouted to very low residual permeabilities and can provide an adequate barrier at substantially lower cost and lower risk. Additionally, at depth, the continuity of grouted cutoffs tends to be of less concern because the zones they treat are intrinsically wider due to the ability to use higher grouting pressures without damaging the formation.

(3) The optimum solution for a site that has been thoroughly explored, tested, and characterized might consist of: (1) pre-grouting for the cutoff wall zones to a minimum verifiable Lugeon value to minimize slurry loss during wall construction, (2) intensive grouting of deeper zones within the foundation that require a cutoff, but that are readily amenable to grouting, and (3) a cutoff wall in the shallower zones where grouting effectiveness may be marginal.

14-6. Environmental Applications.

a. General. Environmental applications for grouted barriers include containment of contaminated materials, deflection of groundwater flows around contaminated zones, and up-gradient barriers to improve the efficiency and reduce the amount of collected water for pumping and treatment alternatives. As with other applications, the specific performance goals for the grouted barrier must be established by modeling the flow regime with and without barriers having specific, realistic characteristics. The following paragraphs describe some design considerations for environmental applications.

b. Maximum Effectiveness. Environmental applications frequently involve achieving the best results permitted by a particular technology. In the case of grouting, there is no universal agreement, but generally it is considered possible to construct multiple-line grouted barriers to a residual water permeability of 1 Lugeon or less. At the Chicago McCook Reservoir site (Chicago District, USACE), it was demonstrated in a full-scale field test that a grouted barrier around the perimeter of an excavated storage facility for combined sewer overflow could be constructed to a consistent residual permeability on the order of 0.1 Lugeon using balanced stable grouts formulated with silica fume and injected to absolute refusal. Given that the geologic conditions were very favorable to grouting and that the best technique was used, this value is believed to be close to the limit of what can be achieved for cement-based grouts. Limited testing on the site using ultrafine cements did not indicate improved results.

c. Compatibility with Other Design Elements. Environmental applications often include a cutoff wall component located in unconsolidated materials. The most common type of cutoff wall is a soil-bentonite cutoff, which typically has a width on the order of 3 ft and a barrier permeability of 1×10^{-7} cm/s. When a barrier is also required in rock below the soil-bentonite cutoff, it is desirable to have a flow impedance (i.e., equal flow under equal head) in the grouted barrier comparable to that of the cutoff wall. Achieving impedance compatibility requires a very low permeability grouted barrier of significant width. For example, a grouted barrier having a flow impedance identical to that of the soil-bentonite cutoff wall described above would require a grouted barrier with a residual permeability of 0.1 Lugeon and a width of approximately 48 ft. Higher permeability results would, of course, require a substantially thicker grouted barrier to achieve a comparable flow impedance.

14-7. Dewatering Flow Control.

a. General. In the majority of cases, dewatering is accomplished by wells and/or pumping from collection sumps without the use of grouted barriers. However, there are exceptions where grouting might be necessary or economically viable as part of a dewatering program. Some of those exceptions might include: (1) situations where long-term flow reductions are of high economic value (i.e., long-term quarry operations), (2) situations where there are contaminants that create both safety hazards and/or extraordinary pumping and disposal costs, (3) thin zones of high permeability that are readily groutable, but that are difficult to dewater through reasonably spaced well penetrations, (4) sites where prevention of groundwater level drawdown is required to prevent adverse effects near the site (e.g., induced settlements of structures, critical groundwater supplies, impact on adjacent streams), (5) situations where permanent seepage reductions along with the use of drains are needed to control both inflows and pressures, and

(6) confined working areas where seepage inflows would substantially impede other work being performed and where there is insufficient space for dewatering or sump facilities. It is not uncommon for grouting to be used as a supplemental measure in localized areas where it is found that other dewatering facilities are not adequately effective.

b. Tunnels. A special case of dewatering flow control exists for tunnels. Tunnel faces in rock are routinely grouted, and the grouting operation functions in an exploratory role, in a face stabilization role, and in an inflow reduction role. While it would be highly desirable to apply all of the same design principles to creating a hydraulic barrier of defined geometry and residual permeability concurrent with tunnel operations, it is not always possible to do so. There are limitations on both the direction that grout holes can be drilled from a tunnel face and on the time available for grouting due to the cycling of tunneling operations. For those reasons, it may not be possible to drill and grout in optimum orientations from within the tunnel, nor is it possible to grout without regard to other construction operations. Accordingly, in tunnel grouting, the approach tends to be to empirically evaluate the results of a program without much flexibility and then to deal with the consequences if the outcome becomes unmanageable.

14-8. Other Factors Considered in the Basis for Design.

a. General. In addition to specific technical issues related to specific applications, other factors, described in the following paragraphs, are often considered as part of the basis of design. Some of these factors are quantifiable, and some are not.

b. Risk and Consequences. From a dam safety perspective, risk is defined in ER 1110-2-1156 as a measure of the probability and severity of undesirable consequences or outcome. The process for determining risk informed alternative selection should be performed as is described in ER 1110-2-1156.

c. Cost-Benefit Analyses. In the absence of specific performance requirements established by technical factors, flow models in combination with cost-benefit analyses can be used to guide the grouting design requirements or to optimize the design of the grouting program. Both the geometry and the required permeability of the grouted zone can be altered, and approximate costs can be assigned to those variations. The geometry can be altered by adding more grout lines. The permeability can be altered by adding more holes or more lines or by increasing the quality of grouting. The quality of grouting can be improved by increased levels of care, sophistication, and quality control, and by using contracting and procurement methods that are tailored to the complexity of the work. Of the three methods for achieving lower residual permeability in the grouted barrier, improvement of quality is generally the most cost effective; the cost premium compared to extremely low grade work may be as little as 15–25%, while the result may be better by an order of magnitude or more. The cost increase for adding holes and lines is proportional to the number of holes added.

(1) Completing a cost-benefit analysis involves quantifying the value of benefits received for various grouting configurations and performance results. For example, in a pump storage project, the value of water lost through seepage can be quantified as there is a direct cost associated with obtaining and pumping water to the reservoir. Similarly, in environmental applications, there can be a direct cost associated with handling contaminated water, the volume

of which can be affected by the grouting program. Where handling inflows has a direct cost, the benefits of reducing the long-term handling can be considered.

(2) The cost-benefit analysis should consider both the total cost-benefit ratio and the incremental costs and benefits of various grouting program alternatives. The nature of grouting is that each additional series of holes will have a diminishing incremental benefit compared to earlier series. This does not imply that the additional series are not necessarily required nor of high value. It merely recognizes that, in most cases, the incremental reductions in permeability are less as the program progresses.

d. Long-Term Durability. All other factors being equal, long-term durability will be most directly related to both the completeness of filling of the fracture system and the width of that zone. Assuming that aggressive chemical attack of the grout is not an issue, most agree that deteriorating performance of grouted barriers over a long period of time is the result of flow through partially filled fractures. The matrix permeability of grout is extremely low and probably on the order of that of concrete, which is approximately 1×10^{-9} cm/s. Therefore, flow through completely filled fractures is essentially nonexistent, and long-term deterioration should not be an issue.

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CHAPTER 15

Hydraulic Barriers: Field Procedures

15-1. General Considerations. This chapter deals with field execution of drilling and grouting operations, techniques that are commonly used to solve typical problems that are encountered, and field evaluation of the completed work.

15-2. Field Decisions.

a. General. Even with thorough investigation programs and well-conceived designs, there are some variables that often require refinement throughout the course of the grouting program and other variables where specific decisions must be made by the contracting officer. Specifically, items such as duration of washing, grouting pressure, refusal criteria, starting mixes, and combining of stages for grouting often evolve as the program progresses and the results are analyzed. Other items, such as final hole spacing, final hole depths, and special treatment zones, require specific decisions by USACE based on analyses of the results with respect to the performance requirements.

b. Decision Making.

(1) Responsibility. Decisions regarding the grouting program are the responsibility of USACE and occur at several levels. Daily decisions that are required for specific holes or stages of holes must be made on site by the contracting officer and the project engineer or geologist in charge of grouting. The decisions often must be made immediately and must consider both technical and contractual implications. More comprehensive decisions regarding the overall effectiveness and general conduct of the program often involve other technical and non-technical professionals who meet regularly at the site to review progress, technical issues, and results.

(2) Basis for Decisions. Making sound decisions for grouting programs requires a thorough understanding of the following items: site geology, the overall design intent and its relationship to other project features, specific quantitative performance requirements for the completed grouting, all elements of drilling and grouting theory and practice, contractual requirements, contractor operational considerations and productivity impacts, risk assessment, and sound interpretive procedures for properly assessing completed results. This comprehensive skill set is obtained through long-term, successful experience. Under the best-case scenario, where an experienced project engineer or geologist is paired with a highly knowledgeable contractor, the grouting program will be successful. Under the worst-case scenario, where a project engineer or geologist deficient in experience is paired with a below-average contractor, the results can be disastrous, either from a technical or a contractual standpoint, or both.

15-3. Field Preparation Activities.

a. Project Review. One of the first items that should be undertaken is a comprehensive project review. The purpose of the review is to thoroughly understand the factual information on which the design was based, assumptions and interpretations that were made from the factual information, the design intent and specific performance requirements, the means and methods that will be used to verify that the completed grouting will meet the performance requirements, and the contract documents intended to facilitate the execution of the design. The review should be critical in nature, and any issues of concern or questions should be identified and addressed before the start of any work.

b. Analysis and Integration of Available Data. A common pitfall is that all available data are not integrated into the project geologic database before the start of construction. In most projects, and particularly in remediation projects, there have been multiple investigations or studies performed at various times for various purposes. In some cases, the investigation record can span as much as 30–50 years. The engineer or geologist in charge of the grouting program should collect all available information from the original design and construction records and reports, investigation and testing programs, instrumentation programs, performance history, seepage records, adjacent site conditions, and all other possible sources, and integrate it into a single body of knowledge as it relates to the grouting program. The information should be consolidated onto plans, profiles, sections, and performance behavior plots to facilitate the maximum understanding of pre-grouting conditions and to aid in interpreting observations made during the drilling and grouting program.

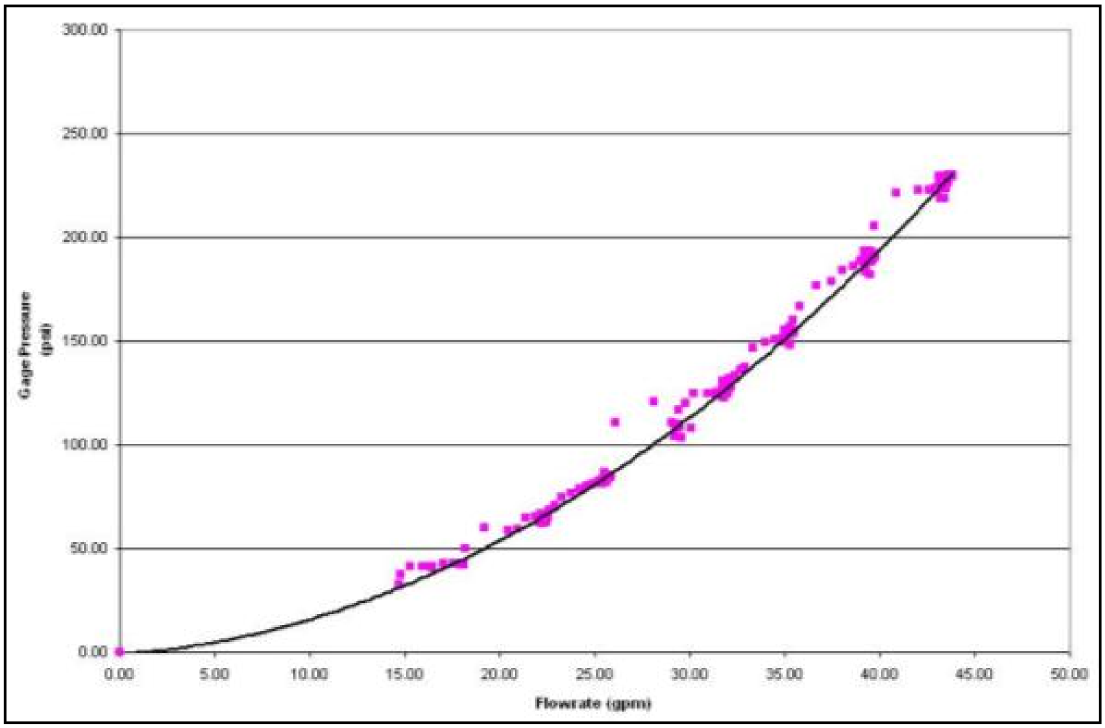
c. Planning of Recordkeeping and Data Interpretation Methods. Considerable planning is required with respect to the data that will be collected, both in terms of form and content, and how those data will be used. The contract documents will assign the respective responsibilities for data collection and reporting, but they often do not define what happens to the data beyond that point. Advance planning of the data management and analysis methods is essential for successful grouting. See Chapter 16 of this manual for additional information on recordkeeping.

15-4. <u>Equipment Checking and Equipment Calibrations</u>.

a. General. All equipment to be used in the work must be checked to determine compliance with the contract requirements with respect to type, size, condition, and adequacy. All equipment calibration certificates that are required should be examined to ensure that they have been properly and freshly calibrated before delivery to the site. It is normal to require the contractor, in advance of starting operations, to assemble all equipment in the system and demonstrate proper system functioning before allowing its use. The following paragraphs describe additional special items that require field calibration.

b. Flow Meters. The accuracy of mechanical or magnetic flow meters is of special concern. These meters are critical to accurate batching of grout mixes; they are used to determine refusal, and are essential pieces of equipment for pressure testing, which is the basis for evaluating the results of grouting. Mechanical flow meters have been found to be highly inaccurate at low-flow rates. Conversely, some magnetic flow meters have been found to have accuracies exceeding the manufacturer's stated accuracy, which enables them to be reliably used for water testing and grouting at very low-flow rates. Regardless of type, the accuracy of flow meters used in the work should be checked frequently. See Chapter 12 of this manual for additional discussion on flow meter accuracy.

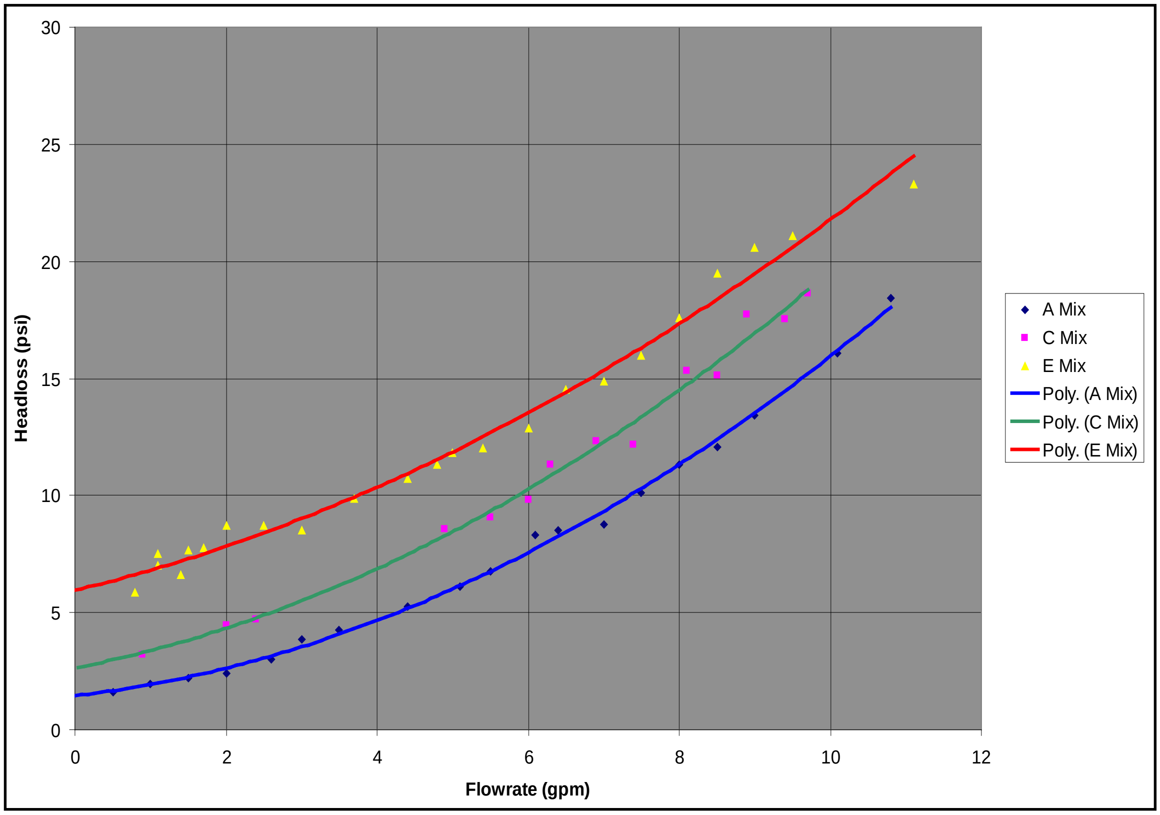
c. Pressure Testing Equipment. Pressure testing equipment must be fully assembled and tested to establish the pressure-flow relationship of the equipment itself. Valid pressure tests can only occur when pressure can be built up within the ground and when the permeability of a zone being tested is less than the flow rate limitations at various pressures of the assembled equipment. The equipment is tested by assembling it on a reasonably horizontal surface and finding the limiting flow rate corresponding to each pressure setting (also known as a head-loss curve for the injection system). If pipe is being used as the transmission line, then the test must be performed multiple times with different lengths of pipe in the system to develop a family of curves. If hose is used as the transmission line, only a single calibration is required since the total length of hose in the system is a constant regardless of the depth of testing. Failure to determine and know the system limitations can give erroneously low permeability values in moderately permeable to high permeable materials. Figure 15-1 shows an example of a system capacity or head-loss curve for pressure testing equipment. The line represents the limiting boundary for flow rates at any given pressure resulting from the hydraulic characteristics of the equipment. Valid tests can only be conducted when the combination of pressure and flow are in the region to the left of the curve, which results in positive pressure being applied to the ground.



(Courtesy of Gannett Fleming.)

Figure 15-1. Example of a limiting capacity curve for a pressure testing system.

d. Grouting System. To determine effective pressures being applied to the formation throughout the entire grouting process and to accurately calculate the Apparent Lugeon value at any point in time, it is also necessary to establish head-loss curves for the grouting system and use them in calculations for effective pressure. As with the pressure test equipment, the complete grouting system is assembled and tested for each grout mix to develop a family of head-loss curves that are used in effective pressure calculations. Also as with pressure testing, if pipe rather than a fixed length of hose is used as the delivery line, it is necessary to also check the variation of the relationship for different pipe lengths. Figure 15-2 shows an example of a set of calibration curves for grouting.



(Courtesy of Gannett Fleming.)

Figure 15-2. Example of field-determined head-loss curves for grout mixes.

15-5. Downstage and Upstage Drilling and Grouting Procedures.

a. General. The drilling and grouting procedures should be determined based on the geologic conditions at the site.

b. Explanation of Methods.

(1) Downstage Drilling and Grouting. The downstage method, sometimes referred to as “descending stage grouting,” involves completing drilling, washing, water testing, and grouting of a stage within the hole and allowing the grout to set for a predetermined amount of time before advancing the hole to the bottom of the next stage. When it is time to drill the next stage, the grout in the overlying stages is redrilled (or jetted out) as part of the process. These steps are repeated until the final hole depth is achieved. The packer for grouting can be repeatedly set at the top of the first stage, and the entire length of hole can be grouted during each downstage cycle. An alternate variation is to set the grout packer at the top of the most recently drilled stage. The procedure for repeatedly setting the packer at the top of the first stage must be done with caution. The controlling allowable pressure will be at the top of the first stage for the entire hole. This limits the ability to increase the pressure with depth or increases the risk of damage if the allowable pressures are mistakenly increased with depth. The various items of drilling, washing, water testing, and grouting equipment are mobilized to the hole for each stage. Stages are predetermined before drilling based on available exploratory data. Normally, all stages on all

series of holes are completed to the depth of the first stage before beginning the second stage of the holes.

(2) Upstage Drilling and Grouting. The upstage method, sometimes referred to as "stop grouting," involves the advancement of the hole to the full depth in one drilling operation, followed by washing the complete length of the hole in one washing operation. The length of the hole is then divided into stages. Stages are individually isolated using a double packer system and are pressure tested beginning at the bottom of the hole and working upwards. Grouting is conducted through a single packer in similar fashion, with the grout packer being moved to the next higher location after grouting has been completed in the prior stage and the pressure within the stage has dissipated. The various items of drilling, washing, water testing, and grouting equipment are mobilized to the hole only one time.

(a) Composite Methods. Performance data have shown that both grouting methods can be effective, and it is quite common to use both grouting methods to best accommodate the geologic conditions within particular portions of a project. Locations requiring downstage grouting might be restricted to particular horizontal reaches of a project (e.g., known foundation discontinuities, faults, or highly broken or weathered zones), particular vertical reaches of a project (e.g., weathered rock overlying competent rock, or an isolated discontinuity at depth), or particular hole series.

c. Advantages, Disadvantages, and Applicability of Methods.

(1) General. Ground conditions and technical considerations govern the selection and use of appropriate drilling and grouting procedures. Cost and schedule implications for the project are simply a result of those factors. Attempts to select a methodology based on cost rather than on suitability for the geologic conditions is technically ill advised and can result in inferior work at the same or higher cost. Several notable authors have commented on this issue:

(a) Houlsby (1990) on Upstage Grouting. "This is the cheapest method on sites where all goes well, but not where failures occur. Its apparent lower cost is often an attraction to specification writers who are trying to minimize costs and are keeping their fingers crossed that all will go well and holes will not collapse too often."

(b) Warner (2004) on Upstage Grouting. "With the substantial limitations this method should probably be reserved for grouting in very competent rock. In sufficiently massive rock with discontinuities generally uniform in width, this is unquestionably the fastest and most efficient method. However, this ideal condition seldom exists in the real world of grouting, so the method is often problem prone, especially where fractured rock zones are encountered. The initially perceived savings can therefore rapidly decrease. … There are also substantial technical shortcomings to this progression. … Obviously, a greater amount of drill cuttings will find their way into higher joints and discontinuities. Significantly, this is usually the zone containing the largest number of discontinuities and where the highest quality of work is needed."

(2) During the course of the grouting program, it may become necessary to shift from the upstage to the downstage grouting method. The contract specifications should specifically state guidance as to

when this should occur. A typical specification requirement that was included in the Center Hill Dam contract defined the conditions that would warrant a change in the grouting method as:

> "When drilling encounters a 50% accumulated drill fluid loss, encounters artesian groundwater conditions, encounters rock conditions that prevent practical advancement of the borehole for upstage grouting, or encounters an accumulated 10 feet of non-rock materials, or where the COR so directs, downstage grouting shall be performed. Upstage grouting is the preferred method of grouting for this work, but the amount of upstage grouting that is practical will vary from section to section, and between the various series of holes. The Contractor shall cooperate to the maximum extent possible in the effort to move to upstage grouting methods."

(3) Table 15-1 summarizes the advantages, disadvantages, applicability, and other considerations for each drilling and grouting method.

Table 15-1. Applicability, advantages, disadvantages, and other considerations for downstage and upstage methods.

Considerations	Downstage Drilling and Grouting	Upstage Drilling and Grouting
Typical site conditions favoring use	• Rock of all types and conditions. • All water loss situations and/or sites with frequent hole connections. • Known problematic zones or reaches (e.g., highly broken or weathered zones, fault zones, weak rock zones, soil-infilled zones). • Karst formations and other void areas.	• Good quality rock (i.e., RQD>40%) that results in non-collapsing boreholes. • Sound rock suitable for sealing of packers on borehole sidewalls. • Minimal number of water losses during drilling (if water loss occurs, operations must cease and zone must be grouted before continuing). • Minimal number of hole connections.
Advantages	• Shallow zones, often the most difficult and most important zones to grout, can be repeatedly grouted. • It is the most flexible method available to accommodate all conditions. • Drill cuttings from lower stages cannot clog fractures in higher zones. • It reduces hole interconnections that can result in incomplete or ineffective grouting.	• Stage lengths can be varied to fit conditions disclosed by drilling and pressure testing (e.g., short stage lengths can be used in problem zones and long stage lengths can be used in uniform, low-permeability zones). • This method is cheaper and faster than downstage grouting, provided conditions are favorable for upstage methods.

Table 15-1. (Continued).

Disadvantages	• It is more expensive than upstage grouting. • There is a potential for heaving surface rock when grouting without a heavy confining load if packer is set at surface, which can be avoided by setting packer at top of most recently drilled stage.	• Low-pressure grouting used in shallow zones. • Drill cuttings can contaminate fractures along entire length of hole. • Grout may bypass packers via the fracture system and re-enter hole above locations of packers. • It is difficult to seal packers in weak or highly fractured rock, and water tests or grouting may cause loss of hole or drill tooling. • Connections with nearby holes may contaminate the holes before being grouted.
Other considerations	• It is common to use this method for upper zones of rock and known problematic zones. • This method is sometimes used to prepare site for upstage operations (e.g., sometimes for primary and secondary holes only, sometimes for first two lines, but not the middle line).	

15-6. Drilling Operations.

a. Hole Layout, Identification, and Marking. Grout hole locations must be established by survey tied to the project control points. The hole designation or identification system to be used requires careful consideration because all records will be tied to the system that is selected. At the time of layout, the hole identification codes are normally painted on a durable surface such as the grout cap or the cleaned foundation rock surface. It is good practice to color code the holes based on hole series. Markings are highly prone to degradation or obliteration, and it is common to refresh markings during each shift whenever required. Figure 15-3 shows an example of typical hole markings in the field.

(Courtesy of Gannett Fleming)

Figure 15-3. Typical numbering and color-coded marking scheme for grout holes.

b. Hole Alignment and Deviation.

(1) Hole Deviation. The deviation of the hole bottom from its planned location is affected by the angle of drilling, the depth of the hole, the type and condition of drill used, the diameter and stiffness of the drill tooling, the condition and type of bit, the accuracy of the drill setup, and the geologic conditions. Relatively few grouting projects have included actual measurements of hole deviations, so there is little actual data available on this topic. It has frequently been assumed that the net effect of all of the factors affecting deviation tends to result in a relatively consistent pattern of deviation and therefore is not of great consequence, particularly with the benefit of increased grout travel at depth resulting from the use of higher pressures. These assumptions might or might not be true in a given situation. Accordingly, potential hole deviation should be carefully considered in the design process, during the preparation of contract documents, and during the execution of the work. The functional deviation requirement for a project, particularly for seepage control, is that the sum of the basic hole spacing and the potential deviations at any depth does not exceed the distance over which grout can effectively travel under pressure to form a continuous barrier. In many projects, where the depth of grouting is in the range of 100 ft or less, experience has shown that relatively little concern exists with normal practice. At a depth of 100 ft, with a total equivalent angular deviation of 5 degrees, the actual bottom separation of holes with a nominal surface spacing of 5 ft would be less than or equal to 22.5 ft. Although this sounds excessive, the grout travel distance is directly proportional to the applied effective pressure (assuming all other factors such as fracture size and grout mix are equal), so the much higher pressure that can be used at depth should produce a result that is roughly equal or better than when using low pressure in the top stage where the total deviation is minimal. Deviation can, however, become critical in the following circumstances: (1) in deep holes with critical grouting

performance requirements, (2) in holes that are intended to either intersect or avoid underground structures or geologic features, (3) and in designs in which holes are terminated at various depths predicated on the benefits of increased grout travel under higher pressures at depth. Figure 15-4 shows the potential hole separation distance at various depths with differing amounts of total equivalent angular deviation for holes intended to have a nominal 5-ft spacing.

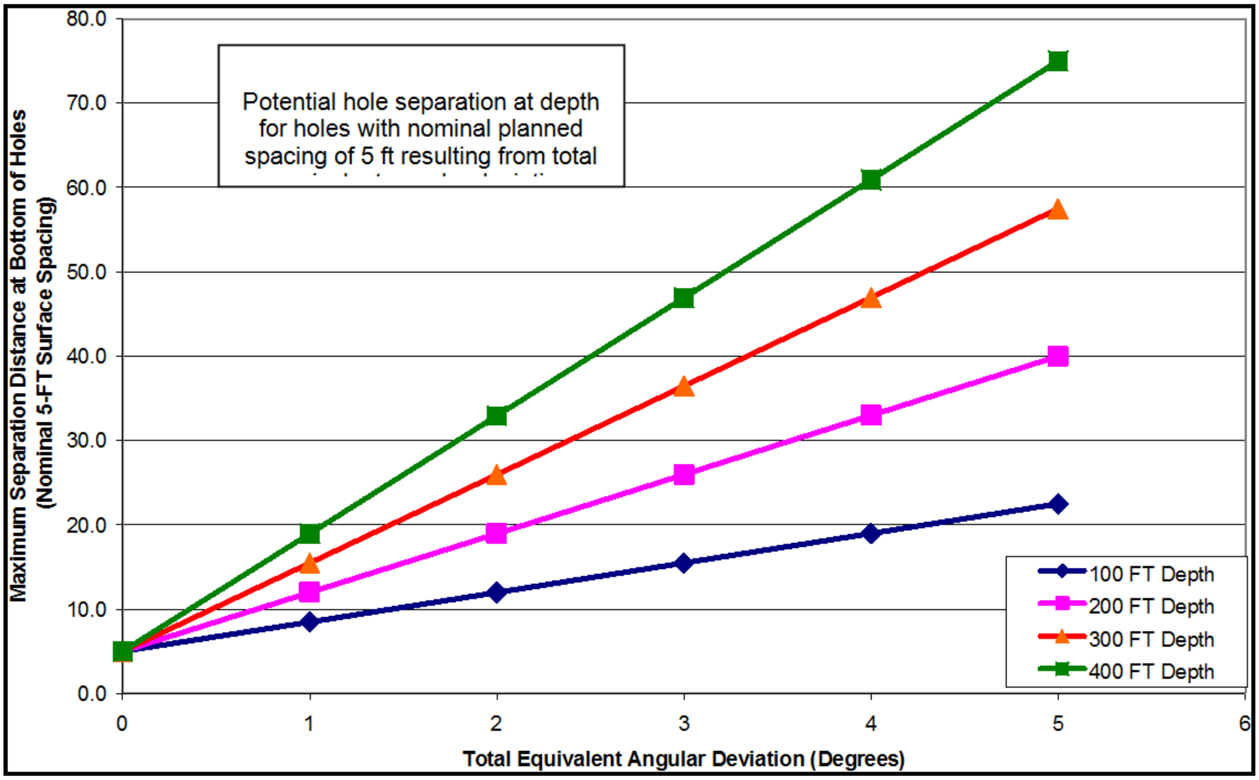


Figure 15-4. Maximum separation distance between adjacent holes for 5-ft nominal surface spacing and for various depths as a function of equivalent angular deviation.

(2) Inclination of Holes. In the majority of seepage control applications, it is desirable to incline borings to maximize penetrations of multiple fracture systems. The efficiency of drilling and grouting is often increased by increasing the inclination as much as practicable. Aside from any potential interaction with geologic features, the planned angle of drilling affects the hole deviation. The drill string is acted on by gravity, so angled holes tend to bend downward with depth. Larger inclinations generally result in greater deviation with depth, sometimes resulting in holes that are nearly vertical at their terminus. It has often been assumed that the holes will bend or curve in approximately parallel misalignment and therefore that the deviation is not of great consequence; however, that might or might not actually be the case and is difficult to quantify.

(3) Drilling Method, Tooling, and Hole Size. The drilling method, along with the diameter of the tooling, use of guide rods, and hole diameter, significantly impacts the hole deviation. Normal top-hole percussion drills, which under favorable conditions can drill to about 200 ft deep, may deviate substantially off course by the time the hole is completed. The use of a larger-diameter guide rod directly behind the bit, in conjunction with larger-diameter drill rods,

will minimize deviations; however, cuttings can sometimes prevent the flushing fluid from exiting between the rod and the borehole sidewall, with undesirable consequences. Even with these provisions, top-hole percussion methods typically generate the largest deviations. Holes to depths of 300–400 ft can be accurately drilled by careful diamond rotary drilling or by using down-hole hammer drills. Rotary core drilling typically is more accurate than rotary blind drilling (solid face bit, no core recovery), and the highest accuracies can be achieved with both rotary and down-hole hammer methods when pull-down or feed forces are minimal. It has become common to use minimum 3-in.-diameter holes in grouting projects, which reduces deviations and may improve access to fractures. Bit selection, rod selection, and drilling technique all affect the deviation of the hole for any of the drill types.

(4) Drill Setup. One of the easiest items to control in the field is the accuracy of the drill setup. Historically, a setup was generally considered acceptable when the drill was positioned within a degree or two of its planned inclination and azimuth. Tools such as a Brunton compass, inexpensive bubble levels, and plumb bobs generally suffice if that is the level of accuracy required. However, digital inclinometers with 0.1 degree of resolution are now readily available, and there is no valid reason to introduce more than 1.0 degree deviation from the planned angle in the initial setup. Transits should be used to verify alignment of the drill in the plane of the grout curtain or borehole azimuth. Holes should be drilled as close as possible to the marked location, but in no case should a starting location deviate by more than 6 in. from its planned location.

(5) Geologic Conditions. Ultimately, the geologic conditions often have the most control on both the absolute amount of hole deviation and the consistency of the hole deviations. Irregular rock surfaces, weak seams, alternating beds of hard and soft rock, and geologic structure interacting with the orientation of the boring can all significantly affect hole deviation.

(6) Normal Deviations to be Expected. As noted previously, there is very little actual data available to assess how much deviation has actually occurred in completed projects. There is a somewhat limited consensus that even under favorable conditions (e.g., vertical holes in relatively homogeneous materials), the equivalent total angular deviation for hole depths of 100–200 ft can easily fall in the range of 3–6 degrees. Complex geologic conditions, introduction of inclinations, and/or the absence of equipment specifically designed to reduce deviations can easily result in actual deviations significantly in excess of this range. Bruce and Davis (2005) provide an excellent summary of measured hole deviations for various drilling methods (Table 15-2).

Table 15-2. Summary of measured drill hole deviations.

Source*	Application	Method	Recorded deviation	
Bruce (1989)	Dam anchors in rock and concrete	DTH hammer and rotary	Target 1 in 60 to 1 in 240; mainly 1 in 100 or better achieved	
Bruce and Croxall (1989)	Deep grout holes in fill	Double head duplex	Achieved 1 in 50 to 1 in 100 (average 1 in 80)	
BS 8081 (1989)	Ground anchors	General	1 in 30 “should be anticipated”	
Houlsby (1990)	Grout holes in rock	Percussion	Up to 1 in 100 at 60 m	
Weaver (1991)	Grout holes in rock	DTH hammer	1 in 100 increasing to 1 in 20 with increasing depth (70 m)	
		“Dry drilled percussion”	1 in 6	
Bruce et al. (1993)	Dam anchors in rock and concrete	DTH hammer	Target 1 in 124, consistently achieved as little as 1 in 400	
Xanthakos et al. (1994)	General in soil	Drive drilling	Up to 1 in 14	
Kutzner (1996)	Grout holes in rock	Percussion	Up to 1 in 20	“Unavoidable”
		DTH	Up to 1 in 50	
		Rotary blind	Up to 1 in 33	
		Rotary core	Up to 1 in 100	
		Wireline core	Up to 1 in 200	
	Horizontal holes in soil	Percussive duplex	Less than 1 in 100	
PTI (1996)	Tiebacks	General statement	Up to 1 in 30 normally acceptable	
FHWA (1999)	General	High-speed rotary	2 to 5 normally acceptable	
		Top-drive percussion	<5 to 20 in 100 depending on depth	
		DTH hammer	Typically 1 to 2 in 100	

*After Bruce and Davis (2005).

(7) Projects with Critical Deviation Requirements. For projects where hole deviation is determined to be a critical issue, a drilling test section should be performed in advance of construction, with actual deviations measured for various types of equipment and/or drilling protocols. The results of the test section should be used in specifying equipment and procedures to be used in the project. Even

with a test section performed in advance, it must be recognized that deviations greater than those measured in the test section may occur on some holes when the full project is undertaken. Using test section results to specify an acceptable maximum deviation, with the consequences of failing to meet that requirement falling solely on the contractor, is not appropriate unless the acceptable maximum deviation exceeds the measured results from the test section. Contract documents that contain deviation requirements that are not clearly achievable can result in extraordinarily high drilling prices or disputes regarding site conditions. It is appropriate to use the test section results to specify equipment, procedures, and initial alignment accuracy. Deviation surveys can be included to assess the actual results being achieved and to assist with determining the need for additional holes. In the relatively rare occasions when borings must be extraordinarily accurate, such as in the proximity of specific underground features, it will be necessary to use periodic DTH alignment surveys as a hole is drilled, and to use special drilling equipment such as wedging and directional drilling, to correct deviations as the hole is incrementally deepened.

(8) Deviation Surveys. When required, deviation surveys should be used as part of both test sections and production drilling activities. Borehole deviations can be measured by a number of different methods, each of which requires inserting some sort of instrument down the hole. Frequently, borehole deviation measurement devices are one of many probes comprising borehole geophysical equipment systems. Systems may be based on magnetometer devices, gyroscopic instruments, optical methods, and/or inclinometer systems. Some devices can be used within casings, and some cannot. Regardless of the method that is selected, it must be capable of providing both the azimuth of the deviation and the amount of deviation at all desired points within the accuracy desired. Measurement and payment for deviation surveys should be tailored to fit its intended use. If, for example, deviation surveys are to be performed only at the beginning of a project to determine that the boring means and methods are producing acceptable deviations, then it would be appropriate to measure and pay for the surveys on a per-foot basis. If, however, it is desired to have equipment and operators available at any time, the measurement and payment should include a monthly cost for maintaining equipment at the site and an appropriate unit measurement for its actual use.

c. Special Drilling Issues.

(1) Washing of Holes While Drilling. For any drilling method, a fundamental requirement is that drill cuttings must be completely removed by a method that will prevent them from entering the fractures that are to be grouted. For holes in fractured rock, the only acceptable flushing fluid is water. Air flush has been found to block fractures, which substantially reduces the effectiveness of grouting. During drilling, a flow of water must be maintained so that the upward velocity removes the drill cuttings and other materials that may be encountered. There is published data illustrating the combination of annular space and flow rate necessary to produce sufficient upward water velocity to remove cuttings of various types. For example, in a 3-in. hole, a flow of approximately 12 gpm will be required to produce adequate velocity. Excessive flows are to be avoided in geologic formations prone to erosion. Upon completion of the hole (or a portion of a hole), it is normal to increase the flow rate of the drill water and allow it to continue to flow while moving the drill tooling up and down at the bottom of the hole until no materials are being brought to the surface. The efficacy of the washing can easily be checked in the field by measuring the hole depth at completion of the washing with the drill and then rechecking after an hour or so. If there is only a minor difference in measurement,

then the washing is effective. If there is a significant difference, then washing at the completion of drilling is not effective, and either materials are settling out of suspension to the bottom of the hole, or caving in the hole is occurring after completion of drilling. If necessary, sampling of materials from the bottom of the hole can resolve whether the issue is one of inadequate washing or of caving materials. For payment purposes, washing that is performed while drilling and at the completion of drilling is normally treated as incidental to the drilling process.

(2) Water Losses and Water Gains. Water losses and water gains while drilling are two of the most critical events that occur during drilling of grout holes. Depending on the nature of the loss (or gain) and the nature of the geologic conditions, different actions are required. The largest number of water losses or water gains will occur on the primary series of holes, and the number of losses normally decreases as subsequent series of holes are drilled.

(a) Abrupt Water Loss in Rock Fractures. When 50% or more of the drilling fluid is abruptly lost, a critical feature requiring grouting has been encountered. Drilling should stop immediately at that point, and grouting of a special stage should be performed at that location before drilling proceeds. Continuing to drill after a sudden loss of the returning flush water creates a huge risk of blocking access to the fracture with cuttings, which will preclude effective grouting of the feature. Additionally, it creates the risk of having the drill tooling becoming stuck in the hole due to cuttings accumulating above the drill bit and around the rods. When there is any abrupt water loss, drilling should be terminated within a foot, and the feature should be grouted to refusal before continuing. The normal procedure is to set a single packer about 5 ft above the point at which the water loss occurred. The locations of water losses should be carefully recorded and plotted on the geologic profile, and the water loss occurrences should be analyzed as the work proceeds. If repeated losses are encountered, the grouting program functionally becomes a stage-down program.

(b) Gradual Water Loss. Inspectors must be alert for partial losses that accumulate gradually as the hole advances. Boring logs should contain an estimated percentage of fluid return as the hole is drilled. A gradual loss generally signals that losses are accumulating in multiple, fine fractures. When water return has decreased to the point that cuttings are not being freely removed from the hole, it is necessary to grout the leaky zone in one or more stages. If this condition is a frequent issue, the grouting program functionally becomes a stage-down program.

(c) Water Loss in Open Voids. When large voids are encountered, such as in karst environments, the technique of terminating drilling and grouting just beyond the point of water loss might or might not be effective. Multiple cycles of short drilling stages followed by grouting may be required. If this is not successful, drilling without return of fluid or installing temporary casing through the feature may be the only options. It is also common to permanently install a sleeve-port casing through these features; the casing must be of sufficient diameter to allow drill tooling to pass through it for continued drilling, and it can then later be used to grout the feature. Packers are seated at intervals within the sleeve-port casing, and grout is forced out of the sleeved ports into the zone. This technique also works well in badly weathered zones and at the soil-rock interface, where the casing may be installed in both soil and rock.

(d) Water Gains. An increase in the amount of flush water indicates artesian conditions. As with water losses, the drilling should be temporarily halted just beyond the location of the zone. If the artesian pressure is sufficient to create a surface discharge, either a standpipe or pressure gauge, as applicable for the situation, should be installed to measure the artesian head. Subsequently, the artesian feature should be grouted as a separate stage. There are typically three appropriate methods for grouting artesian zones. The first method requires the water-gaining stage to be grouted to absolute refusal. After the packer is deflated and removed, the hole should be visually monitored to determine if the water flow was stopped. The second method is similar to the first, but involves the addition of an accelerator in the grout to dramatically shorten the set time. The third method involves grouting the stage to absolute refusal, grouting the remaining stages above the artesian zone in an upstage method, and if necessary filling the remainder of the hole with grout to the ground surface. The intention is to maintain a head on the artesian zone by continuing grouting above it. Upon completion, the hole should be visually monitored. If the water continues to flow, a mechanical packer should be installed at the surface. It is recommended that inflatable packers not be left in the ground until the grout takes initial set, to avoid accumulation of grout in the packer and injection hose and the subsequent loss of the entire hole.

(3) Caving Conditions. Portions of a hole with caving conditions might or might not have a high permeability associated with them. Regardless, when the driller detects caving conditions, drilling should be terminated and the caving zone must be grouted to prevent losing tooling in the hole and to prevent gradual enlargement of the hole, which would prevent packers from sealing properly. In thick zones of caving material, it will be necessary to repeatedly interrupt the drilling and perform grouting in short downstages. Under extreme conditions, it may be necessary to grout through sleeve-port pipes installed entirely through collapsing zones. The sleeve-port method has replaced an older, problematic technique known as circuit grouting.

(4) Erodible Zones. Certain geologic formations may contain soft, erodible rock materials, unconsolidated materials, and/or soil-infilled fractures. The nature and location of these conditions must be discerned from observation of the drill behavior and the materials flushed from the hole. The features should be carefully logged and the information entered on the geologic profiles. The need for special treatment of these features, and for the specific protocols to be used must be assessed and developed in the field. In some cases, they will require grouting of special stages, while in other cases, it may be acceptable to continue drilling and deal with the features in later operations.

d. Inspection and Logging of Borings.

(1) Inspector Qualifications and Responsibilities. Inspection of drilling operations and collection of data from the borings should be performed by a qualified geologist or geotechnical engineer with prior experience in foundation grouting. One inspector is generally required for each drill crew. When multiple drill crews are operating within a short reach, one inspector can reasonably cover inspection of a maximum of two rigs by alternating between them at critical points in time and by maintaining constant radio contact with the drill operators. However, it should be expected that some logging needs must be sacrificed when one inspector is responsible for multiple drilling operations. The drilling inspector’s duties include: (1) verifying that a particular hole is eligible for drilling according to the sequencing rules, (2) verifying that the drill is on the correct hole, (3) checking the inclination and azimuth of the drill setup, (4) verifying that the drilling depths are being correctly measured, (5) observing the drill cuttings, (6) determining when drill water is lost or reduced to the extent that drilling should be stopped, (7) observing the

behavior of the drilling equipment, (8) checking water levels in the borings, (9) observing drill water connections to other borings or to surface discharge points, (10) making determinations as to when washing of holes after drilling has been satisfactorily completed, (11) keeping track of items that are to be measured for payment, and (12) preparing logs of the borings.

(2) Logging of Exploratory and Verification Borings. Exploratory and post-grouting verification borings are normally carefully sampled, cored, and tested. They should be treated as geotechnical investigation borings and, as such, should be carefully logged in accordance with all requirements and recommendations contained in EM 1110-1-1804, *Geotechnical Investigations*. Digital photos should be taken of all soil samples, all individual core runs, and the filled core boxes. When triple-tube coring is used, which is recommended for all grouting projects, the core runs should be photographed before removal from the split inner liner. Phenolphthalein should be used for detecting grout on fracture surfaces since recovery of relatively fresh grout can be very difficult. Logging of exploratory borings is particularly time consuming when done correctly, so one drilling inspector should be dedicated per drill rig.

(3) Logging of Production Borings. Production borings will, in general, be performed by destructive drilling techniques without recovery of samples. However, there will still be substantial data collected, and it should be recorded on standard log forms. If data are collected manually, the information to be recorded should include: (1) incremental penetration rates, (2) the nature and character of drill cuttings from the return flush, (3) any unusual drill behavior such as rod drops or extremely slow progress, (4) the estimated percentages of lost circulation, (5) the connections of drill flush to the surface or to other holes, (6) any zones of caving or difficult drilling, (7) the location of soil or highly weathered seams, (8) the water levels before the start of drilling for each shift, (9) and the time required for washing. If drills are equipped with devices to measure and record drilling parameters in real time, additional data such as torque, thrust, rotation speed, and rate of advance can be recorded to provide an indication of the relative drilling energy, which may be able to later be related to specific geologic features or conditions.

(4) Incorporation of Data into the Grouting Program. Collection of data, whether from exploratory or production holes, is only of value when the data are used effectively. During the planning of the field program, careful consideration should be given to the entire information management system, and to the effort required to execute it. At the completion of each boring, a final boring log should be prepared. The originals of the logs should be maintained in an organized filing system for each boring. Photos should be downloaded into the file for the boring. A copy of the completed boring log should be provided to the staff charged with updating project geologic drawings.

15-7. <u>Hole Washing After Drilling</u>. Drilling costs may be on the order of 50-70% of the total grouting program costs, and the sole purpose of the drilling is to provide access to fractures to be grouted. For that reason, every effort should be made to maximize potential access of grout to the fractures in the hole. Requiring that water be used as the only drilling fluid to remove cuttings is one part of that effort because it minimizes blocking of the fractures while drilling. However, the flushing fluid is laden with suspended solids of varying particle sizes, and all of the fractures are exposed to hydraulic heads that tend to drive those solids into those fractures. Accordingly, every hole should receive a separate fracture washing operation with clear water after drilling has been completed. It has become customary to use special washing tools that are easily fabricated. Specifically, the critical feature is to have drilled holes in the side of the washout bit so that washing water is directed in a radial direction to the sides of the hole. For a 3-in. hole, it is common to require a bit with a 2-in. diameter and closely spaced 1/8-in.-diameter holes, and a limited bottom discharge. Typically, it is required that the pumping capacity be sufficient to provide 100 psi in the system measured at the top of the hole. Washing should begin at the top of the hole and progress downward. Washing of any interval of the hole should be accomplished by raising and lowering the washing bit in short increments, and washing should continue until the flow is clean. This process is repeated to the bottom of the hole. If the wash water is not returning to the surface, the hole is usually washed for a minimum of 5–10 minutes. Washing is measured and paid for on a time basis, so the grouting inspector has as much latitude as desired regarding the duration of washing required. Chapter 9 of this manual includes photos of typical special hole washing equipment and washout bits.

15-8. Pressure Testing.

a. General. Pressure testing consists of isolating a segment of a hole by packers, injecting water at a known effective pressure, monitoring the behavior during injection, interpreting the results, and calculating a permeability value for that stage. The permeability unit that is normally used is the Lugeon value, which has been found to be very convenient and is in widespread use in grouting. The results of the test are, of course, affected by many factors, including whether the interval of hole being tested is representative of the overall rock mass

b. For the test results to reasonably reflect the overall characteristics of the rock mass, the holes must be oriented such that they intersect the fracture systems, a sufficient number of tests must be performed to ensure that the rock mass is reasonably characterized, and the stage lengths must be sufficiently short to ensure that tests are performed within zones of similar nature (i.e., similar fracture size and/or spacing). Grossly non-representative results and highly erroneous interpretations can occur if these basic requirements are not met.

c. Lugeon Unit.

(1) The Lugeon value for a test stage in a hole is calculated by:

Lugeon value = water take in L/m/min × (10 bars / effective test pressure in bars).

(2) In English units, the Lugeon value can be calculated by:

$$\text{Lugeon value} = (Q/L) \times (1801/P_{\text{eff}})$$

where:

Q = flow rate in gallons per minute

L = stage length in feet

1801 = conversion factor

P_{eff} = effective pressure applied to test stage in psi.

(3) The approximate conversion equations for Lugeon units to other common permeability units are:

$$1 \text{ Lugeon unit} = 1.3 \times 10^{-5} \text{ cm/s}$$

$$1 \text{ Lugeon unit} = 2.6 \times 10^{-5} \text{ ft/min.}$$

d. Exploratory Phase Testing. Much of the exploratory phase testing should have been performed as part of the design investigations, but it is quite common to continue or supplement that work early in the grouting project to both expand and refine the site characterization. Specifically, the goal of exploratory phase testing is to thoroughly define the site geology as it pertains to grouting. This includes having a thorough understanding of the geologic formations, the geologic structure, the fracture systems, and the permeability characteristics, particularly how the permeability varies by formation, depth, and horizontal location. Exploratory phase pressure testing should be performed on cored holes; it is also highly recommended to use borehole video on those holes.

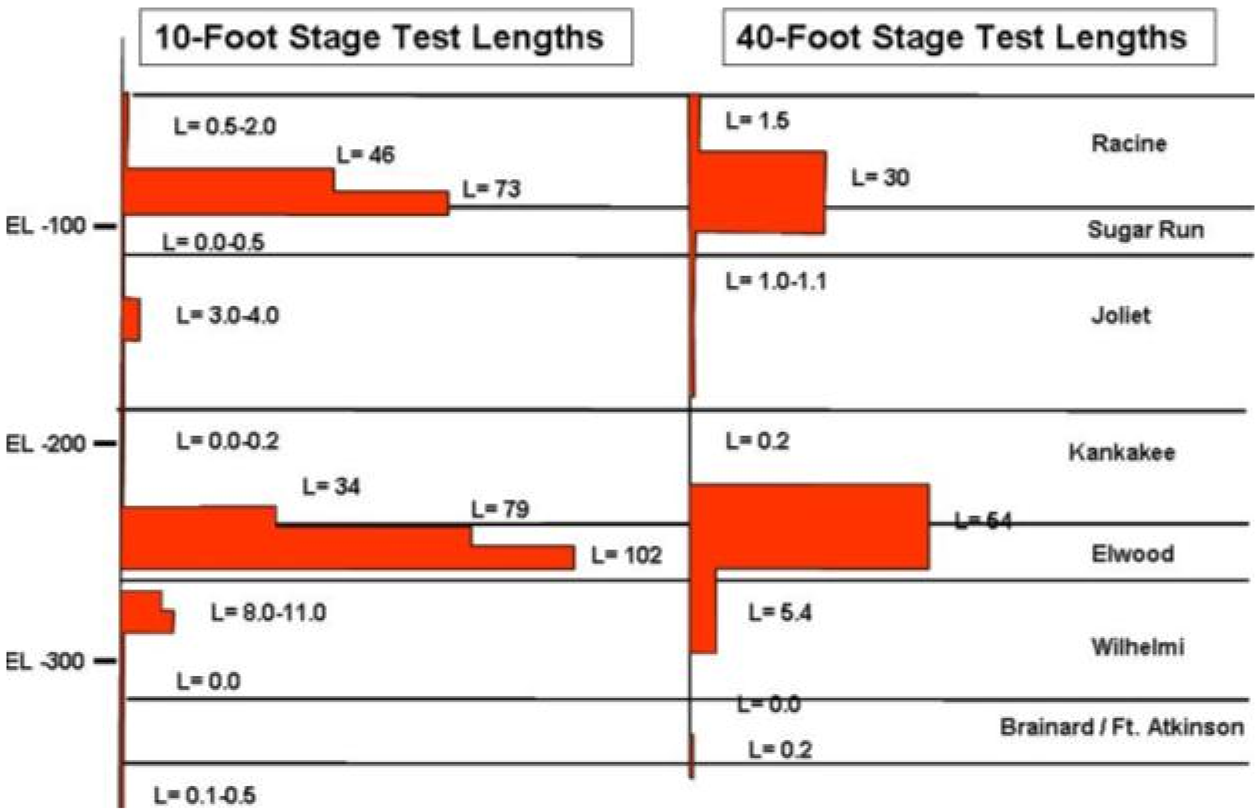
(1) Stage Lengths. Exploratory phase pressure testing should be performed in short stages to maximize the resolution of permeability variations. Long stages create an averaging effect that may lead to an erroneous understanding of actual conditions. Normally, the maximum stage length for pressure testing for this purpose should be 10 ft, and it may be of value to use even shorter stages for special conditions, particularly where geologic interfaces or other special features are of concern. Figure 15-5, which displays calculated permeabilities in the same hole for tests performed in 10-ft and 40-ft intervals, shows the effect that stage length has on making accurate determinations of permeability and on the understanding of the site conditions. Where a 40-ft interval was used, the permeability profile and the interpretation of the site conditions would differ substantially. The use of real-time monitoring systems to display pressure testing data will significantly reduce the duration of pressure tests since the behavior of the stage and the permeability can often be obtained with test durations of 2–5 minutes per stage. Under these circumstances, and assuming the pay item for pressure testing is measured as a unit of time, the use of short stage lengths can provide a particularly detailed permeability profile of the ground at reasonable cost.

(2) Stepped Pressure Tests.

(a) Stepped pressure tests are tests in which the pressure is incrementally increased and then decreased, usually in five discrete steps, with each pressure increment being a complete test. The results for each test are plotted, usually as a bar graph. The variation of the Lugeon value at different pressures provides much information about the nature of the fracture systems. Specifically, the tests can clearly disclose: (1) whether the fractures are clean, fine or coarse in nature, (2) the pressure required to lift the rock, dilate the fractures, and/or hydrofracture through unconsolidated materials, (3) the presence of infilling or weathered materials that are prone to being washed out, or (4) whether the fractures are prone to clogging. A sufficient number of stepped pressure tests should be performed so that the nature of each geologic unit is understood. The duration of testing for each step should be as required for the results to stabilize or, for non-stabilizing increments, at least 5 minutes. Figures 15-6 through 15-11 show schematic examples of ideal characteristic modes of behavior, along with their interpretation. With the exception of Figure 15-9, these schematics are based on illustrations by Houlsby (1990). Note that these examples are also helpful in interpreting grouting behavior when Apparent Lugeon values are being monitored on a real-time basis.

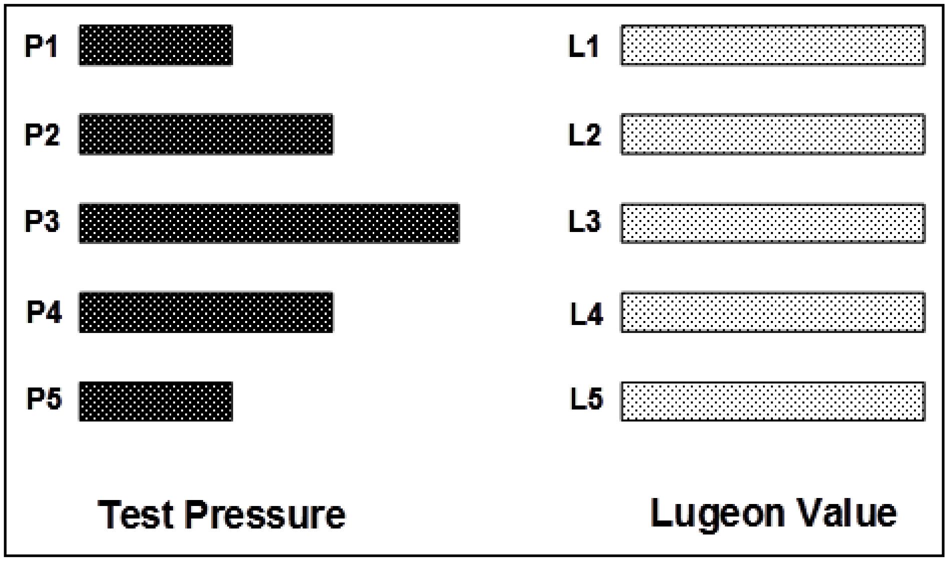
(3) When Lugeon values decrease with increasing pressure, but then increase to prior values as the pressure is subsequently reduced, it generally indicates the presence of larger fractures with turbulent flow, causing increased head loss in the fracture and reduced permeability at the higher pressures.

(4) When Lugeon values abruptly increase at a particular pressure increment, but then return to a constant lower value at lesser pressures, it indicates dilation of fractures followed by a return to the normal condition after reducing the pressure. It can also indicate reversible hydrofracturing through void infill materials or into embankment materials. During the exploratory phase, before production grouting is undertaken, the stepped pressure test can be intentionally structured to achieve these results so that the pressures at which dilation begins to occur is known. Normally, dilation is avoided in North American practice, and the dilation pressure can be used to help establish the "safe" pressures for production pressure testing and grouting.



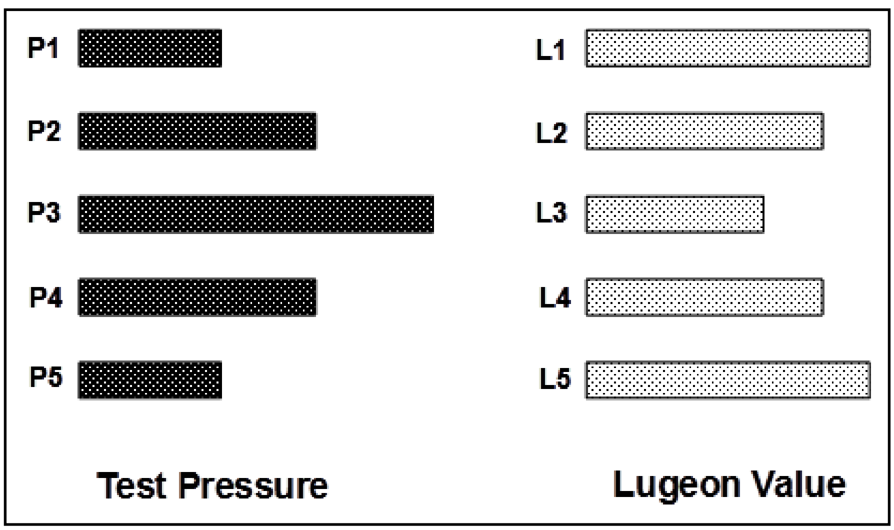
(Courtesy of Gannett Fleming.)

Figure 15-5. Effects of stage length on measured Lugeon values and interpretation of site conditions.



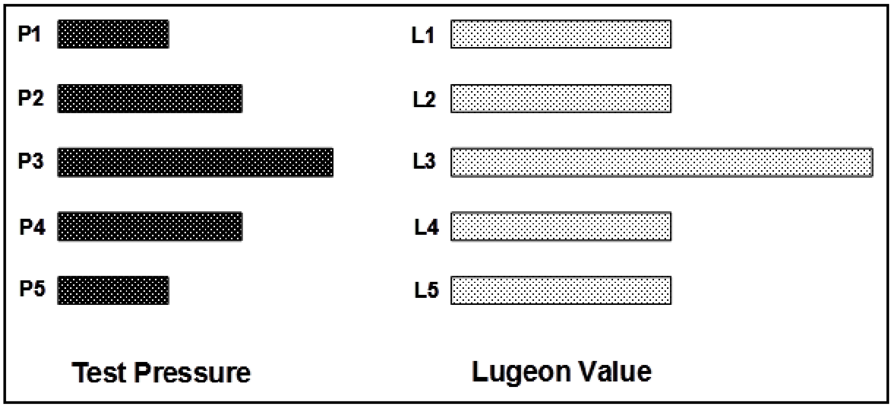
(After Houlsby 1990)

Figure 15-6. Laminar flow behavior. When the test section is composed of clean, finer fractures, there will normally be little variation of the Lugeon value with pressure because it is dominated by simple laminar flow. The permeability is constant within the range of pressures used.



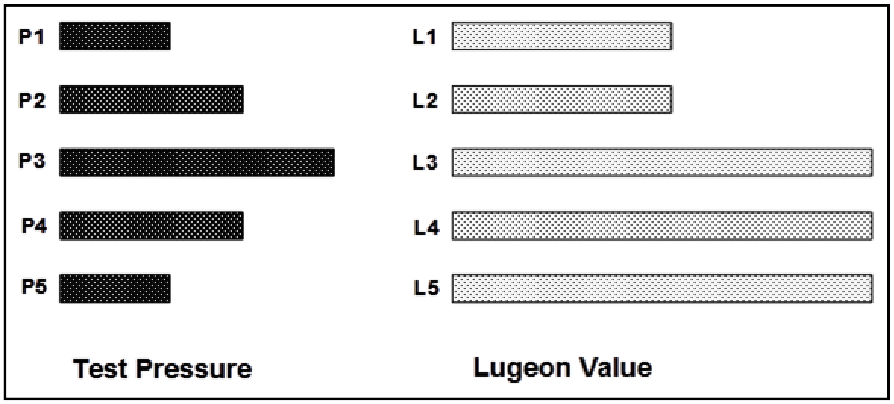
(After Houlsby 1990)

Figure 15-7. Turbulent flow behavior.



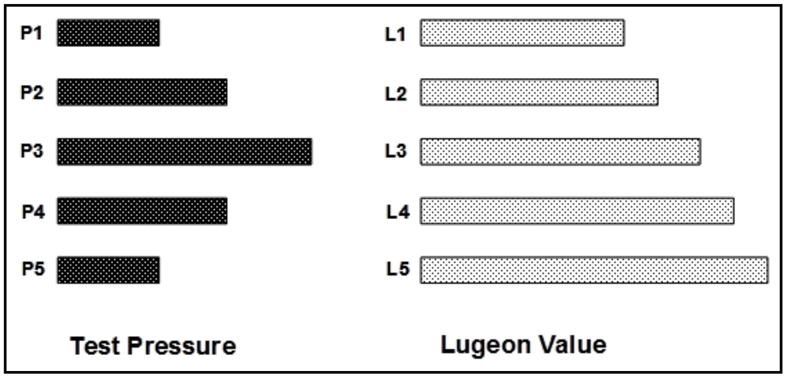
(After Houlsby 1990)

Figure 15-8. Dilation behavior.



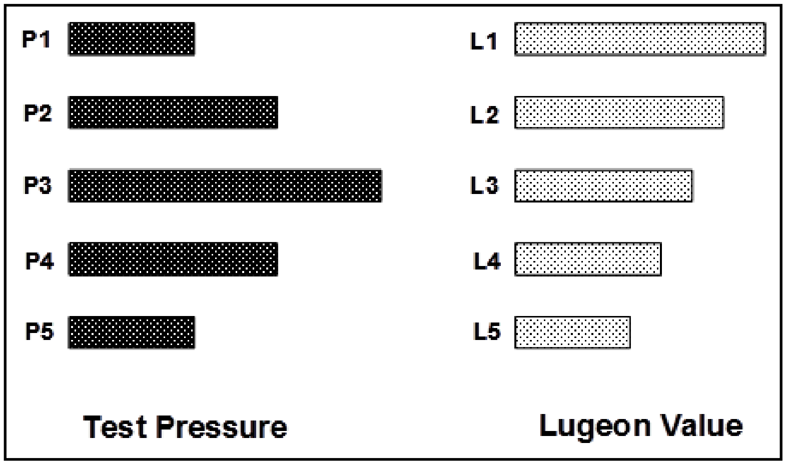
(After Houlsby 1990)

Figure 15-9. Permanent uplift behavior.



(After Houlsby 1990)

Figure 15-10. Washout behavior.



(After Houlsby 1990)

Figure 15-11. Clogging behavior.

(5) If the dilation results in "locking open" of the fracture, the permeability will stay at the increased level as pressure is reduced.

(6) When Lugeon values continually increase throughout the testing sequence, it suggests that washout of material is occurring with time.

(7) When Lugeon values continuously decrease through the testing sequence, it suggests that clogging of the fracture is occurring with time.

e. Production-Phase Pressure Testing. Pressure testing in the production phase of the grouting program serves numerous purposes, as described in the following paragraphs. In general, the stage length for production-phase testing should be the same as the length of stages to be grouted. The pressures for production-phase testing should, in general, be the pressures that will be used for grouting. These pressures should be selected with consideration of the maximum water pressure that the rock could experience in the design life of the project.

(1) Identification of Hole Connections and Surface Leaks in Advance of Grouting. Pressure testing will allow advance identification of connections of the stage to be grouted with other holes and with surface leakage points. This allows the contractor to assemble, in advance, the necessary equipment and personnel that will be needed to effectively respond to these conditions.

(2) Lubrication of Dry Fractures. Injection of water in advance of grouting will wet fractures located above the water table. Injecting into pre-wetted fractures is desirable to prevent rheology changes in the grout.

(3) Completion of Pre-Grouting Geologic Data. Production-phase tests on the first series of hole, in advance of any grouting operations, provide extremely good coverage of the virgin permeability conditions of the site. These data will confirm, refine, or alter the interpretation of site conditions that was developed in the exploratory phase. If the conditions are found to differ substantially, it may result in alteration of the grouting program.

(4) Guidance on Selection of Initial Grout Mix. In most cases, grouting is always started with the thinnest mix to be used for the project to ensure that the fine fractures are filled before the thickening required for larger fractures in the stage. However, using a thicker initial mix may be appropriate for some formations. Pressure testing in advance can provide guidance in those cases where variable starting mixes are used.

(5) Verification of Suitability of Initial Grout Mix. When balanced stable grouts are being used on a project, a calculated parameter called the Apparent Lugeon value is routinely monitored during injection of the grout. In simple terms, the Apparent Lugeon value is identical to the Lugeon value, except that it is calculated based on corrections for viscosity differences between grout and water. (See Chapter 5 of this manual.) Comparison of the water-test-determined Lugeon value with the grout-determined Apparent Lugeon value immediately after grouting has been started on a stage is an important indicator that the grout mix is penetrating fractures properly. When the two values are closely matched, it is clear that the grout is initially entering the fracture with nearly the same ease as water. When the two values are badly mismatched, it indicates that grout is not freely penetrating the fracture from the outset, and reconsideration of the mix constituents or proportions may be required. Without monitoring the relationship of these two values, there would be no way of knowing that the grout penetration may be inadequate until unsatisfactory end performance is discovered. This is one of the most important reasons for routinely conducting water tests on all stages in advance of grouting.

(6) Combining Stages for Grouting. Normally, grout stages in the primary and secondary series of holes are fixed in length, and each stage is grouted individually because the stages may contain fractures of different sizes. However, by the time the tertiary or quaternary series of holes are being grouted, there has normally been travel of grout into the regions around those holes, particularly in the larger fractures. Therefore, in general, the conditions will have become more homogenous, and open fractures will tend to consist of smaller fractures that will be grouted to completion using only the thinnest project mixes. It is not uncommon to find completely tight stages at that point. Pressure testing of each stage provides a sound basis for logical decisions about combining stages for grouting in these final series of holes. When multiple stages are found to have essentially the same Lugeon value, it is acceptable to combine two or more of them, which may result in considerable economy in the grouting operation. The number of stages to be combined must consider the impacts of pressure variations that will occur from a longer, combined stage. In general, it is recommended that combined stages should not exceed 40 ft in length. Unless pressure testing has verified the suitability of combining stages, it should not be performed at all.

(7) Closure Analysis and Program Verification. The fundamental goal of any curtain grouting program for a hydraulic structure is to produce a grouted zone of the desired width and residual permeability. When those results are achieved with certainty, satisfactory closure has been achieved. The process by which it is determined that closure has been achieved is termed "closure analysis." Closure analysis involves intense analysis and scrutiny of the data, and an extrapolation process. Verification holes are drilled and tested after closure is deemed to have occurred to test the closure decision and extrapolations. Much emphasis is placed on performing very high-quality water pressure tests to calculate the permeability in every stage of every hole series and then using those data as the principal method to determine whether the grouting has, in fact, met the design residual permeability. It is logical to place great reliance on the pressure test

data because the purpose of the grouting is to act as a barrier to water. Closure analyses are discussed at length in Section 15-14.

f. Pressure Washing. Pressure washing is a special operation, but it is included under pressure testing for three reasons: (1) the need for pressure washing usually becomes evident while performing pressure testing, (2) the same equipment is normally used, and (3) payment for pressure washing is usually made under the pressure testing payment item. Pressure washing should only be conducted under the guidance of a qualified person.

(1) Determination of Need. Pressure washing is normally performed when a pressure test indicates that there is a connection from the stage being tested to one or more other holes and when the discharge is muddy, indicating an erodible seam of material. When that occurs, the pressure testing operation is extended as necessary in an attempt to remove the materials from the fracture. Under ideal conditions, the operation continues until the flow from the discharge points is clear.

(2) Efficacy of Pressure Washing. In general, the efficacy of pressure washing has been found to be low. Removal of material will cease when the flow velocity in the fracture, which is controlled by the rate of water injection, is insufficient to carry the material to the surface. Unless extraordinary measures are taken, it is more likely that a somewhat limited "rat hole" erosion will be the result. The visual results may be deceptive because even a small amount of suspended solids in the wash water will give the appearance of effectiveness, and the flow will eventually become relatively clear as the limits of erosion are reached. When it is absolutely critical to remove materials from a seam, the following may be required: closely spaced holes (i.e., 1.25- to 2.5-ft centers), the use of high-capacity pumps to increase the rate of water injection, and periodic injections of air with the water to maximize the agitation. Water pressures used during pressure washing must be controlled to prevent hydrofracture or erosion damage to the soil-rock interface.

(3) Alternatives. Effective treatment of erodible seams is critical for the long-term performance of the grouting. While pressure washing should always be included in the treatment program, other alternatives are also available.

(a) Excavation of Seams. Whenever possible, the excavations to the surface from which grouting is to be performed should extend below the depth of erodible seams. While designers are often tempted to limit excavation based on a perceived cost savings, this is normally a false economy compared to the potential costs of unsatisfactory long-term performance.

(b) Intensive Grouting. Even though the majority of problems can be prevented by additional excavation, there may still be seams that cannot be reached by that measure. Pressure washing, though known to be problematic, should still be conducted as effectively as possible. This measure should, however, be accompanied by intensive grouting of the seam without relaxation of refusal pressures or criteria. This may require many more holes and may result in repeated hydrofracturing of the seam, making one think that normal refusal will not eventually be met. However, the repeated hydrofracturing, coupled with closely spaced holes, will result in confinement of the seam and/or mechanical and chemical alteration of the materials to the extent that it will eventually be grouted to the same criteria as normal fractures. Due to the potential for structural damage, hydrofracturing shall not be considered appropriate for remedial grouting in existing dam or levee foundations unless measures have been taken to provide a very high level of confidence that the seams that are being hydrofractured are completely isolated from the foundation and embankment soils.

15-9. Injection Pressures.

a. General. The radius of spread of grout is controlled by the fracture size, the cohesion (actually Bingham yield point) of the grout, and the injection pressure. The rate of flow of grout into the fracture is controlled by the size of the fracture, the viscosity of the grout, and the injection pressure. Both the radius of spread and the rate of flow are directly proportional to the injection pressure, so higher grouting pressures are beneficial in increasing the extent of the zone of treatment and in decreasing the time to accomplish that treatment. The only issue, therefore, is establishing safe grouting pressures to be used in treatment. Other activities, such as pressure washing and water pressure testing, have the potential to damage the embankment by using pressures that are too high and should always be performed at pressures less than the established safe grouting pressure.

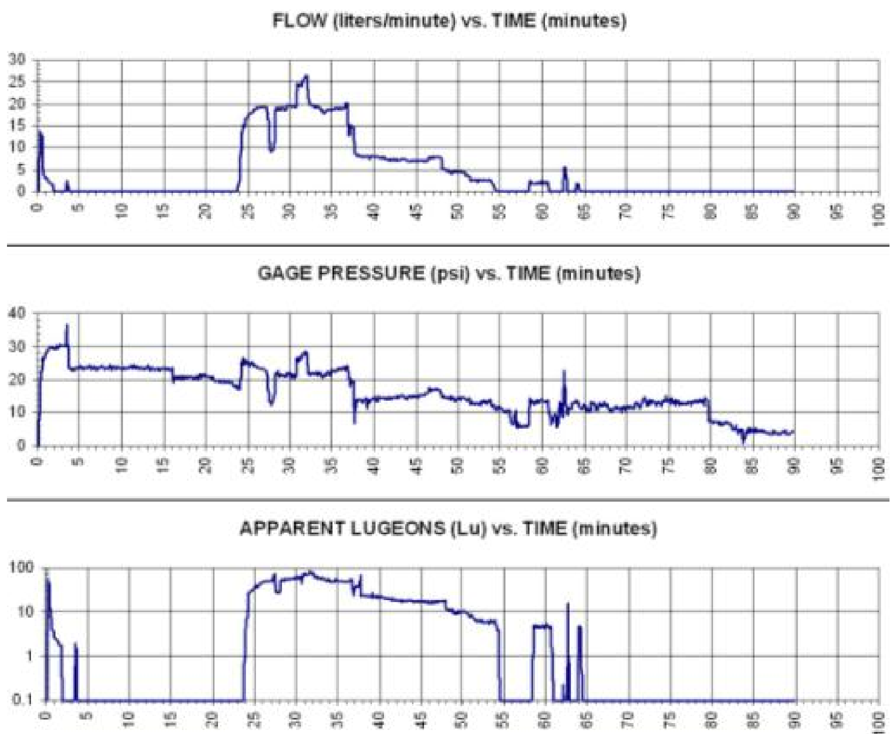
b. Uplift vs. Dilation. Safe injection pressures for rock are pressures that will not lift or separate fractures to the extent that they are irreversibly locked into a more open position. This “locking open” of fractures is termed “uplift,” although it could conceivably occur on fracture planes other than the nearly horizontal planes that are normally visualized. Dilation is similar, but is reversible. Dilation might or might not occur as a first indication of uplift.

(1) Uplift. Uplift is to be carefully avoided, both during water pressure testing and during grouting. By definition, it permanently increases the permeability of the rock mass by resulting in an irreversible movement of rock. If it occurs during water pressure testing, as a minimum it will require repair by more extensive grouting. Depending on the horizontal and vertical extent of the rock movements, which will generally not be known except by installing numerous check holes, it may damage already completed portions of the work. Similarly, uplift during grouting may damage adjacent areas of work already completed and will require extensive effort to determine the extent of damage and the amount of regrouting necessary to repair the damage. It is easier to cause uplift during injection of water than during grouting because water injection can generate larger forces at an equivalent pressure. One of the greatest concerns in grouting is causing uplift that is undetected and therefore not repaired, resulting in unsatisfactory performance.

(2) Dilation. Dilation is the temporary and reversible opening of fractures by fluid forces. Dilation can be the first stage of uplift developing, but it can also be a separate phenomenon representing elastic deformation of the rock mass. The fractured and ungrouted rock mass has a bulk modulus of elasticity affected, in part, by the presence and nature of the fracture system. It is possible to cause detectable and recoverable deformation of that rock mass by applying pressure. In European practice, many consider dilation to be highly desirable because it can improve penetration of grout into very fine fractures, resulting in better fracture filling when the pressures are reduced and the rock closes on the grout that has been placed. The most serious issues are the practicality of monitoring dilation effectively and ability to absolutely differentiate it from uplift. United States practice has, in general, not embraced the concept of intentional

dilation of fractures. Even if the deformations are recoverable, they may still damage previously grouted sections and potentially any overlying existing structures. Dilation may have valid application in specific situations, such as for formations containing numerous, but very fine, fractures, which preclude entry of normal grout mixes, requiring the use of low viscosity, but more expensive solution grouts or extraordinary long amounts of time at very slow injection rates. However, dilation should only be undertaken on a production basis after being carefully evaluated and proven in a full-scale, highly controlled test section.

(3) Monitoring Uplift and Dilation. Until recently, monitoring the occurrence of dilation and uplift has been neither practical nor effective. Surface monitoring for uplift is cumbersome, time consuming, and, in general, insufficiently accurate and/or timely to be of much practical value. Detection of dilation has not normally even been considered. Currently, with the advent of extremely accurate pressure transducers and flow meters, balanced stable grouts, and computer monitoring, both dilation and uplift are easily detected on each stage on a real-time basis within seconds of initiation. Figure 15-12 shows an example of a stage record that exhibits the opening of a fracture under pressure followed by closing of the fracture after the pressure is reduced. The dilation, which in this case was interpreted as the beginning of uplift (as opposed to elastic dilation within the rock mass), occurred 25 minutes into the grouting under a relatively minor pressure increase. As the pressure was gradually reduced, the dilation gradually closed, followed by abrupt closure at about 55 minutes.



(Courtesy of Gannett Fleming.)

Figure 15-12. Grouting record with dilation that closed on reduction of pressure.

c. Effective Pressure. Regardless of how safe injection pressures are determined for a project, it is important to accurately know the actual pressure that is being applied to a water pressure testing or grouting stage. For that reason, the effective pressure that is being applied should be calculated for each stage. In the past, it has been common to simplify the calculations and not account for every factor that enters into the calculation. The widespread use of computers in grouting projects now makes that simplification unnecessary, and it is recommended that calculations include correction for all factors to bring the accuracy into the same range as the accuracy of other operations. The effective pressure is usually calculated at the midpoint of the stage, but for long stages it may be necessary to calculate and check the effective pressure being applied at both the top and bottom of the stage. The effective pressure applied at any point in the hole at any given time is calculated as:

Effective Pressure = Transducer or Gauge Pressure + Static Head of Grout or Water
– Static Head of Groundwater – Dynamic Losses

and is illustrated in Figure 15-13. When very low pressures are required, it is common to use gravity grouting from the agitator without applying any pump pressure. In that condition, the effective pressure is calculated as:

Effective Pressure = Static Head of Grout + Agitator Head
– Static Head of Groundwater – Dynamic Losses.

If this pressure exceeds the maximum allowable pressure for the stage, then an agitator must be provided at approximately the elevation of the ground surface at the collar of the grout hole. For additional information on calculating effective pressures, see Appendix B.

(1) Static Head of Grout. The static head of grout is the pressure applied by the column of grout in the hole measured from the elevation of the pressure gauge to the elevation of the point of interest. The static head of grout varies based on the density of the grout mix. One foot of water head is equivalent to 0.433 psi (the density of water is 62.4 lb/ft^3, divided by 144 sq in./ft^2, yields 0.433 psi/ft of head). The static head of grout equals 0.433 psi/ft times the elevation difference between the pressure gauge and point of interest times the specific gravity of the grout mix. In the case of a water pressure test when water is the injected fluid, the specific gravity equals 1.

(2) Groundwater Head. When the groundwater level is above the point of interest, it applies a pressure that must be accounted for in the calculation. In this circumstance, the groundwater head is calculated as the pressure created by the difference in elevation between the groundwater level and the point of interest times 0.433 psi/ft. Groundwater levels across the site are normally known from the exploration phase work, but the levels should be verified or checked before a water testing or grouting operation is performed. When groundwater levels are located at or below the point of interest, then the groundwater head is zero.

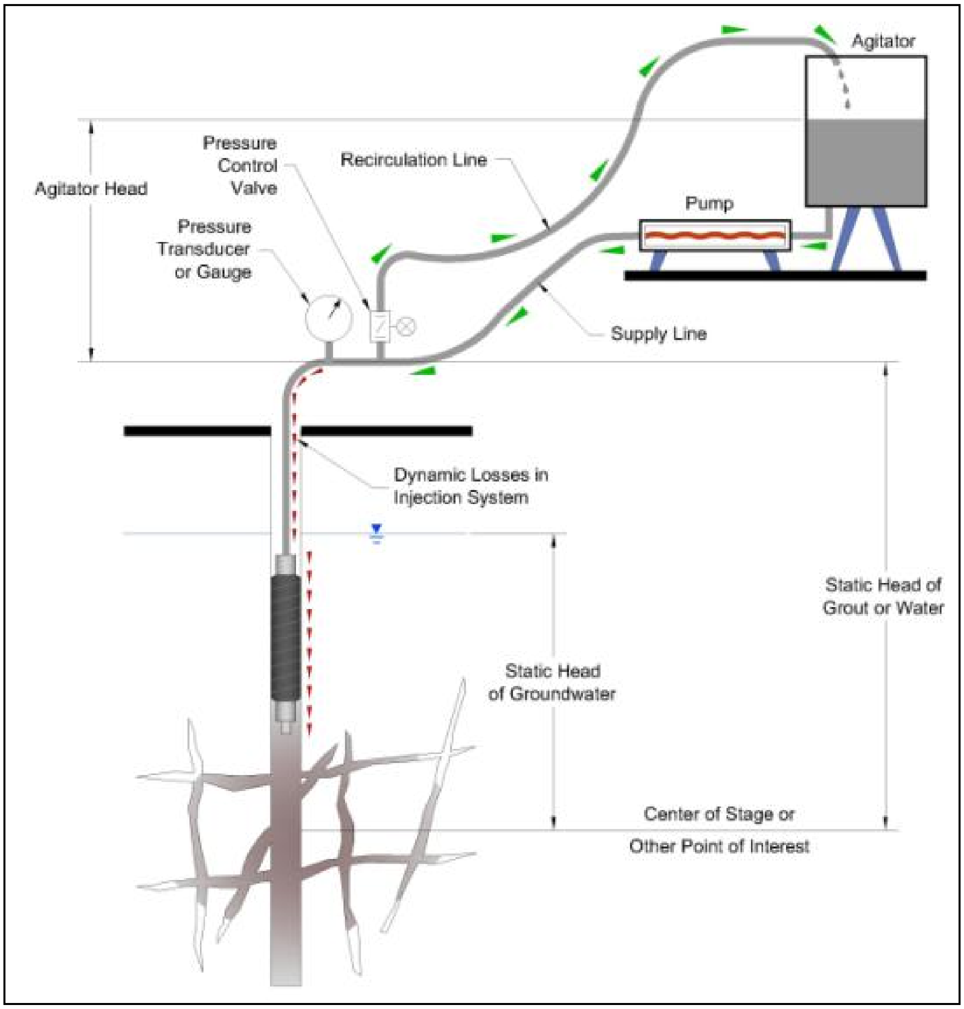
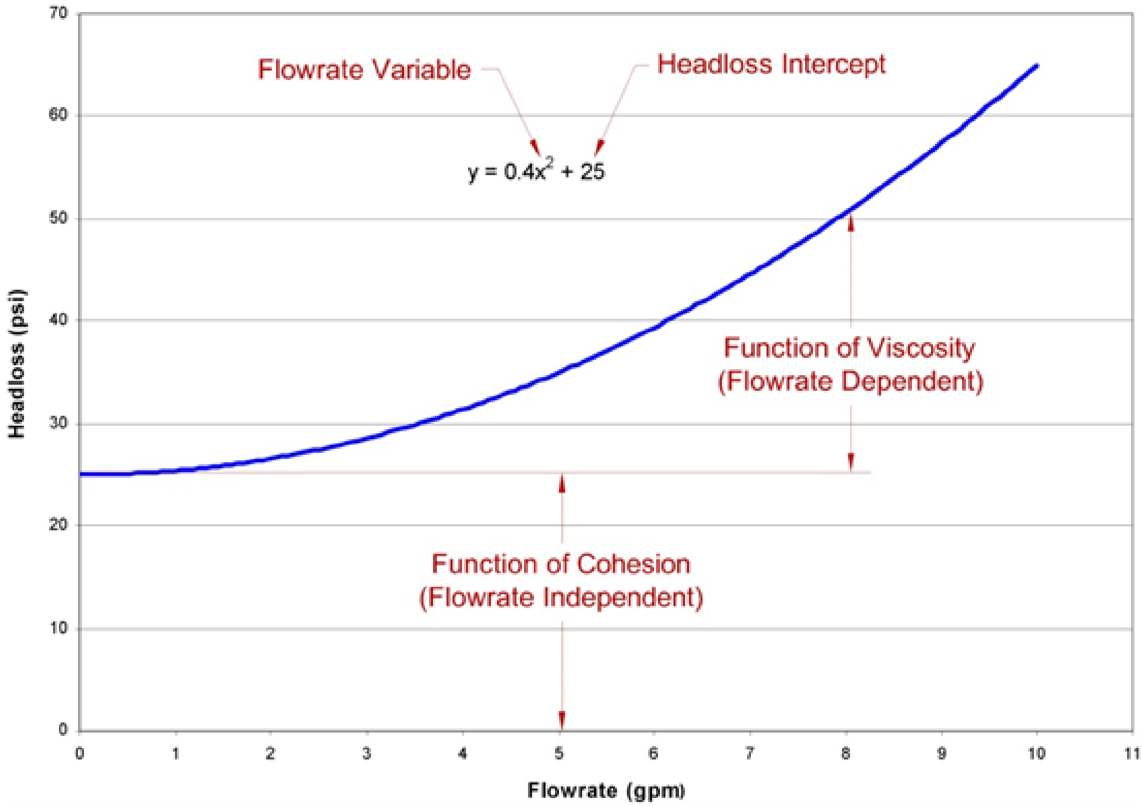


Figure 15-13. Schematic illustrating effective pressure calculation. (Flow meter not shown; see Fig. 10-16 for configuration.)

(3) Dynamic Head Losses. When fully automated monitoring is being used, it is normal to also include a correction for dynamic head losses. The dynamic head losses are the pressure losses that occur in the system and that vary according to the characteristics of the fluid being pumped (cohesion and viscosity) and the rate of pumping. Including dynamic losses is important for the water pressure tests because high rates of pumping may be involved. For grouting, the dynamic head loss may be very high at high pumping rates, depending on the viscosity of the grout. Thicker mixes will generate higher head losses. On projects where multiple grout mixes are used in a stage, there is a transition period where there will be some of two mixes in the grout lines and the correction for dynamic head loss may be somewhat inaccurate. Near refusal, the cohesion plays a more important role, since the amount of pressure required to initiate movement of the fluid through the system is cohesion controlled. This initiating pressure is often referred to as the "head-loss intercept." The head-loss intercept for water is zero. Including the dynamic head losses for grout is significant in assessing the initial grouting behavior of the hole. Dynamic losses, whether for water or for various grout mixes, are determined by fully

assembling the equipment, pumping the fluid through the system with a free discharge condition, and developing curves showing flow rate versus pressure for each fluid. Figure 15-14 shows an example of a dynamic loss curve for one specific grout mixture.



(Courtesy of Gannett Fleming)

Figure 15-14. Illustration of grout head-loss graph.

(1) Direct Measurement. For projects where an accurate measurement of the effective pressure is desirable, such as dam remediation, it is possible to measure the pressure directly in the stage being grouted. During remedial grouting for Wolf Creek Dam in 2012, a down-the-hole pressure sensor with a gauge saver was incorporated into the packer assembly and provided real-time measurement of the pressures at the top of the grout stage. The measurement assembly proved to be reliable and functioned for over 1000 stages. This method measures the actual total pressure and eliminates the need for estimating dynamic losses. The inaccuracies associated with the transitions between different grout mixes are also eliminated. The effective pressure can be determined by subtracting the measured groundwater pressure before grouting from the total pressures measured during the grouting. When using direct measurement, it is recommended to also estimate pressures using the conventional methods for redundancy.

d. Rules of Thumb. “Rules of thumb” have been established for safe grouting pressures in soil and rock. Weaver (2000) presents a good summary of the rules of thumb. On recent USACE projects, maximum safe grouting pressures have been established as 0.5 psi/ft for the overburden soil thickness and 1 psi/ft for depth into rock. These rules of thumb were developed based on experience. The weight of the material over the zone being grouted was the primary consideration when developing the rules of thumb (USACE 1984). If these guidelines are followed, then the pressure applied to the grouting stage will be less than the weight of the overlying materials, thus preventing lifting or “heave.” An additional margin of safety is afforded by the strength of the rock. However, factors such as depth to the water table, sloping ground conditions, discontinuities that cause lower in-situ ground stress, and defect connections to the soil were not considered. Blindly using rules of thumb can be dangerous, especially when grouting through existing embankment dams where defects in the rock are connected to the foundation soils and low confining stress conditions are present. A review of numerous past grouting projects performed by USACE and the Bureau of Reclamation (Patoka, Mississinewa, East Branch, Wolf Creek, Hop Brook, John Martin, Norfolk, Allatoona, Hartwell, Oologah, Alvin Bush, Abiquiu, Efaula, Dworshak, Libby, Clarence Cannon, Longview, Morrow Point, Flaming Gorge, Hoover, Heron, Kortes, Hungry Horse) indicates that these rules of thumb have not been applied consistently and were often misunderstood. The grouting pressures ranged from 0.5 to 2.0 psi/ft of depth. On some projects, the guidelines were interpreted to be the required pressure rather than the maximum. On other projects, only the gauge pressure at the top of the hole was used, with no regard for the additional pressure from the static head of the grout column. Sherard (1973) documents numerous cases where dams were damaged from drilling or grouting operations. USACE grouting projects with incidents of damage or hydrofracture include Red Rock Dam, Kentucky Lock, Patoka Saddle Dam, Center Hill Dam, Wolf Creek Dam, Addicks Dam, and Barker Dam.

(1) Safe Pressures for Soils. Grouting pressures should be maintained lower (with a margin of safety) than the pressure that could cause unwanted damage or hydrofracture. There are many theories and models proposed in the literature to estimate borehole fracture pressure in soils and rocks. For soils, a review of measured data in the published literature from field and laboratory studies indicates that the borehole fracture pressure (P_f) can be approximated by the sum of the minor principal total stress (σ_3) plus the undrained strength.

(2) This seems relatively simple, but there is a lot of uncertainty involved in estimating the confining pressures under dams. The minor principal stress is a function of the geometry (dam slope), abutment configurations, the smoothness or irregularity of the soil-rock interface, the differential settlement, the soil density, the overconsolidation ratio, the pore pressures, the soil strength, k_0, Poisson’s ratio, etc. Many of the geometric features of a dam, such as embedded conduits and concrete walls, result in confining stresses that are typically much lower than equivalent depths in flat ground conditions. Numerical stress analysis using finite element or finite difference software can be performed to parametrically evaluate the range of confining stresses that may be present under a dam or other structure. Some simplifying assumptions will need to be made, and ranges of parameters that are difficult to measure (such as Poisson’s ratio and k_0) will need to be estimated.

(3) When grouting through existing dams and levees, the following practices are recommended:

(a) Avoid grouting through dam or levee cores when possible.

(b) Consider doing a numerical stress analysis when proposing grouting through or near areas that are known to create low confining stress.

(c) Only use gravity grouting in the soil or soil-rock interface.

(d) Isolate embankment and foundation soil with protective casing and grout bags.

(e) Backfill borings, casing, or piezometers only with gravity grouting.

(f) Proceed with single-stage backfilling unless the groundwater table and the undrained strength are both low.

(g) Carefully monitor backfill grout. If significant losses occur, use multistage backfilling with an accelerator to minimize production impacts.

(h) Perform downstage grouting using only gravity through the interface zone.

(i) Evaluate the geology carefully. Use a borehole televiewer to determine the extent and continuity of rock fractures.

(j) Use this information to determine the depth of gravity downstages.

(k) Only apply pressure above gravity when the foundation soil is reliably isolated.

(l) Limit the applied pressures. There is rarely a need to exceed pressures equivalent to the water to the top of dam.

(m) Use real-time measurement of grouting parameters, including DTH pressures in the stage being grouted.

(n) Address allowable grouting pressures, hydrofracture potential, and grouting procedures in the Drilling Program Plan.

(4) Safe Pressures for Rock. In situations where there is no overburden or existing structure over the rock being grouted, or where the rock has been reliably isolated from the soil, the 1.0-psi-per-foot-of-depth rule of thumb is likely a safe guideline to use for most intact rocks. Massive formations provide few avenues for rock block movement due to minimal fracture frequency. Highly fractured and weak rock obviously warrants more conservative pressures. For conditions where the risk of damage is minimal and the rock is massive, higher effective injection pressures can be used with confidence. Trial values for bedrock can be established at the higher end of published guidance (i.e., in the range of 1.5–3.0 psi/ft, depending on the characteristics of the rock), and the suitability of those values can be evaluated on every stage. It is recommended that caution be used in grouting the uppermost stage and that pressures be limited to approximately 1.0 psi/ft in that zone.

(5) Advance Evaluation Programs. If a full-scale test section is performed as part of the design process, the program should include testing to determine the actual limiting pressures for dilation, uplift, and/or hydrofracturing, which then can be used to establish safe injection pressures for production work. The advantage of this approach is that, if high pressures are found to be workable, it presents the opportunity to either increase the hole spacing or possibly terminate some series of holes at reduced depths.

15-10. Grouting Operations.

a. Starting Mixes. The starting mix depends on the geology and the objective of the project. Grouting of each stage should, in general, be started with the thinnest mixes that are being used on the project. They are the appropriate mixes for the finer fractures, and it should always be assumed that fine fractures exist within the stage being grouted. Assuming pressure can be built up in the hole, using the thinnest mix allows grout to be injected into these fine fractures for a period of time before considering thickening of the grout in response to the presence of coarser fractures.

(1) Unstable Grout Mixes. Starting mixes for unstable grout mixes, composed of neat cement grouts or grouts that have additives, but are not stable throughout all mix consistencies, should have a volumetric water/cement ratio of 2:1 or 3:1, with the choice being based on early experience at the site. If a 2:1 mix is readily accepted, then it is suitable as a starting mix. If trial injections of a 2:1 mix result in refusal in 30 minutes or less, then typically a 3:1 mix should be used as a starting mix. Historically, starting grout mixes as thin as 5:1 have been used successfully in certain geologic conditions.

(2) Balanced Stable Grouts. There is no standard convention for designating balanced stable grouts at this time. A suite of mixes will be developed that cover a range of marsh funnel flow times. Normally, the thinnest mix has a 1-L marsh flow time of 35–40 seconds. This is the mix that should be used as the starting mix in all stages. Water, by comparison, has a marsh flow time of 28 seconds.

b. Management of Grout Injection.

(1) Initiation of Grout Injection. After the grout has been mixed and is in the agitator, it must be circulated through the entire system to ensure complete filling of the lines and to allow checking of the system for leaks or other problems. After circulation is established, the control valve on the header is gradually opened and adjusted until the pressure value is stable. Operators should check for surface leaks and connections to other holes and detect any other problems.

(2) Observation of Injection Behavior and Typical Conditions and Responses. The goal of grouting, under ideal conditions, is to inject the grout smoothly and continuously. The reality of grouting is that the time it takes to reach refusal is highly variable. The grouting is actively managed by the geologist or engineer in control of the injection operation based on a review and interpretation of data being obtained and plotted while injection is in progress. Grouting of the stage is actively managed by changing mixes and/or other factors to produce the desired result. The information gathered during drilling and pressure testing of the stage should be used in the decisionmaking process while managing the injection. The following paragraphs summarize typical behaviors commonly observed while grouting and typical responses to those behaviors.

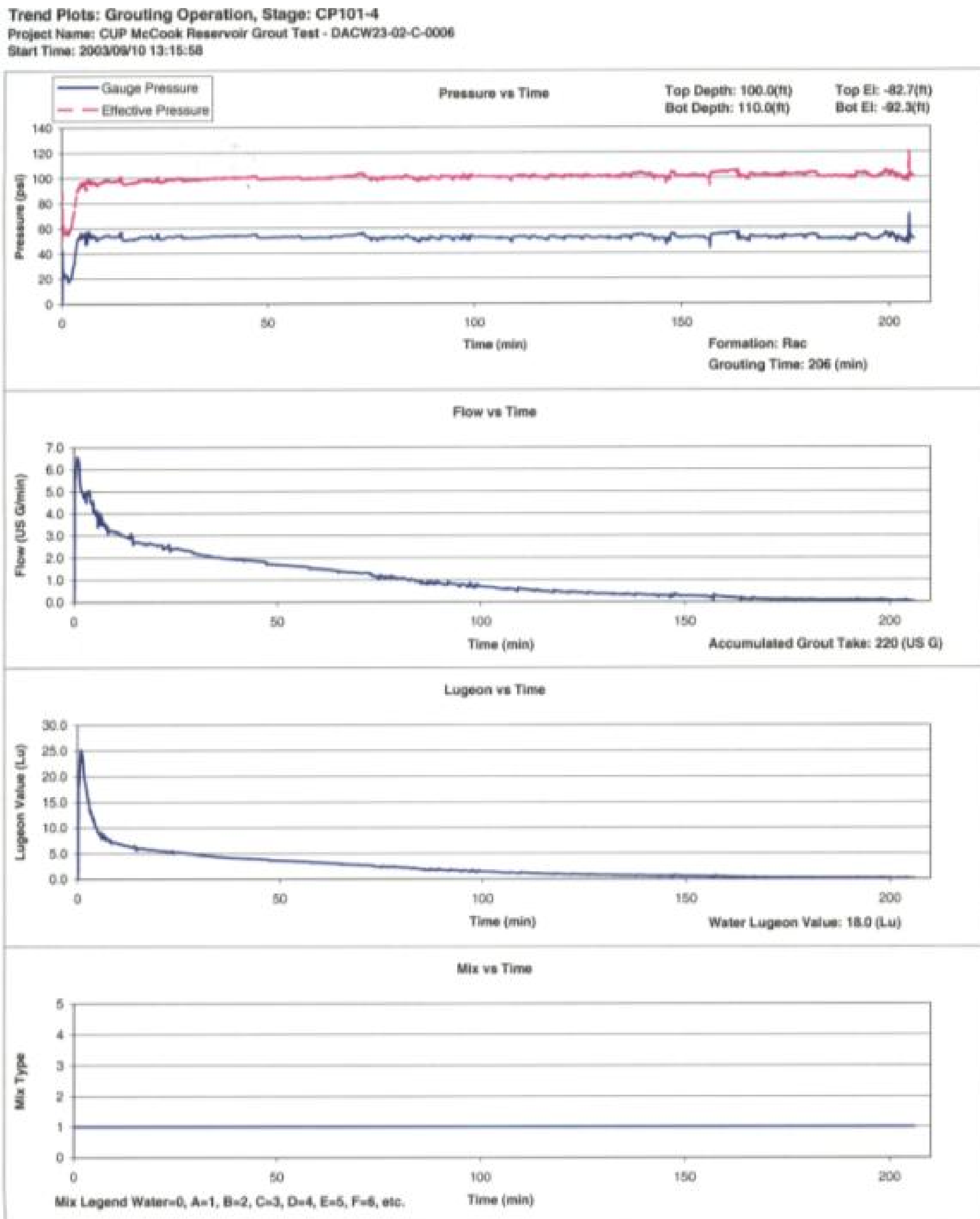
(a) Gradual Decline in Rate of Injection. When the rate of grout injection is declining, however slowly, it is normal to continue injection with the same mix since the stage is moving in the direction of refusal. Thickening of the mix under this condition may result in rapid and premature termination of grouting without filling of the fractures. A similar interpretation —that grouting is proceeding satisfactorily under the current mix— applies when grout is being injected at the maximum rate and the pressure is gradually increasing toward the maximum target pressure.

(b) Constant Rate of Injection. When the rate of grout injection continues for an extended period of time without decreasing, it generally indicates the grout mix must be thickened. The point at which the decision to thicken is made will vary, depending on the rate of grout take. In a stage with a slow rate of injection, it may be appropriate to inject for an hour or more before changing mixes. In a stage with a relatively high rate of injection, the decision to thicken the mix may be made after three or four batches of the initial mix are injected.

(c) Sudden Decrease in Take. A sudden decrease in take typically indicates either that the grout was too thick initially (if the sudden decrease occurred with the initial mix) or that thickening of the mix was not appropriate for the fractures. There are two possible remedies for this event: an attempt can be made to inject water to revive the stage, which requires wasting of grout in the agitators and lines and switching to water injection; or, as an alternative, noting the location and adding extra holes on either side to remedy the zone where premature termination occurred. Geological factors causing a sudden decrease could include displacement of water from a void through fine fractures; the sudden decrease in take would indicate that the void is completely filled with grout, and subsequent injection at the reduced rate represents grout permeation of the fine fractures.

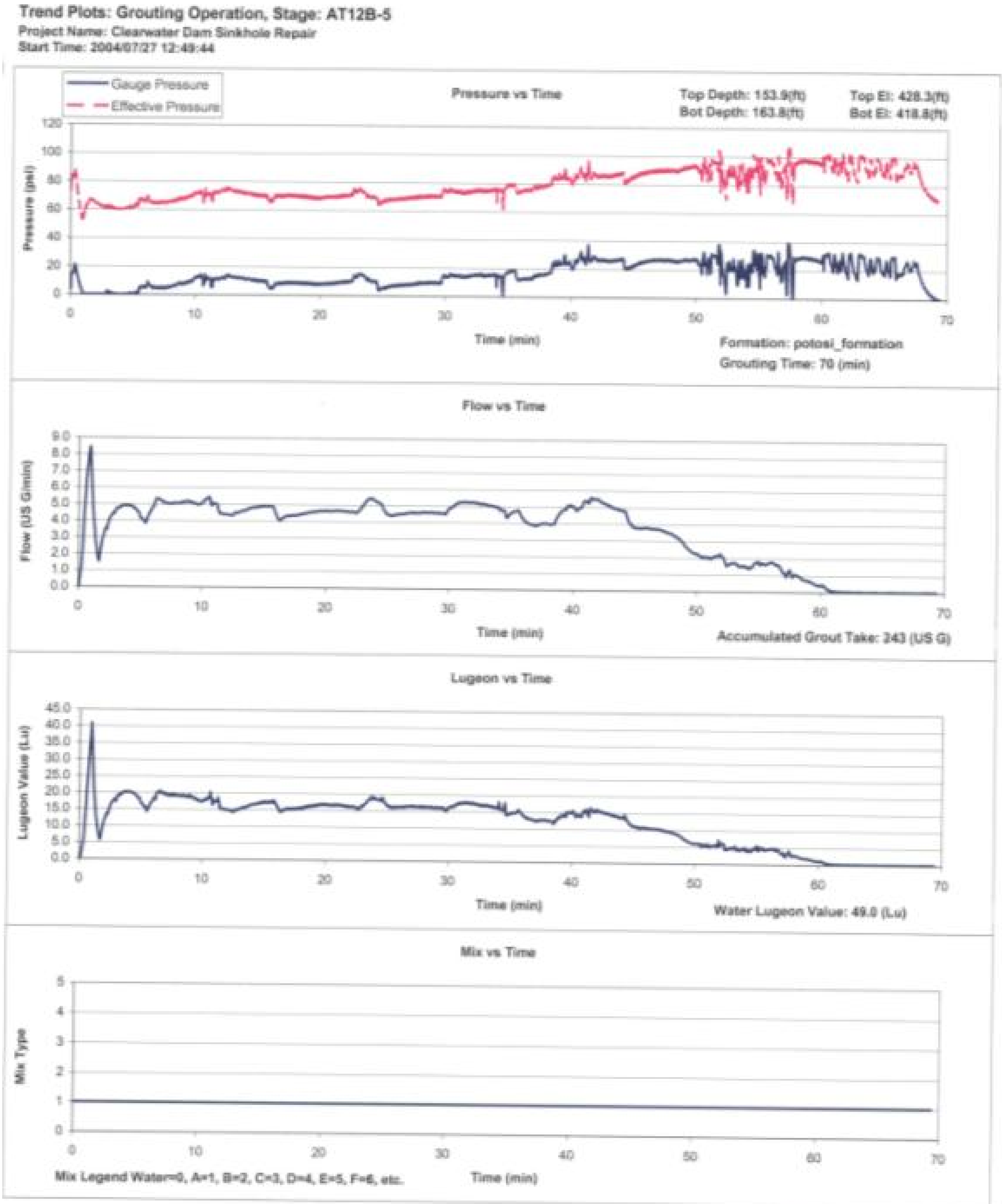
(d) Sudden Increase in Take. A sudden increase in take could indicate one of the following types of conditions: (1) dilation or uplift of the rock, (2) breakout of surface leaks or connections to other holes, (3) or hydrofracturing through infilled materials. Assessment of the most likely event and determination of the appropriate response are made based on knowledge of the conditions of the zone being grouted. For example, the proper response for suspected uplift may be to reduce pressures without changing mixes. An appropriate response to leaks or connections may involve special measures at those discharge points with or without subsequent thickening of the mix. Hydrofracturing through infilled materials might require no change in operations. However, if the grouting program is being performed in an earthen embankment dam or levee, the suitable response could be quite different. Figures 15-15 through 15-23 show examples of grouting records that illustrate various time behaviors of the injection and the points of mix changes.

c. Mix Changes. Grouting references frequently contain extensive tables for thickening and thinning grout mixes. These tables are normally applicable for unstable grouts composed of only cement and water. In using these tables, it is necessary to know whether they are based on weight or volume relationships. The tables do not apply to balanced stable grouts. In general, it is not very common to alter a batch of grout that has already been mixed. Normally, thickening is performed when a fresh batch of grout is mixed rather than by adding materials to an existing, partially used batch. Thinning, if used at all, is typically performed only when there is a substantial amount of grout remaining in the agitator after completion of a stage and it is time to begin another stage with a thinner starting mix. Depending on the age and the amount of grout left from the previous stage, it is not uncommon to simply waste that grout and mix a fresh batch of the starting mix. Regardless of which procedure is followed, the injection lines and pump must be fully flushed of the thicker mix before filling them with the thinner starting mix.



(Courtesy of Gannett Fleming)

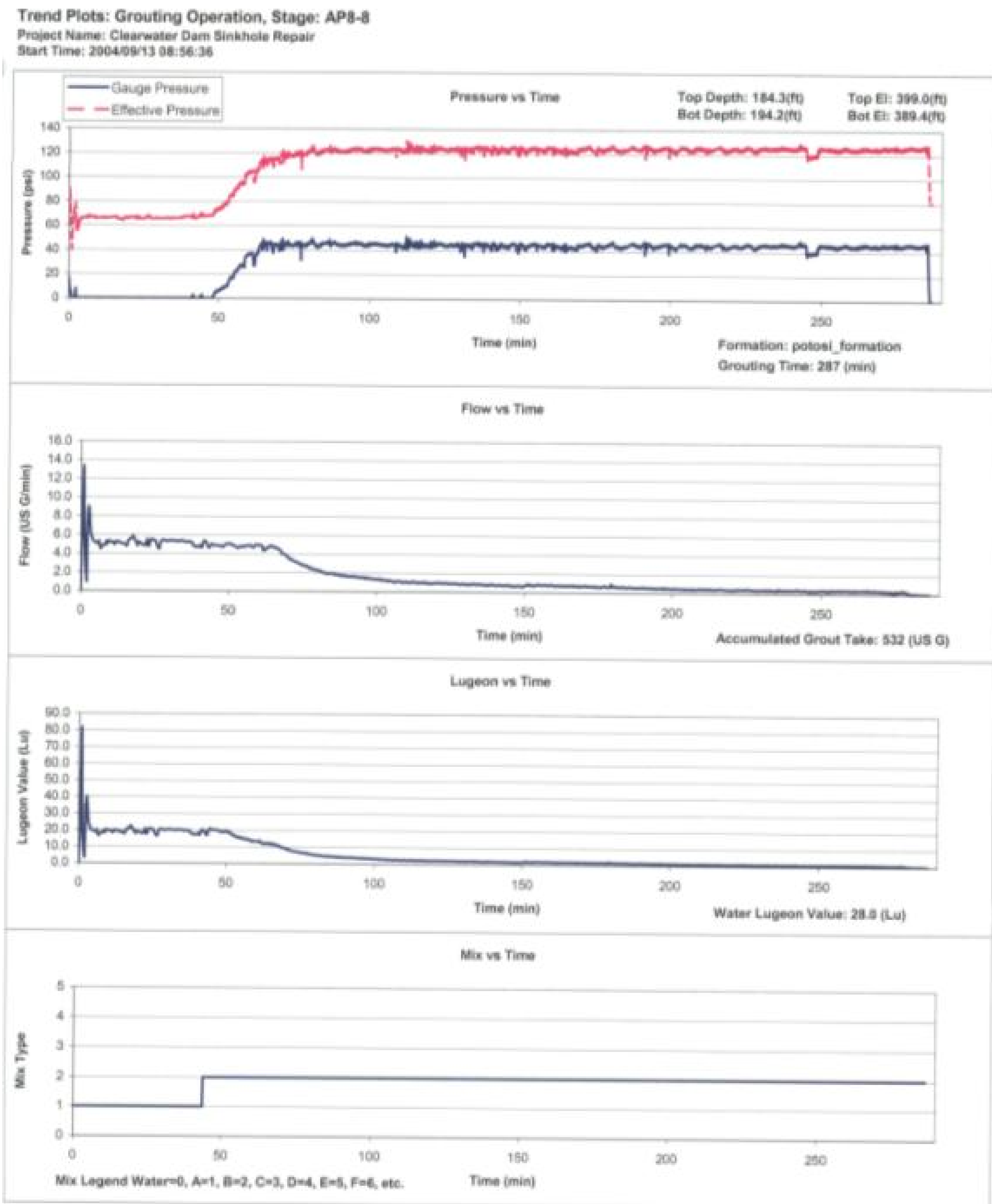
Figure 15-15. Grouting to refusal under a single mix, with a gradual, steady decline of grout take and Apparent Lugeon value.



AT12B-5	7/27/2004 12:50	Gator header station - Cart 1. Tremied 7 gallons. Packer set at 150'.
AT12B-5	7/27/2004 12:51	Pressure grouting begins at minute 2.
AT12B-5	7/27/2004 13:57	Stage complete.

(Courtesy of Gannett Fleming)

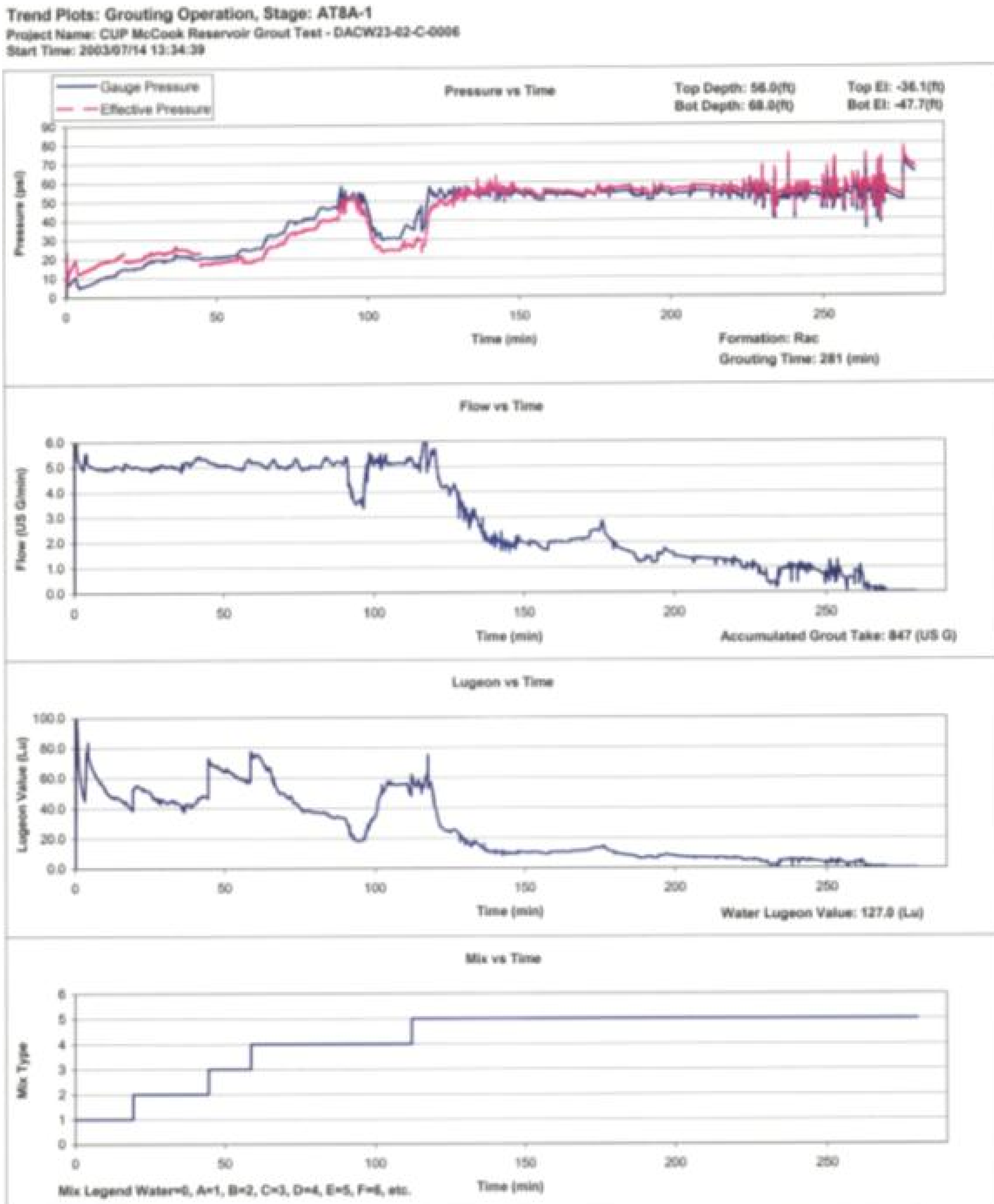
Figure 15-16. Grouting to refusal under a single mix. The mix was not thickened because injection at the maximum rate showed that the pressure was building to desired pressure.



AP8-8	9/13/2004 08:58	Begin pressure grouting at 2 minutes.
AP8-8	9/13/2004 13:42	Stage complete.
AP8-8	9/13/2004 08:56	Gator header station - Cart 1. Tremied 10 gallons. Packer set at 175'.

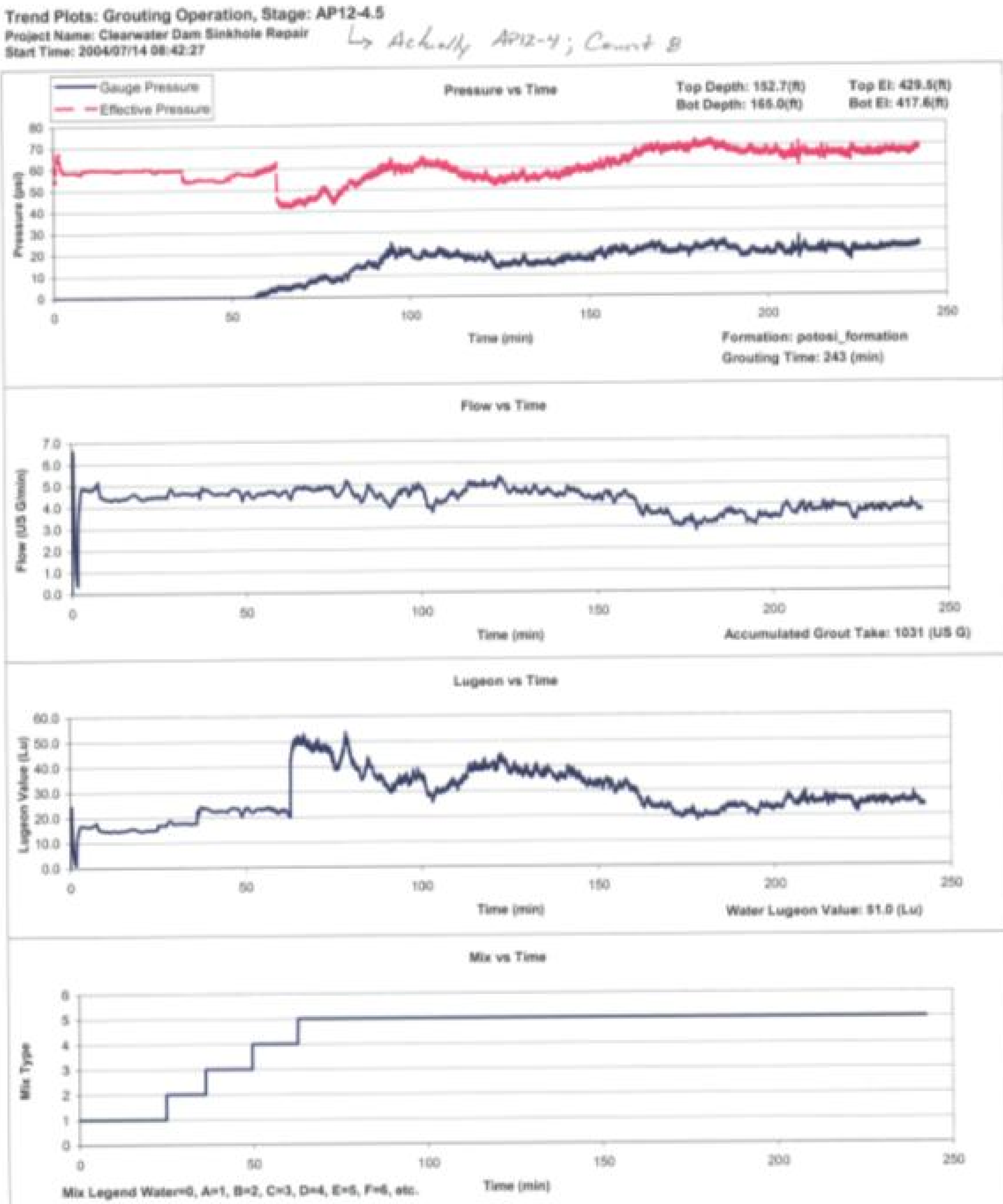
(Courtesy of Gannett Fleming)

Figure 15-17. Gradual refusal after one mix change. The mix was changed after there was no pressure buildup at the maximum rate of injection.



(Courtesy of Gannett Fleming)

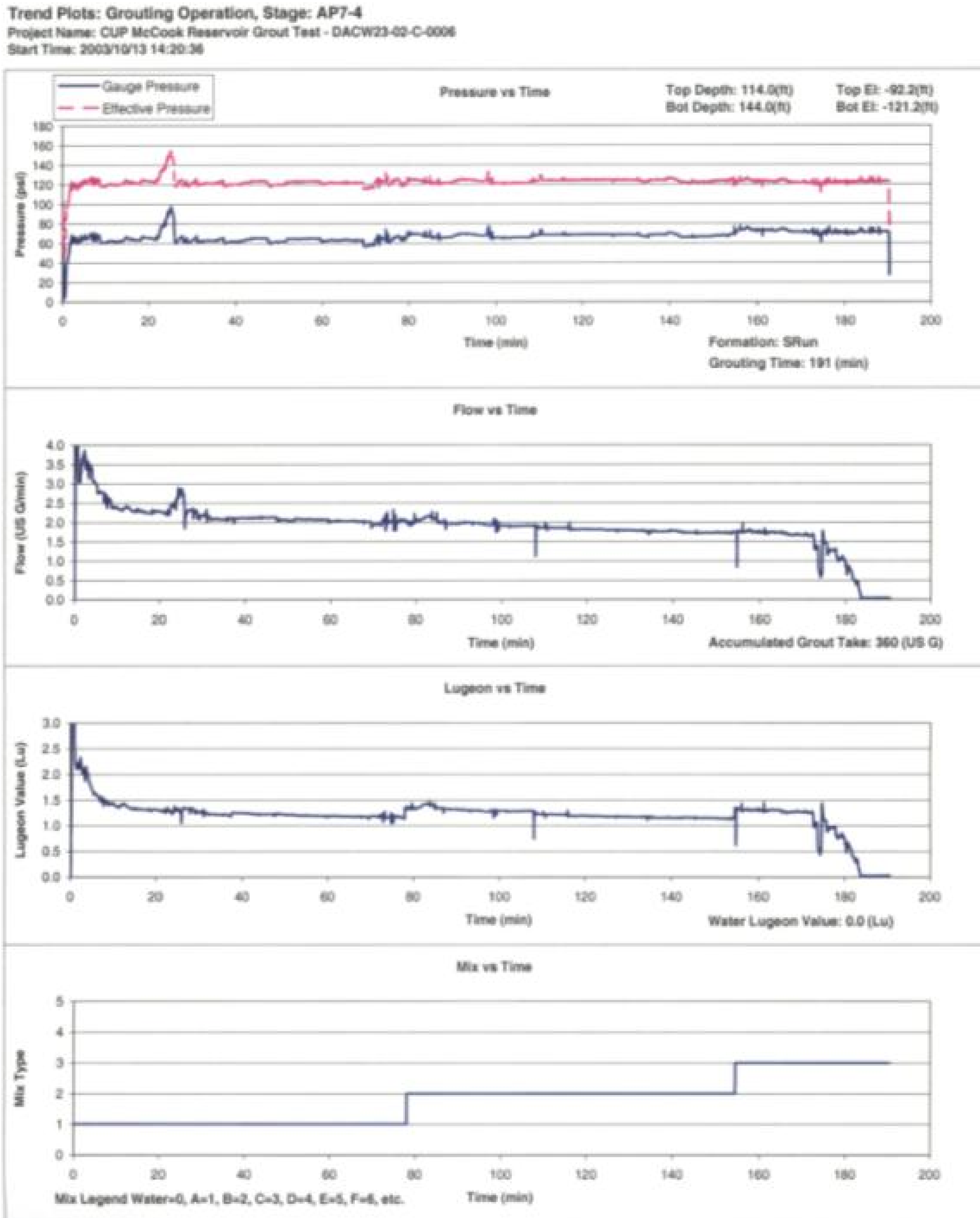
Figure 15-18. Gradual refusal after four mix changes. The mixes were changed in response to pressure plateaus while injecting at maximum rate. The sudden decrease in pressure near minute 100 could be interpreted as hydrofracturing through clay in fracture.



AP12-4.5	7/15/2004	Tremied 4 gallons from time 0 to to 1 minute. Inflated packer and started to Re-Grout at 2 minutes.
AP12-4.5	7/15/2004	2 tablespoons of welan gum per 20 gallons of grout was added to the E-mix to create a mix with a
		higher viscosity.
AP12-4.5	7/15/2004	Stage NOT Complete.
AP12-4.5	7/15/2004	Gator-Yellow Header Station - Cart 1.
AP12-4.5	7/15/2004	After completion of grouting on 7/14, lost water during washing operations. Hole measured open to 154'
		using packer.
AP12-4.5	7/15/2004	Stopped after approximately 1,000 gallons for the day.

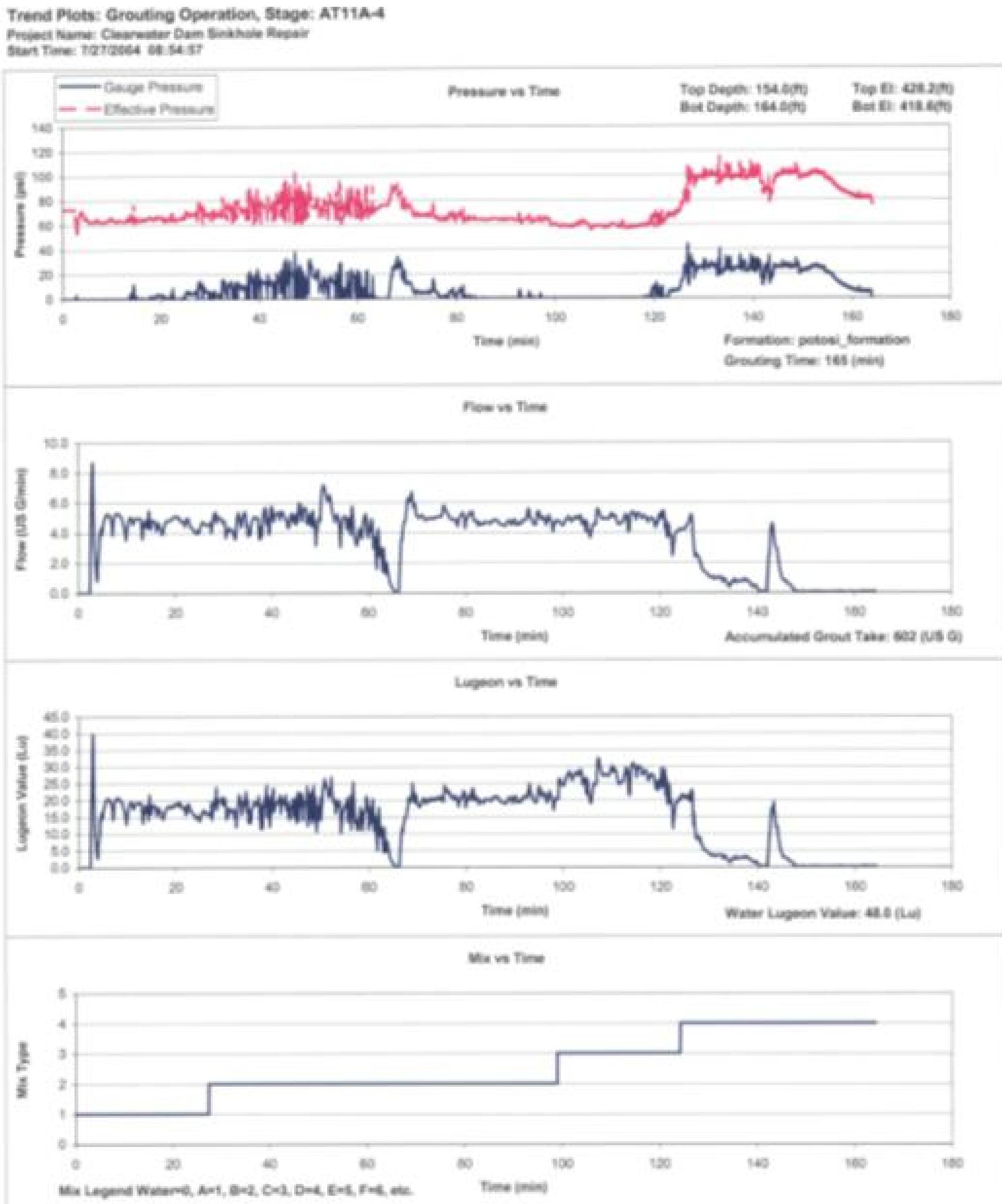
(Courtesy of Gannett Fleming)

Figure 15-19. Termination of grouting without reaching refusal at a predetermined maximum stage injection quantity of 1000 gallons. There were four mix changes without any indication of approaching refusal. The rapid thickening was based on early stage response.



(Courtesy of Gannett Fleming)

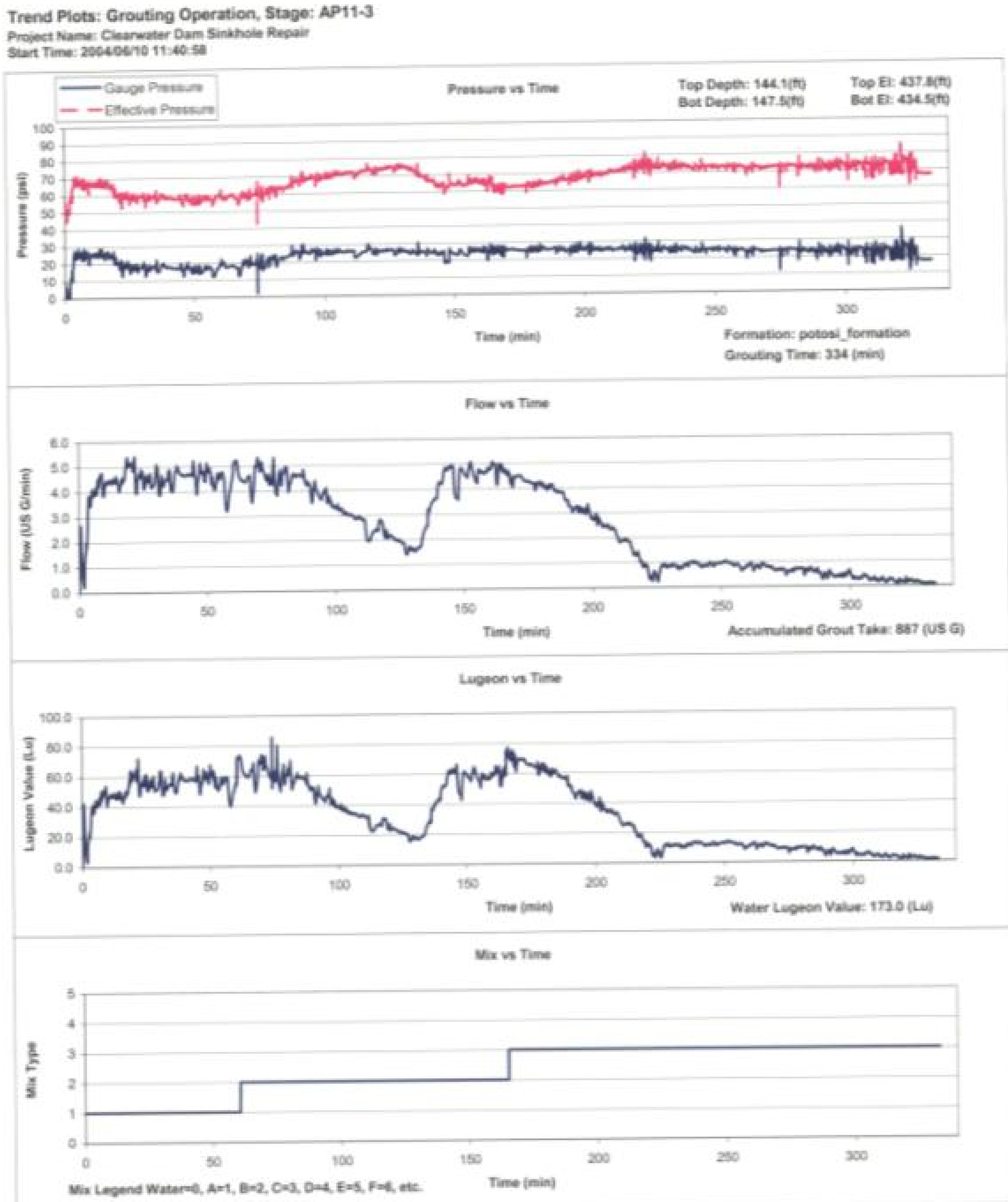
Figure 15-20. Abrupt refusal of a low Lugeon stage after a second mix change. The decision to thicken was based on time management of the stage and the extremely slow rate of approaching refusal under Mix 2.



AT11A-4	7/27/2004 08:54	1st Regrout of stage. Tremied 7 gallons. Packer set at 150'
AT11A-4	7/27/2004 10:01	Switched to remote agitator at minute 64.
AT11A-4	7/27/2004 11:23	At minute 144 - Packer check.
AT11A-4	7/27/2004 11:30	Stage complete.
AT11A-4	7/28/2004 15:10	During the initial grouting of AT11A-4 holes AT11B, AP12, and AT12A were filled with water to check for connections. All lost water. During the regrout of AT11A-4 the next day, hole AT11B held water when filled, while holes AP12 and AT12A lost water. After AT11A-4 refused, holes AP12 and AT12A held water when filled. Connections between AT11A, AT11B, AP12, and AT12A are believed to have existed prior to the grouting of AT11A-4. [This note was added after conversations with field personnel who witnessed the above events]

(Courtesy of Gannett Fleming)

Figure 15-21. Abrupt refusal of a moderate Lugeon stage after a third mix change. The decision to thicken to Mix 4 was based on no indication of progress toward refusal under Mix 3.

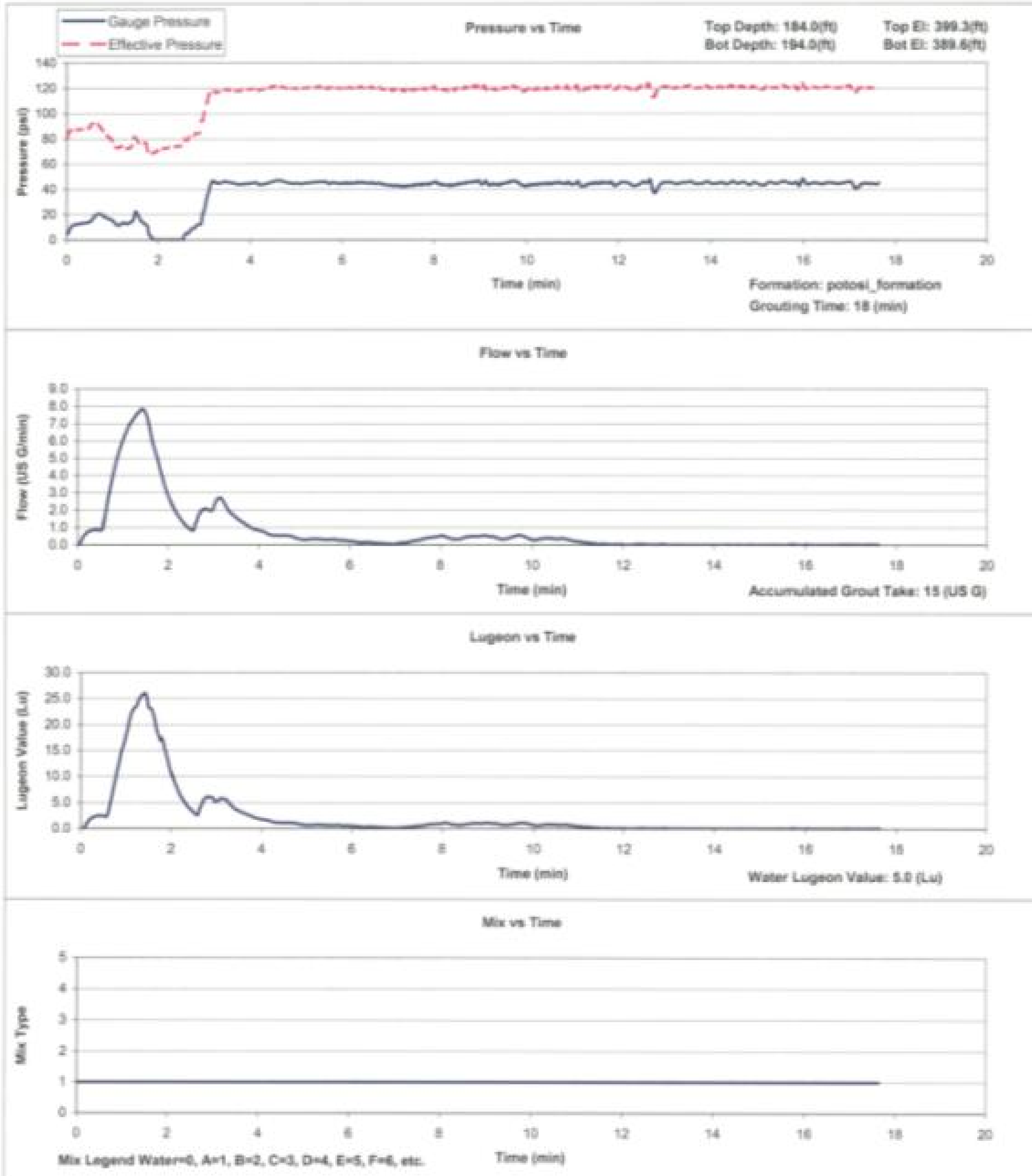


AP11-3	6/10/2004	Tremie 3 gallons.
AP11-3	6/10/2004	Packer set at 142'.
AP11-3	6/10/2004	Cart 1 - Gator
AP11-3	6/10/2004	Stage Complete.

(Courtesy of Gannett Fleming)

Figure 15-22. Sudden increase in grout take as the desired pressure is first approached at 130 minutes in karst geology, interpreted as hydrofracturing through infilled materials. Refusal was reached after extended grouting with Mix 3.

Trend Plots: Grouting Operation, Stage: AT8B-8
Project Name: Clearwater Dam Sinkhole Repair
Start Time: 2004/09/24 10:36:31



AT8B-8	9/24/2004 10:36	Gator header station - Cart 1. Tremied 10 gallons. Packer set at 175'.
AT8B-8	9/24/2004 10:39	Began pressure grouting at 2 minutes.
AT8B-8	9/24/2004 10:54	Stage complete.

(Courtesy of Gannett Fleming)

Figure 15-23. Rapid refusal on a tertiary hole.

d. Maximum Sustained Injection Rates.

(1) Normally, the rate of grout injection is controlled by the allowable injection pressure. Pressure is built up to the maximum effective injection pressure and the stage will take grout at the rate the geologic features allow. However, in some stages (or perhaps in the early phases of grouting a stage), it may not be possible to attain the desired pressure, and the injection rate may be limited by the delivery capacity of the equipment and/or the personnel performing the grout mixing. In the past, when grouting operations were paid for on the basis of the number of bags of cement injected into a stage, specifications did not, in general, contain requirements for injection rates. However, with measurement and payment now being structured on the basis of grouting time, which eliminates some historical disincentives to quality work, it is necessary to specify the maximum continuous injection rate that will be required. Sustained rates for mixing, delivery, and injection are limited by a number of factors, including such items as human endurance and environmental factors. Provided that the plant and associated delivery and injection equipment is sized appropriately, and that sufficient labor is available to support the operations, there is no practical limit on maximum sustained injection rates if high rates are required. However, as discussed in Chapter 10 of this manual, there are significant disadvantages to having oversize equipment on the project.

(2) A maximum pumping rate should be established for injecting grout to restrain grout travel within reasonable limits and to have better control of the job. A reasonable maximum pumping rate for most foundation grouting is considered to be 3ft3/minute, or 25 gpm,. The rate of the injection into the hole must be controlled by the Government. The maximum rate is only used when there is no pressure buildup. As the pressure rises, the injection rate is reduced. The rate of injection may also be reduced to restrain grout travel under open-hole conditions. The specifications should clearly indicate that the rate of injection will be controlled by the Contracting Officer's Representative and will vary, from the specified refusal criteria to the maximum rate based on pressures, and from 0.5 ft3 per minute to the maximum rate regardless of pressures.

e. Bleeding. When grouting with unstable grouts, periodic bleeding of the hole is recommended to remove water that separates from the mix. This separation and accumulation of bleed water will occur as the grout takes reduce to a near static condition. The bleed water will tend to rise to the top of the hole due to density differentials. Bleed water is removed by including a bleed valve on the high side of inclined standpipe fittings. The valve is periodically opened to remove the bleed water. Figure 15-24 shows an example of the standpipe setup with a bleed valve.

15-11. Refusal Criteria.

a. General. Specification of a refusal criterion must be carefully considered when preparing bid documents. Therefore, borrowing what has worked on another job is not recommended. Items to be taken into account include: What is the purpose of the grouting? Is it to be standalone or supplemental to subsequent barrier wall construction? What is the desired resulting permeability? How variable is the geology (for example, will close-order holes be necessary regardless of the grout takes on more widely spaced holes)? How many grout lines are planned? How large is the grout job? What are the consequences of a "less-than-perfect" grout job? Additional factors may also be considered. Given all this and the inherent variability of geology, this task is not easy or straightforward. Equating any refusal criterion, no matter how stringent, to a resulting permeability during the planning and design phase is rough at best. In practice, a specified refusal criterion must be coupled with the judgment of the construction staff gained during actual execution of the work on a hole-by-hole basis. This approach is particularly effective when coupled with the automated systems described in Chapter 12 of this manual.

(Courtesy of Gannett Fleming)

Figure 15-24. Standpipe arrangement for unstable grouts equipped with a bleed valve.

b. Historical Criteria. Historically, there have been many definitions of the point at which a stage is considered to have been grouted to refusal. In some cases, the definitions are based on reaching a certain rate of grout take or reaching zero measurable take. Sometimes the definitions incorporate rates of take that vary with the applied pressure, and in some cases, they use a rather more complicated approach in which grouting can be terminated while there is still take occurring at the maximum pressure, but no measurable take at some specified reduced pressure. Additionally, the time frames over which these items are to be measured differ from source to source, depending on whether grouting is to continue for a minimum specified amount of time beyond the point of reaching the refusal criterion. Further, some sources differentiate refusal criteria based on whether the work is considered to be a critical hydraulic application. Obviously, all of the criteria are also subject to the accuracy of the method of measurement. Based on a review of various historical criteria, using some assumptions about the accuracy of measurements, it appears that until recently the point of refusal has been defined as when the rate of grout take was somewhere between 0.10 and 1.5 gpm, which represents a variation by a factor of 15. More commonly, the range has likely been between 0.1 and 1.0 gpm, a variation of one order of magnitude. Holding times after reaching the defined point of refusal have typically ranged from 0 to 15 minutes.

c. Current Trends. The tendency over the last 20 years, and particularly within the last decade, has been to use more stringent refusal criteria. Given the capabilities of the automated systems, this approach has become more straightforward and reliable. While refusal has historically been defined by a limiting flow rate, the power of the automated systems and borehole imaging makes it possible to use other criteria. The effective pressure, grout mix properties, fracture characteristics, and stage length all influence the volume, penetration distance, and flow rate.

(1) Performance. Performance is wholly controlled by the residual permeability that will exist after grouting is complete. The residual permeability obviously depends on the efficacy of the fracture filling, which, in turn, is very dependent on using low refusal criteria.

(2) Durability. The durability of the grouting is greatly affected by the completeness of the fracture filling. A total absence of flow channels after grouting is completed produces a more effectively grouted zone.

(3) Economics. The entire purpose of grouting is to treat the foundation to the extent necessary to provide the desired reduction in permeability at the least cost. This depends on numerous factors other than the refusal criterion, which is difficult to quantify with precision. In some geologic environments and for some grouting applications, applying a stringent refusal criterion will provide the greatest economy. This makes sense in these cases because, by the time a stage is reaching the point of refusal, virtually all of the program costs have been incurred and the additional effort may affect overall program costs by only as little as 2–5%. However, in some (notably karst) geology, or when the purpose of the grouting does not require an exceptionally tight foundation, this extra 2–5% represents an unnecessary expenditure; on larger jobs, it can amount to several million dollars and a longer construction schedule.

d. Refusal Criteria Recommendations.

(1) Low-Tech Monitoring and Control. For those relatively few projects still using low levels of technology for both grouting mixes (i.e., unstable grouts) and for monitoring and control of injection (i.e., dipstick and gauge technology), Houlsby (1990) offers a simple yet effective solution in which he recommends that refusal be defined as the point at which there has been no measurable take at the desired pressure within a 15-minute period. He also recommends that the full pressure should then be maintained for an additional 15 minutes, which, in effect, provides an additional 15 minutes of grouting. After grouting is terminated, the injection valve is closed and the pressure is maintained until excess pressure naturally bleeds off. Assuming that measurements are as accurate as possible for the equipment being used, the flow rate at the defined point of refusal is probably in the range of 0.10–0.15 gpm. Since grouting is specified to continue for 15 minutes beyond that point, the actual rate of take at completion is often less than that value. Based on information interpreted from a variety of sources, it is estimated that multiple-line curtains grouted with this criterion can routinely achieve a grouted zone permeability of less than 5 Lugeons, provided all other aspects of the design and execution are of high quality.

(2) High-Tech Monitoring and Control. Aside from the many other benefits of using state-of-the-art monitoring and control technology, the use of pressure transducers and flow

meters substantially improves the accuracy of measurements while allowing much earlier detection of when designated refusal points are reached. Therefore, the technology permits grouting to lower refusal criteria with confidence, and the additional time required to reach refusal is largely or completely offset by more rapid detection. This results in the ability to create less permeable grouted zones at little or no premium in cost. At the Chicago District's McCook Reservoir Test Grouting Program, where extremely low residual permeability was required and careful evaluations of accuracy were performed, it was found that real-time observation of injection rates as low as 0.05 gpm was practical. Below that flow rate, pressure variations made further resolution impractical. In that case, the refusal point was defined to occur when the 0.05-gpm flow rate was observed for 5 minutes, at which point the injection valve was closed and the pressure held for 15 minutes or until the excess pressure dissipated. Post-construction testing indicated that grouting with balanced stable grouts in conjunction with state-of-the-art monitoring and control produced a barrier with a residual permeability of 0.1 Lugeon. It is estimated that multiple-line curtains grouted with these materials, technology, and refusal definition can routinely achieve a grouted zone permeability of 1 Lugeon or less, provided all other aspects of the design and execution are of high quality. For this project, an extremely low residual permeability was required because of environmental considerations that may not be relevant on other projects. It does, however, show the capability of the modern systems.

15-12. Routine Problems During Grouting.

a. Surface Leaks. Surface leaks of grout are normal on most grouting projects, and they can be difficult to deal with. The potential for surface leaks can be greatly reduced by establishing the depth of foundation excavation below highly fractured zones and by using substantial surface grout caps. Where foundations are not of the best quality, placing a thin layer of slush grout mixed to a volumetric water/cement ratio of 0.5 over the fractured surface can be quite effective in limiting the number of leaks. Regardless of frequency, sufficient manpower and materials must be on site to handle leaks when they occur. Surface leaks are typically managed by one of the following methods:

(1) Self-Sealing Leaks. Some leaks will be sufficiently small that no action is required. Grout may flow from them for a period of time, and the leaks may stop without intervention.

(2) Caulking. Leaks that continue without reduction can be caulked using a variety of materials, including wooden wedges, oakum, or bits of cloth that are forced into the cracks. On occasion, the materials are soaked in sodium silicate to help initiate setting of the grout behind the temporary barrier. Leaks should not be caulked until the grout coming from the leaks is the same consistency as the grout being injected. Caulking is often problematic because leakage points may move during caulking or because the condition of the leakage points is not amenable to being caulked effectively. Injection rates can be decreased while caulking if required, but injection should not cease.

(3) Choking with Thick Mixes. For surface leaks that cannot be brought under control with other means, it may be necessary to progressively thicken to very thick grouts and/or reduce pressure. If leakage does not cease with those measures, then grouting should be terminated for a period of time, followed by regrouting after the grout has had time to take at least a thixotropic

set. When any of these methods are used, it should be assumed that grouting of the rock mass in that zone has not been effective, and additional holes must be drilled and grouted to whatever extent necessary for verification of effective treatment. The method of choking surface leaks using thick mixes should only be used when all of the methods described in Section 14-2. d. (2) are unsuccessful.

b. Hole Communications. Another common event that must be dealt with is communication that occurs between holes due to intercommunication of a fracture network of sufficient size that the grout will travel from the hole being injected to one or more adjacent holes. This is not desirable because it may not be readily obvious that it is occurring, and it can result in blinding of fractures by grout in the second hole without the benefit of application of pressure needed to ensure good injection. In a grout curtain program that is appropriate for the geologic conditions, the primary hole spacing will be sufficient to make this a rare occurrence. If primary holes are too closely spaced for the conditions, it can occur many times. Pressure testing will often, but not always, identify the presence of communications that must be handled. Equipment and personnel must always be available to properly deal with communications. The normal treatment may be to install a packer assembly with a pressure transducer and flow meter in the communicated hole at the top of the stage being grouted in the first hole and inject grout either into both holes simultaneously or individually. It is common to leave one hole open until after undiluted grout appears to be exiting the hole. If communications are still found to exist, the packer in the second hole is moved up one stage dimension, and grouting on both holes is resumed. This procedure may result in successive communications, and it is normally a requirement to have sufficient equipment on hand to deal with at least three holes concurrently.

c. Excessive Takes. On occasion there are stages that show no signs of progressing toward refusal, even after going to the thickest mix and injecting a substantial quantity of that mix. When that occurs, it is highly likely that the hole has intersected a long, wide fracture and that the grout is traveling a considerable distance. In that case, excessive injection may have little benefit for performance. An exception is that, in karst geology, a very large feature might exist that will require complete filling. In that case, extensive injections may be appropriate, or it may be necessary to change from a high-mobility to a low-mobility grout. The Government Grouting Engineer should make the decision for the appropriate action after careful consideration of the available geologic and water test data.

15-13. <u>Other Special Issues</u>.

a. Impact of Grouting on the Groundwater Regime. Before construction of a grouted barrier, there will be a groundwater flow regime that has reached equilibrium with the geologic conditions and the boundary conditions controlling that flow regime. Constructing a low-permeability barrier within that regime will progressively alter that regime; the potential consequences, both during and after construction, must be thoroughly understood in advance. A simple example is a grouted barrier constructed across a valley for new dam construction, where, quite commonly, there may be a relatively uniform flow field in the rock, with flow down the valley. As the barrier is constructed, it should naturally cause a rise in groundwater levels on the upstream side of the grouted zone. This effect may be delayed because the initial response to a partially completed curtain can be a flow concentration through fractures in the uncompleted portion of the work. As the barrier is brought to completion, artesian conditions may develop where none existed previously, and it may increase the amount of water that must be handled by the site's dewatering system. Provided proper data are available for design, the expected results at various stages of construction can be reasonably predictable, and the design of the dewatering and water diversion systems should take this into account. In other geologic and boundary condition situations, the impact can be more complex.

b. Impact on Dewatering Systems, Drains, and Instrumentation. The distance that grout will travel in the fracture system depends on the size and interconnection of the fractures, the characteristics of the grout mixes, the pressures used in grouting, and the duration over which grout is injected. Aside from the impact that grouting will have on groundwater regimes and the potential increase in the amount of water that must be handled by a dewatering system, the grouting always has the ability to adversely affect the performance of dewatering systems, drains, and instrumentation. Under worst-case conditions, they can be rendered totally ineffective. Accordingly, the contract documents should always anticipate these possible effects. Preventive measures include locating dewatering wells and instrumentation beyond the anticipated travel limits of the grout and never drilling or constructing drains until after grouting has been completed. As a guide, it is desirable that dewatering wells and instrumentation be located a distance away from the line of grout holes that equals or exceeds the primary hole spacing. Even with that precaution, the contract documents should include provisions for compensation of the contractor for replacement of dewatering wells and instrumentation that are rendered ineffective by the grouting process. If there are existing drains within that distance, it should normally be assumed that they will likely be damaged and will require replacement.

15-14. <u>Closure Analyses and Program Verification</u>.

a. Closure Analysis. Closure analysis, in the most basic terms, involves maintaining data of one form or another for pressure test results and grouting results for each stage tested and grouted. At the least sophisticated end of the spectrum, which is a minimal manual program, the records are limited to writing pressure test results and grout takes on a profile and making qualitative judgments regarding whether the program is complete or not. At the most sophisticated end of the spectrum, there will be scaled color plots of the results on a profile, with the different hole series shown on different CADD layers; multiple plots of pressure test data and grout takes by hole series, with each plot representing what has been determined to be similar geologic conditions, which may be further subdivided by horizontal station and elevation if found to be appropriate; and statistical analyses. Chapter 16 of this manual includes a variety of examples of closure diagrams and analyses.

b. Pressure Test Data. If grouting is to be designed at all, then the design must be based on the goal that the grouting will produce a particular value of residual permeability in the grouted zone. Therefore, the only way the grouting results can be verified to meet the design intent is by applying permeability-based closure standards. For that reason, pressure test results form the backbone of the closure analyses for critical hydraulic structures. When plotted by hole series for similar geologic zones, they show the sequential evolution of decreased permeability resulting from the grouting. A weakness in this approach comes from the fact that water pressure testing precedes grouting. Therefore, the water pressure test data must be extrapolated to estimate the residual permeability after grouting of the final series of holes. When balanced stable grouts are being used, the Apparent Lugeon values that can be measured at the beginning of grouting of each stage can be compared to the water pressure test Lugeon. If there is good agreement, the initial Apparent Lugeon can also be used to extrapolate residual permeability. To confirm the extrapolated residual permeability, water pressure tests should be performed in verification holes after the final series. It is logical to place great reliance on the pressure test data because, in fact, the purpose of the grouting is to act as a barrier to water.

c. Grout Takes. Despite primary reliance on water pressure tests for verification of performance, the measured grout takes are also fully used. The takes are plotted in a similar manner and examined in parallel with the pressure test data. Before the widespread use of balanced stable grouts, it was conventional wisdom that there was poor correlation between water pressure tests and grout takes. The generally vague correlations that were observed to exist are certainly related to both the variation in grout rheology during the injection period and the difference between the rheology of various grout mixes and that of water. This issue may have also been compounded by a myriad of other factors that relate to how the work was actually being executed, to the grout mixes being used, and to the spectrum of geologic conditions. While there is still no reliable correlation available for general use, the following statements are generally true: (1) better correlations have been found to exist with the adoption of balanced stable grouts, (2) fairly good correlations are possible within a specific geologic unit for particular mixes, (3) a vast amount of data are being collected from projects where all of the work was carefully executed, (4) and it is possible that general correlations of value may eventually evolve.

d. Common Errors In Closure Analyses. A number of common errors can contribute to incorrect performance or interpretation of closure analyses:

(1) Inappropriate Averaging. It is common in closure analyses to use averaging to assess closure over a particular region of the grout curtain, whether that region is defined by a geologic unit (or interface of units) and/or by horizontal and/or vertical limits. For example, it is certainly possible to average the results over the entire project using all the data without regard for any differentiating factors, and, surprisingly this often produces a “well-behaved” closure plot. This practice may be acceptable after all the appropriate regions have been individually analyzed and brought to proper closure, but it is inappropriate if that first step has not occurred.

(2) Misunderstanding of the Basic Site Geology. Some of the most egregious errors may arise from not having a sound understanding of the basic site geology and, in particular, the relationship of grout holes intersecting the geologic features and the fracture systems. For example, closure analyses on well-fractured rock tend to work very well because the fracture system is penetrated frequently, and because there is good communication between fractures. Conversely, rock that is massive and with few fractures may have fracture spacings that will not be penetrated by each series of holes. For example, the fractures may be penetrated by only some of the primary and secondary holes, and penetrated by most of the tertiary holes. In that scenario, it may result in a closure analysis showing higher average tertiary test values than the prior series of holes.

(3) Analyses of Inappropriately Sequenced Work. Closure analyses tend to work well on normally sequenced work, whether performed in an upstage or downstage mode. Closure

analyses become very difficult to reasonably prepare and interpret when work is performed out of sequence or in multiple modes.

(4) Lack of Accounting for Every Anomaly. Each anomaly is potentially significant and should be accounted for in the closure analysis in one way or another. Aside from any averaging that is performed, the results for every hole need to be carefully scrutinized individually.

(5) Lack of Sufficient Verification Holes. Closure based on permeability criteria requires interpolation and extrapolation, unless the work is so critical or conditions require that split-spaced holes be extremely close together. Typically, the permeability is assumed to decrease between the grouted holes, but the extent to which this residual permeability remains depends on the physical characteristics of the site and the closure criteria and grouting protocols followed. Although closure criteria are the numerical determinants of when grouting is to be finished, the success of the program is determined by a combination of:

(a) An evaluation of the records of the production grout holes.

(b) The performance of the final series of grout holes during water pressure test and grouting.

(c) An evaluation of verification hole testing and grouting (if necessary).

(d) Observations and evaluation of instrumentation and seepage monitoring associated with the structure.

(6) The number, depths, and location of verification holes vary with the number of grout holes, the extent of grout curtain(s), and the number and locations of critical areas of the structure. The number and design of verification holes should be based on conditions, and these holes should not be arbitrarily placed within the alignment of the curtain. Because most grout holes are not typically cored, documented/logged, or tested to the same level as verification holes, verification holes may be best located in critical areas, especially where grouting did not proceed as expected, either because grout takes were large or because they were lower than expected. If anomalous grout or water pressure test results occur and are significant enough to consider placing a verification hole in the vicinity, the depth and alignment should be optimized to try to intercept the geologic features that may be responsible for the anomaly (if known or suspected.) It is preferable to extend the boring slightly beyond the area where the anomaly is observed to account for the inherent variability in geologic characteristics. A rudimentary estimate for the number of final verification holes is in the range of 5–10% of the number of holes in the final series of split-spaced holes, with the understanding that site-specific conditions should be the determining factors rather than the results of a rule-of-thumb approach. The number of verification holes is often established during the contract development to determine a budgeted cost, but the contract should allow the Contracting Officer’s Representative to adjust the locations, number, and depth wherever practical.

(7) Seepage monitoring, whether at springs, galleries, or relief wells, should be performed before grouting and, where possible, throughout a year’s seasonal variations to establish baseline conditions. Although continued monitoring during larger grouting projects provides an ongoing measure of performance, it is also important to correlate the observations with the baseline and

the seasonal fluctuations that may have occurred before grouting, and to note significant weather events that may have occurred around the time of the observations. The size and scope of seepage analysis will vary by necessity from project to project.

(8) Instrumentation, specifically piezometers, provide data that can be used to assess the performance of grouting, both during grouting and as a reservoir fills. The type and placement of piezometers should be carefully considered. Where there is a risk that grout could seal off, limit or destroy their usefulness, fully grouted vibrating wire piezometers are recommended. As with seepage analysis, baseline data are essential in evaluating the subsequent data. Correlating piezometric data with the sequence of grouting and with seasonal (or significant variations in) weather is important in determining the manner in which grouting has altered and controlled seepage. Abrupt increases in piezometric levels may indicate that a larger or more extensive permeable area has been successfully treated, whereas gradual increases may be more often associated with the treatment of smaller, more disseminated features that contribute to the permeability. Abrupt increases in piezometric level may have stability repercussions and should be brought to the attention of those involved in the design of embankments or structures that may be impacted.

CHAPTER 16

Hydraulic Barriers: Records and Reports

16-1. Introduction. This chapter contains examples of various project records required for grouting and provides guidance on selecting and instituting recordkeeping methods pertinent to grouting programs. Automated grouting activities and equipment will generate a lot of information. Although the Government's rights concerning information gathered or generated by contractors are defined in Federal Acquisition Regulation (FAR) Part 27 and Defense FAR Part 227, the specifications should reinforce and emphasize these rights and clearly define the formats in which the information is to be supplied. Some of the data gathering is achieved through proprietary software or equipment. Contractors are entitled to protection of these trade secrets, but the Corps should have sufficient information on the internal workings to evaluate the validity of the outputs. Confidentiality agreements between the Corps and contractor have been used and may be sufficient to satisfy this concern. One should not assume that the information will be made available in a usable format if it is not required in the specification of contractor submittals. It may also be appropriate for the Corps to require access to the raw data outputs from the contractor's equipment to provide the greatest assurance of quality.

16-2. Purpose of Records.

a. General. Grouting programs demand an intensive and detailed recordkeeping process. Records are required not only for tracking pay items and quantities, but even more importantly, for verifying that project goals are being achieved.

b. Assessment of Results. Grouting results for hydraulic applications are evaluated through a combination of indirect means, such as estimates based on extrapolations of water pressure test data and grouting data, supplemented by direct assessment using post-grouting permeability testing; and measurement of project performance regarding instrumentation and measured seepage, compared to a pre-grouting baseline. Direct testing, which is performed in verification holes, is used to validate the reasonableness of the extrapolation procedures and the estimated results. Except in the most critical of applications, direct verification of all work is generally not practicable due to cost. The number of verification holes will vary, but it is common to have one verification hole for every 50–200 ft of barrier length. These spacings result in directly testing only on the order of 2.5–10.0% of the barrier. Therefore, continuous scrutiny of the records and evaluation of the grouting program as the work progresses are necessary to identify locations where anomalies have been encountered, where additional work is definitely required, where estimated results suggest that the residual permeability might be larger than desired, where target permeabilities have been achieved, and where verification testing is appropriate. Detailed recordkeeping throughout the course of the project is necessary to achieve this goal.

c. Quantity Tracking. As with any other contracts that include unit price pay items, tracking of quantities is necessary to process invoices, to prepare contract modifications, and to track the financial health of the contract. Many of the records required for project assessment and technical documentation described herein contain pay item quantities. Depending on the complexity of the contract, solicitation method, and specific type of work performed, additional quantity tracking records might be required.

d. Equitable Adjustments. Equitable adjustments on grouting projects are common due to the nature of the work, changes in the grouting program and procedures that may be found to be necessary or advantageous, site conditions that differ from those represented in the contract documents, and certain solicitation methods. Detailed project records are essential for evaluating and fairly resolving contractor requests for equitable adjustments.

e. Communication of Interim Results to Others. On most projects there is a need to be able to communicate grouting results to other parties on an ongoing, real-time basis. The other parties' interests might be technical, schedule, contractual, or financial. The basis for providing the needed information is the basic project records. However, for the information to be of value, the records must be translated into summary documents that are clear and easily understood by all. With the advent of advanced data management and analysis tools, it is common that the results take graphical form. These legitimate needs for information require that all project records be maintained up to date at all times, and also require that the resources be kept available to produce the summary documents on demand and distribute them in appropriate media.

f. Project Completion Reports. On most civil work projects, a completion report is required. Detailed grouting project records aid in the completion of these reports. Summary analyses produced throughout the course of the work are particularly useful when preparing a completion report.

16-3. Records Keepers. On grouting projects, both the contractor and the Government are responsible for keeping records. The contractor is typically tasked with generating certain records, the details of which must be outlined in the contract documents. The required records typically include drilling logs, water testing and grouting logs, quality control logs, etc. The contractor may also be tasked with performing other duties, which could range from generating simple summary sheets for particular tasks (e.g., a summary of water test results for a recently completed section of curtain), to creating detailed project reports including the contractor's independent analysis and interpretation of the conditions encountered and results of the work performed. Ultimately, the Government is responsible for project decisions and is therefore responsible for its own interpretation of data or at least approval of interpretations. If the contractor is not tasked with keeping records of sufficient detail to allow analyses on the work, the Government must do so with its own forces. If the majority of the records are generated by the contractor, Government personnel need not keep records of individual work items (e.g., water testing logs for individual stages). However, Government personnel should keep a daily record of the work performed in sufficient detail to spot check the contractor's records. Depending on the nature of the project, it may be necessary for Government personnel to keep more detailed records and, in some cases, to produce all technical records.

16-4. Recordkeeping Methods. Traditional recordkeeping methods for grouting projects consisted of volumes of hand-written field reports, summary records, and hand-drawn profiles and sections. These records are time consuming to produce, and in many instances the field copy is the only copy available. Computer technology, in particular spreadsheets and databases, has proven to be an invaluable tool for grouting projects. The ability to track running totals of contract quantities and project technical data in electronic format lends itself to query reporting methods that can produce accurate and specific information. Contractors using sophisticated computer grouting systems may come equipped with such databases, spreadsheets, and queries.

If such levels of technology are not specified for the project, an adequate number of Government personnel will need to be assigned to manage, analyze, and maintain project records.

16-5. Information Dissemination. Work activity reports generated by the contractor, including all technical and pay item data, should be provided to the Government on a daily basis. The contract documents clearly specify the content of the documents that the contractor will provide, including the interval, number of copies, and format. Hard copy reports are the norm, but electronic data transmission has certain advantages, particularly if Government personnel are tabulating information in spreadsheets and databases. If electronic copies of daily reports, or portions thereof, are required, the contract documents should clearly state the documents (and electronic formats) that are required.

16-6. Pre-Contract Information. Before award of the grouting contract, significant effort has typically been expended to investigate, research, and design the project. The information generated as part of this design process can prove invaluable during the production grouting. These records could include the original design and construction documents, previous project completion reports, subsurface investigation and laboratory testing data, core photographs, bedrock mapping and discontinuity data, instrumentation readings, and geophysical investigations. Much of this information would likely be documented in the solicitation documents, although sometimes in a summary format. Typical grouting contract drawings would include a plan view of the site that shows the grout hole locations and other pertinent grouting information. Profiles and sections cut through and along curtain locations typically provide the best avenue of conveying existing site information. This information could include: (1) investigative drilling boring logs and interpreted conditions, (2) permeability test results, (3) locations where drilling difficulties such as weathered zones, water losses, or solution features were encountered, (4) and grout hole locations, designations, spacing, and inclinations. On-site Government personnel in charge of the grouting program should request copies of all information that could provide insight into the site conditions. When the contractor is providing sophisticated grouting information management systems, this information can be incorporated into the system as part of the start-up operations. However, in that event, the Government must, as part of the contract documents, define the specifics of the information to be incorporated.

16-7. Reporting and Technology.

a. Technology Levels. Grouting records and recordkeeping methods can vary significantly, depending on the type of grouting being performed, and from project to project, depending on the level of technology specified for the work. The specified recordkeeping methods could be as simple as hand-written logs or as sophisticated as web portals containing interactive, electronic files providing project-specific, custom, graphical summaries of all data and results. The following paragraphs discuss the three recognized levels of grouting technology for rock permeation grouting and summarize the records that should be kept for each level.

(1) Level 1—Dipstick and Gauge Technology. This technology is the basic level required for rudimentary recordkeeping. It is referred to as “dipstick and gauge technology” because volumes are typically measured with a graduated dipstick placed in the agitator and pressures are manually read from a dial pressure gauge. Readings are typically obtained every 5–15 minutes. Automation is completely absent with this technology, and all field records are manually

generated by inspectors in the field. An improvement in technology at this level would include the addition of electronic flow meters and pressure transducers (along with the use of balanced, stable grouts). These instruments provide instantaneous readouts of flow rate and injection pressure at the instrument location. However, no transmission or recording of the data is provided, and all water pressure testing and grouting records are still manually recorded. At this level, the contractor provides few records beyond quantity information, and the vast majority of the recordkeeping tasks are the responsibility of the Government. Under the best circumstances, adequate inspection staff is available to fully cover all field inspection tasks, and the data collected is manually entered into spreadsheets and databases for further analyses. This work is time consuming, laborious, and frequently subject to human error. All pay items are manually tracked, and as-built grouting profiles are manually updated.

(2) Level 2—Data Collector and Display Technology. Level 2 technology consists of automated data collection and display units for recording flow rates and pressure. These units record field instrument data at a frequency typically between 2 and 15 s. At the low end of this level, the units simply consist of data recorders that log injection flow rates and injection pressures at the instrument locations and display the data as they are obtained. More advanced units at this level automatically correct the data for head losses and gains as necessary to calculate the effective injection pressure and other parameters such as the Apparent Lugeon value and cumulative grout take. In either case, the data are automatically recorded by the system and displayed in real time, typically at a central monitoring location. With this level of technology, hard copy plots of injection rate, injection pressure, and other parameters calculated by the system and plotted versus time are easily obtainable either automatically or with minimal manipulation. The raw data used to generate the hard copies is also readily available and should be transmitted to the Government. In some cases, pay items such as injection time and volume of material injected are tabulated by the system. After the data collection is complete, all other project records, including data input for subsequent analyses and as-built grouting profiles, are manually entered and manually generated.

(3) Level 3—Integrated Systems Technology. This level of technology represents the current state of the art in grouting monitoring and evaluation. In fully integrated systems, all field instruments are monitored in real time through a computer interface; all necessary calculations are performed automatically; grouting quantity information is tabulated and summarized electronically; program analyses are conducted automatically by the system using numerous variables; and multiple, custom as-built grouting profiles are automatically generated and maintained in real time. This level of technology provides the most reliable and highest quality project records with minimal operator effort. Since all data are in electronic format, these systems lend themselves to customization of project records for specific project needs and sharing of project data with remote experts. Features that may be included in fully integrated systems include: (1) automatic tagging of data with geologic units, (2) graphical stage records showing mix changes, (3) 3-D representations of data, (4) CADD fully integrated with the data collection and database systems, (5) easily customizable displays to provide the most effective graphical representation of data and results, (6) creation of project-specific, custom drawing sets illustrating the relationships between one or multiple variables, (7) a broad range of database query functions for analysis of results and work planning, (8) automatically generated closure plots with user-defined limits based on geologic units or specified horizontal and vertical limits,

(9) electronic links to project data including boring logs, drawings, and photos, and (10) full on-site capability to produce any desired output product within minutes.

b. Applicability of Technology Alternatives. Level 1 technology is applicable for very small projects of limited scope, projects in which unstable cement grouts are being used, and projects that do not involve constructing critical hydraulic barriers. Level 2 technology is applicable for small projects (i.e., up to $1 million in size) using balanced stable grouts where extensive analyses and reports are not expected to be required. The performance of Level 2 technology can be boosted by dedicating additional human resources such as staff exclusively assigned to data entry, data analysis, and on-site CADD capability. Level 3 technology is applicable for medium to large projects where comprehensive recordkeeping, complex analyses, and extensive reporting on a real-time basis are desired as the norm. Due to developments in the technical capabilities of GIS with mature graphic depiction, the current trend is to develop the data management system using this technology.

16-8. Production Records.

a. General. Regardless of the complexity of the project and the level of technology specified for performing the work, certain core field records are always required for adequate documentation. The contract documents should clearly specify the reporting requirements for each activity record that is required and mandate the inclusion of example reports meeting the requirements in the proposal or as a separate submittal before production. Government personnel should review the example records the contractor provides to ensure that they comply with the specifications and conform to the project's intent. The following paragraphs describe production records in detail.

b. Drilling. Three basic types of drilling operations are common on grouting projects: overburden drilling, rock drilling, and exploratory drilling. Exploratory drilling documentation procedures should be conducted in general accordance with EM-1110-1-1804, Geotechnical Investigations, and should use the USACE standard ENG Form 1836. Overburden and rock drilling logs should be documented using either ENG Form 1836 or other record formats submitted by the contractor and approved by the Government. At a minimum, drilling records should include: (1) basic hole data such as the hole designation, inclination, and total drilled depth; (2) specifics regarding the drilling method and tooling such as top-hole hammer rotary percussion, 3-in.-diameter hole, soft rock button bit, and drill steel size, (3) details regarding the response of the materials to the tooling advancement such as soft rock, hard rock, and tool drops, (4) the color of wash water or cuttings if no samples are being obtained, and (5) special conditions such as water losses, artesian conditions, and voids. The types of data recorded on the drilling log may vary, depending on the type of equipment used and the level of detail required. The contract documents should specify the drill logging and recordkeeping methods required to satisfy the intent of the contract. These methods could take the form of hand-written drill logs prepared by the drill operator, logs prepared by Government staff, typed logs using ENG Form 1836 and prepared by a registered geologist or engineer provided by the contractor, or drill logs prepared using software that is GIS compatible. The drilling log should include all applicable pay item quantities specific to the particular drill hole. Figures 16-1 and 16-2 show completed example drilling logs.

Hole No. WP11

DRILLING LOG	DIVISION: LRD	INSTALLATION: McCOOK RESERVOIR GROUT TEST		SHEET 1 OF 9
1. PROJECT: CHICAGO UNDERFLOW PLAN (CUP)		10. SIZE AND TYPE OF BIT: HQ3 3.75-inch impregnated		
2. LOCATION (Coordinates or Station): W-Line 1+50		11. DATUM FOR ELEVATION SHOWN (TBM or MSL): Chicago City Datum		
3. DRILLING AGENCY: Advanced Construction Techniques, Ltd		12. MANUFACTURER'S DESIGNATION OF DRILL: Boyles B15		
4. HOLE NO. (As shown on drawing title and file number): WP11		13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED: --	UNDISTURBED: --
5. NAME OF DRILLER: Denis Lafleur		14. TOTAL NUMBER CORE BOXES: 26		
6. DIRECTION OF HOLE: ☐ VERTICAL ☒ INCLINED 15 DEGREES FROM VERTICAL; ☒ AHEAD STATION ☐ BACK STATION DIRECTION OF INCLINATION		15. ELEVATION GROUNDWATER	0 hr.	24 hr. -36.6 Ft
		16. DATE HOLE	STARTED 02-12-04	COMPLETED 02-14-04
7. THICKNESS OF OVERBURDEN: 48.0 Ft		17. ELEVATION TOP OF HOLE: 14.22 Ft.		
8. DEPTH DRILLED INTO ROCK: 366.0 Ft		18. TOTAL CORE RECOVERY FOR BORING = 366.0/366.0 = 100%		
9. TOTAL DEPTH OF HOLE: 414.0 Ft		19. INSPECTOR: Cliff Wright		

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE REC / % RQD e	BOX OR SAMPLE NO. f	Remarks (Drilling time, water loss, depth of weathering, etc., if significant)	
-32.1			48.0' to 68.9': Light gray to olive-gray, locally mottled dark gray to light olive-gray DOLOMITE (Racine Fm - Subdivisions D&C), very hard, slightly weathered to unweathered, Cherty with gray and green Clay stringers, 63.0' to 68.0', thin to thick bedded, RD=15 °, intensely to moderately fractured RQD = 18.9'/20.9' = 90%	100 100	R-1	R-1: 48.0' to 49.0' Recovery = 1.0'/1.0'=100% RQD = 1.0'/1.0'=100%	
	50		48.0' to 49.9': undulating vertical fracture, healed 48.0' to 48.9', rough and tight 48.9' to 49.9' Bedding joints 49.6', 49.7', and 49.8' 50.2' to 50.7': tight vertical joint, RD=70' Bedding joints: 51.2', 51.9', 52.4', 52.5' (tight with green clay coating) Tight stylolitic bedding joints: 53.2', 53.8', 54.9', 56.7', 57.2', 58.1'	100 96	R-2	R-2: 49.0' to 59.0' Recovery = 10.0'/10.0'=100% RQD = 9.6'/10.0'=96% R-2 recovered 0.1' of core initially missing from R-1	50
	55						55
	60		Fairly tight bedding joints in slightly vessicular Dolomite at 59.3', 60.0', 60.7', 61.1', 61.6'			R-3: 59.0' to 69.0' Recovery = 10.0'/10.0'=100% RQD = 8.3'/10.0'=83%	60
			Undulating joints along Clay stringers and Chert nodules at 63.1', 63.6', 64.3', 64.8', 65.3', 65.9', 66.5'	100 83	R-3		
	65		Generally very broken to broken with green and gray Clay (up to 1" thick) from 66.6' to 68.8'				65
-52.3							
	70		68.9' to 91.0' Vessicular to vuggy dark gray DOLOMITE, locally mottled light gray (Racine Fm - Subdivision B), very hard, slightly weathered to unweathered, thin to thick bedded, stylolitic, intensely to highly fractured, bedding joints generally rough or stylolitic, vertical joints generally healed, tight, or Clay filled RQD = 20.1'/22.1'= 91%			R-4: 69.0' to 79.0' Recovery = 10.0'/10.0'=100% RQD = 9.7'/10.0'=97%	70
	75		Numerous rough, undulating vertical fractures 69.0' to 79.0', most are healed or partially healed and tight Vertical joint 74.8' to 75.7' is tight and Clay-filled (Clay approximately 1/8" thick)	100 97	R-4		75

ENG FORM 1836 | Project: CHICAGO UNDERFLOW PLAN (CUP) | Hole No. WP11

Figure 16-1. Completed drilling log using ENG Form 1836.

ROCK DRILL LOG

Hole No: AP21 Date: 01/16/04

Drill Hole Identification

Station:	1+10	Previous Drilled Depth:	87.5
Hole Inclination:	15 degrees	Total Rock Drilled:	22.5
Hole Top Elevation:	982.6	Total Drilled Length:	110.0

Equipment		Drill Crew	
Drill Rig:	IR Airtrack	Driller:	Joe Smith
Tooling:	T-45	Helper:	Ben Snyder
Bit Size:	3.0 inches	Start Time:	9:50 AM
Bit Type:	Soft rock button	Finish Time:	11:30 AM

Drilling Conditions / Observations

From (foot)	To (foot)	Comments
87.5	95	Soft drilling, brown/tan return
95	97	No rotation or hammer, very soft, lost water
97	103	Hard drilling, no water return
103	110	Hard drilling, regained 50% water return, gray wash

Figure 16-2. Completed drilling log using a form provided by a contractor.

c. Hole Washing. Hole washing logs are often overlooked as a technical record and are frequently considered simply a pay item record. Hole washing is completed after drilling and before water testing and is the first time that relatively weak tooling, compared to the stiff drill tooling, is inserted into the hole. That being the case, the washing operation can provide valuable information regarding the subsurface conditions before inserting more expensive equipment and conducting costlier water testing and grouting operations. For example, if the wash tooling is difficult to insert, if obstructions are encountered, or if difficulty in extracting the tooling indicates caving of holes, then it should be presumed that difficulties may also be experienced during the water testing and grouting operations. At a minimum, the washing record should clearly convey zones that required multiple washing sessions before the return water became clear, zones where difficulty was experienced during wash tooling insertion or withdrawal, zones where collapsing material was likely encountered, and all pay items applicable to the washing operation particular to the specific hole. Hand-written washing logs are typically completed by the wash equipment operator and suffice for the majority of grouting projects. Figure 16-3 shows a sample hole washing log.

WASHING LOG

Date: 01/17/04

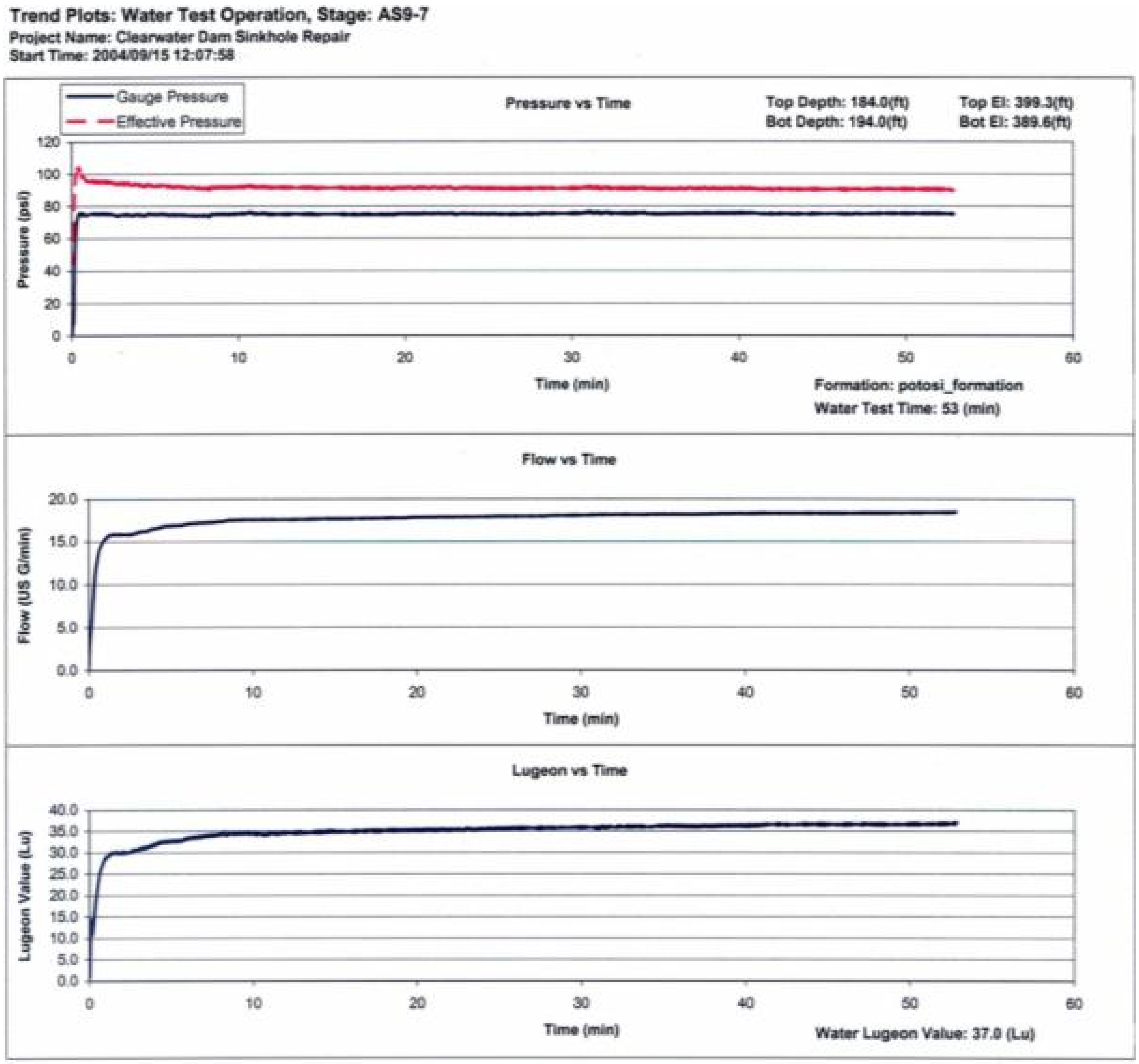
Washing Equipment	Washing Crew
Washout bit and pipe Water off supply line	Washer: Joe Smith Helper: Ben Snyder

Conditions / Observations

Hole I.D.	Time Starting (hh:mm)	Time Ending (hh:mm)	Time Duration (min.)	Depth From (foot)	Depth To (foot)	Comments
AP 19	12:00	12:35	0:35	0	55	gray return
				55	95	brown return
				95	110	gray return, drill cuttings
AP20	12:45	13:25	0:40	0	70	gray return
				70	105	brown return
				105	110	gray return, drill cutting
AP21	13:30	13:50	0:20	0	95	brown wash, 50% return
				95	100	lost all water, hole caved
						at 100 ft

Figure 16-3. Hole washing log using a form provided by a contractor.

d. Water Testing. Water testing, also commonly referred to as “pressure testing,” constitutes the primary means of assessing hydraulic conductivity and evaluating curtain closure. Appropriate records are vital for this activity. Water testing records should include all appropriate pay item information, which typically consists of the test duration or number of tests. The type and level of sophistication of the records generated during the water testing operation depend on the level of technology available. Level 1 records consist of manual gauge and dipstick or mechanical flow-meter readings at regular intervals, typically on the order of 5–15 minutes between readings. Hole- and stage-specific information, including grout hole number, hole collar elevation, hole inclination, top and bottom stage depth, appropriate geologic unit for the stage, water table depth, and height of instruments above the hole collar, are manually recorded on the log. The Lugeon value or stage permeability is calculated manually in the field during testing or after completion of the test. The raw data records produced at Levels 2 and 3 are identical and consist of time history plots of injection pressure at the collar and injection rate. Higher-end Level 2 systems and all Level 3 systems perform further calculations, including the effective stage injection pressure based on the hole geometry, the depth to the water table, the height of the field instruments above the collar, and head-loss factors, allowing the Lugeon value to be calculated automatically in real time. The effective injection pressure and calculated Lugeon value are also plotted in time history format along with the raw field instrument data. The information manually recorded on the paper field log using Level 1 systems is input into a computer interface at Levels 2 and 3 before testing the stage and is populated onto the hard copy trend plot for the test. Figure 16-4 shows an example Level 3 or high Level 2 stage water testing record. Regardless of the level used, it is important to make provision for remarks concerning unusual conditions or activities that help to explain what occurred during the injection, including grouting.



AS9-7	9/15/2004 12:22	Trend indicates erosion.
AS9-7	9/15/2004 12:07	Mobile header station - Cart 2. Packer set at 175'.
AS9-7	9/15/2004 12:09	Unable to keep hole open to full stage depth of 194' after several hole washings. Hole open to ~189'.

(Courtesy of Gannett Fleming)

Figure 16-4. Stage water pressure testing log from a Level 3 or high Level 2 system.

e. Grout Batching. Grout batch logs are typically produced manually by the batch plant operator. The records required are minimal and consist of the mix type and the time of mixing and transfer to the agitator tank. If grout in the system exceeds the maximum allowable time before injection and requires disposal, the batch log should note the approximate volume of grout wasted and if a system cleanout was performed. Automated batching allows for the production of more detailed records, such as the weight or volume of individual ingredients added during mixing. Both records provide sufficient data for reconciliation of grout material quantities if compensation for individual ingredients is a requirement of the contract. Figure 16-5 shows an example of a manual grout plant log.

f. Grout Quality Control. Documentation of the quality control tests performed during initial mix design testing and production grouting is necessary to ensure that the grout properties are in compliance with the contract requirements. Testing results could include compressive strength of cured grout cubes, grout viscosity, specific gravity, bleed, and pressure filtration. The quality control logs should note the date, time, testing agent, grout mix tested, time batched, and test results. Figure 16-6 shows an example grout quality control log.

GROUT BATCH LOG

Date: 1/18/04

Station 1								
Time	MIX A	MIX B	MIX C	MIX D	MIX E	MIX F	MIX G	MIX M
13:49	✓							
13:55	✓							
14:03		✓						
14:10		✓						
14:16		✓						
14:21			✓					
14:29			✓					
14:40				✓				
14:51				✓				
15:09				✓				
15:32				✓				
Cleanout of agitator and circulation loop at 15:40								
Waste approximately 30 gallons of D-Mix								
15:51	✓							
15:59	✓							
16:14	✓							
16:31		✓						
16:58		✓						
Cleanout of agitator and circulation loop at 17:10								
Waste approximately 20 gallons of B-Mix								

Figure 16-5. Grout plant batch log.

GROUT QUALITY CONTROL REPORT

Tested By: Mike Smith Date: 1/18/04

Grout		Test Types					
Batch Time	Mix Type	Test Time	Marsh Time (sec)	Specific Gravity	Bleeding at 2hrs (%)	Pressure Filtration (min-1/2)	COMMENTS
13:49	A	14:00	36	1.42	2	0.049	Hole AP18, Stage 2
14:03	B	14:09	44	1.46	0	0.042	Hole AP18, Stage 2
14:21	C	14:27	56	1.51	0	0.039	Hole AP18, Stage 2
14:40	D	14:44	65	1.59	0	0.037	Hole AP18, Stage 2
15:51	A	15:58	37	1.43	1	0.046	Hole AP18, Stage 1
16:31	B	16:35	44	1.45	0	0.039	Hole AP18, Stage 1

Cubes cast for mix A batched at 13:49 and mix B batched at 16:31

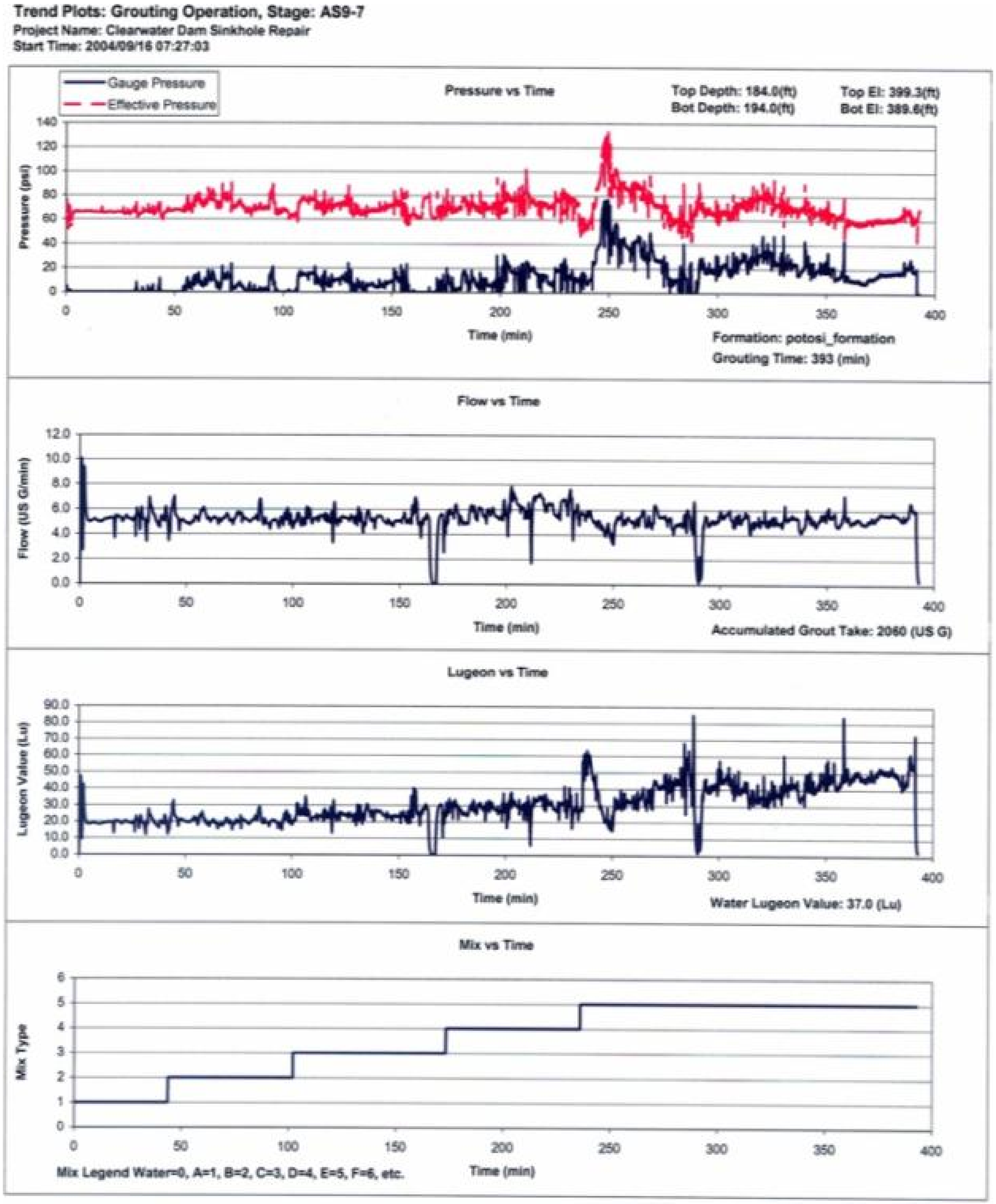
Compressive strength tests for cubes cast on 1-11-04 attached

Figure 16-6. Grout batch quality control log.

g. Grout Injection. A majority of the inspection, logging, and recordkeeping effort is associated with producing detailed grout injection logs. The format of these logs is similar to that of the water testing log except that the injection fluid consists of grout rather than water and tracking of grout mix changes is required. For Level 1 systems, the injected mix type is recorded manually on the field log. On some automated systems, the injected mix type may be recorded as an additional time history plot on the hard copy log. The total volume of grout injected and the total injection time should be recorded on the log. Figure 16-7 shows an example grout injection record produced by automated methods. Grout stage logs should include the allowable pressure for the stage along with the input assumptions for the calculation of the estimated effective pressure.

h. Specialty Records. Specialty exploration, testing, or other production activities required for a project require appropriate, specific documentation. These activities could range from simple tests such as falling or constant head permeability testing to borehole deviation surveys, borehole sidewall imaging, and down-hole or surface geophysical testing methods where the records produced are limited by the available processing software. The contract documents should clearly state the reporting requirements of the specialty testing records.

16-9. Quantity Reconciliation. Tracking of quantity growth and authorization of progress payments require accurate tracking of the work performed and quantities consumed. Quantities should be reconciled with the contractor daily. In their daily report, the contractor should be required to submit the total of each quantity consumed that day, in addition to reports for individual operations (i.e., drilling, washing, grouting logs, etc.). Government personnel should compare each line item quantity to the contract quantity to determine in advance if the accrued quantity will likely exceed the contract quantity and if a change order is required. This task is best performed using spreadsheets and plots such as percentage of quantity consumed versus percentage of work completed. This type of analysis easily allows for projections of estimated final quantities. Quantities should be entered into the spreadsheet by date to allow for future querying of the data and to ease production of summary quantity reports for processing progress payments. Figure 16-8 shows sample spreadsheet input.



AS9-7	9/16/2004 07:29	Hole AS9 was washed to ~ 190' prior to grouting.
AS9-7	9/16/2004 07:27	Gator header station - Cart 1. Tremied 10 gallons. Packer set at 175'.
AS9-7	9/16/2004 07:29	Began pressure grouting at 2 minutes.
AS9-7	9/16/2004 12:18	From minutes 288 - 291: Switched to remote agitator.
AS9-7	9/16/2004 13:58	Stopped grouting due to maximum injection volume of 2,000 gallons. Stage not complete.

(Courtesy of Gannett Fleming)

Figure 16-7. Stage grouting log from a Level 3 system.

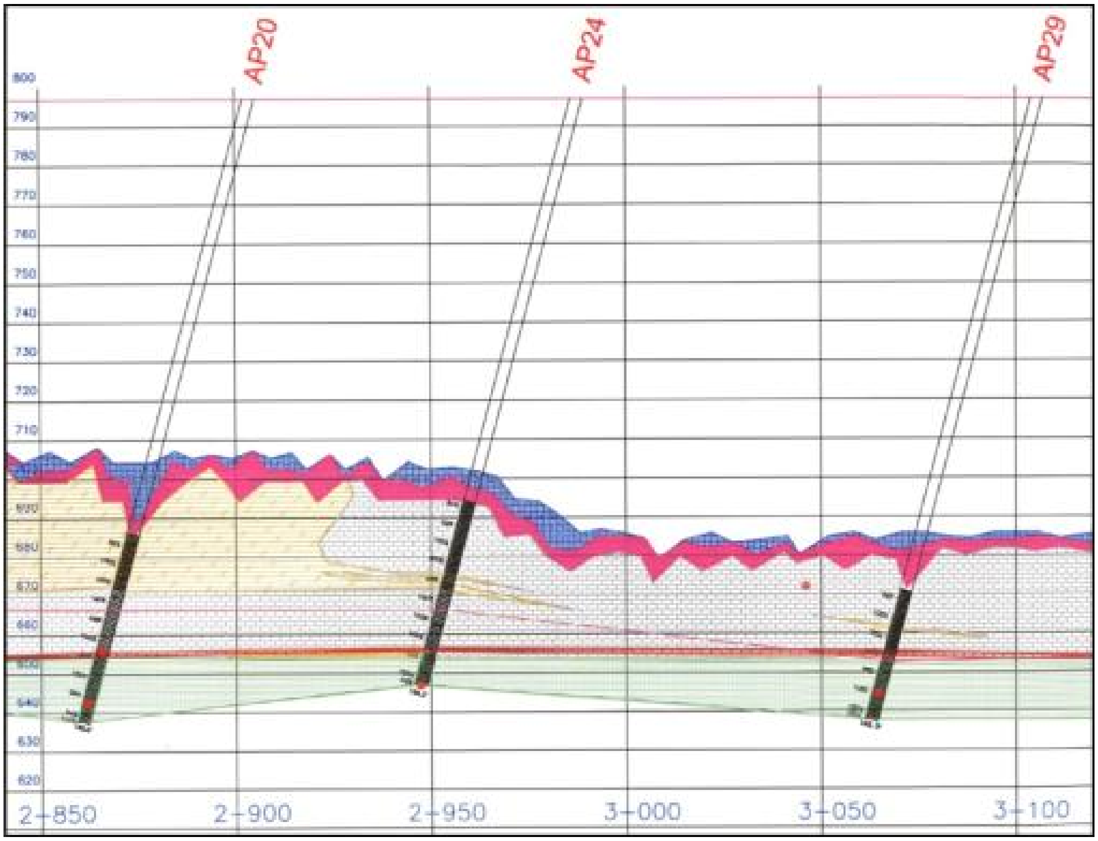
Description	Mobilize	Demobilize	Drill Setups	Exploratory Drilling	Production Drilling	Connections to Grout Holes	Pressure Testing	Washing	Placing Grout	Portland Cement	Flyash	Bentonite	Viscosity Modifier
Scheduled Quantity	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Unit	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Unit Price	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Scheduled Value	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Quantity to Date	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Value to Date	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Percent of Scheduled Value	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Figure 16-8. Quantity summary spreadsheet for quantity projections.

16-10. Summary Project Records.

a. General. Effective grouting information management starts with a preconstruction data set with interpretations based on that data and involves continuously building on and refining both the original data set and interpretations through collection and use of large amounts of diverse types of additional data acquired through the grouting program. When properly managed, interim data summaries will be maintained and available at all times to refine geologic interpretations, facilitate analysis of the grouting program effectiveness, and guide execution of the program. Interim summary records are the basis for periodic technical reviews by on-site and off-site staff. If the records are continuously updated and maintained, many of the project completion records will be complete at the end of the project. Described here are examples of summary records used on previous grouting projects.

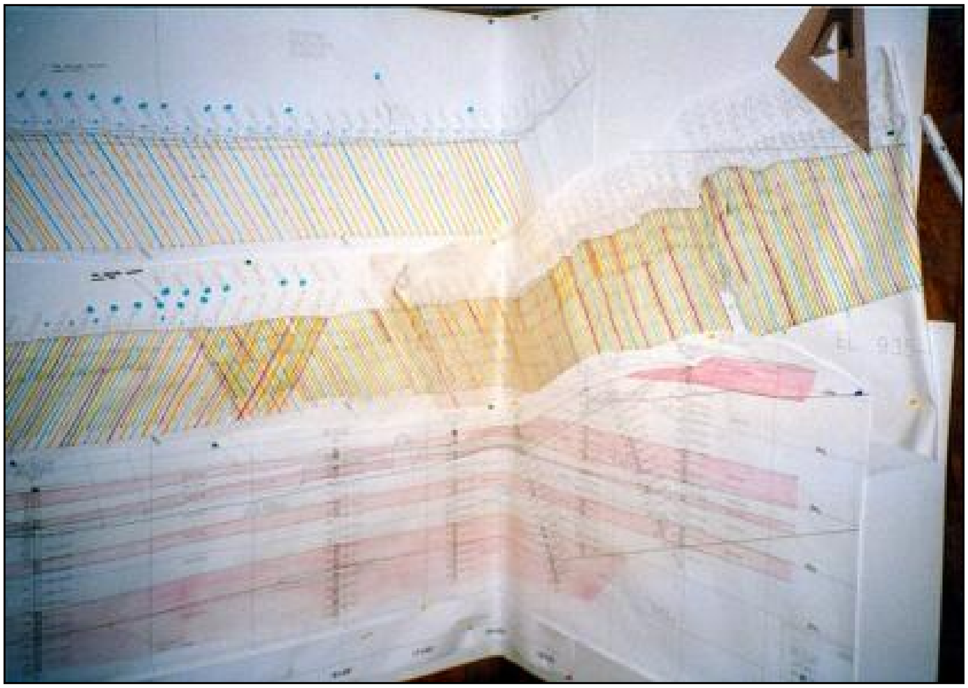
b. Geologic Profiles. Because the geology generally controls the extent of required treatment, geologic profiles generated as a result of the grouting program provide significant insight into how the site conditions are affecting the treatment process. Typically considerable effort and money are expended for exploratory drilling and testing, or for detailed logging of production grout holes on a grouting project, and the final step in summarizing these data is the detailed profile or section. The profiles should clearly illustrate the locations of known geologic contacts and anomalies such as weathered zones and solution features. Hand-drawn profiles have been the norm, but computerized drafting programs lend themselves to easy editing of the profile throughout the course of work, easy transmission of information to off-site personnel, and a more professional-looking product. The use of GIS technology, which is also capable of generating the graphic representations, can add analytical power and data management flexibility. Stick boring logs showing testing values, core recoveries and RQD, and interpreted conditions from both preconstruction logs and investigations conducted during the course of the grouting work should be included. Figure 16-9 shows an example of an as-built geologic profile being developed as the work progresses. For illustration purposes, only the locations of the primary holes are shown.



(Courtesy of Gannett Fleming)

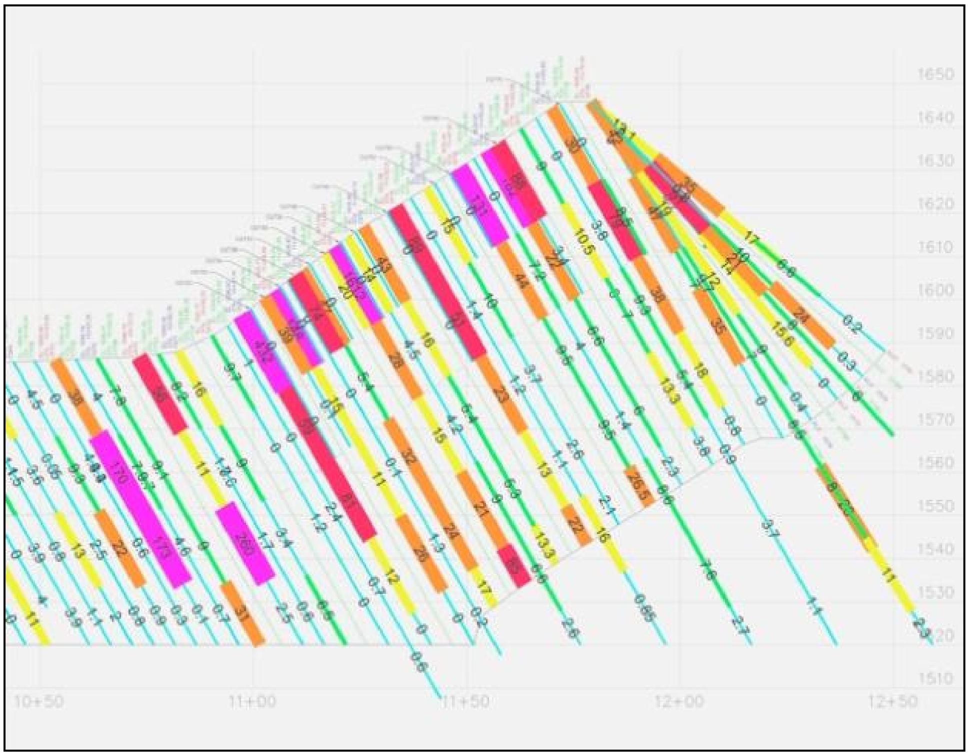
Figure 16-9. Detailed geologic profile being developed as part of grouting.

c. Pressure Testing Graphs and Grouting Profiles. Graphical representations of the water pressure testing and grouting results provide an excellent summary of where high-permeability zones were encountered and where areas of more extensive treatment, as evidenced by the volume of grout placed, is required or was performed. The intent of the profiles is to evaluate correlations from the water pressure testing and grouting information, and other available data such as locations of geologic units, depth of weathering, water loss, and connected zones, and to generally provide an overall view of the conditions encountered and work performed. Hand-drawn profiles may be produced to illustrate such information, but they require significant effort, and the final product often appears as a confusing mix of numbers and lines that often do not easily convey the conditions encountered. Computer drafting programs produce a more flexible and professional-looking product, and the layering capability of the software allows for selective display of the plotted data that greatly simplifies the visual correlation of various data sets. Figure 16-10 shows an example of a hand-drawn profile. Figures 16-11 and 16-12 show examples of computer-generated profiles.



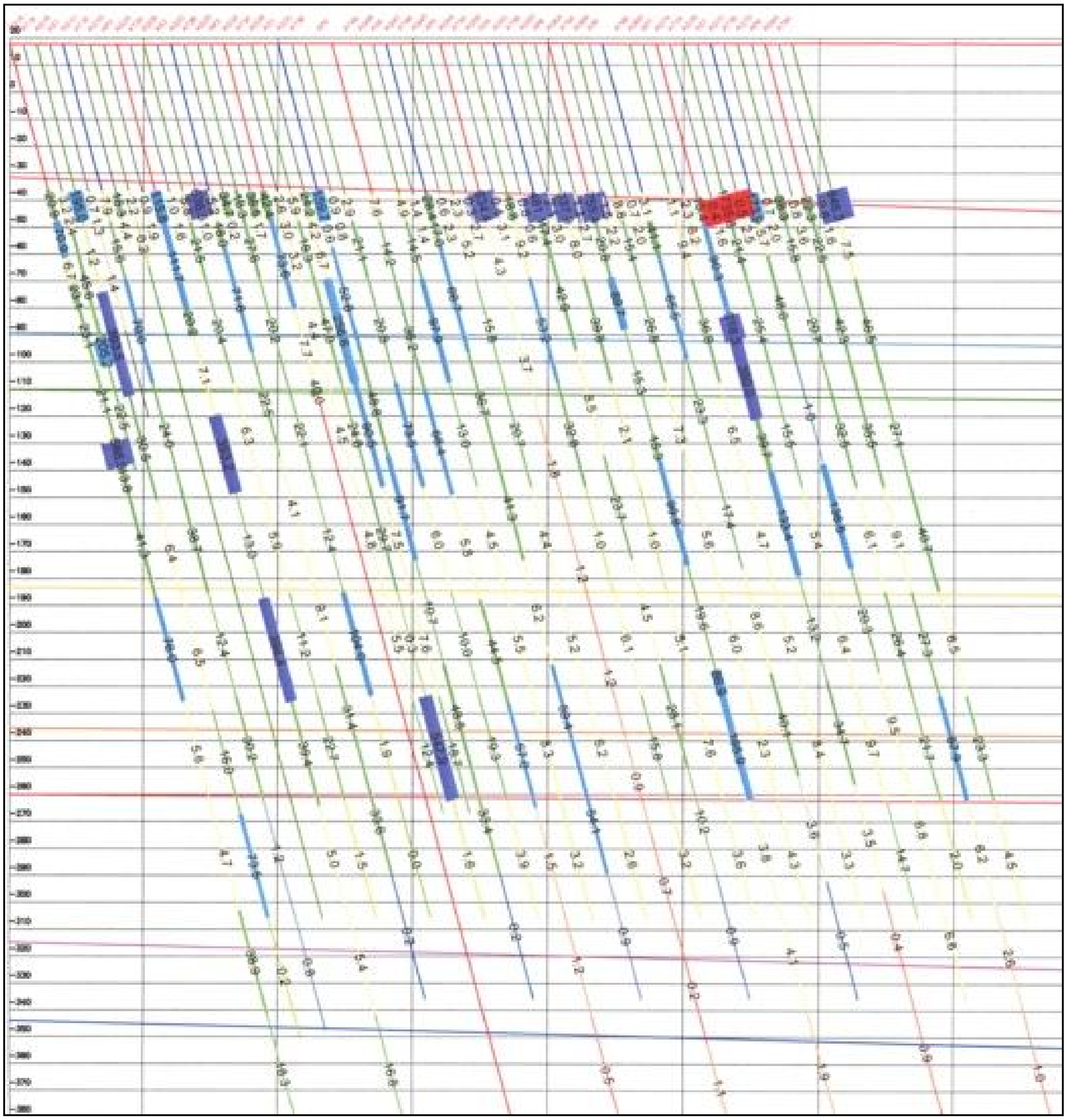
(Courtesy of Gannett Fleming)

Figure 16-10. Hand-drawn profile containing all progress and results on a single sheet.



(Courtesy of Gannett Fleming.)

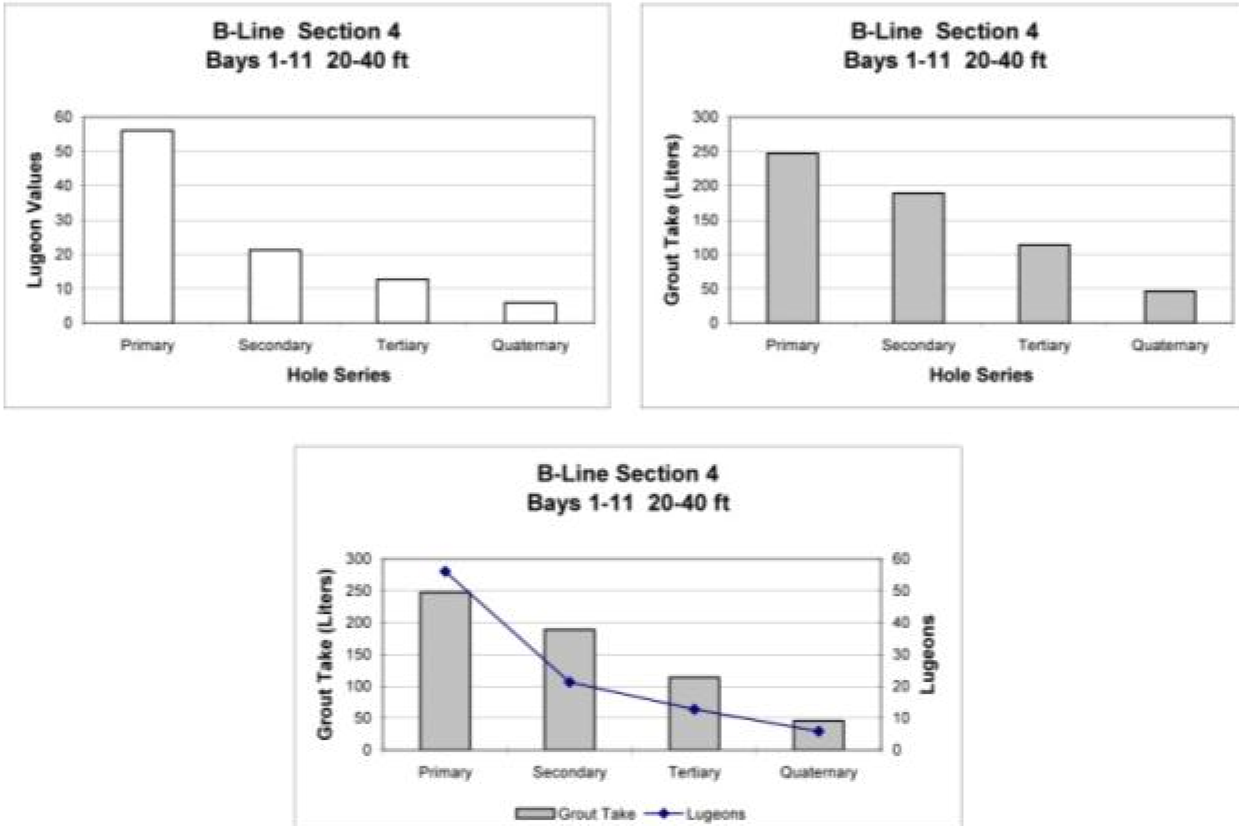
Figure 16-11. Computer-generated profile showing water pressure testing results with scaled color coding.



(Courtesy of Gannett Fleming.

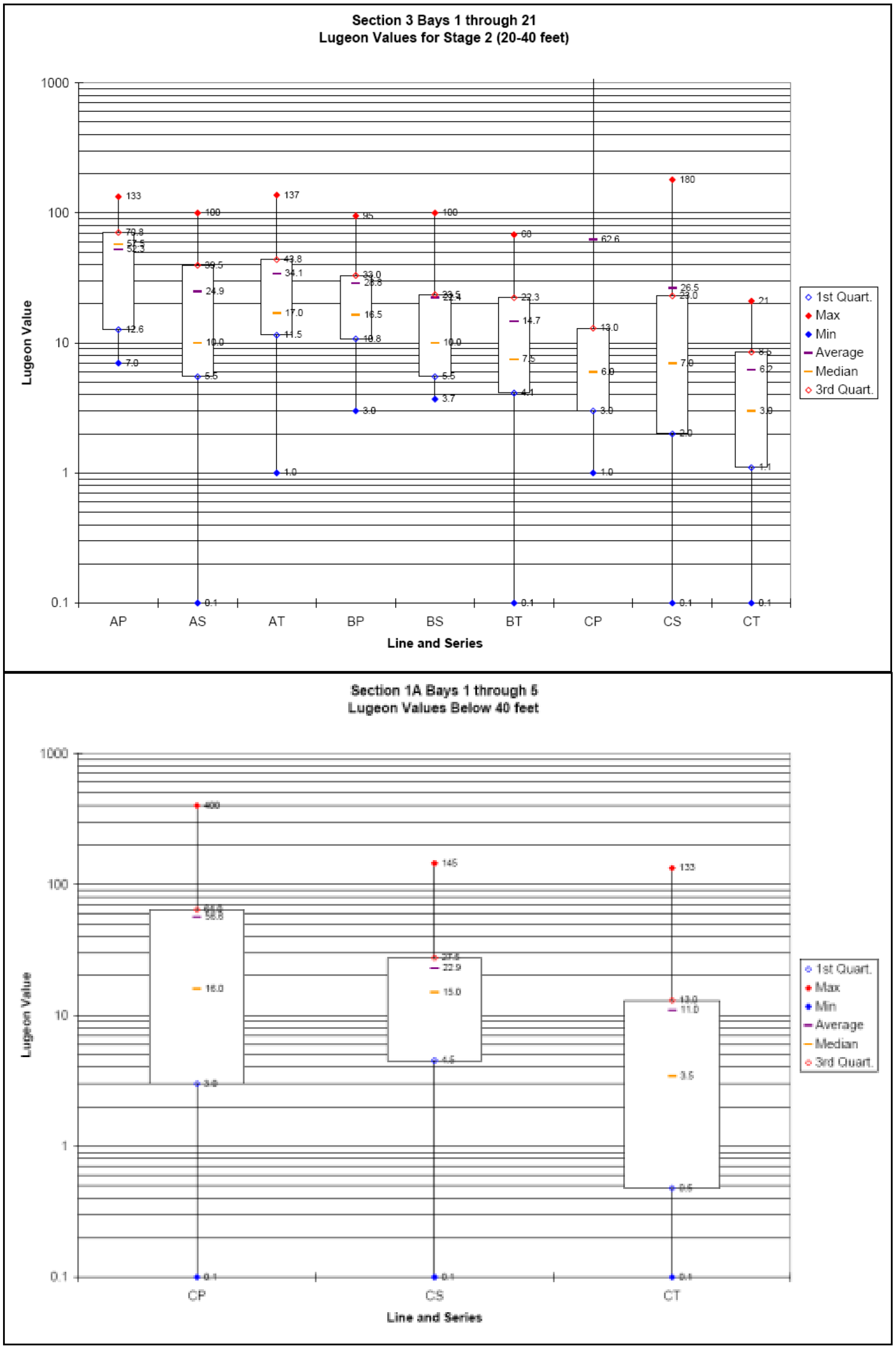
Figure 16-12. Computer-generated grouting results profile.

d. Closure Reports. Under ideal circumstances, split spacing of subsequent hole series throughout the grouting program will systematically reduce the permeability and grout takes. Evaluation of the magnitude of reduction of Lugeon values and grout takes and estimation of the final permeability within the curtain is done using closure analyses. Chapter 15 of this manual discusses closure analyses, including pitfalls in performing them. There are many ways to present the results of the analyses, and the best way for a particular project usually evolves over the course of the program. It is also common to look at “closure” of related items such as water levels, incidence of water loss, and grouting times, in a similar fashion (and concurrently) to ensure that all data are consistent in showing that the desired closure is, in fact, being achieved. Figures 16-13 through 16-23 show examples of various types of closure reports.



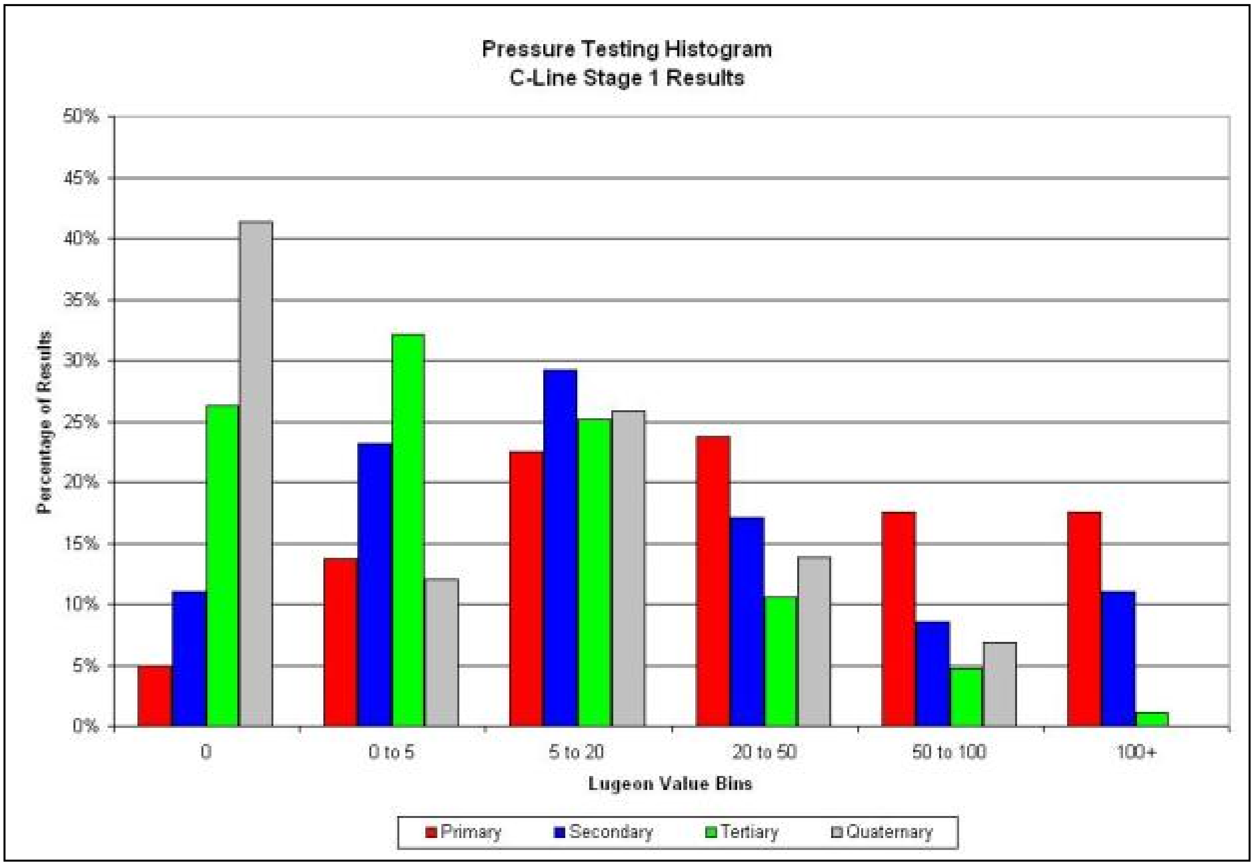
(Courtesy of Gannett Fleming)

Figure 16-13. Simple average value closure diagrams: pressure test results, grout take, and combined plot.



(Courtesy of Gannett Fleming)

Figure 16-14. Closure report graphs produced using spreadsheets and statistical functions.



(Courtesy of Gannett Fleming.)

Figure 16-15. Closure report histogram summarizing the frequency of measured Lugeon values.

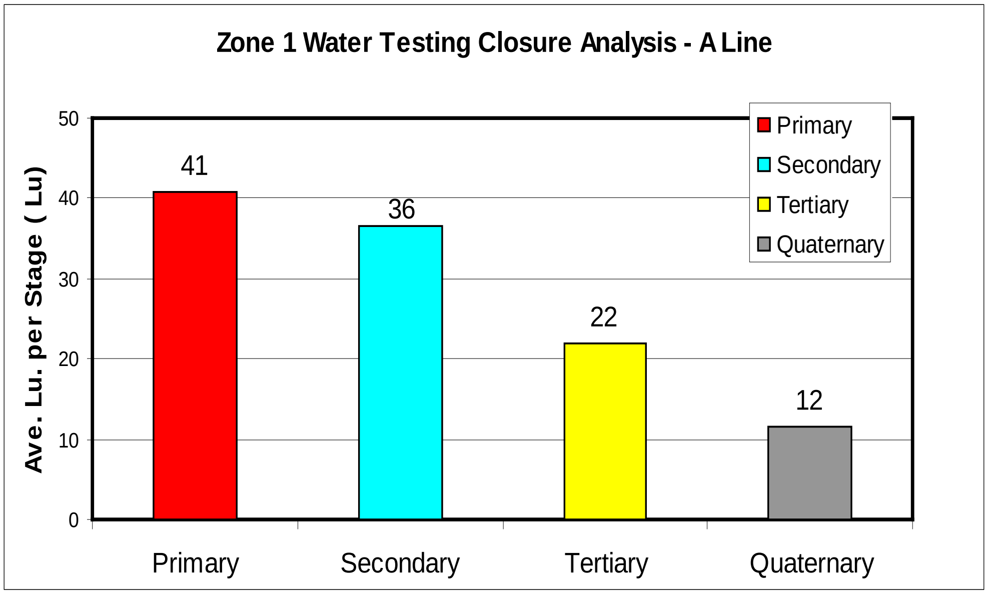


Figure 16-16. Closure plot for water testing.

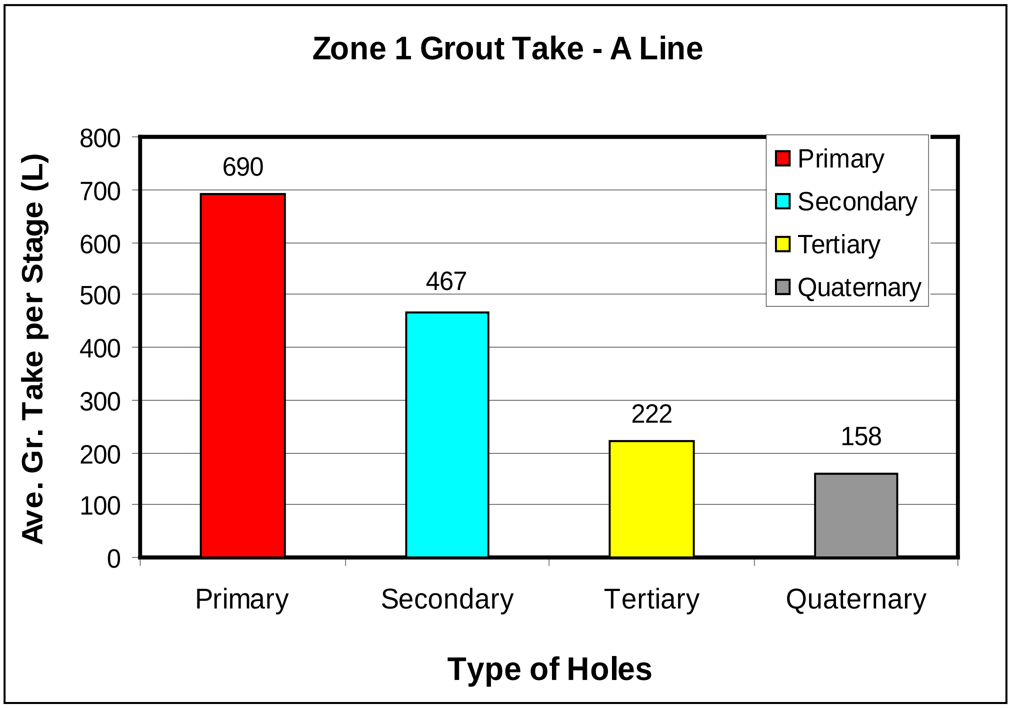
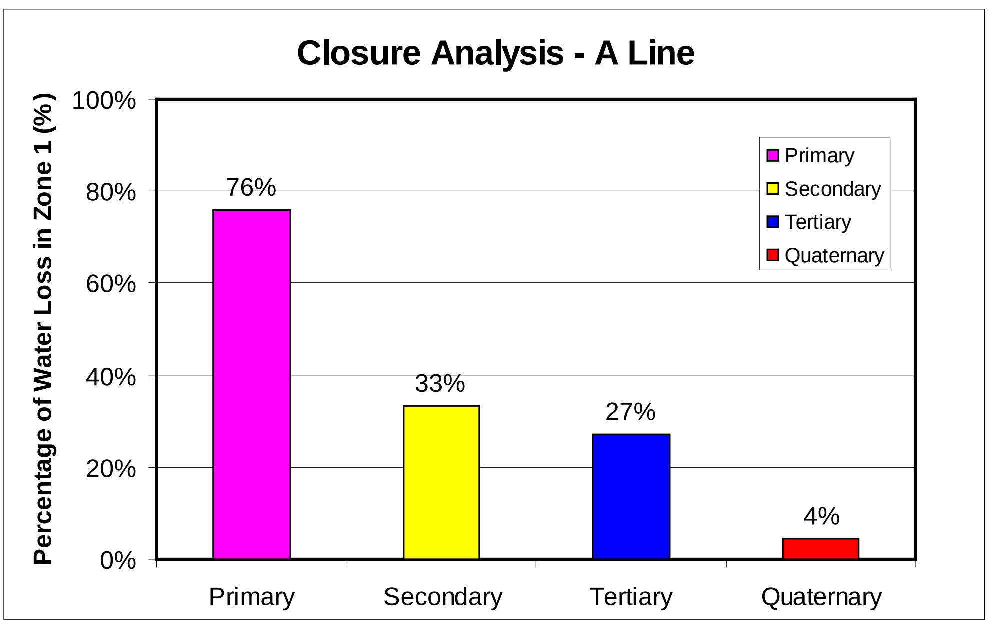


Figure 16-17. Closure plot for grout take.



(Courtesy of Gannett Fleming.)

Figure 16-18. Closure plot for the percentage of stages with water loss during drilling.

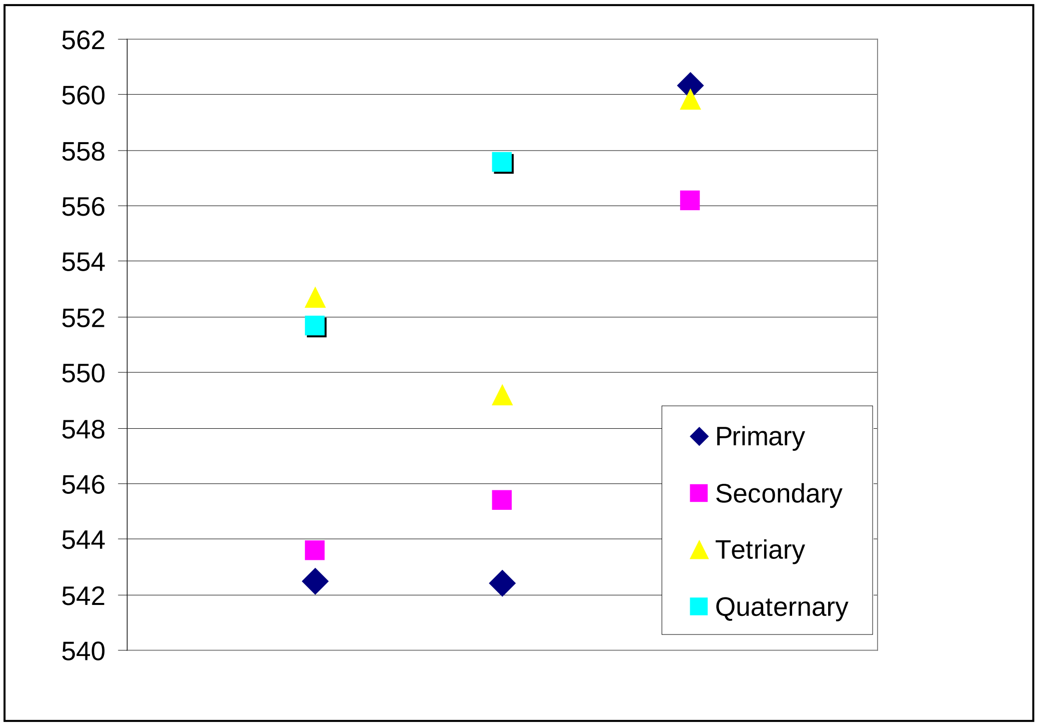
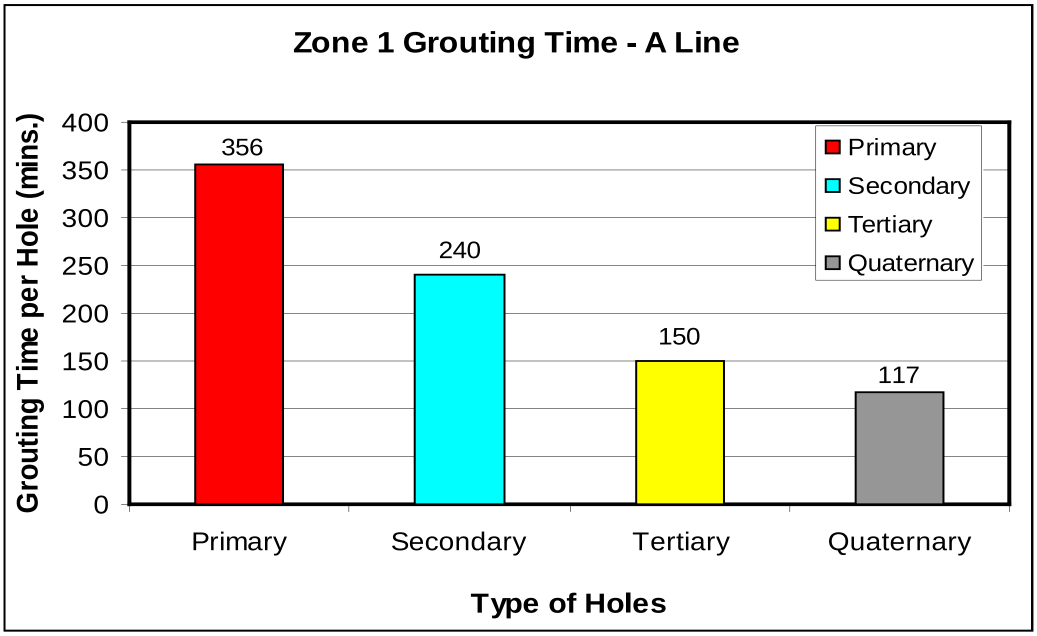
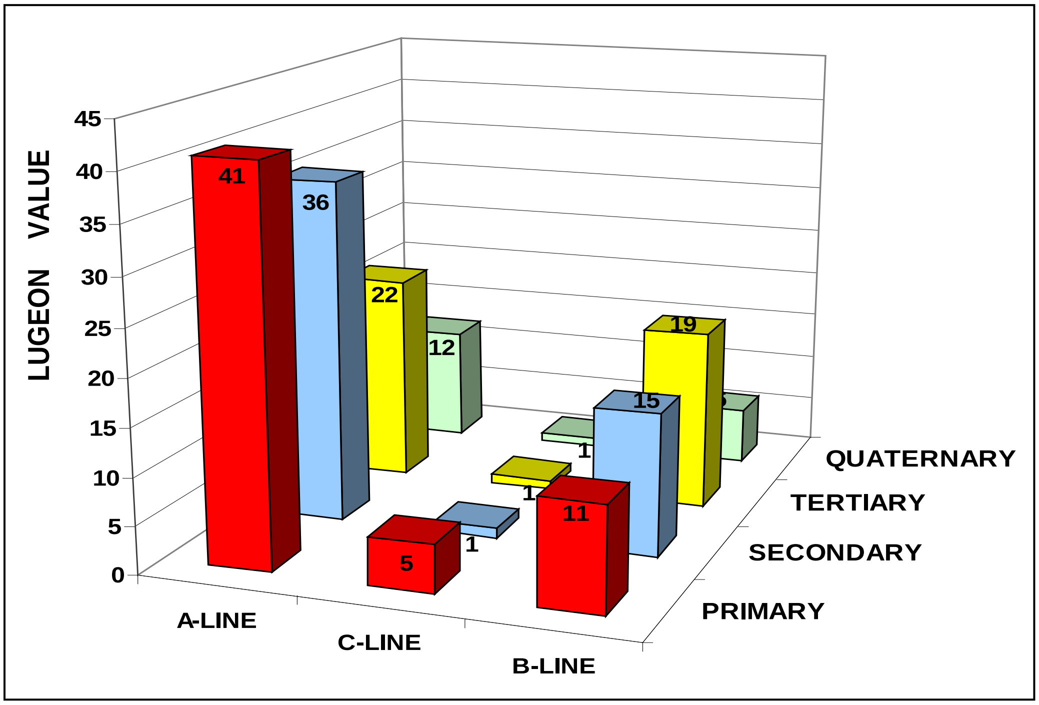


Figure 16-19. Water level profiles upstream of a barrier indicative of changes in piezometer readings after the completion of each series of holes.



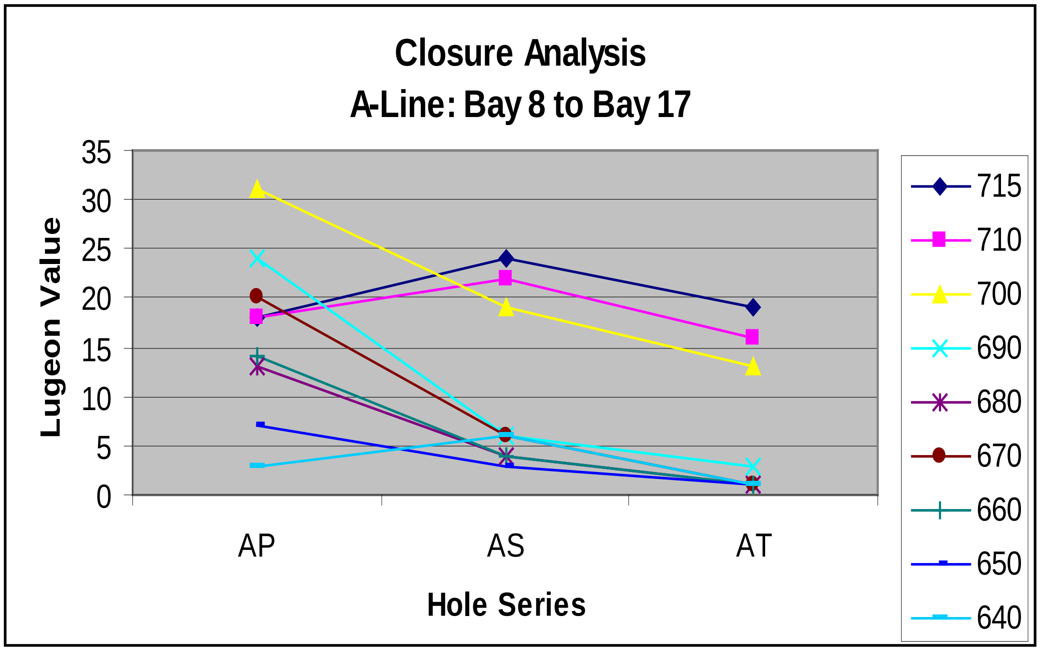
(Courtesy of Gannett Fleming)

Figure 16-20. Closure plot for grouting time per hole.



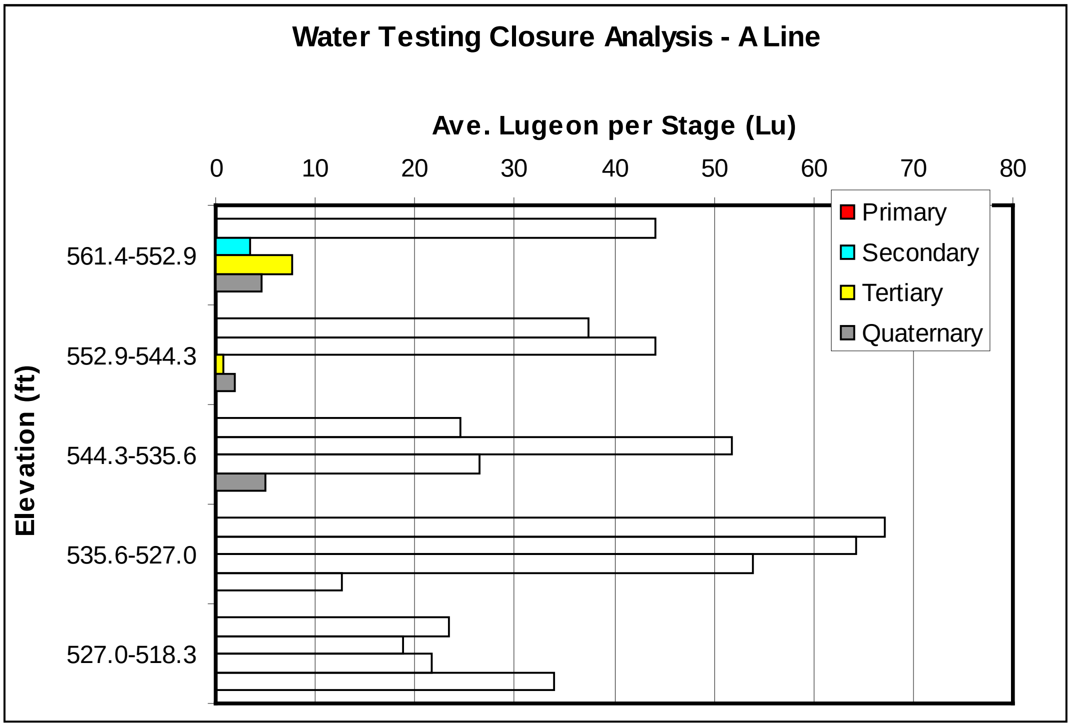
(Courtesy of T. Flaherty, P.G.)

Figure 16-21. Closure plot for multiple lines (work sequence was A-Line, B-Line, C-Line).



(Courtesy of Gannett Fleming)

Figure 16-22. Closure analysis by hole series and by elevation of center of stages.



(Courtesy of Advanced Construction Techniques)

Figure 16-23. Closure plots by vertical elevation and hole series.

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CHAPTER 17

Grouting in Karst Environments

17-1. General. Karst formations are three-dimensional landscapes that develop within and over soluble rocks. Karst features are the result of mildly acidic water acting on soluble rock such as limestone, dolomite, gypsum, and anhydrite. Over geologic time, the acidic water dissolves rock surfaces, and flowing water within the rock fracture system enlarges the fractures. Eventually, it can form extremely complex, heterogeneous conditions within the overlying soils, along the bedrock surface, and within the rock that can be problematic for the construction of structures of all types.

17-2. Dissolution Rates and Their Impacts on Grouting Approach.

a. Limestone and Dolomite. The rate of dissolution of limestone and dolomite is normally quite slow. While effective treatment of existing dissolution features might be essential in structural foundation grouting applications, ongoing dissolution of rock is of no practical consequence over the life of the structure in normal environments where flow gradients are low. Until relatively recently, the dissolution of limestone or dolomite and the enlargement of fractures beneath dams were also considered to be processes that occurred so slowly that it was not a major issue over the life of a dam. However, numerical modeling studies of fracture systems under high gradients now suggest that it is possible for active dissolution to be of sufficient magnitude to produce unacceptable seepage rates well within the life of a dam (Romanov et al. 2002, 2006). Although many factors are involved, the initial open fracture size is a key parameter affecting dissolution rates. Studies by James and Kirkpatrick (1980) suggest that the smallest fracture size that might be significant for progressive dissolution to occur in limestone is in the range of 0.5 mm. The erosion of infill material far outweighs the concern for additional dissolution over the life of the project.

b. Gypsum and Anhydrite. Gypsum and anhydrite formations are less common than limestone and dolomite, but they are subject to highly significant active dissolution during the useful life of structures. In gypsum, it has been estimated that cavities on the order of 1 m can be created in a 100-year period (Waltham and Fookes 2002). Additionally, the smallest initial open fracture size that is significant for progressive dissolution to occur is less than for limestone and dolomite and is on the order of 0.2 mm for gypsum and 0.1 mm for anhydrite.

c. Grouting Implications.

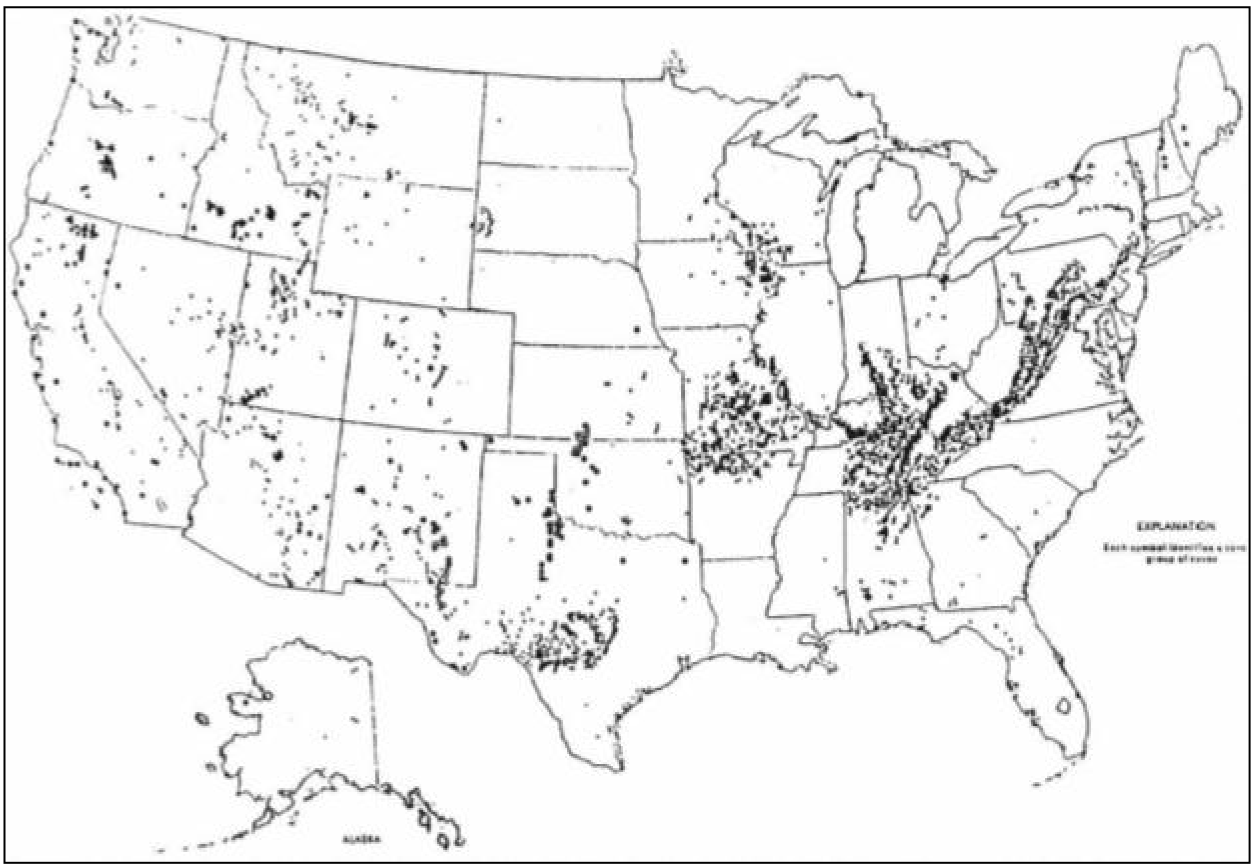
(1) Importance of Grouting Fine Fractures. The potential for ongoing dissolution during the useful life of a dam is a serious factor for its long-term performance. The practical implication is that intensive grouting of the entire fracture system is critical. The importance of properly treating small fractures should not be overlooked or overshadowed by the presence of more dramatic features. Normal cement grouting has the ability to fill the critical fracture sizes in limestone, dolomite, and gypsum, and if properly performed, it will prevent progressive dissolution. Sealing the smaller critical fractures in anhydrite would require ultrafine cements, chemical grouts, or cutoff structures as an alternative to grouting.

(2) Refusal Criteria. A second grouting implication is that the grouting refusal criteria should be based on achieving absolute refusal when the trend plot for the stage supports this action. The use of criteria that permit termination of grouting before that point can result in residual permeabilities that may affect the long-term performance of the dam.

(3) Termination Depth for Grouting. Careful consideration must be given to the termination depth for grout curtains in soluble rock. Whenever possible, the curtains should tie into zones with a hydraulic conductivity comparable to the completed curtain. If that is not possible, the potential long-term effects of seepage beneath the bottom of the curtain must be carefully evaluated and considered.

17-3. Karst Areas, Profiles, and Features.

a. Karst Regions. Karst topography develops in predictable limestone and dolomite regions in the United States. Figure 17-1 shows a map of the distribution of cavern areas in the United States. Table 17-1 lists the major karst areas along with the general characteristics of each region.



(From EM 1110-1-2908, *Engineering and Design – Rock Foundations*.)

Figure 17-1. Karst areas of the United States.

Table 17-1. Summary of major karst areas of the United States.

Karst Area*	Location	Characteristics
Southeastern Coastal Plain	South Carolina, Georgia	Rolling, dissected plain, shallow dolines, few caves; tertiary limestone generally covered by thin deposits of sand and silt
Florida	Florida, southern Georgia	Level to rolling plain: tertiary, flat-lying limestone; numerous dolines, commonly with ponds; large springs; moderate-sized caves, many water filled
Appalachian	New York, Vermont, south to northern Alabama	Valleys, ridges, and plateau fronts formed south of palaeozoic limestones, strongly folded in eastern part; numerous large caves, dolines, karst valleys, and deep shafts; extensive areas of karren
Highland Rim	Central Kentucky, Tennessee, northern Georgia	Highly dissected plateau with carboniferous, flat-lying limestone; numerous large caves, karren, large dolines and uvala
Lexington-Nashville	North-central Kentucky, central Tennessee, southeastern Indiana	Rolling plain, gently arched; lower palaeozoic limestone; a few caves, numerous rounded shallow dolines
Mammoth Cave-Pennyroyal Plain	West-central Kentucky, southwestern Kentucky, southern Indiana	Rolling plain and low plateau; flat-lying carboniferous rocks; numerous dolines, uvala and collapse sinks; very large caves, karren developed locally, complex subterranean drainage, numerous large “disappearing” streams
Ozarks	Southern Missouri, northern Arkansas	Dissected low plateau and plain; broadly arched lower palaeozoic limestones and dolomites; numerous moderate-sized caves, dolines, very large springs; similar, but less extensive karst in Wisconsin, Iowa, and northern Illinois
Canadian River	Western Oklahoma, northern Texas	Dissected plain, small caves and dolines in carboniferous gypsum
Pecos Valley	Western Texas, southeastern New Mexico	Moderately dissected low plateau and plains; flat-lying to tilted upper palaeozoic limestones with large caves, dolines, and fissures; sparse vegetation; some gypsum karst with dolines
Edwards Plateau	Southwestern Texas	High plateau, flat-lying cretaceous limestone; deep shafts, moderate-sized caves, dolines; sparse vegetation

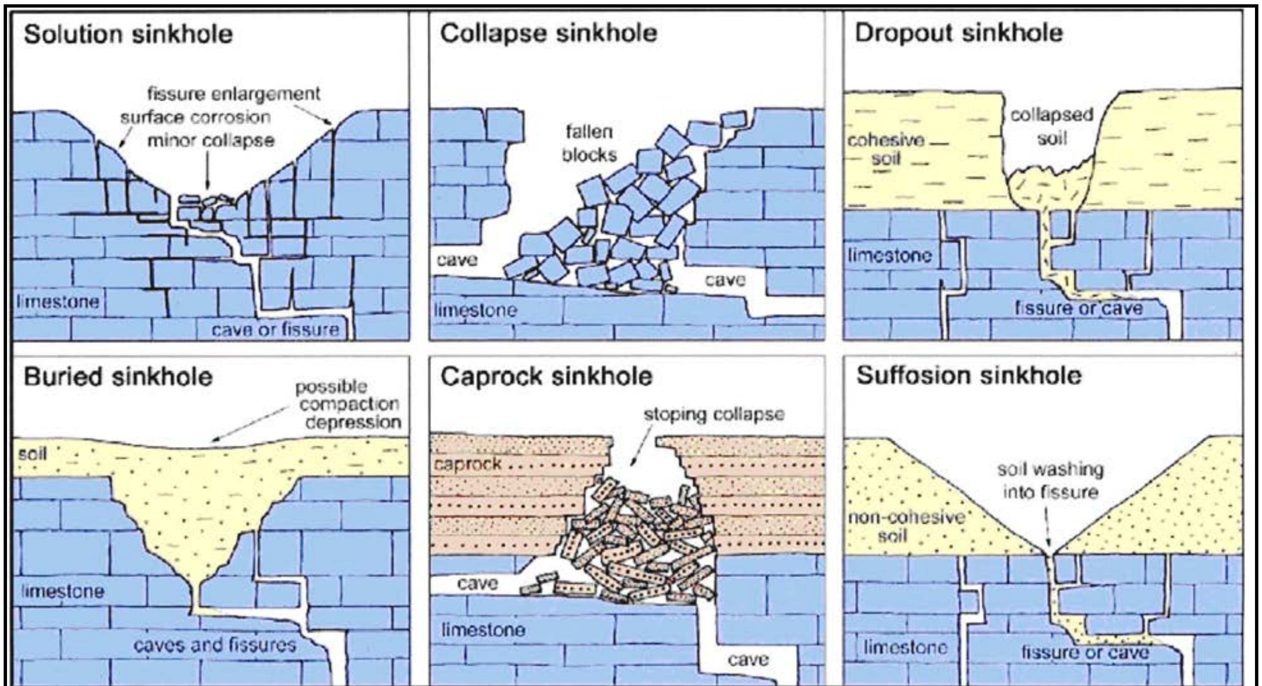
Table 18-1. (Continued).

Black Hills	Western South Dakota	Highly dissected ridges; folded (domed) palaeozoic limestone; moderate-sized caves, some karren and dolines
Kaibab	Northern Arizona	Partially dissected plateau, flat-lying carboniferous limestones; shallow dolines, some with ponds; few moderate-sized caves
Western mountains	Wyoming, northwestern Utah, Nevada, western Montana, Idaho, Washington, Oregon, California	Isolated small areas, primarily on tops and flanks of ridges, and some area in valleys; primarily in folded and tilted palaeozoic and mesozoic limestone; large caves, some with great vertical extent, in Wyoming, Utah, Montana, and Nevada; small to moderate-sized caves elsewhere; dolines and shafts present; karren developed locally
*From EM 1110-1-2908, *Engineering and Design – Rock Foundations*.		

b. Karst Physical Features. Karst features within any given region are extremely variable, and both the nature of features and the distribution of features can vary greatly even within the limits of any given project site. The nature and size of karst features that might be present are a function of many factors associated with particular formations, including their age, chemical composition, structure, climatic region, prior exposure or protection, fracture system, and hardness. The following paragraphs describe common types of karst features that might be present and of concern. (Not all of these features are necessarily present on any given site.)

(1) Undetected Overburden Voids. Voids can exist in the soil above the bedrock surface with no visible surface expression. Water flowing into or through fractures at the bedrock surface has the ability to gradually remove materials, and the soil can form an arch above the point of initial material loss. With time, the void will gradually enlarge through cycles of wetting, drying, and incremental collapse.

(2) Overburden Sinkholes. Sinkholes, also termed “dolines,” are closed depressions in the overburden above the rock and are the expression of material loss into the fracture system. Sinkholes can be broad and shallow or abrupt and steep; they can form gradually or suddenly; they can be uniformly distributed over an area or they might follow specific trend lines or trend zones; and their bottoms can extend to the rock surface or be contained within the overburden. The maximum diameter of sinkholes that can form is generally related to the depth to rock. Figure 17-2 shows various sinkhole types.



A classification of sinkholes, with respect to the mechanisms of the ground failure and the nature of the material which fails and subsides; these features are also known as dolines (in the same six classes). The two types on the right may be known collectively as subsidence sinkholes. The structures, cave patterns and sinkhole profiles tend to be more complex in dipping limestone, but the concepts remain the same as those shown by these examples in horizontal limestone; except that the caprock sinkhole cannot exist in conformable vertical beds.

(From Waltham and Fookes 2003, with permission from the Geological Society, Piccadilly, London.)

Figure 17-2. Sinkhole types.

(3) Loose and/or Wet Overburden Zones. In some karst environments, it is common to find moderately dense soil materials overlying loose and/or wet overburden zones above the rock surface. These zones may also contain voids and/or remnants of solutioned rock. These sometimes occur between rock pinnacles, but they can also extend for a considerable distance above the rock surface. These zones are a result of material loss into solution cavities in the immediately underlying bedrock.

(4) Pinnacled Rock Surfaces. Carbonate rocks can develop a profile with an abruptly changing depth to the contact between the residual soil and the weathered rock surface. Jagged, pinnacled rock surfaces develop due to weathering along nearly vertical fracture systems. The pinnacled surfaces can be closely spaced and frequent, or they may occur as somewhat isolated features.

(5) Detached or Semi-Detached Rock Masses. Depending on the nature and extent of weathered fracture systems, detached or semi-detached rock masses can exist in the foundation. Under extreme conditions, they can behave as floating boulders or as very weak ledges of material.

(6) Large Variations in Top of Rock Level. Aside from containing pinnacled rock surfaces, some sites can also exhibit areas where the top of rock level varies dramatically on a

larger scale. There might not be any visible surface indications that these large variations in depth to rock exist.

(7) Soil-Filled Fracture Systems. Infilled or partially infilled fracture systems are common in karst. The infilling materials can consist of insoluble weathering residue or materials that have been transported into the fractures. Possible materials include stiff clay seams, soft clays, silts, sands, and rock detritus.

(8) High-Permeability Fracture Systems. Even in the absence of discrete cavities, bedrock in karst regions normally contains high-permeability fracture systems. The permeable fractures may have highly preferential orientations with strong interconnections to other fractures.

(9) Cavities. Cavities form in the bedrock through long-term dissolution widening of specific fractures, joints, or bedding planes. Lesser fractures feed water to these larger fractures through the fracture system interconnections. Cavities can range from small individual features to extremely large interconnected systems. Cavities can contain all types of materials including collapsed rock.

c. Karst Feature Illustrations. Some of the most useful aids in understanding and appreciating the range of features that can be encountered in various karst environments are schematics and photos. Table 17-2 and Figure 17-3 show an engineering classification of karst, and Figure 17-4 shows a schematic of a karst profile. Figures 17-5 to 17-25 show many kinds of karst environments.

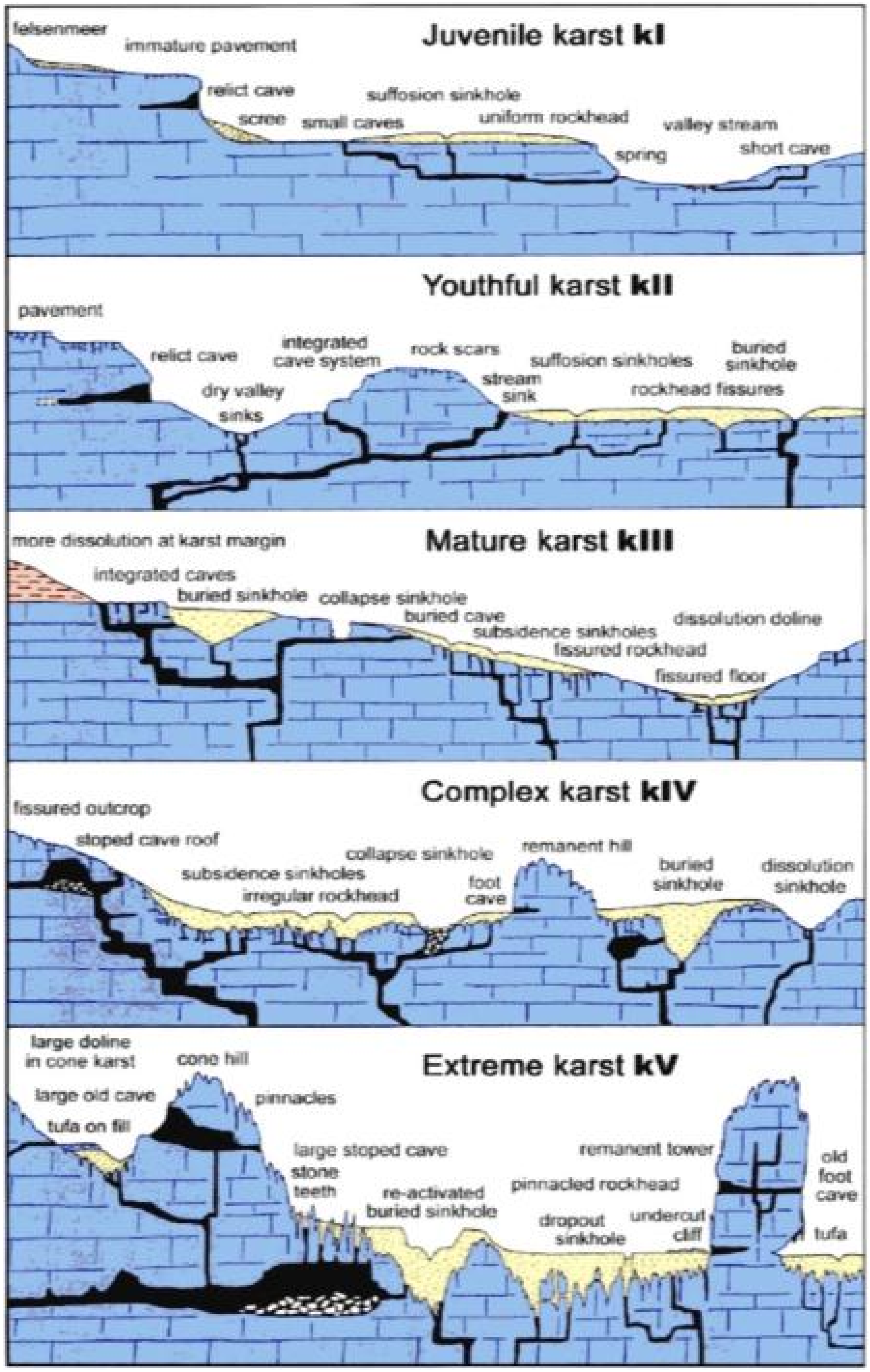
Table 17-2. Engineering classifications and characteristics of karst types, best viewed in conjunction with Fig. 17-3.

Karst class*	Locations	Sinkholes	Rockhead	Fissuring	Caves	Ground investigation	Foundations
kI Juvenile	Only in deserts and periglacial zones or on impure carbonates	Rare NSH <0.001	Almost uniform; minor fissures	Minimal; low secondary permeability	Rare and small; some isolated relict features	Conventional	Conventional
kII Youthful	The minimum in temperate regions	Small suffusion or dropout sinkholes; open stream sinks NSH = 0.001–0.05	Many small fissures	Widespread in the few meters nearest the surface	Many small caves; most <3 m across	Mainly conventional, probe rock to 3 m; check fissures in rockhead	Grout open fissure; control drainage

Table 17-2. Engineering classifications and characteristics of karst types, best viewed in conjunction with Fig. 17-3.

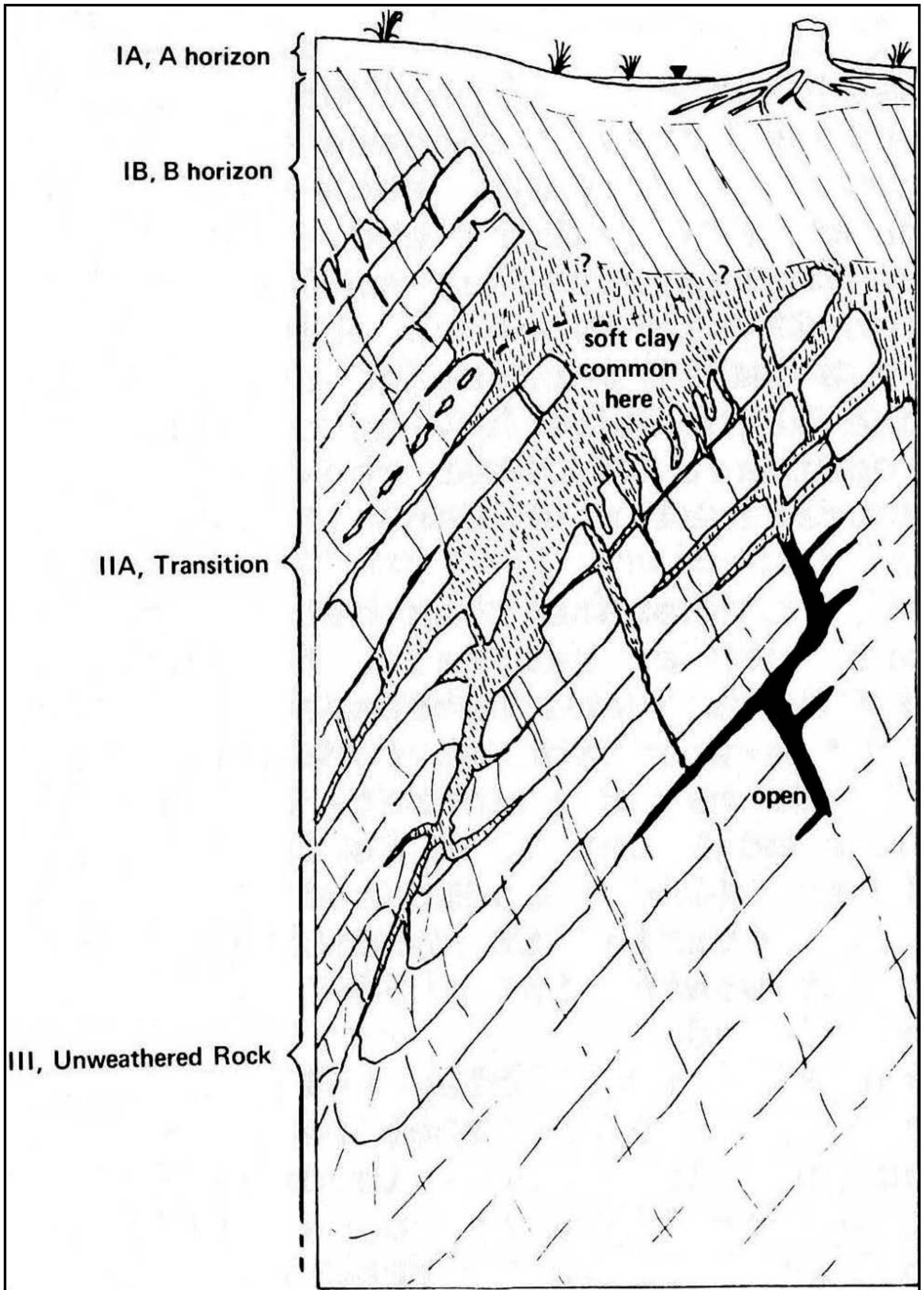
kIII Mature	Common in temperate regions; the minimum in the wet tropics	Many suffosion and dropout sinkholes; large dissolution sinkholes; small collapse and buried sinkholes NSH = 0.05–1.0	Extensive fissuring; relief of <5 m; loose blocks in cover soil	Extensive secondary opening of most fissures	Many <5 m across at multiple levels	Probe to rockhead; probe rock to 4 m; microgravity survey	Rafts or ground beams; consider geogrids, driven piles to rockhead; control drainage
kIV Complex	Localized in temperate regions; normal in tropical regions	Many large dissolution sinkholes; numerous subsidence sinkholes; scattered collapse and buried sinkholes NSH = 0.5–2.0	Pinnacled; relief of 5–20 m; loose pillars	Extensive large dissolutional openings, on and away from major fissures	Many >5 m across at multiple levels	Probe to rockhead; probe rock to 5 m with splayed probes; microgravity survey	Bored piles to rockhead or cap grouting at rockhead; control drainage and abstraction
kV Extreme	Only in wet tropics	Very large sinkholes of all types; remnant arches; soil compaction in buried sinkholes NSH >>1	Tall pinnacles; relief of >20 m; loose pillars undercut between deep soil fissures	Abundant and very complex dissolution cavities	Numerous complex 3-D cave systems; galleries and chambers >15 m across	Make individual ground investigation for every pile site	Bear in soils with geogrid; load on proven pinnacles or on deep bored piles; control all drainage and control abstraction
*After Waltham and Fookes (2003)							

Table 17-2 provides outline descriptions of selected parameters; these are not mutually exclusive and give only broad indications of likely ground conditions that can show enormous variation in local detail. The comments on ground investigation and foundations are only broad guidelines to good practice in the various classes of karst. NSH = rate of formation of new sinkholes per km^2 per year.



(From Waltham and Fookes 2003, with permission from the Geological Society, Piccadilly, London)

Figure 17-3. Schematics of karst classifications. Typical morphological features of karstic ground conditions with the five classes of karst (Table 17-2). These examples show horizontal bedding of the limestone; dipping bedding planes and inclined fractures add complexity to most of the features and also create planar failures behind steep cliff faces. The dotted ornament represents any type of clastic soil or surface sediment.



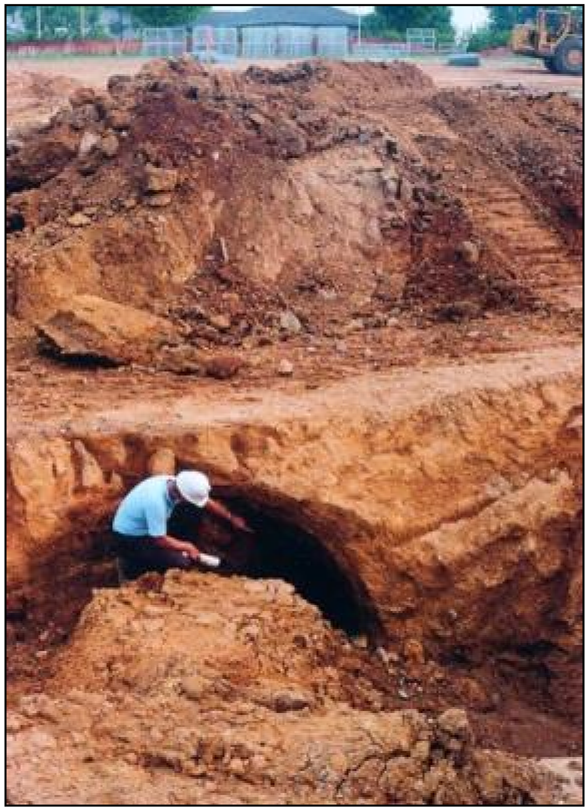
NOTES:

1) Very impure (sandy or silty) carbonates may develope a saprolite, IC zone

2) A partly weathered chalky limestone, IIB zone is sometimes present.

(From Deere and Patton 1971, with permission from ASCE.)

Figure 17-4. Schematic of karst profile.



(Courtesy of Gannett Fleming)

Figure 17-5. Soil arch in an incipient dropout sinkhole in cohesive soil exposed during excavation in Hershey, PA. No surface expression of void was visible. Before excavation, the surface had been proof-rolled with loaded scrapers and compacted. The site had approximately 35 sinkholes with surface expressions.

Figure 17-6. Sandbagged dropout sinkhole that developed in an emergency spillway approach channel during a flood at Patoka Dam (Louisville District).

(Courtesy of Gannett Fleming)

Figure 17-7. Karst profile at a Pennsylvania quarry wall in folded limestone.

(Courtesy of Gannett Fleming)

Figure 17-8. Near-surface weathering conditions in Pennsylvania limestone.

Figure 17-9. Limestone exposure at Mississinewa Dam (Louisville District).

Figure 17-10. Cambrian dolomite outcrop at Clearwater Dam, Missouri (Little Rock District).

Figure 17-11. Cambrian dolomite outcrop at Clearwater Dam, MO (Little Rock District).

Figure 17-12. Near-surface vertical feature at Center Hill Dam (Nashville District).

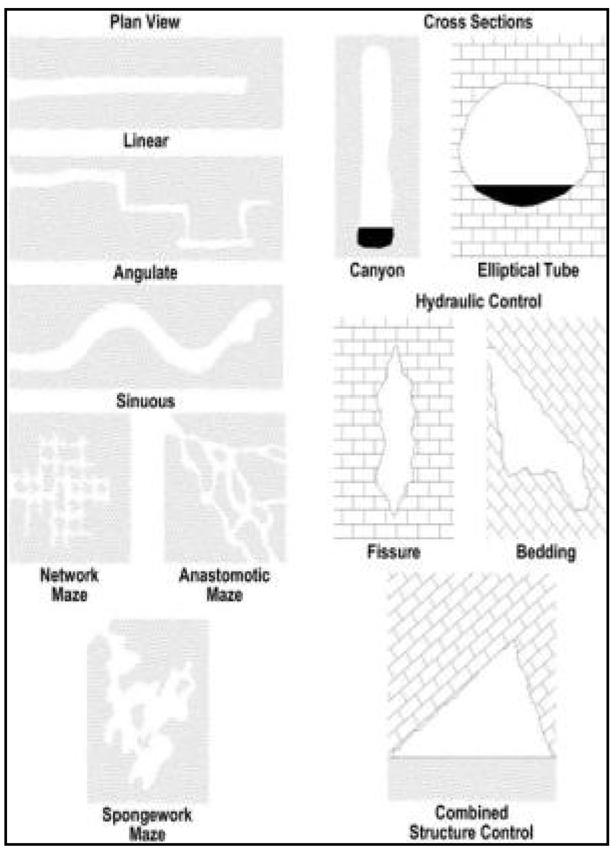
Figure 17-13. Near-surface soil-filled cylindrical features at Center Hill Dam (Nashville District).

Figure 17-14. Outcrop exposure downstream from the right abutment at Center Hill Dam (Nashville District).

Figure 17-15. Isolated dissolution features along the structure at Wolf Creek Dam (Nashville District).

Figure 17-16. Isolated near-vertical dissolutioning along the structure at Wolf Creek Dam (Nashville District).

Figure 17-17. Large expanse of minimally dissolutioned rock at Center Hill Dam (Nashville District).



(After Ritter 1978.)

Figure 17-18. Plan and section schematics of cave type features.

Figure 17-19. Elliptical tube cave features at Center Hill Dam (Nashville District).

Figure 17-20. Entrance to Robert Hall Cave, a fissure-type cave near Patoka Lake Dam (Louisville District).

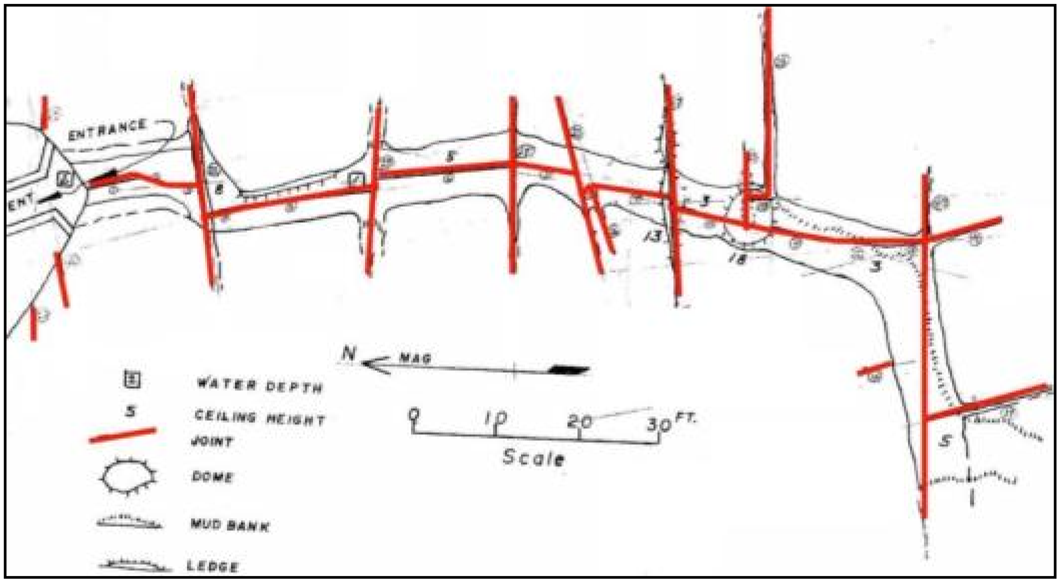
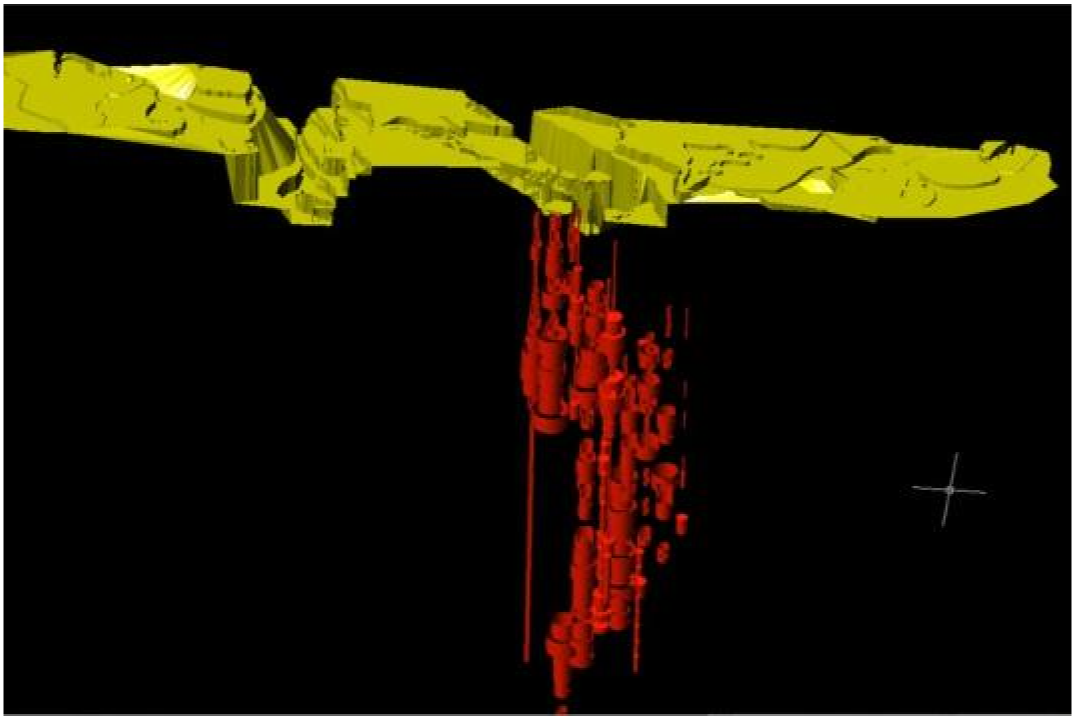


Figure 17-21. Geologist's field map of Robert Hall Cave near Patoka Lake Dam (Louisville District).

Figure 17-22. Triangular dissolution feature in the foundation of Mississinewa Dam, treated at the time of original construction (Louisville District).

Figure 17-23. Triangular dissolution feature in the excavation sidewall of Mississinewa Dam, treated at the time of original construction (Louisville District).

Figure 17-24. Foundation treatment of a network of caves and dissolution channels at Wolf Creek Dam as part of original construction (Nashville District).



(Courtesy of Gannett Fleming)

Figure 17-25. Deep dissolution feature beneath the original foundation of Clearwater Dam. The foundation rock profile shows locations where large intersecting dissolution features crossed the foundation and were treated during original construction. Note that the deep feature was small immediately below the rock foundation, but expanded to large size at depth. The deep feature delineation is based on remedial grouting results.

d. Epikarst.

(1) An additional geologic concept that is gaining widespread use and acceptance, particularly in hydrogeologic applications, is the term “epikarst.” While numerous definitions vary considerably, epikarst is generally agreed to be a karst subsystem composed of the uppermost zone in certain karst profiles that can be differentiated from the rest of the rock mass on the basis of a number of special characteristics. The thickness of the epikarst zone varies considerably, but it is commonly estimated to be between a few meters and 10–15 m thick. Compared to the underlying rock mass, the epikarst zone has a more closely spaced network of fissures. Its hydraulic conductivity can be high and more spatially homogeneous than the rock mass below, which is characterized by more widely spaced fractures between rock blocks. Porosity is typically significantly greater than in the bulk rock located below. Epikarst allows rapid infiltration and can result in detainment or storage of infiltration water. The epikarst funnels water to points of concentration, facilitating the creation of deeply penetrating collector structures.

(2) In the Clearwater Dam cutoff wall pre-grouting program (Little Rock District) in Missouri, the epikarst zone was identified as part of the karst system that required treatment by grouting. Despite the high soil content within this weathered zone, permeabilities were determined to be high. During installation of overburden casings, frequent and total loss of

drilling water occurred. Additionally, it was found that large quantities of grout were required during the overburden casing grouting operation. Based on these conditions, it was determined that separate grouting of the epikarst was required, and it was accomplished using sleeve-port grouting methods using three-port pipe sections in the epikarst and single-port pipe sections in the soils above the epikarst. The thickness of the epikarst zone in each hole was defined on the basis of several drilling characteristics, including occurrence of water losses, mixed soil and rock materials as indicated by intermittent percussion operation of the drilling equipment, and rapid advancement of the drill tooling in weathered rock. The thickness of the epikarst was typically 20–30 ft, and its hydraulic conductivity was often in the range of 50–100 Lugeons. Figure 17-26 shows an epikarst exposure at Clearwater Dam.

Figure 17-26. Clearwater Dam epikarst exposure (Little Rock District).

17-4. <u>Investigation and Characterization of Karst Sites</u>.

a. General. Primary engineering issues for structures in karst areas include providing adequate long-term support for structure foundations, protecting structures against the possible loss of soil beneath or adjacent to structures, and providing hydraulic barriers for dams. General procedures for site investigation and site characterization are contained in EM 1110-1-1804, *Geotechnical Investigations*, and EM 1110-1-2908, *Engineering and Design – Rock Foundations*. EM 1110-1-1802, *Geophysical Exploration for Engineering and Environmental Investigations*, provides additional guidance on other methods. These manuals also provide some specific information related to investigation of karst sites and the efficacy of certain methodologies. However, there are no simple guidelines for investigating sites in karst areas. The intensity of the investigation and the methodologies used must take into account the specific characteristics of the local formations and must be consistent with the nature and importance of the structure. In general, investigations in karst areas will be more intense and may use a much wider range of investigation methods than at other types of sites. However, it is also the nature of karst settings that the residual uncertainty after exploration will still be substantial. Frequently, the exploration data are not interpreted as having specifically defined all critical features, but instead are interpreted as having been adequate to define the types of features that are present at the site and that might be found at any location. Designs that can accommodate these types of features are then selected.

b. Down-Hole Cameras. The extensive use of down-hole cameras on several USACE projects (Clearwater Dam, Missouri; Portuguese Dam, Puerto Rico; and Wolf Creek Dam, Kentucky), combined with coring and water pressure testing, has proven to be extremely useful in understanding the conditions within the rock masses and in guiding how the grouting can best be performed. Figure 17-27 shows typical rock core collected from a grouting project. While collection and hands-on examination of the core are extremely useful, the information available by direct down-hole examination of the rock and fracture systems, as illustrated in Figures 17-28 through 17-38, is being found to be of extraordinary value.

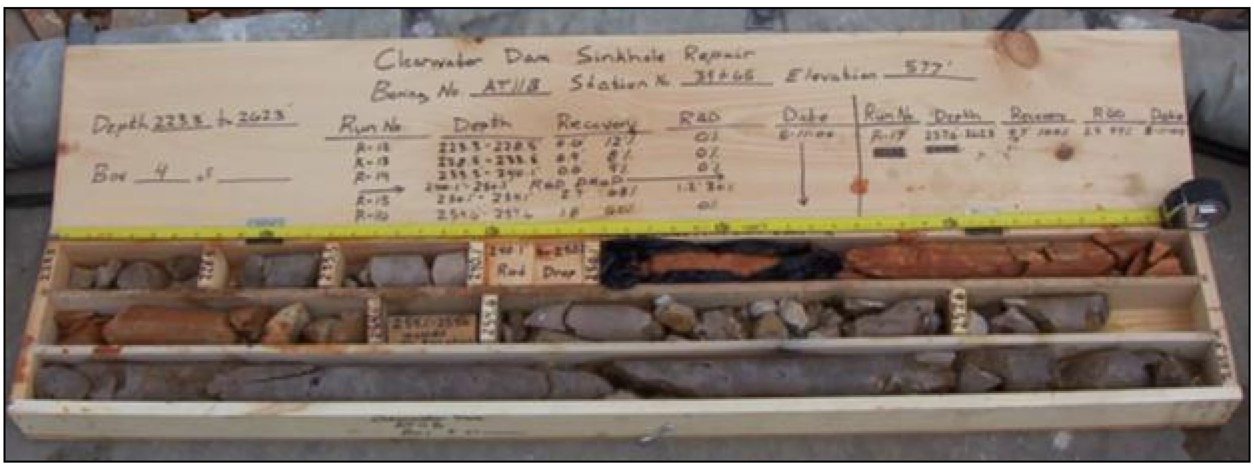


Figure 17-27. Typical rock core from limestone with dissolution features (Little Rock District).

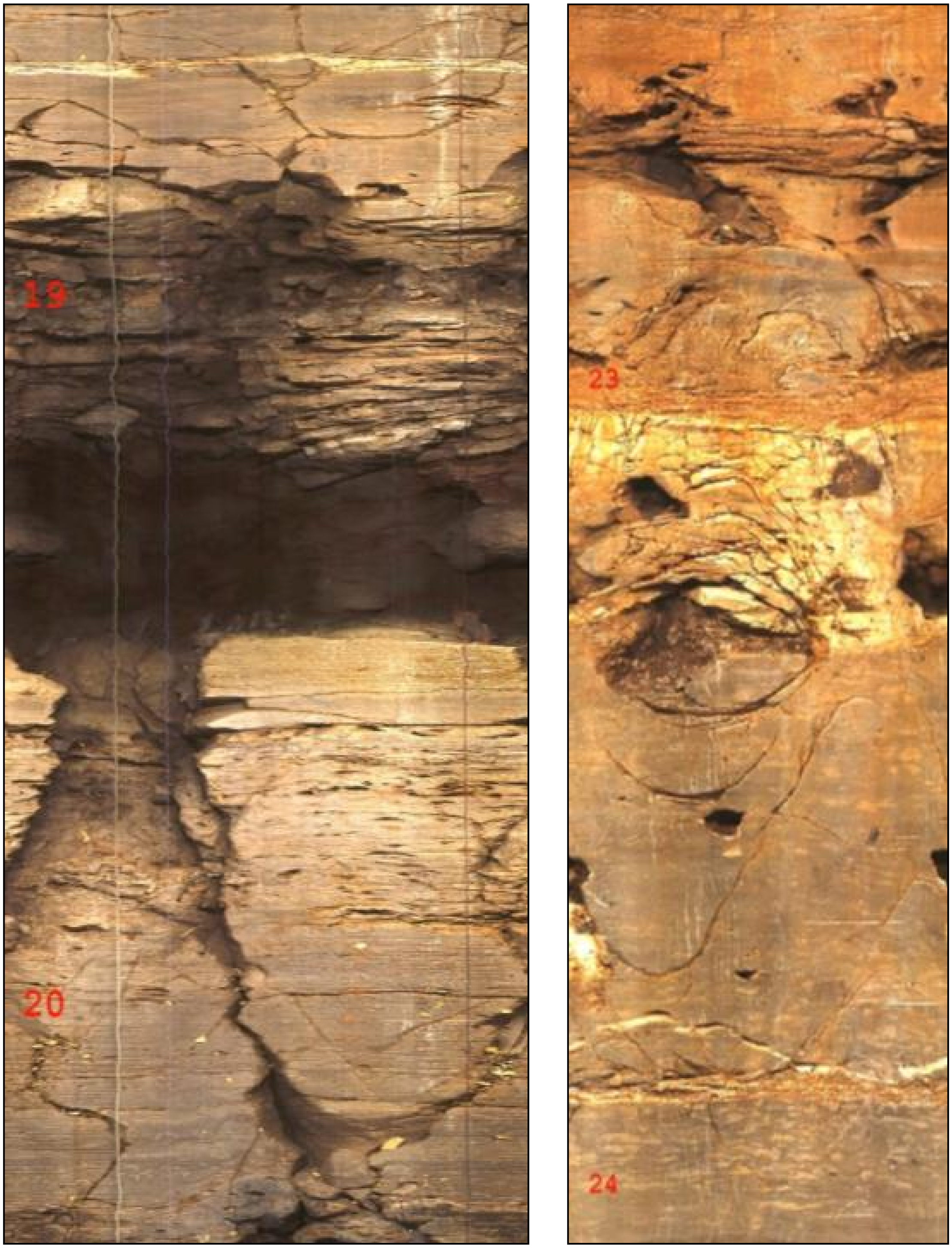


Figure 17-28. Clearwater Dam at shallow depth into rock (Little Rock District). The vertical scales are meters. Note the relatively clean, high-angle fractures and dissolutioning along bedding and as vugs.

Figure 17-29. Clearwater Dam at moderately shallow depth into rock (Little Rock District). The vertical scales are meters. Note the very large feature on the right photo.

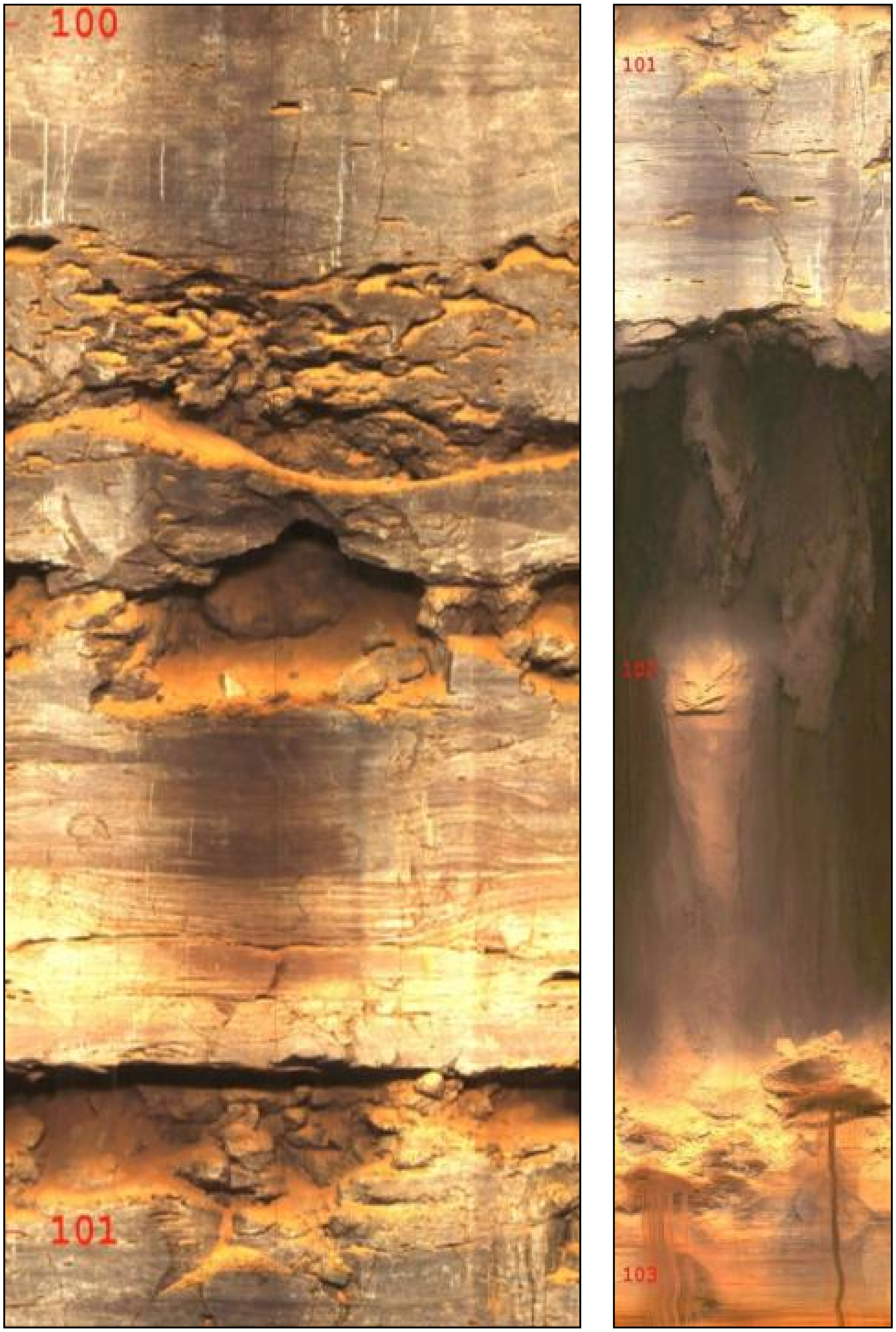


Figure 17-30. Clearwater Dam at substantial depth into rock (Little Rock District). The vertical scales are meters. Note the extent of soil infilling in the left photo and the approximately 1-m feature in the right photo.

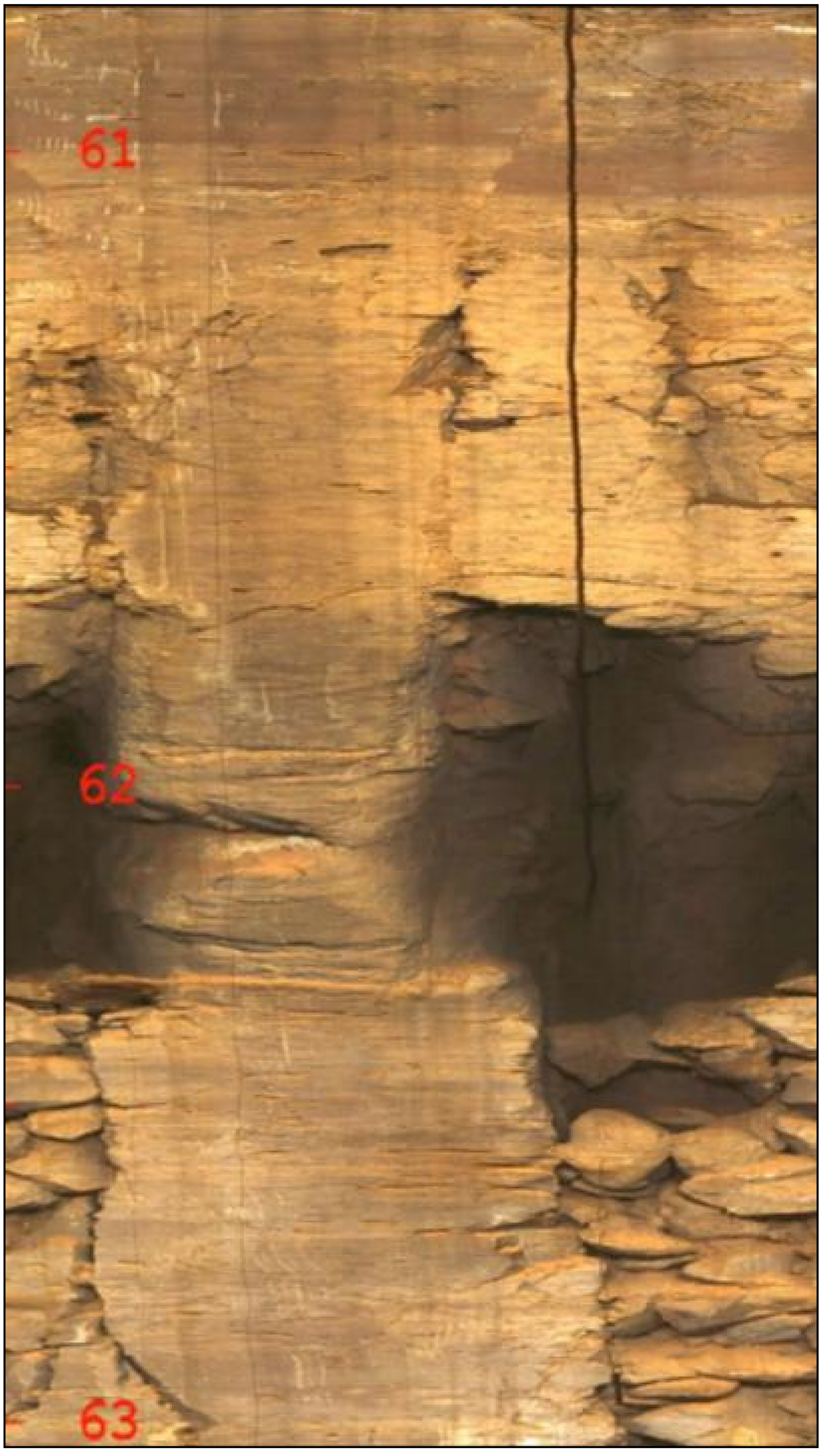
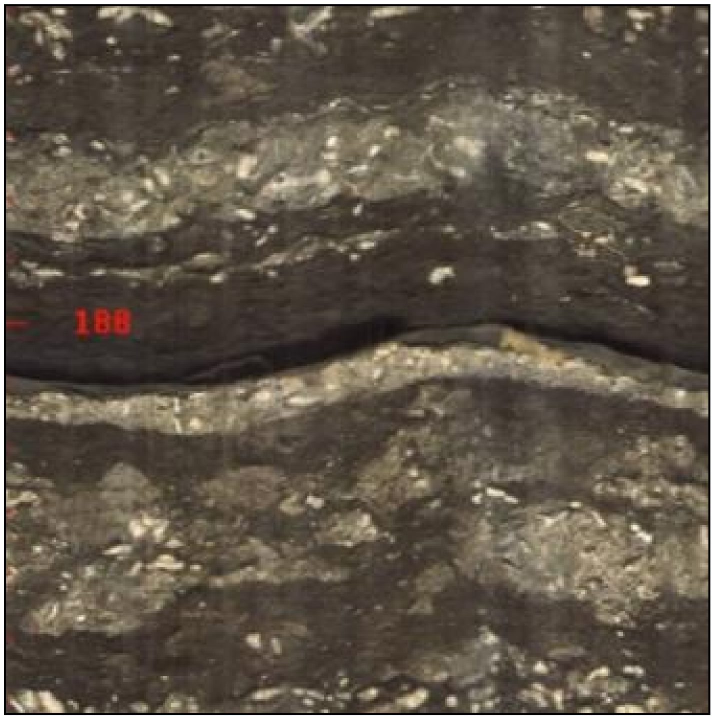


Figure 17-31. Clearwater Dam at moderately shallow depth into rock (Little Rock District). The vertical scales are meters. Note the significant dissolution feature.

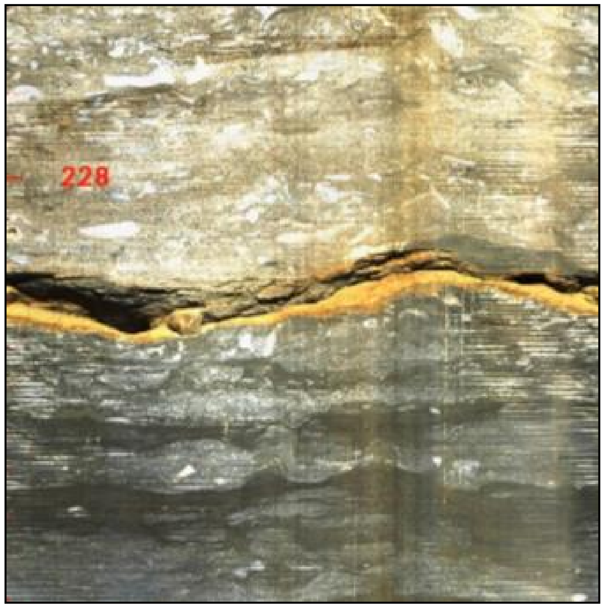


(Courtesy of Advanced Construction Techniques)

Figure 17-32. Clean bedding plane fracture at Wolf Creek Dam, Leipers formation. The vertical scales are feet.

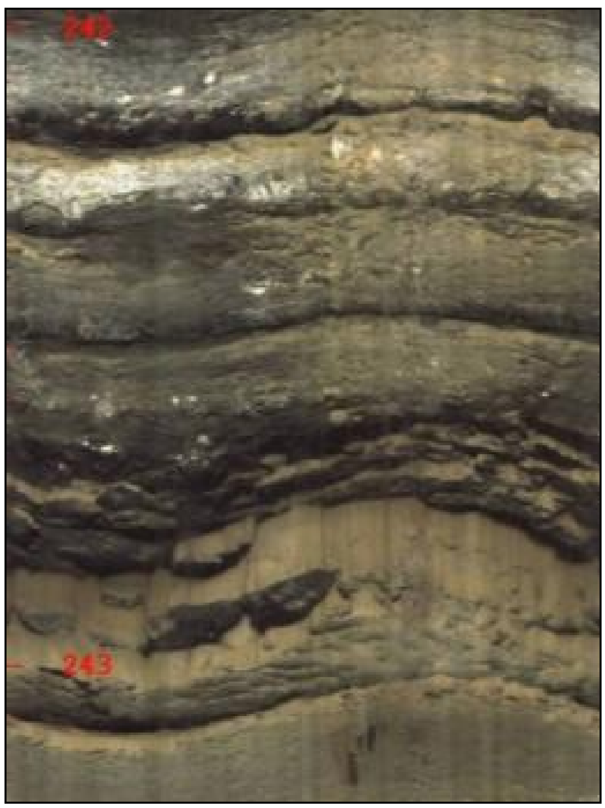
(Courtesy of Advanced Construction Techniques.)

Figure 17-33. Multiple bedding plane fractures with localized soil filling at Wolf Creek Dam, Leipers formation. The vertical scales are feet.



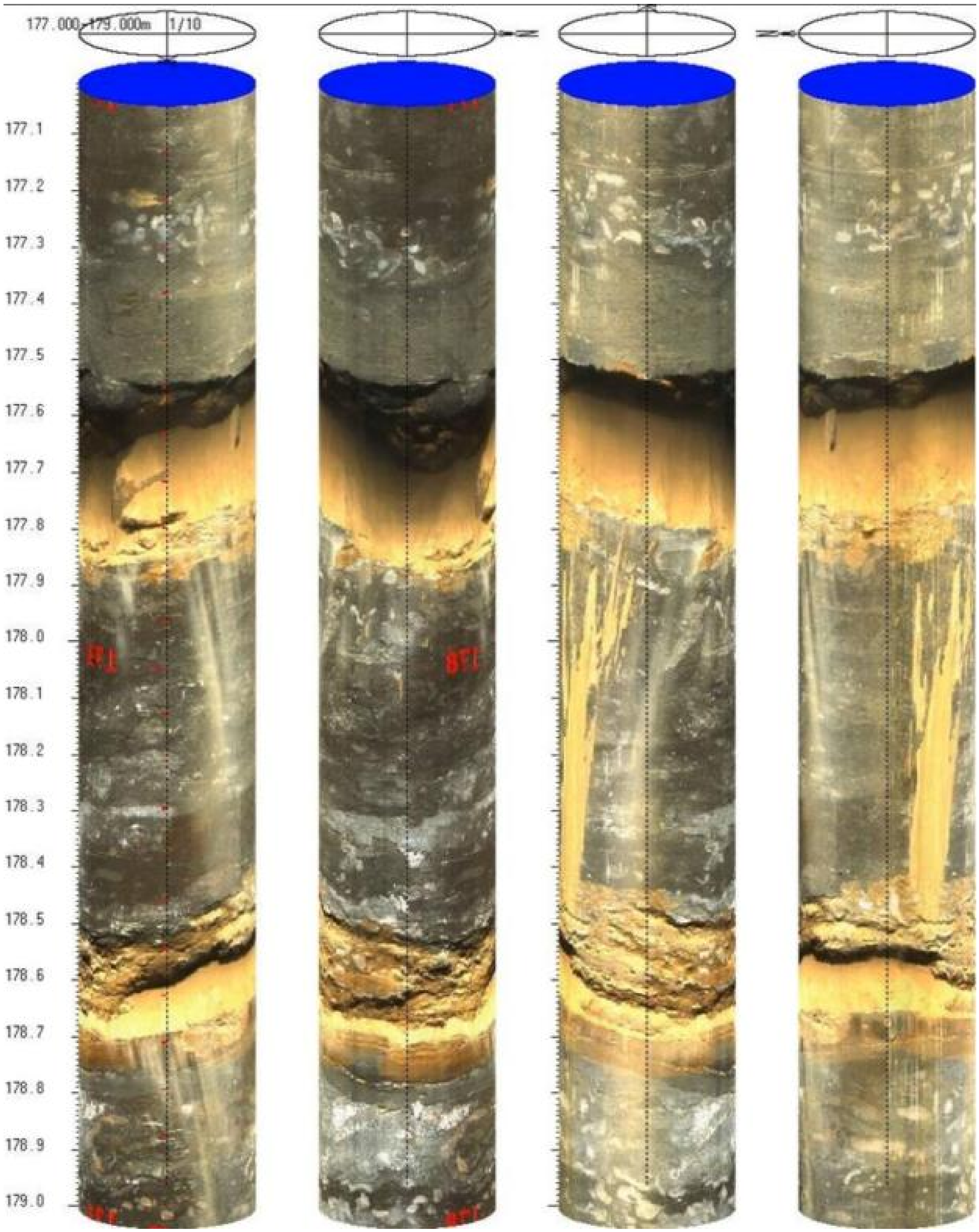
(Courtesy of Advanced Construction Techniques.)

Figure 17-34. Weathered and partly infilled bedding plane fracture at Wolf Creek Dam, Leipers formation. The vertical scales are feet.



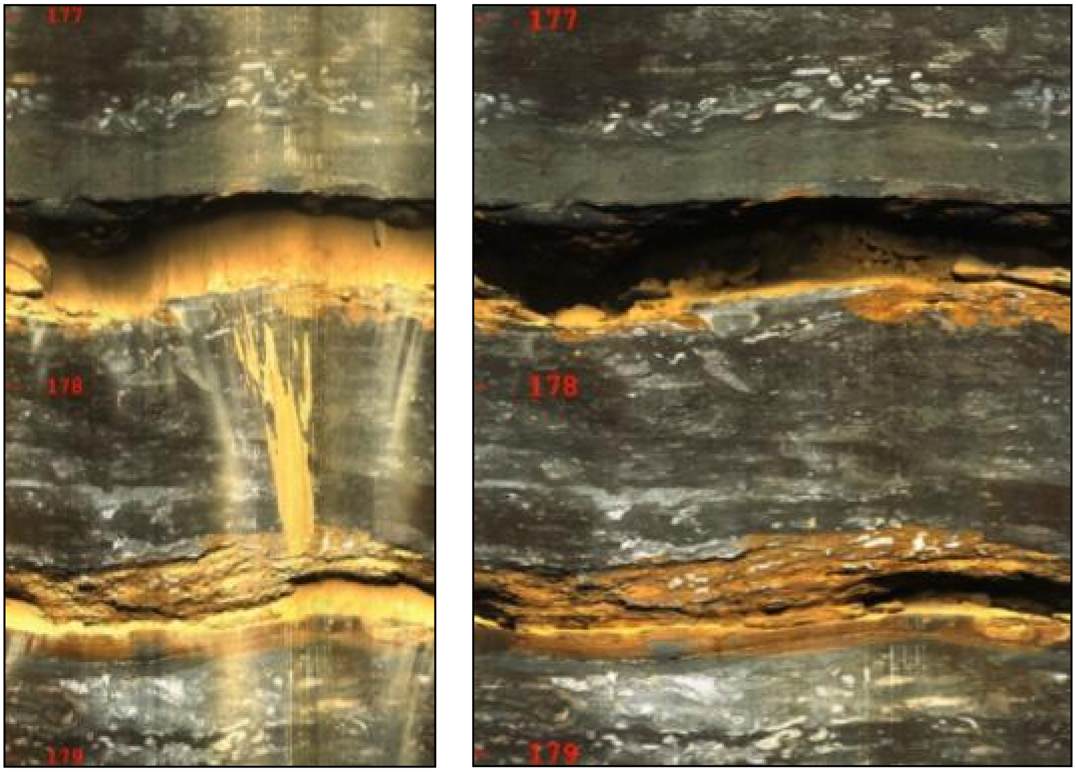
(Courtesy of Advanced Construction Techniques.)

Figure 17-35. Bedding plane fractures at 0.1- to 0.2-ft intervals at Wolf Creek Dam – Leipers formation. The lower fractures are infilled. The vertical scales are feet.



(Courtesy of Advanced Construction Techniques)

Figure 17-36. Virtual core (four views; see compass at top of each core) showing large, soil-filled bedding fractures at Wolf Creek Dam, Leipers formation. The vertical scales are feet.



(Courtesy of Advanced Construction Techniques)

Figure 17-37. The same fractures, as shown in Fig. 17-36, but showing conditions before washing (left photo) with high-pressure radial water jets and after washing (right photo) of fractures. The vertical scales are feet.

(Courtesy of Advanced Construction Techniques)

Figure 17-38. Clean 1-ft feature at Wolf Creek Dam, Leipers formation. The vertical scales are feet.

17-5. Grouting for Improving Foundation Support.

a. General. Grouting performed with the intention of either providing or enhancing foundation support has been used in a wide variety of situations for both shallow and deep foundations. Within the three-dimensional influence zones in the soil and rock that carry the applied pressures, grouting can serve one or more functions. These functions can include: (1) filling near-surface fractures to reduce the potential for water-induced soil migration beneath the structure, (2) filling open rock fractures to consolidate the rock mass and thereby increase its bearing capacity, and (3) compacting and/or displacing and replacing weak materials with grout. In some cases, grouting is used to permit the use of shallow foundations instead of deep foundations such as micropiles, driven piles, or caissons, which, in their own right, can be highly problematic and very expensive to construct in karst environments. In other circumstances, it is used as a supplemental measure for deep foundation systems. The following paragraphs give examples of applications for foundation support. In some instances, multiple methods are used.

b. Cap Grouting. Cap grouting refers to a treatment method performed with the intent of filling voids and sealing openings only at the rock surface (Warner 2004). Borings are typically advanced only to the rock surface or sometimes to a very shallow depth of penetration into the rock. The goal is to fill existing cavities and open fractures at the interface to eliminate existing voids in the soil and to prevent soil from entering the fracture system in the future. The intent is to create a continuous barrier at the soil-rock interface. Sowers (1996) suggests that this treatment is most applicable for sites where there is a widespread soft zone with numerous small erosion domes and few or no rock pinnacles. Grout mixes range from ready-mixed concrete or mortar to LMG to sanded HMG mixes to thick neat cement HMG. Grouting is normally performed on a split-spaced grid pattern. Applied pressures range from gravity heads to pressures approximately equal to the overburden pressure, and grout can be injected either to refusal or to predetermined volumes. Initial hole spacings are typically on the order of 10–30 ft. Final hole spacing is typically on the order of 5–10 ft or less to reasonably assure overlap and coverage. Cap grouting can be an attractive alternative where foundation conditions are generally favorable for shallow foundations, but additional precautions are desired. This method can also be used to treat large areas beneath structures where structure loads might be carried by deep foundation systems, but where floating floor slabs are vulnerable to sinkholes. The efficacy of the treatment generally cannot be determined because of insufficient data about treated vs. untreated sites.

c. Consolidation Grouting. Consolidation grouting of a rock mass within a predetermined region of concern is sometimes performed for heavily loaded spread footings constructed on questionable rock. It has also been used as a supplemental measure beneath deep foundation elements when conditions are unreliable or suspect, as an alternative to constructing the deep foundation elements to great depths, or as an alternative to reducing the design load capacity of the deep foundation elements.

d. LMG Ground Improvement. LMG improvement of soil and rock has been used to permit construction of spread foundations on soil and on rock in karst areas with conditions that would normally be considered unsuitable for that type of foundation. Gomez and Cadden (2003) describe three recent projects where this approach was successfully used. While the grouting process used was similar to compaction grouting methodology, LMG ground improvement in karst involves different mechanisms and goals because of the geologic conditions. LMG injections into rock offer the following potential benefits: strengthening the rock mass, improving the consistency of the rock mass, densifying weathered rock materials, and filling some joints and voids. LMG injected into rock will typically not result in significant sealing of fractures that might carry water and soil. LMG injection into soil foundations results in a more complex, composite behavior that involves creating a mass of soil and grout with increased strength, stiffness, and uniformity within the zone of stress influence. This is accomplished by constructing uniformly distributed grout inclusions within the soil mass that fill voids, displace soft materials, and act as vertical reinforcement and load distribution elements. The result is a zone of treatment that is stronger, stiffer, and less compressible. While these benefits can be intuitively understood and appreciated, ascribing specific improvement parameters for karst sites is not really possible. For the sites with injections in soil, Gomez and Cadden report that grout injections were in the range of 3.3–3.9 ft^3 of grout per foot of hole, but they indicate that injection volumes in karst are often in the range of 3–5 ft^3 per foot. Split-spacing methods are used, with the primary hole spacing in the range of 10–15 ft and with a final hole spacing in the range of 4–5 ft. Termination of grouting for any given stage is based on reaching refusal pressure, observing surface displacement, and/or attaining injection of specific volumes at lower than maximum pressures.

17-6. Karst Features and the Performance of Dams.

a. Introduction. The performance spectrum for dams on karst foundations ranges from successful, to dams that developed extreme leakage problems, but that were remediated, to dams that were never able to successfully store water. Certainly, karst-related problems at dams are so common that they have become notorious as the source of numerous, serious, and costly long-term performance and safety issues. One of the most historically interesting projects was Hales Bar Dam on the Tennessee River. It was privately constructed between 1905 and 1913 for hydroelectric power and was located about 33 miles downstream from Chattanooga. The foundation for the dam was Mississippian-age Bangor limestone. Over 25 years of study, it was found that the Bangor limestone contained numerous clay-filled cavities and open interconnected solution features. Between 1905 and 1910, three different construction contracts were terminated due to the difficult foundation conditions. Ultimately, a fourth contractor completed the project on a time and materials basis. Shortly after completion, excessive leakage developed at one of the abutments, and attempts were made to treat the leakage by surface plugs on the riverbed. In 1919, drilling and grouting from the gallery commenced using hot asphalt injected into the foundation voids. More than 2,900 yd^3 of asphalt was injected, and by 1922 it appeared that leakage control had been successful. By 1929, the leakage gradually increased to levels approximately the same as existed in 1919. By 1939, which is when the structure was assimilated into the TVA system, seepage was estimated to equal about 10% of the entire river flow. In 1941, the TVA began installing a cutoff trench comprising 750, 18-in.-diameter lined secant piles extending more than 100 ft below the riverbed. This cutoff was supplemented with another barrier located immediately upstream comprising diamond-drilled holes on 10-in. centers filled with asphalt. A third barrier, constructed downstream of the first two, was composed of 13-in.-diameter secant piles filled with concrete along with 3-in.-diameter grouted holes between the secant piles. Construction of this triple cutoff barrier was completed in 1945. In April 1963, TVA announced that Hales Bar Dam would be abandoned due to increasing leakage and the need for larger locks. Nickajack Dam was constructed between 1964 and 1967 to replace Hales Bar Dam, and the remnants of Hales Bar Dam now lie within Nickajack Lake.

b. USACE Dams on Karst Foundations.

(1) The USACE owns and operates 610 dams that serve navigation, flood control, water supply, irrigation, hydropower, recreation, environmental enhancement, and combinations of these purposes. In 2005 and 2006, the Corps performed an initial screening of more than 130 dam projects that were believed to represent the highest potential risk. Subsequently, within this group of 130 dams, the USACE identified six dam projects that were judged to be critically near failure or have extremely high life and/or economic risk. These six dam projects rose to the level of national priority for funding, studies, investigations, and remedial work. Interim risk reduction measures were implemented at each of the six dams. Three of the six dams having the highest risk and priority are earth dams on karst foundations: Clearwater Dam in Missouri, Wolf Creek Dam in Kentucky, and Center Hill Dam in Tennessee. Immediately preceding action on these projects, major cutoff projects were constructed at the abutment and emergency spillway area of Patoka Dam and across a substantial length of Mississinewa Dam, both of which are located in karst areas of Indiana and had experienced seepage-induced performance problems. A cutoff wall has been completed at Clearwater Dam and at Wolf Creek Dam. A cutoff wall at the Center Hill Main Dam and an RCC berm at the Center Hill Saddle Dam are currently in construction. Many of the USACE embankment dams are founded partly on excavated and treated rock surfaces and partly on overburden materials. Where the foundations are on rock, the foundation treatment programs normally included cleaning out karst features at least within the limits of the cutoff trench, placing backfill concrete on all or a portion of the foundation, and backfilling the features with either compacted soils or concrete. Grouting was also normally performed beneath the treated rock foundation. However, often the grouting was relatively ineffective compared to current practice and often consisted of vertical holes (which potentially missed vertical fractures and joints) that sometimes went to insufficient depth and/or were grouted with mixes and methodologies that are not considered today to be the most effective practice.

(2) The vast majority of grouting currently performed by the USACE is for remediation of seepage problems at existing dams. In some instances, grouting is the only solution that is applied; in other instances, extensive pre-grouting is used to facilitate construction of cutoff walls. Recently, combined solutions are being used, with cutoff walls providing long-term protection in some portions of the foundation and grouting providing long-term protection in other portions. In these combined solutions, the grouting program is used to define the required depth for the cutoff wall, provide pre-treatment for construction of the cutoff wall, and act as the sole long-term protection beyond the limits of the cutoff wall.

c. Typical Problems and Hazards Created by Karst Conditions at Dams. In most cases, performance problems at dams on karst foundations are related to seepage. It is relatively rare for structural issues to be the primary problem. Typical problems and hazards include:

(1) Excessive rates of seepage.

(2) Excessive pore pressures and/or adverse distribution of pore pressures.

(3) Internal erosion of the embankment and/or foundation materials.

(a) Internal erosion due to backward erosion and piping or stoping of embankment and/or foundation materials, internal erosion of the embankment into the foundation, or internal erosion due to scour at the abutments can result in the formation of unknown cavities within cohesive soils or the creation of soft, loose zones in non-cohesive materials that weaken the structure both hydraulically and structurally. Surface expressions of loss of materials can include the formation of sinkholes, lateral deformations, and/or settlement.

(b) The problems created are often concealed and may not be detectable by visual inspection. In many cases, diagnosing the long-term health of the structure relies on extensive instrumentation systems monitored over a long period of time to detect gradual changes. Whenever changes in behavior are detected, whether gradual or abrupt, it is clearly a sign that the site and conditions are actively deteriorating. Even with extensive instrumentation systems, problematic changes may not be detected because of the location of the instruments with respect to the karstic features, which tend to be unpredictable.

(c) A separate but related type of problem for new reservoirs is the potential for unacceptable loss of water from the reservoir through avenues other than leakage through the dam foundation itself, such as the abutments and reservoir rim. The potential for this type of problem must be separately examined by obtaining a thorough understanding of the regional groundwater regime before construction and assessing the changes that will likely occur upon filling of the reservoir.

17-7. Field Drilling and Grouting Issues in Karst Features.

a. There are numerous issues that make grouting in karst difficult. The following paragraphs briefly summarize these issues.

b. Soil/Rock Interface Zone. The soil-rock interface zone can be particularly problematic. The nature of the issues will vary, depending on whether the interface zone is a compacted embankment on a prepared rock foundation or whether it is natural overburden overlying the original rock profile.

(1) Compacted Embankment/Prepared Rock Foundation Interface. Problematic conditions are less common when properly compacted embankment materials were placed over an excavated and treated rock foundation. In these cases, weathered rock materials will have been removed, and there will be a distinct interface between the embankment materials and sound rock. Drill casings can be seated a short distance into the foundation rock (i.e., 1–3 ft). The primary issues are treating any soft or loose embankment zones and protecting the embankment materials during drilling and grouting operations.

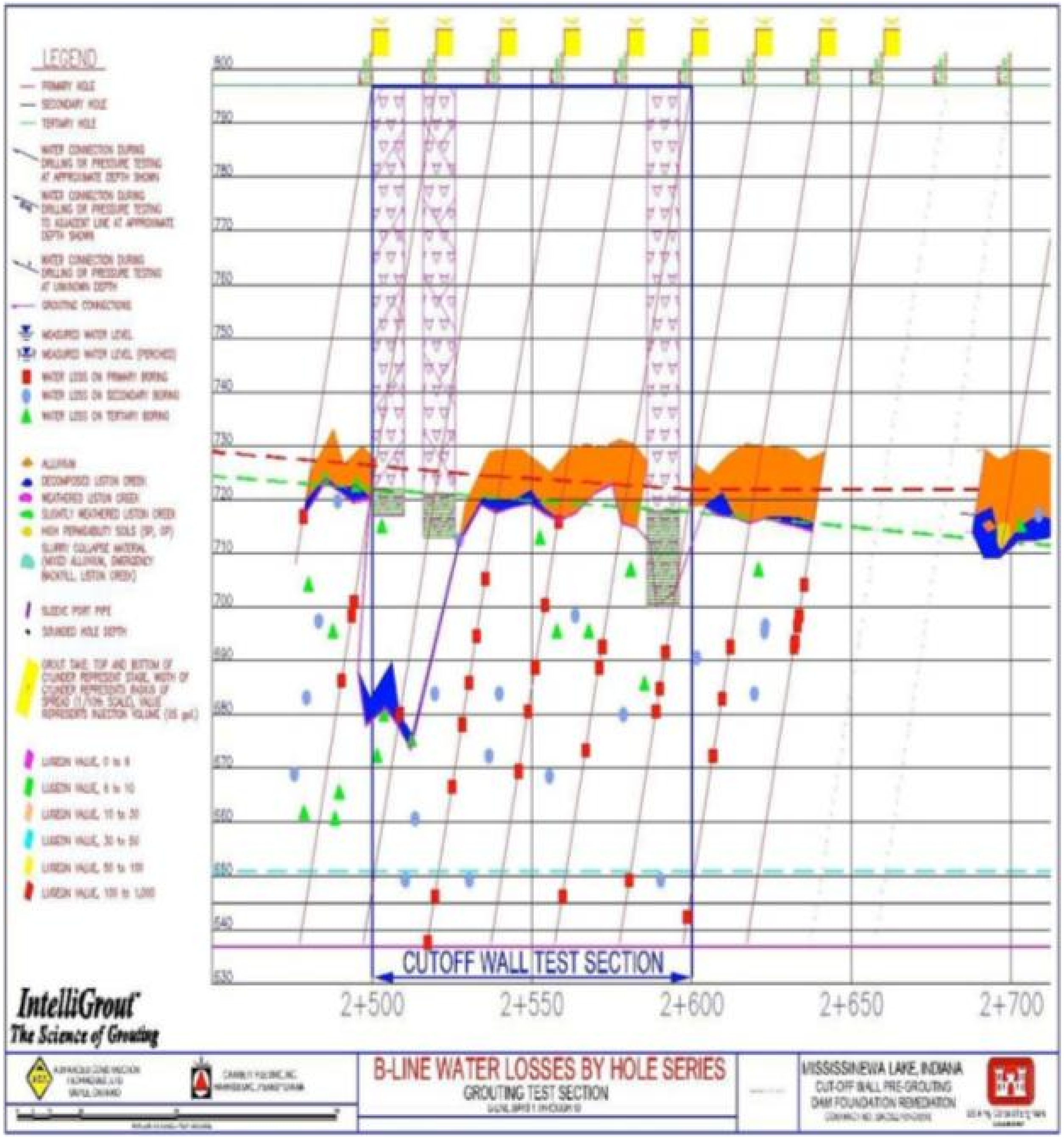
(2) Natural Soil/Rock Interface Zones. When embankments have been constructed on natural overburden materials rather than on prepared rock foundations, the drilling and grouting problems are normally more numerous. There can be pervious zones within the overburden; there might be loose, soft zones of material near the rock interface resulting from loss of materials into the rock; and there can be an extensive transition zone of mixed soil and rock

materials with voids and/or soft infilling that have high permeability and that cause grout hole stability problems. These materials must be treated and at the same time protected from induced damage during drilling and grouting operations.

c. Water Losses During Drilling. Water losses during drilling are extremely common when grouting karst sites, particularly in the early portions of the program. When water losses occur, it is necessary to stop drilling and grout the loss area as a special stage before proceeding. Failure to do so can result in the loss of equipment in the hole, and in contamination of the fracture system with drill cuttings, which can subsequently prevent effective grouting. As an alternative to constant production interruptions due to water losses, downstage grouting is often performed in karst formations. Figure 17-39 illustrates the pattern of water losses by hole series that occurred on a small segment of the Mississinewa Dam pre-grouting program.

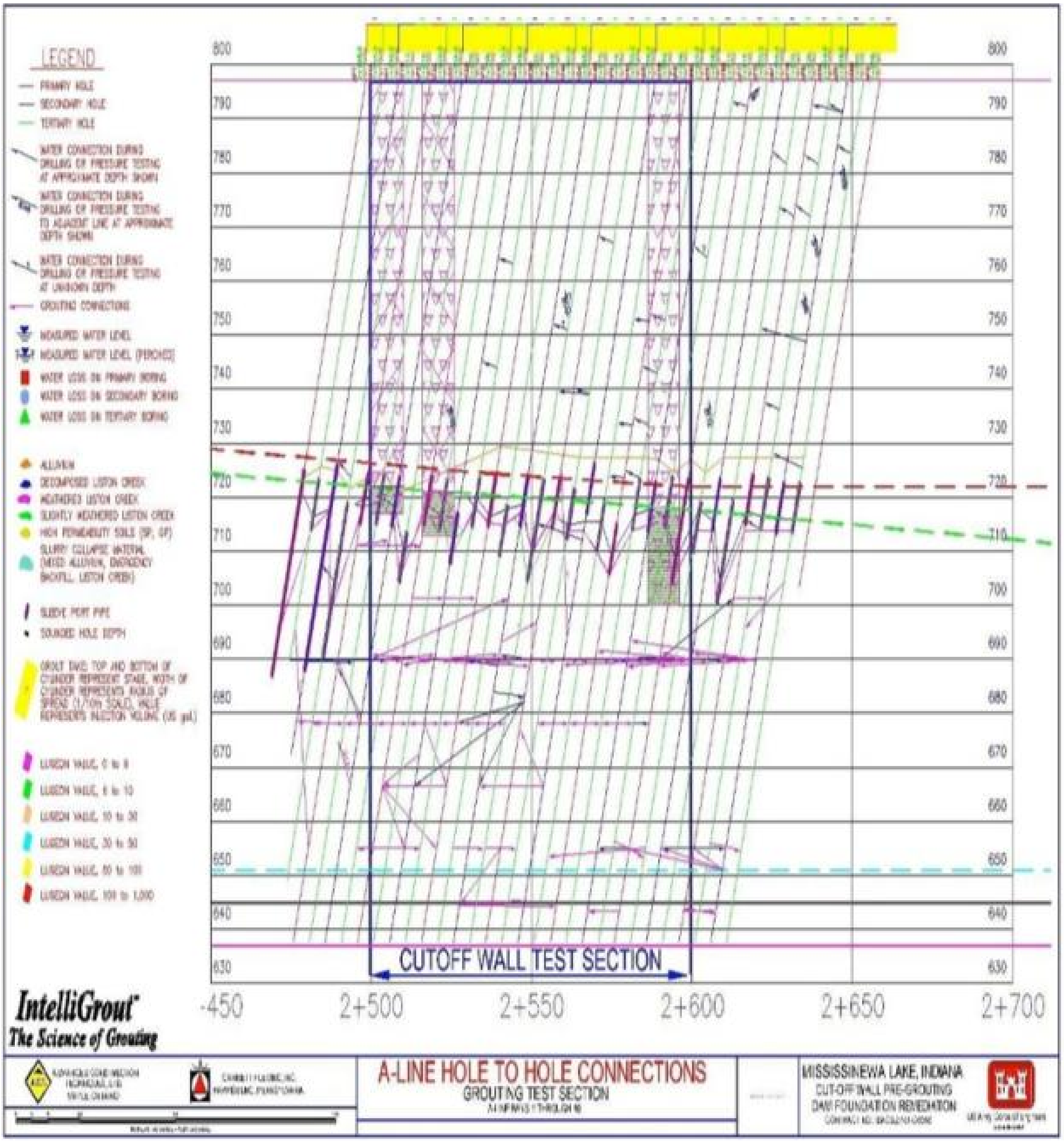
d. Rod Drops/Cavities. Rod drops during drilling can be common on karst sites. Drops are identified by the driller and/or inspector, and they can indicate the intersection of a soil-filled zone or cavity. Unless these locations are examined with a down-hole camera, the nature of the feature causing the rod drop will be extremely uncertain. Even with a camera, it may not be clear whether a feature has been penetrated along its margin or whether it has been penetrated at a more representative location. Hole collapse can be a common problem where rod drops occur.

e. Hole Interconnections. Interconnection of grout holes can be common if the hole spacing of the initial series of holes is not sufficiently wide. In general, interconnections are undesirable because they can result in contamination of fractures at the connected holes and can prevent effective grouting from the adjacent holes. Interconnections might or might not be readily apparent when injecting grout into any given hole. For example, the interconnection can result in an adjacent hole being slowly filled with grout without the grout rising all the way to the surface, and the filling of the adjacent hole might not be detected until a later time. In other cases, multiple hole interconnections can occur, which makes handling the interconnections a difficult field process. Figure 17-40 shows hole connections that occurred on a small segment of the Mississinewa Dam pre-grouting program.



(Courtesy of Gannett Fleming and Advanced Construction Techniques)

Figure 17-39. Water loss locations at Mississinewa Dam in a cutoff wall test section. Water loss symbols are color coded and vary by hole series.



(Courtesy of Gannett Fleming and Advanced Construction Techniques)

Figure 17-40. Hole connection diagram at Mississinewa Dam in a cutoff wall test section. Arrows indicate the origin and extent of hole connections.

f. Non-Penetrated Dissolution Features. The probability of penetrating dissolution features is directly related to the hole spacing and hole pattern being used. In almost every case, it should not be assumed that all dissolution features have been directly penetrated. Therefore, grouting of many dissolution features must be done through connecting fractures. These connecting fractures are often smaller features, so the rate of grout injection can be entirely controlled by the flow capacity of the fractures feeding the larger dissolution feature. One indicator of this condition can be when low Lugeon zones accept large quantities of grout without appearing to be heading toward refusal. The grout should be thickened very cautiously and in small increments, with large quantities pumped between thickening points to prevent reaching refusal in the smaller fractures without properly grouting the larger dissolution feature.

g. Features with Hydraulic Conductivity Exceeding the Capacity of Water Testing Equipment. It is essential to know the pressure and flow characteristics of the equipment being used for water pressure testing. In the course of performing pressure tests, the test becomes invalid whenever the combination of injection rate and injection pressure approaches the operating curve for the injection system. At that point, the injection of water is system limited, and the Lugeon value of the stage cannot be determined.

h. Infilled Seams in Rock. Soil-infilled seams are one of the most problematic issues in karst grouting. The borehole walls can be washed using wash probes with high-pressure radial jets, but the actual depth of washing into the joint is likely to be limited. During pressure testing, additional infilled material can be removed, but the removal of material will cease when an opening is created that has a capacity equal to the injection capacity of the equipment. In general, therefore, it is not possible to completely remove infilled soil with normal operations.

(1) If deemed necessary, it is possible to create washed channels that continuously connect from hole to hole if a group of closely spaced holes is drilled (i.e., 2.5 ft or less) and adjacent holes allow washwater to exit. The efficacy of the washing can be improved by periodic injections of air into the wash water. Subsequently, the entire group of holes must be sequentially pressure grouted (i.e., grout is injected into one hole until fresh grout returns from other holes and then grout is applied under pressure into those holes). The potential benefit of this approach is to create at least a small continuous barrier of grout from hole to hole within a particular seam.

(2) In the absence of the washed channel approach, which is rarely used, infilled seams are grouted to absolute refusal using normal techniques with considerable uncertainty about the mechanism of refusal. What is known empirically is that it was brought to refusal under applied pressures. The applied pressures should exceed the expected service heads. It is common that there will be episodes of hydraulic fracturing in the soil seams in this process, as indicated by a buildup of pressure followed by pressure drops. Although hydrofracturing carries a negative connotation in most grouting, this is considered a beneficial process, and it has been demonstrated on many projects that refusal can be reached after a series of hydrofracturing episodes. These methods have a rather high potential to cause damage, so they are not recommended when working in or under existing dams or similar critical structures.

17-8. <u>Grouting Concept Mechanisms and Reliability of Grouting in Karst</u>.

a. Concept Mechanisms. Various mechanisms are believed to occur during grouting of karst features, depending on the nature of each type of feature. The reliability and confidence in grouting as a long-term barrier also differs with the feature types.

(1) Clean Fracture Systems. Wherever regions of the site are composed of clean fracture systems or dissolution features, it should be expected that grouting can be brought to a

satisfactory conclusion. Provided best practices in all operations are used, there is no reason to expect that the fractures cannot be effectively filled with grout or that grouting will not provide satisfactory long-term service. Figure 17-41 shows a large clean fracture that was completely filled with grout in a core extracted at depth from the foundation of Mississinewa Dam in Indiana.

(Courtesy of Gannett Fleming)

Figure 17-41. Large, clean fracture in limestone completely filled with grout.

(2) Infilled Fracture Systems. Significant uncertainty exists regarding the long-term performance of grouting in fracture systems that have soil infilling, whether that occurs in a near-surface transition zone or at depth within the rock mass. It has been demonstrated on several projects (Patoka Lake Dam, Mississinewa Dam, and Clearwater Dam) that the grouted zone can be brought to refusal at the desired pressures, which may be one to two times the service heads that might be imposed by the reservoirs. Pressures used should be low enough to prevent damage when working under existing structures. The long-term effect of reservoir pressure on infilling has resulted in degradation of neat cement grout treatments in the past. The longevity of the modern procedures is yet to be proven although following current day best practices is expected to provide superior durability.

(a) Simultaneous mechanisms that are believed to occur during drilling and grouting operations in these regions of the foundation include filling of open fractures, partial removal of infilled materials, displacement of soft materials, densification of infilled materials, and hydrofracturing of infilled materials. These mechanisms are believed to work together to provide stability under applied gradients, the time reliability of which is unknown. Filling of clean fractures limits water access to the grouted zone and provides confinement to infilled fractures; partial removal and displacement of soft materials may provide compartmentalization of the infilled fractures; densification may increase resistance to piping; and hydrofracturing can provide both compartmentalization of infilled materials and a chemical improvement in clayey materials. The effectiveness of any of these methods is uncertain.

(b) Hydrofracturing may be one of the more important elements of this combined mechanism. Zuomei and Pinshou (1982) reported effective grouting of extensive karst caves filled with soft clay and fine sand at the Wujiangdu Dam project in China. Applied reservoir heads were up to 540 ft. Grouting was accomplished using HMG mixes under very high pressures, and hydrofracturing was observed. Extrusion of soft clays was also observed during field operations, with soft clay sometimes oozing from holes adjacent to the hole being injected. After grouting, specimens were removed from grouted caves, with the following observations and conclusions: clay samples contained a closely spaced network of grout lenses, with the volumetric ratio of grout to total volume of the samples on the order of 57%; clay strength was increased within the grouted zone; grouted clay samples tested under high gradients up to 500 showed no tendency for seepage failure; and a layer of grout was found to exist around the cave boundary between the cave walls and the clay. It was concluded that the hydrofracturing process can provide barriers with sufficient strength and stability to resist high water pressures. Zuomei and Pinshou (1982) recommended beginning the injections near the center of the feature and working outward rather than using a normal split-spacing procedure as a means of keeping extrusion paths open.

(c) At Patoka Lake Dam (Louisville District, USACE), Walz et al. (2003) reported observing frequent hydrofracturing during grouting of infilled fractures and modifying the closure criterion based on those field observations. Specifically, grouting was performed at heads well in excess of the maximum reservoir service heads that might occur. Closure was not considered to be achieved until water testing of the verification holes indicated that design Lugeon values had been achieved and that no further hydrofracturing was possible at the designated grouting pressures. It is important to note that the Patoka grouting work was not done on the dam or through any embankment material, but was on the abutment and spillway section where higher pressure was less critical.

(d) At Mississinewa Dam (Louisville District, USACE), pre-grouting performed for a cutoff wall included successfully grouting some large solution features using HMG and sleeve-port pipes. Hydrofracturing was part of the mechanism, and it was possible to bring the stages to refusal at the required pressures. Figure 17-42 shows a sample of sandy soils with phenolphthalein indicating the extensive presence of grout.

b. Reliability. In summary, a high degree of long-term reliability can be expected when clean karst features have been properly grouted. For infilled features and fractures, the long-term reliability is still uncertain despite some encouraging results on certain projects. There are unresolved long-standing concerns that soil-filled seams and features will either extrude or gradually be washed out over a long period of time under high heads. For that reason, in many cases, grouting is not relied upon as a sole long-term solution in these zones.

(Courtesy of Gannett Fleming.)

Figure 17-42. Soil sample from a sleeve-port grouted zone, with grout presence indicated by phenolphthalein and recovered grout lenses.

17-9. General Protocols for Remedial Drilling and Grouting in Karst.

a. General. Following are recommended protocols for planning and conducting effective remedial drilling and grouting in karst environments. These general protocols are based on procedures that evolved on recent USACE projects where grouting objectives in karst have been successfully met. While there are undoubtedly valid exceptions to any element in the protocol due to particular circumstances and applications, it provides a general outline that should facilitate success in remedial grouting.

b. Site Subdivision. Sites are often non-uniform in character. Every effort should be made to identify regions within the foundation that are believed to be similar or different so that appropriate grouting methodologies can be established for each region. Boundaries of similar regions might be related to specific formations, but they can also vary within individual formations as a function of either depth or spatial location. Differing areas can be extensive in size, or they can be isolated within an otherwise relatively homogeneous rock mass.

(1) Site Subdivision by Karst Characteristics. The fundamental nature of karst feature development is that it originates from a well-distributed network of small fractures and generally a few larger ones. Over geologic time, the larger fractures can become major dissolution features due to flow concentration by their action as a drain for the smaller features. This is not meant to imply that the flow capacity of the smaller network of fractures is small, as it can, in fact, be enormous. However, there are often relatively isolated large features or regions containing a network of large features that will need to be identified and treated with special measures and techniques. The locations of these features can often be identified well in advance by analysis of lineaments and topographic features.

(2) Subdivision by Depth of Infilling. Another important subdivision discriminator is the depth of soil-infilled fractures. Below the depth of infilling, complete filling of the fracture system with grout is readily possible, which results in the ability to achieve minimal residual permeability that should perform well over the life of the structure. Above the depth of infilling, special measures are normally required to achieve effective, long-term performance. These measures can include extensive pressure washing effort, more closely spaced holes, more lines of grouting, and intentional hydrofracturing of the soil infillings. In combination, the measures would remove more of the infilled materials, contain and compartmentalize the remaining soil materials, and reduce gradients across the zone. Another common alternative is to install a cutoff wall through the depth of infilling.

c. Final Hole Spacing, Number of Grout Lines, and Performance-Based Verification. All other factors being equal, grouting in karst will be more effective with increased numbers of holes and increased numbers of grout lines. The minimum number of lines that should be considered for “temporary” applications such as pre-grouting for cutoff walls is two, and the maximum final hole spacing should be 5 ft. Three to five lines should be used in “permanent” applications. Locally, wherever adverse conditions are encountered, both the number of lines and the number of holes per line should be intensively increased. The key element that ultimately controls the program is performance-based verification. The required Lugeon value for the grouted zone must be established in advance, and a sufficient number of verification holes into the grouted mass must show that the required performance has been achieved.

d. Overburden Drilling, Hole Casing, and Sleeve-Port Grouting. Drilling through embankments must be accomplished using equipment and methods consistent with the requirements of ER 1110-1-1807. Methods that have successfully been used include sonic drilling and rotary duplex systems. If continuous sampling is required and some sample disturbance is not an issue, sonic drilling methods are very effective. Rotary duplex equipment does not provide continuous sampling, but standard penetration sampling equipment can be introduced at any time. Recovered soil samples should be logged and examined for loose, soft, wet zones that indicate damage associated with piping. A pocket penetrometer is recommended for quantifying the consistency of the materials. Any pervious soil zones should also be identified, and the thickness of highly weathered transition zones must be determined. The overburden or epikarst drilling should continue until it penetrates a minimum of 2 ft into sound rock materials. The casing installed in the hole should be solid pipe in zones not requiring grouting (i.e., dense embankment soils or dense overburden soils without pervious lenses) and sleeve-port casing pipe in all other areas (i.e., soft soil zones, pervious zones, and all transition materials). Installation and grouting of all overburden casings and any sleeve-port grouting of the overburden, epikarst and transition material stages should be completed in advance of rock drilling and grouting to protect the embankment and/or overburden materials. If desired, barrier bag systems can be used on the sleeve-port pipe to isolate the sleeve-port stages. Figure 17-43 illustrates a typical overburden sampling program, and Figure 17-44 shows sleeve-port casing for installation in required zones.

e. Initial Hole Spacing in Rock. The initial hole spacing of holes drilled in rock should be such that a minimal number of connections occur between holes. In general, a minimum primary hole spacing on the order of 80 ft or more is recommended.

Figure 17-43. Overburden sampling program with a sonic drill at Wolf Creek Dam (Nashville District).

Figure 17-44. Sleeve-port casing to be installed in overburden and soil-filled dissolution features at Mississinewa Dam in Indiana (Louisville District).

f. Downstage Grouting and Upstage Grouting. The number of water losses, hole interconnections, and hole collapses that occur in an early series of holes will normally be substantial. Since the procedure for dealing with water losses is to terminate drilling and grout the water loss zone as a separate stage, the grouting program functionally becomes a downstage drilling and grouting program until the frequency of water losses is minimal. Redrilling collapsed holes also functionally results in downstage grouting. Additionally, protection of the base of embankment dams during drilling and grouting is of primary concern, and better protection can be provided by downstage grouting at least a certain distance beneath the embankment. For those reasons, the following protocol is recommended for rock grouting beneath existing embankment dams:

(1) After the installation and grouting of all overburden casings and any sleeve-port grouting of the overburden, epikarst, and transition material stages, downstage drill and grout two stages in every hole on each line. This will provide maximum protection for the base of the embankment.

(2) After completing the first two downstages on all lines, downstage drill and grout holes using a normal line sequence and series until a hole spacing of 20 ft is achieved.

(3) If conditions permit, upstage drill and grout the remaining series of holes.

g. Stage Lengths. Stage lengths of 10 ft are recommended for the first two stages in rock and should not be more than 15–20 ft below that depth where downstage drilling and grouting are used. Long stages will result in grouting being controlled by the characteristics of larger features and may preclude effective grouting of smaller ones. Ultimately, grouting of the smaller features is equally critical for minimizing flow through the interconnected fracture system. Where upstaging is used, the final stage length for grouting will be determined by the results of water pressure testing. It is common to combine stages, provided they have similar Lugeon values. However, because of the maximum allowable injection pressures for a particular stage or depth interval, not more than two stages should be combined.

h. Pressure Washing. When pressure testing data indicate that washout of material is occurring during the test, the pressure testing should continue until flow stabilizes, which may be at the capacity of the equipment.

i. Use of Down-hole Cameras. It is recommended that a down-hole camera be available on site at all times. The camera should be used frequently in the early part of the program as an exploration tool, and whenever needed to examine locations of water loss, rod drop, and soil seams, and locations where the capacity of the pressure testing equipment is exceeded.

j. Starting Grout Mixes. Except where camera examination clearly dictates otherwise, grouting should always be started with the thinnest mix being used on the project.

k. Thickening of Grout Mixes. Thickening of grout mixes should be done cautiously and gradually. Adjustments to the grout mix shall be directed based on guidance from the Government dependant on pressure and flow data acquired, recorded, and analyzed by the automated system.

l. Inability to Reach Refusal at Thickest HMG Mix. In some stages, there will be an inability to reach refusal even after pumping large quantities of the thickest HMG mix. In most cases, this will occur in stages where the fracture or feature permeability was so high that Lugeon values were not able to be determined during pressure testing operations. Assuming that cameras were not used to evaluate the conditions within the stage, a conclusion of probable failure to reach refusal should be determined to exist only when very large quantities of grout have been pumped into the stage and when it is clear that no reduction in the Apparent Lugeon value is being achieved. If the Apparent Lugeon value is declining even minimally, pumping should continue without interruption. If operations are being conducted in the blind (i.e., without the use of down-hole cameras), the conditions will generally be completely unknown, and there is little rational basis for determining an injection volume at which grouting should be temporarily suspended. If cameras have been used, there may be a more logical basis for setting a maximum injection volume. Regardless, at a certain point of non-progress, it is normal to suspend the injection, allow sufficient time for the grout to take an initial set, and then attempt to re-inject the stage. This intermittent grouting process may be performed multiple times as necessary to reach refusal. Where this process has been required, additional closely spaced holes should be drilled and grouted in the entire area to verify completion of the grouting process.

17-10. Treatment of Large Cavities and Caves.

a. General. It is common that large cavities will be partially filled with residual clays, silt, sand, and/or rock debris. Cavities can be highly irregular in dimension and alignment, and they may have substantial or limited continuity. Accordingly, treatment of each large cavity or cave that is encountered is essentially a sub-project of its own that requires careful formulation of a design, an execution plan, and a verification program. While a wide array of methods, materials, and techniques have been used, they ultimately share some common characteristics: (1) exploration of the feature and preparation of a design and execution plan, (2) initial filling and/or blocking of the cavity using materials and methods that are compatible with cavity size, flow, and head conditions, (3) intensive conventional grouting after initial filling, and (4) verification.

b. Exploration, Assessment, and Design. Whenever a large feature is encountered that cannot be effectively grouted by HMG and normal grouting methods, the feature should be explored extensively to determine its size, alignment, and condition so that an effective treatment program can be designed and executed. Assessment of condition includes determining the extent and nature of infilling materials, evaluating the size and type of interconnected fractures, understanding how the feature is affecting the local flow regime, and assessing flow velocity and flow rate through the feature. After the feature has been characterized, the design requirements must be established in terms of the physical extent of treatment required and the performance requirements for the treatment program. In some cases, the intent might be to create a barrier of limited dimensions, while in other applications, it may be desirable to maximize the physical extent of treatment. The design and field approach must also consider how the treatment will alter the flow regime during and after treatment, as it must be successful under the initial flow conditions and under higher heads and higher velocities that may develop at interim points in the treatment. Treatment materials and methodologies must be consistent with the conditions and the performance requirements for the design. When high-flow rates are present with discharge points in environmentally sensitive areas, careful consideration must be given to the environmental controls that are workable with the chosen design.

c. Cleaning of Cavities in Advance of Treatment. More frequently than not, large cavities are not cleaned before treatment despite the obvious desirability of doing so. Where cleaning has been performed, it is normally accomplished through arrays of closely spaced holes (i.e., 2.5–3 ft), with air and water being injected in pairs of holes and with one or more of the other holes being left open for venting the washed materials (Milanovic 2004). The process is difficult, and the efficacy of washing can be low, depending on the extent and nature of materials within the cavity. Cleaning is rarely performed on projects involving seepage remediation beneath existing dams due to concerns about destabilizing what may be an already fragile situation, and due to concerns about possible damage to the dam itself.

d. Initial Filling of Cavity and/or Construction of Cavity Flow or Grout Containment Barriers. Depending on the nature of the cavity, the flow and head conditions, and the intent of the design, various courses of action can be considered, including initial filling of the cavity and/or construction of one or more barriers intended to either reduce flows through the cavity and/or provide containment for subsequent grouting operations. Various reference sources (Houlsby 1990, Weaver and Bruce 2007, Warner 2004, Milonovic 2004) describe a wide array of techniques and materials and provide numerous case histories for this phase of the treatment program. Under emergency conditions, it is considered acceptable to use anything that can be placed down the hole that will temporarily stop the flow sufficiently to proceed with grouting. Under non-emergency circumstances, it is preferred to consider the materials more carefully so that the cavity plug does not contain materials that would be objectionable or detrimental to the long-term quality. Materials and/or systems that have been used are:

(1) Concrete with anti-washout agent.

(2) Low-mobility grout with anti-washout agent.

(3) Sanded HMG mixes with anti-washout agent.

(4) HMG mixes with co-injection of sodium silicate.

(5) Hot bitumen with co-injection (or immediate subsequent injection) of HMG.

(6) Urethane grouts. For large void filling, it is desirable to inject the urethane grout into pre-placed gravel.

(7) Gravel and/or rock materials.

(8) Grout-inflated geotextile bags attached to pipes penetrating the cavity.

(9) Secant piles constructed with casings that create overlaps when successfully drilled.

e. Note that the order of the listed Materials and/or systems does not indicate preference for one method over another. Some clearly have applicability to smaller openings; others are applicable to large cavities. Combinations of materials might also be appropriate. Chapter 18 of this manual provides additional related discussion on this topic.

f. Final Grouting and Verification. The sole purpose of the initial filling program described above is to prepare the zone for final grouting with HMG, which is necessary to both complete the process and provide the best assurance of long-term performance. Grouting should be intensive, with as many holes as necessary. All holes should be grouted to absolute refusal under the normal maximum program pressure. Numerous verification holes should be used to directly check the quality of the completed work.

CHAPTER 18

Grouting Under High-Head and/or High-Flow Conditions

18-1. Introduction. Certain challenging natural or man-made seepage conditions are difficult to remedy using standard grouting methods. In many instances, high-head and/or high-flow conditions are emergencies that require immediate attention. In these cases, innovation and experience provide the best chance of success. This chapter provides examples of situations where high-flow and/or high-head conditions might be encountered and of methods that have been employed to address such situations.

18-2. High-Flow and/or High-Head Conditions.

a. General. Certain geologic formations, particularly karst and evaporites, lend themselves to the development of potential seepage paths that can convey enormous amounts of water under a relatively low head. Underground construction such as tunnels, shafts, and mines can extend to great depths and encounter features within the rock that apply extremely high heads and convey large volumes of water into the structure. Above-ground construction such as dams and quarries may induce or encounter high-flow and/or high-head situations. The following paragraphs describe some of the circumstances and mechanisms under which high heads and/or flows may develop.

b. Karst Formations. Dissolution-enlarged fractures and large, open dissolution features such as caves within karst formations can be problematic to treat. The proximate permeability of these features may be many orders of magnitude higher than the typical permeability of the formation. These features can extend for long distances, act as an internal drain within the formation, interconnect with surface water sources, and have the capacity to convey enormous amounts of water. Structures that might encounter these types of situations in karst include nearly all forms of above- and below-ground construction. High-flow conditions are quite common and well documented in karst.

c. Underground Construction. Seepage inflows into below-ground construction can be particularly problematic. Deep, below-ground structures run the risk of encountering joints or other features within the rock that convey seepage at very high heads. Pressures may be on the order of hundreds of pounds per square inch, depending on the depth of the structure and the location of the water table, and even small joints and fractures within the rock may convey large amounts of water under these pressures. Seepage into underground structures can be disastrous, requiring extraordinary efforts to address and sometimes resulting in abandonment of the structure. Seepage into salt mines is particularly troublesome, since the inflowing water rapidly dissolves the salt, sometimes resulting in a runaway situation that cannot be remedied. Claims on tunneling projects are often the result of excessive seepage into the excavation during construction (Figure 18-1).

d. Above-Ground Structures. Above-ground structures, particularly water-retaining structures, can induce problematic high flows or high heads. Situations that might contribute to such circumstances include: (1) leaky sheet pile joints in cellular cofferdams (Figure 18-2), (2) joints in dam galleries or concrete monoliths, (3) honeycombed or unconsolidated concrete in power houses, intake structures, or locks, and (4) alkali-silica reactions induced cracking in structures. The number of potential mechanisms that could contribute to a high-flow and/or high-head condition in above-ground structures is nearly limitless.

(Courtesy of Gannett Fleming)

Figure 18-1. Seepage through rock bolt drill holes into a rail tunnel during rehabilitation.

(Courtesy of Cianbro and Colonial Pipeline Company)

Figure 18-2. Seepage under and through a cellular cofferdam structure driven by approximately 20 ft of head.

18-3. Categorization of Conditions. Evaluation of the particular flow and pressure conditions to be remedied is necessary for proper selection of the treatment methods. These conditions can generally be divided into three distinct categories.

a. High Head, Low Flow. As noted above and elsewhere in this manual, even tight joints or fractures may convey large quantities of water. However, compared to flows discussed elsewhere in this chapter, these flow rates are considered low. Conditions such as these are common in below-ground construction at moderate to great depth. A single small fracture or a network of small fractures hydraulically connected to a water-bearing zone can result in significant head and therefore flow to the structure. The gradient into the structure can be extremely high, as pressures just beyond the face or wall of the structure can be at full hydrostatic pressure and rapidly dissipate to atmospheric pressure as the seepage enters the structure. The result may range from a high-velocity, pressurized stream of water to a slow seep, depending on the ability of the fracture network to supply seepage to the entrance point. Heads can be on the order of hundreds of psi, and flows may range from a seep to hundreds of gallons per minute.

b. High Flow, Low Head. Conditions such as these are common in karst at relatively shallow depths. Since the permeability of a dissolution feature may be several orders of magnitude greater than the parent material, the feature may behave as an infinite drain from a limited supply. Therefore, the head driving the flow will likely be low, but the amount of flow may be extremely high. Heads may be negligible (under pre-treatment conditions), but flows might be on the order of hundreds or thousands of gallons per minute. After successful treatment, substantially higher heads might develop upstream from the treatment zone due to elimination of the flow path.

c. High Head, High Flow. These conditions are typically the most difficult to treat. They have been encountered in quarries located in karst environments adjacent to surface water sources. Large, open joints in rock abutments at dams can apply the full reservoir head and convey enormous amounts of water downstream. A relatively large fracture or fractures, or a conduit connected directly to an unlimited water supply, is required to generate these conditions. Flows may be on the order of thousands of gallons per minute, and the pressure is limited only by the driving head generated by the supply.

18-4. Assessment of Conditions.

a. General. The features or mechanisms that contribute to the seepage condition must be known to adequately design treatment measures and implement the measures in the field. In some cases, there might be obvious field evidence regarding the source of the seepage (Figure 18-3). In other cases, the source of the inflow may ultimately never be determined because of its tortuous path or because it may not be possible to investigate certain locations. The properties that ultimately control the design of the remedial grouting method are the geologic and/or man-made conditions, the flow rate, and the driving head.

(Courtesy of Dr. Donald Bruce)

Figure 18-3. Source of major inflow into an adjacent open pit quarry excavation through a solution feature.

b. Geologic and/or Man-Made Conditions. Previous information such as structure drawings, inspection reports, construction photographs, reconnaissance photographs, geologic reports, site investigations, and site characterization reports should be reviewed for any indication of the possible seepage path. It is unlikely that there will be a general consensus among practitioners regarding the exact seepage path simply by review of this information. Even if there is a high level of confidence in the data, additional investigations should be conducted for confirmation. Numerous methods may be employed, including drilling of exploratory borings, pressure testing at short stage intervals, down-hole camera logging, geophysical methods, groundwater temperature measurements, groundwater chemistry and pH measurements, and dye tracing.

c. Flow Quantity and Driving Head. Seepage quantities and piezometric heads should be measured at the discharge location and within the recharge area. Weirs can be erected to measure large discharges and can be instrumented to so that real-time flow rate measurements are available. Other sources of flow measurements include the pumping rate from a dewatering system and borehole flow meters. Piezometric heads can be obtained from exploratory drilling, nearby wells, and the elevations of nearby surface water. The installation of automated piezometers in exploratory holes and nearby wells allows real-time measurement of groundwater levels before, during, and after the remediation.

18-5. Common Treatment Methods.

a. General. The treatment methods described below involve temporarily stopping the flow, followed by construction of a long-term, and (one would hope) permanent, solution. Temporary flow stoppage is typically the more difficult task and requires innovative techniques, while the permanent repair typically involves more conventional grouting methods such as LMG and/or HMG.

(1) Temporary Flow Stoppage. Temporary flow stoppage is required to enable construction of the permanent plug, which is typically LMG and/or HMG or the construction of a bulkhead. Many methods have been employed, some of which are discussed in the following paragraphs. Essentially, the only requirement is to stop the flow, effectively, quickly, and cost effectively.

(2) Upstream Entrance Control. Whenever the upstream entrance source of water is well-defined and accessible, blocking the flow at that point can be highly advantageous. Geomembrane systems installed over a properly prepared surface can greatly reduce or eliminate problematic flows.

(3) Blocking Agents. Injection of cement grout with a high concentration of blocking agents has been effective on some projects. There are commercially available products that can be added to cement grouts during mixing. However, nearly any available material that can be pumped and that provides some blocking properties may be used. A review of notable grouting literature (Warner 2004, Weaver and Bruce 2007, Houlsby 1990) indicates that the following materials have been used as blocking agents: sawdust; wood shavings; almond husks; ground walnut shells; newspaper; scrap plastic; synthetic fibers including polypropylene, nylon, polyethylene, and polyolefin; plastic tabs and punchings; plastic grocery bags; feed grains; bran; straw; cornstalks; cane fibers; grass; clay; sand; gravel; shredded rubber; perlite; and plaster. In some cases, the injection of grout in combination with one of these materials is intended to be the permanent repair; however, more commonly it is intended to create a temporary flow blockage. Temporary blocking agents might or might not prove suitable for the flow and head categories described above. Some experimentation may be required to determine what form of blocking agent will be effective or to evaluate whether the blocking agents will work at all.

(4) Batten Bars and Plates. Batten bars and plates have been used when the seepage entrance point is confined to a relatively small area or defect. High-head, low-flow conditions are most amenable to this method. Common applications include rock fractures in tunnels and underground construction and leaking joints in concrete dams. The method involves anchoring or jacking steel plates against the seeping surface. Gasket material between the plate and seeping surface can be employed to develop a nearly watertight seal, provided the plate is anchored sufficiently such that the developed head does not extrude the gasket. In many cases, valves installed on the plate at regular intervals allow the seepage to pass through during plate installation. After being installed, the valves are closed, and pressure behind the plate will increase until equilibrium is reached, which might result in the development of additional seepage exits at other locations. Grouting is conducted by injection through the valves or by drilling additional grout holes equipped with blow-off preventers to intersect the feature. If treatment of the primary defect results in re-emergence of the seepage at secondary locations, the same methodology can be employed. Numerous applications of the method at varying locations may be required to achieve the desired result. After grouting, the plates can either be removed or remain in place. The procedure described above constitutes the construction of both the temporary flow stoppage measures and the permanent plug; therefore, it is unlikely that additional methods will be necessary provided that unsatisfactory seepage conditions do not reappear.

(5) Structural Plugs. TM 5-818-6, *Grouting Methods and Equipment*, describes two methods for constructing a structural plug within features conveying water at depth. The first method involves drilling closely spaced, cased holes though the feature. The casings are grouted or concreted and left in place, and then a secondary series of cased holes is drilled such that they overlap the primary holes. The casings must be of a material other than steel to permit the drilling of the secondary holes. The result is a series of fully penetrating overlapping casing columns that greatly reduces the conveyance capacity of the feature. Cement grouting is then initiated upstream. This method may be suitable for any of the high-flow conditions described above. The second method of constructing a structural plug involves introducing inflatable geotextile or barrier bags into the feature attached to lengths of pipe. The bags are pumped full of grout or mortar until they expand and contact the adjacent bag or feature sidewalls, thereby greatly reducing flow within the feature. After the inflation grout achieves its set, additional grouting is initiated upstream. Construction of barrier bag systems may not be possible if extremely large features or high heads are encountered, because the bag could be torn free from the injection system.

(6) Bitumen. Injecting hot bitumen is the most common method for stopping high flows with or without high head. Bitumen grouting is discussed in detail in Chapter 11 of this manual.

(7) Polyurethane Grouts. Naudts (2003) describes the growing use of polyurethane grouts in combination with cement grouts for treatment of high flows and high heads. Polyurethane grouts have controllable set times that can be varied from seconds to minutes (Weaver and Bruce 2007). They are also expansive when unconfined, with some reports noting expansion of 30 times the injected volume (Warner 2004). The abilities to expand and rapidly set are greatly beneficial under most high-flow, high-head circumstances. For additional information on polyurethane grouts, refer to EM 1110-1-3500, *Chemical Grouting*, Warner (2004), and Naudts (2003).

(8) Set Accelerators.

(a) The primary difficulty in using cement grouts to control high-flow situations is the rapid expulsion of the grout from the target feature before setting. In these instances, the use of set accelerators can provide great benefit. The set times are highly variable, depending on the accelerator and the dosage. In some cases, flash set can occur, which can be beneficial when grouting a flowing water condition, but can also be catastrophic for equipment. Weaver and Bruce (2007) identify several accelerators, some common and some uncommon, and describe in detail both the beneficial properties they impart and the drawbacks of their use.

(b) The two most common accelerators are calcium chloride and sodium silicate. Calcium chloride is typically proportioned in the range of 1–6% by weight of cement; 2% is a common dosage (Weaver and Bruce 2007). Both Weaver and Bruce (2007) and Warner (2004) note that higher concentrations have been known to reduce set times to as little as a few minutes, and in some cases, flash setting can occur. Sodium silicate is another accelerator that can be used to control set times. Warner (2004) notes that cement-sodium silicate grouts are very effective where water is moving through the host deposit. Weaver and Bruce (2007) note that sodium silicate in concentrations of less than approximately 20% by weight of cement causes the cohesion of the mix to rise initially, but then remain constant until initial set. Concentrations above approximately 20%

may cause flash set. Table 18-1 (from Warner 2004) shows a range of sodium silicate concentrations and the approximate set times and associated applications for such mixes.

Table 18-1. Range of approximate grout set times for cement-sodium silicate grouts.

Cement-Silicate Ratio by Volume*	Time of Set	Typical Usage
1:1	1–10 s	Controlling flowing water
10:1	1–10 min	Controlling flowing water and limiting grout mobility
30:1	10–20 min	Compounding rapid-setting cement grout
50:1	30–60 min	
*After Warner (2004).		

(c) In the application for water control where very fast set times are desired, the potential for grout flash setting in the mixing, pumping, and injection equipment is a real hazard. In these instances, it is common for the two prime components (grout and sodium silicate) to be mixed immediately after the components exit the injection equipment. This can be accomplished in one of two ways: injection of the two component into separate, but adjacent boreholes into aggressively running water such that the high water velocity mixes the components; or twin-stream mixing where both components are injected through a multi-conduit packer and mix immediately at the base of the packer. Some authors note that the fast setting of cement grouts may produce a weak and brittle grout and that a more substantial plug must be constructed upstream.

b. Permanent Plug Construction. As noted above, temporary flow stoppage is necessary simply to allow construction of the permanent plug. Typically HMG and LMG are used in constructing the permanent plug, but in some cases concrete and flowable fill may be injected, followed by grouting. The issues with respect to construction of the permanent plug include the required length of plug necessary to resist blow-out or extrusion from the induced head upstream, and the required level of seepage reduction necessary to achieve the desired results. In some cases, relatively high levels of residual seepage may be acceptable so tight grouting of the plug may not be necessary. On the other hand, if only minimal seepage is desired, multiple rows of grouting adjacent to, beneath, and upstream of the permanent plug may be required.

c. Other Considerations. One practical matter to consider is that the temporary plug is, as its name denotes, a short-term measure. Construction of the permanent plug should be initiated immediately after stoppage of flow to take best advantage of the circumstances. In some cases, the temporary flow stoppage may have a short life span and may quickly deteriorate, resulting in exponentially increasing flow rates. Depending on the head driving the seepage, after the temporary plug is installed, a rapid rise in piezometric levels might occur. As a result, adjacent structures could be adversely impacted and seepage outbreaks may occur at other locations. These potential impacts require that frequent and sometimes continuous monitoring and inspection of the site features be conducted. Real-time monitoring systems can reduce the manpower requirements for these efforts. If seepage outbreaks do occur, they require quick decision making and flexibility from both the engineer and contractor.

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CHAPTER 19

Special Considerations for Cutoff Wall Pre-Grouting Programs

19-1. Introduction.

a. Cutoff Walls. Cutoff walls have become an important tool for remediating seepage problems at dams. Bruce et al. (2006) report that since completion of the Wolf Creek Dam cutoff in 1974–1978, approximately 30 such projects have been completed in North America using a variety of techniques and materials, including concrete diaphragm walls constructed either in panels or as secant pile structures, soilcrete walls constructed by deep soil mixing equipment, and soil-bentonite walls. In the majority of applications, the purpose of the cutoff wall has been to reduce seepage occurring either through the dam embankment or through pervious soils beneath the embankment. These types of cutoff walls typically terminate either at the top of rock or at a short penetration distance into rock.

b. Cutoff Wall Depths. A small number of walls have had the primary purpose of reducing seepage occurring at substantial depths within the underlying bedrock. Most commonly, these projects are concrete cutoff walls located at karst sites (Mississinewa Dam in Indiana, W.F. George Dam in Alabama, Beaver Dam in Arkansas, Clearwater Dam in Missouri, Center Hill Dam in Tennessee, and Wolf Creek Dam in Kentucky) or at sites with other types of permeable rock (Fontenelle Dam in Wyoming and Navajo Dam in New Mexico).

c. Cutoff Walls in Fractured Rock. When cutoff walls are intended to penetrate fractured rock, special technical issues arise due to the potential for sudden loss of slurry through fractures or cavities in the rock. Such slurry losses can result in trench collapses endangering and/or damaging the embankment, creation of safety hazards for personnel, and potential loss of equipment. When these conditions and risks are known to exist, it has become common to use a systematic pre-grouting program in advance of cutoff wall construction. This chapter discusses some of the general aspects of cutoff walls and provides guidance on some of the special considerations involved in systematic pre-grouting programs.

19-2. Aspects of Cutoff Wall Construction and Performance.

a. General. Cutoff walls have been installed with verified satisfactory verticality and continuity to depths of at least 223 ft using clamshells, 280 ft with secant piles, and 402 ft with hydromills (Bruce et al. 2006). Clamshells are suitable for unconsolidated materials, secant piles are better suited for hard rock conditions, and hydromill panel walls are best suited for soft to moderately hard rock. Technological advancements in this method are increasing the capabilities of the equipment and the rock strengths that can be excavated. Figures 19-1 through 19-3 show the clamshell and hydromill excavation equipment used for constructing the cutoff wall at Mississinewa Dam.

Figure 19-1. Hydraulic clamshell and hydromill machines at Mississinewa Dam (Louisville District).

Figure 19-2. Cable-operated clamshell for overburden excavation at the Mississinewa Dam cutoff wall (Louisville District).

Figure 19-3. Hydromill cutter wheels for limestone excavation at the Mississinewa Dam cutoff wall (Louisville District).

b. Excavation Stability. Stability of cutoff wall excavations is primarily the result of maintaining an adequate head of bentonite slurry in the excavation at all times. In soils, the bentonite slurry penetrates a short distance into the soils and forms a low-permeability skin on the side walls of the excavation, which acts as a membrane upon which the slurry exerts a stabilizing pressure. The depths of penetration are reported to range from 1 to 3 in. in sand, 3 to 6 in. in sand and gravel, and up to 12 in. in gravel (EM 1110-2-1901). No data are known to exist that relate rock fracture size to slurry penetration distance or slurry loss rates. Equations for analyzing trench stability during excavation under various soil and groundwater conditions are contained in EM 1110-2-1901, *Seepage Analysis and Control for Dams*. It is not uncommon for the excavation factors of safety to be quite low. Excavation stability can be enhanced by limiting the size of each excavation such that arching effects provide stability. Maintaining proper slurry levels at all times is critical to the excavation stability, and uncontrollable loss of slurry will commonly cause trenches to collapse.

c. Cutoff Wall Construction Issues. Cutoff walls are often termed “positive cutoffs” because they are designed to be continuous, interconnected elements constructed of low-permeability materials. Under ideal conditions, the concrete elements provide their barrier function independently of the foundation conditions. However, experience has shown that satisfactory construction requires skill and care in every aspect of the operations, and even under those circumstances, there can be constructability and performance issues. Besides simple geometric misalignment, other issues that have been identified in cutoff wall construction, both in dams and in urban excavation retaining structures, include:

(1) Uncontrolled Slurry Loss in Embankments. Uncontrolled slurry losses have occurred during construction of cutoffs in embankment dams. Such slurry losses can occur in low-stress zones that exist in the embankment and at locations where the embankment has been damaged by prior seepage. The slurry loss in these situations is attributable to hydrofracturing, which can induce cracking of the dam over large regions.

(2) Uncontrolled Slurry Loss in Rock. Uncontrolled slurry losses have occurred during construction of cutoffs in pervious foundation rock. Such slurry losses occur when the fracture size is such that the slurry, with or without suspended solids, is not capable of blocking the fractures.

(3) Trench Collapse or Enlargement Due to Extremely Poor Ground Conditions. As indicated previously, the factors of safety for slurry trench excavation are normally not high, and the ability of the trench to be supported is a function of soil type, soil properties, and slurry characteristics. However, zones or strata can be encountered where the factor of safety is inadequate, and trenches can collapse.

(4) Segregation of Concrete. Mix design and field control of concrete mixes are critical to prevent segregation, honeycombing, and other defects in the cutoff wall. These types of defects have been observed in projects where subsequent excavation has exposed the slurry walls and where post-construction exploratory drilling of the wall was conducted.

(5) Soil or Slurry Inclusions Within Panel Elements. The occurrence of soil and/or slurry inclusions can be highly significant if they completely penetrate the panel elements. Inclusions can occur as a result of local soil sloughing and/or disruption in the flow of fluid concrete during panel construction. Soils particularly prone to creating inclusions include weak silts, weak silty clays, and highly fissured over consolidated clays.

(6) Vertical Panel Joints. Vertical panel joints are a recognized source of cutoff wall leakage. Depending on many factors related to the construction process, the rate of leakage can range from minimal to severe. Under best practices, where the joint surfaces are thoroughly cleaned while fresh bentonite slurry is circulated, leakage may be minimal due to a thin layer (i.e., < 1 mm) of contaminated bentonite on the joint surfaces. Under less ideal conditions, bentonite seams of up to ¼ in. have been found to exist at panel joints (Bruce 2007). Systematic cleaning of the these joint surfaces with specially designed brushes proved to be very effective at Wolf Creek Dam in Kentucky.

d. Performance Efficiency. In 1986, EM 1110-2-1901 indicated that the experienced performance efficiency of concrete cutoff walls, determined in terms of head reduction efficiency, was on the order of 90% or better for properly constructed walls. Flow rate reduction efficiency was not reported. No additional data are available on recent projects nor is there an update available on the long-term performance of the walls constructed before 1986.

19-3. Systematic Pre-Grouting Programs for Remedial Cutoff Wall Projects.

a. General. When remedial cutoff walls are to be constructed through embankments and into permeable foundation rock, systematic pre-grouting programs have been found to provide significant benefits beyond reducing the possibility of slurry loss. This section discusses the various functions that the programs can serve.

b. Embankment Characterization. One high-value function of systematic pre-grouting programs is intensive exploration. Normally when a remedial cutoff wall is being considered, the embankment is either known or suspected to be damaged, but neither the extent nor the cause of damage is fully known. Closely spaced grout holes (i.e., 5-ft centers or closer) drilled through the embankment and into the rock allow identification of damaged zones in the embankment and/or problematic ground conditions that might adversely affect cutoff wall construction. Advance knowledge of the conditions allows the selection of appropriate contingency or supplemental embankment preparation measures.

c. Soil/Rock Interface Characterization and Treatment. Systematic pre-grouting borings provide thorough characterization of the soil/rock interface zone, including its nature, variability, and permeability, all of which are important in cutoff wall construction.

d. Rock Characterization. Systematic drilling, pressure testing, and pre-grouting of the rock foundation is the most comprehensive method for determining the required depth of the cutoff wall, determining zones where grouting is acceptable as the sole long-term solution, and nearly eliminating the risk of slurry loss during subsequent excavation into the bedrock. Systematic pre-grouting can also provide immediate "emergency" benefits to the project far in advance of construction of the cutoff wall.

e. Composite Barrier Action of Grouting and a Cutoff Wall. An additional benefit of systematic pre-grouting is that it will promote a combined barrier action, resulting in higher effectiveness and a longer service life. Grouting and cutoff walls can both have defects, but it is substantially less likely that the defects will occur at the same locations. Therefore, grouting can either impede or prevent flow at cutoff wall defects, and the cutoff wall in turn can reduce stress on grouted zones (see Figure 19-4).

Figure 19-4. Pre-grouting for the Mississinewa Dam cutoff wall.

19-4. Basis of Design.

a. General. The basis of design for pre-grouting programs for cutoff walls is essentially identical to the design of other grouting programs. The physical dimensions of the zone to be grouted and the required permeability of the grouted zone must be appropriate for the desired performance.

b. Slurry Loss Prevention. No published information appears to exist that provides specific guidance on the size of rock fractures in which slurry loss cannot be controlled. At Mississinewa Dam (Louisville District), a simplified analysis indicated that if a completely grouted zone having a residual permeability of 10 Lugeons or less extended 5 ft beyond the walls of the slurry trench, the slurry loss rate would be manageable for the infrastructure system being used on that project. The calculation was based on a simple $Q = kiA$ model for the proposed panel size with adjustments for slurry density and viscosity. The calculation essentially modeled a panel excavation filled with clean slurry. On the Mississinewa project, pre-grouting was not included in the original contract, but was found to be necessary when, shortly after the foundation rock was penetrated, the test section area could not be completed without sudden and complete loss of slurry. The 10-Lugeon criterion was adopted for the project, and after pre-grouting, the concrete cutoff wall was successfully completed without any further incidents of slurry loss. Grouting to a residual permeability of less than 10 Lugeons would have been possible, but it was not required since the cutoff wall depth had been predetermined and it was decided to extend the wall down to the full depth of required treatment. Final hole spacings varied between 5 and 10 ft, depending on the area, and less than absolute refusal was found to be acceptable to achieve the 10-Lugeon result.

c. Composite Barrier Action and Composite Barrier Systems. Where pre-grouting is being used to define the depth of the cutoff wall or as the sole treatment in certain zones of the foundation, or when pre-grouting is desired to provide composite barrier action in concert with the cutoff wall, the design approach is slightly different. The required dimensions and the residual permeability must simultaneously satisfy the needs of a permanent grouted barrier and also slurry loss requirements in the cutoff wall zone. In general, this will lead to a tighter grouted barrier system, often in the range of 1–5 Lugeons. Additionally, all holes should be grouted to absolute refusal.

19-5. Number of Lines, Hole Spacing, and Line Spacing.

a. General. It is highly desirable to minimize drilling within the proposed footprint of the slurry trench excavation and within the zone of expected drilling deviation. The loss of drill tooling in the excavation area is the primary concern. Whenever possible, drilling in that region should be limited to verification holes performed after grouting is believed to be complete. Drilling risks are substantially reduced after pre-grouting is complete.

b. Number of Lines. Experience to date indicates that, in most cases, one line of grouting on each side of the cutoff wall location is required to produce a suitable grouted zone for cutoff wall construction. At Mississinewa Dam, one element of the grouting test program was to evaluate whether one line would suffice or not. It was found that two lines were in fact necessary. Where cutoff wall depths are extremely deep and line spacings are wide due to the wall depth and allowances for drilling deviation, it might be necessary to include an additional line of grouting within the proposed excavation area if indicated by the verification hole results. However, it is preferable to try using higher grouting pressures and longer grouting duration to maximize grout travel rather than to add an interior grout line.

c. Hole Spacing. Holes must be spaced sufficiently close to intersect all fractures at a sufficient frequency that grout will penetrate at least the half distance between the penetration points. Where special features, such as dissolution features, are known to exist, both the size and the frequency of the features must be considered in determining hole spacing. The efficacy of any given hole spacing can only be evaluated through extrapolation of closure results and verification testing.

d. Line Spacing. The line spacing should be the minimum necessary to prevent hole deviations from entering the footprint of the proposed excavation (e.g., see Figure 19-5). Wide spacing of lines creates a risk of an improperly grouted zone between the lines, with a potential to cause slurry loss along the axis of the dam. When line spacings are in question, it is recommended that a tighter spacing be used initially, along with borehole deviation surveys, to determine the actual extent of deviation. If the initial spacing results in penetration of the excavation space, the line spacing should then be increased.

Figure 19-5. Pre-grouting lines for the Mississinewa Dam cutoff wall (Louisville District).

19-6. Grouting Pressures. Grouting pressures for systematic pre-grouting are established using guidelines provided elsewhere in the manual, with the exception that the refusal pressure should be checked to ensure that it substantially exceeds the excavation slurry pressure that will be applied. Pre-grouting for cutoff walls will involve grouting of zones containing mixed materials or infilled fractures that are not particularly favorable for grouting. Grouting pressures are applied for a very brief time, whereas slurry pressures might be applied for days or even weeks in a particular panel. Accordingly, it is recommended that holes be grouted to refusal at least twice the applied slurry pressure, provided that it is otherwise safe to do so. Careful consideration must be given to the potential to cause damage using these methods when working in or under existing dams or similar critical structures. Higher pressure multiples, when possible, are desirable. The slurry pressure used in the calculation should be based on the maximum density of slurry that is permitted.

CHAPTER 20

Blanket Grouting and Consolidation Grouting

20-1. Introduction.

a. General. "Blanket grouting" and "consolidation grouting" are terms that describe grouting programs that are performed over relatively broad areas and to relatively shallow depths for the purpose of improving the properties of rock. While these terms are sometimes used interchangeably, Weaver and Bruce (2007) and Warner (2004) recommend drawing a clear distinction between the two terms because their purposes and methods differ substantially. This manual recognizes and follows the recommended differentiation based on these definitions:

b. Blanket Grouting. Blanket grouting is defined as shallow, off-curtain grouting performed beneath dams to decrease permeability, reduce flow velocities, and/or redirect flow lines away from critical areas. Functionally, it is an integral extension of the grout curtain.

c. Consolidation Grouting. Consolidation grouting is defined as grouting performed to improve the mechanical properties of rock foundation materials. While consolidation grouting might also alter the hydraulic conductivity, its primary function is to beneficially alter the behavior of the rock mass under applied loads.

20-2. Blanket Grouting.

a. Applications. As described above, blanket grouting is an integral extension of the grout curtain for dams. It is a common and normally recommended element for embankment dams, but it is also often used for upstream portions of concrete dam foundations. Figure 20-1 shows a layout for blanket grouting from Portugues Dam, Jacksonville District, USACE.

(1) In embankment dam applications, the lateral extent of blanket grouting is normally the width of the core contact area unless further extension is warranted to protect other zones of the embankment. The most vulnerable portion of earth dams is normally the core contact area, and it is often the most difficult zone to treat. Blanket grouting the entire core contact area of embankment dams substantially increases the level of protection for the core by decreasing flow rates and flow gradients. If the full width of the core contact area is not blanket grouted, the grout curtain will produce full hydrostatic heads at its upstream limit and can induce very high gradients at the base of the core. Under some circumstances, this can result in hydrofracturing of the embankment at that location or erosion of embankment material resulting in the potential for loss of core material into the downstream fractured rock.

(2) In concrete dam applications, the benefit of blanket grouting is enhanced seepage reduction rather than internal erosion protection. Typically, the blanket grouting zone will extend from the grout curtain upstream to the heel of the structure. This zone is sometimes treated after the concrete dam has been constructed so that the blanket grouting can be performed under higher pressures while simultaneously filling fractures in the rock foundation and grouting the contact between the concrete and the rock.

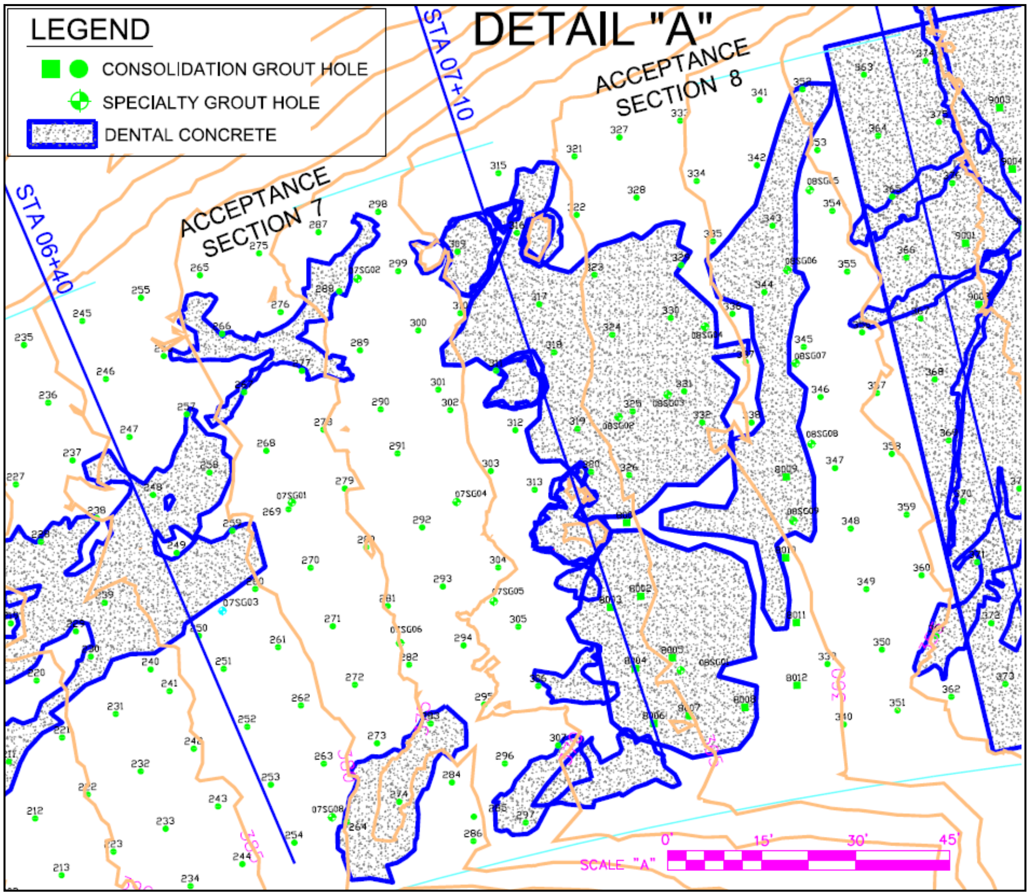

Figure 20-1. Blanket grout hole layout from the Portugues and Bucana Rivers Project, Jacksonville District, USACE.

b. Depth of Treatment. The required depth of blanket grouting is determined by evaluating the geologic conditions and assessing the potential risks and benefits for a particular structure. The depth can be as little as one stage, which would result in treatment of 10–15 ft of rock, but for large dams that have problematic foundations or contain extremely sensitive core materials, it might consist of multiple stages extending to depths of 30–100 ft. Even on smaller structures, more than one stage depth may be of value if a particularly high-permeability zone exists at shallow depth. Blanket grouting should always be included in seepage models for the dam to evaluate both its overall effect on behavior and the benefits of various vertical and horizontal treatment zones.

c. Materials, Means, and Methods. Blanket grouting should be viewed as an integral extension of the grout curtain, and as such, the intensity of grouting; the materials, means, and methods used (balanced stable grouts, absolute refusal, etc.); the degree of care and control exercised; and the level of verification should be identical to those used for the curtain.

d. Sequencing. Normally, it is preferable to complete blanket grouting in advance of the curtain grouting. This sequence will reduce difficulties in curtain grouting and may facilitate the use of higher pressures in the curtain construction.

20-3. <u>Consolidation Grouting Principles and Basis of Design</u>.

a. General. The intent of consolidation grouting is to fill open fractures to improve the structural properties of the rock mass. It can be used to reduce deformations associated with closing of fractures under applied loads, it can be used to treat locally fractured zones and thereby homogenize the foundation, and it can be used to fill fractures for the purpose of reducing movement of rock blocks that might otherwise loosen during excavation and/or tunneling operations. Consolidation grouting can also be used for foundations in karst areas to increase the level of assurance of adequate foundation support. The enhancement value of consolidation grouting depends on the rock mass conditions. Greatest benefit will occur in highly fractured rock masses with a predominant number of open joints.

b. Deformations in Fractured Rock. Rock masses are composed of intact blocks of rock separated by fracture discontinuities. Under applied stress, the rock mass behavior is governed by the interaction between the intact blocks and the discontinuities. As a result, fractured rock masses seldom behave as an ideal elastic material, and the stress-strain responses are typically not linear. The total vertical deformation of a fractured rock mass under applied load consists of an initial non-linear strain (represented by the initial tangent modulus), reflecting the effects of discontinuity closure, followed by a linear elastic deformation (elastic modulus). The modulus of deformation, which is a representation of total combined strain at any given stress level, is determined from the slope of the secant line on the stress-strain curve between zero and some specified stress. EM 1110-1-2908, *Rock Foundations*, provides illustrations of these components and provides much more detail on deformation behavior, and it describes numerous test methods and empirical methods for estimating the in-situ deformation modulus. It also provides guidance on circumstances when it might also be necessary to consider lateral deformation in analyses.

c. Basis of Design. The basis of design selected for any particular application will be determined by the geologic conditions, the load intensities, the sensitivity of the structure, and the criticality of the structure.

(1) Small Non-Critical Applications. Consolidation grouting for many routine applications may be non-critical, but it is used as a precautionary measure or to increase the level of confidence that foundation conditions will be both suitable and uniform. Typical applications include improvement of localized zones of highly fractured rock and treatment beneath individual foundation elements. In these types of applications, the cost for treatment is relatively low for the level of protection provided, and the basis for design is often intuitive rather than analytical.

(2) Critical Applications. Critical applications can include large concrete dams, intake towers, and structures containing movable features such as lock and flood control gates. In these types of applications, the basis for design can be extensive testing, including in-situ deformation tests, finite element deformation and stress analyses, and properly scaled field pilot tests to measure the benefits produced by consolidation grouting. Guidance on approaches to these elements is contained in EM 1110-1-2908.

(3) Intermediate Applications. Intermediate applications fall between the non-critical and critical applications. Intermediate applications include those situations where the cost of

consolidation grouting is sufficiently high that a purely intuitive basis is insufficient for making a decision on whether to grout, but where either the size or the sensitivity of the project is not sufficient to substantiate the time and effort required for the comprehensive process used for critical applications. Concrete dams less than 100 ft in height commonly fall into this category. For intermediate applications, it is common to estimate the rock deformation modulus based on empirical methods that are related to one or more rock quality or rock rating system, calculate the differential settlements, and compare them to allowable values for the type of structure to be built. If it is found that the differential settlements are clearly tolerable, then consolidation grouting is not required. If it is found that the estimated differential settlements approach the point of concern, a second iteration is performed by assuming the fractures to be grouted, adjusting the rock rating parameters accordingly, estimating a deformation modulus for the improved condition, and recalculating differential settlements.

d. Evaluation of Impact on Flow Behavior. In most applications, consolidation grouting does not have the potential to create negative impacts. However, that is not necessarily true for dams, particularly when consolidation grouting is performed on the downstream side of the core trench in embankment dams or downstream from the grout curtain in concrete dams. Although the intent of the consolidation grouting might only be structural alteration of the rock mass, it will, in fact, alter the hydraulic conductivity and the seepage flow paths. In concrete dam foundations, it can render a planned drainage system ineffective unless the drains extend well below the grouted zone. In earth dam foundations, it can block seepage from entering the blanket drain system and direct seepage to exit at the downstream toe. Whenever consolidation grouting is being considered for dams, the consolidation grouting zone must be included in the seepage model so that the effects are clearly understood and the design properly accommodates those effects.

20-4. Consolidation Grouting Equipment, Materials, and Methodology.

a. Grouting Equipment. The equipment for mixing, handling, and placing consolidation grout is the same equipment normally used for HMG.

b. Mixes. The grout mixes used for consolidation grouting are normally either balanced stable grouts or neat cement grouts. Although neat cement grouts generally develop a higher compressive strength and have a shorter set time, either type of mix would normally suffice, and the choice of one versus the other is predominantly based on the project size and type rather than on specific property differences. For example, on a dam project where balanced stable grouts are being used extensively in a hydraulic barrier application, it would be logical to use the same grouts for consolidation grouting. On a foundation improvement project where the only activity is consolidation grouting, it might be more logical to use neat cement grouts to limit execution complexity. Where neat cement grouts are used, the starting mix should not be thinner than a 2:1 water-cement ratio (by volume). If it is found that thickening is routinely required during the grouting, the starting mix should be changed to a 1:1 water-cement ratio.

c. Staging. Grouting can be performed using either upstage or downstage methods as required by the foundation rock conditions. Stage lengths of 10–15 ft are recommended.

d. Water Pressure Testing. Water pressure testing is normally not used in consolidation grouting programs.

e. Pressures and Refusal. Grouting pressures should be similar to those used in curtain grouting. Stages should be grouted to absolute refusal and then locked off at the desired pressure and held for 15 minutes or until all measurable pressure has dissipated in the stage.

f. Hole Patterns. Holes are drilled and grouted using the split-spacing method. Primary hole spacings are commonly on 40- to 80-ft centers, and final hole spacings are typically on 5- to 10-ft centers, depending on closure.

g. Closure. The determination of satisfactory closure is based on records of grout take evaluated in a manner similar to curtain grouting. Verification holes should be used to confirm closure extrapolations.

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CHAPTER 21

Grouting Considerations for Conduits

21-1. Introduction. Grouting is conducted within or adjacent to conduits for a wide variety of reasons, ranging from treatment or repair of the conduit to complete abandonment. Described in this chapter are common conduit grouting applications and special considerations that pertain to conduits.

21-2. Annular Space Grouting.

a. General. Annular space grouting occurs in many applications. The scope of annular space grouting discussed in this chapter is limited to grouting the annular space between a liner (also commonly referred to as a “slip liner”) and an existing host pipe. Similar applications can include the grouting of the annulus between host and primary conduits in micro-tunneling and pipe jacking applications. However, in those applications, the annulus is commonly not grouted to facilitate future replacement of the primary conduit, if replacement should become necessary.

b. Applications. Slip lining of conduits is a trenchless pipe replacement method. Liners are commonly installed in existing conduits as a remedy for deterioration of the host pipe. Liners also can be used to improve the hydraulic capacity of an existing conduit. The space between the host pipe and liner is typically grouted to restrain the liner, prevent seepage between the conduits, and strengthen the liner system.

c. Liner Materials. The most common liner materials in slip lining applications are high-density polyethylene (HDPE) with fused joints, HDPE with mechanical joints, structural steel pipe, and polyvinyl chloride (PVC). Cured-resin-impregnated felt liners (inversion lining) and folded and formed PVC liners have been used in conduit repair; however, grouting of these liners is not required. The type and size of liner will dictate the methods used for injecting grout into the annulus.

d. Grout Materials. Materials used for annular space grouting are similar to those used in other HMG applications. Like other HMGs, zero bleed should be a design requirement. The only other concern specific to the grouting of annular spaces is the density of the injected material. Liners must be restrained from movement during injection. The movement is due to the buoyant forces imposed on the conduit during grout injection. In essence, the liner floats within the injected grout before setting of the grout. The amount of force necessary to provide restraint depends on the displacement volume and the density of the grout. Denser grout results in a higher buoyant force, so larger restraints are required at more frequent intervals.

(1) To minimize buoyant forces on conduits, many designers specify that foamed grouts be used (Figure 21-1). The addition of a closed cell foam to the grout during the mixing process results in a lower-density material. Grouts with specific gravities less than 1.0 are easily achieved. Compressive strengths are typically suitable for most applications, but should be verified by testing. The only concern with regard to using such low-density material is the ability to displace water in the annulus. If the specific gravity of the injected material is less than that of water, the grout will float on top of any water in the annulus, resulting in areas of

incomplete filling. Unless a dry annular space can be guaranteed, it is recommended that all grouts used for grouting of annular spaces be formulated with a minimum specific gravity of 1.1.

(2) For slip lining of a conduit for structural repair, the strength of the grout may be an important consideration. Grout strength can vary widely, depending on the mix. It is recommended that, if a specific minimum grout strength is required, a performance-based specification should be used to make best use of the contractor's resources and experience. Frequent casting of grout cylinders is necessary for verification of the production work. The entire annular space between the HDPE or steel liner and the existing conduit should be grouted. This decreases the possibility of point loads on the liner and adds strength to the composite system.

(Courtesy of Advanced Construction Techniques)

Figure 21-1. Double-drum paddle mixer with a cement auger feed and foaming device (lower left).

e. Grouting Methods. Annular space grouting is typically conducted using one of two methods. If the pipe is accessible, the interior of the liner can be equipped with injection ports such that the grouting is performed from inside the liner. The limiting liner size for performing this work depends on equipment and labor access, and a conduit approximately 3 ft in diameter is generally considered a lower limit. For smaller conduits, or conduits that otherwise prohibit direct access, slick lines are used. Slick lines are injection conduits fixed to the exterior of the liner before liner insertion. They allow the grouting to be performed from outside the conduit.

(1) Injection Port Method. Threaded couplings or ports are installed through the pipe liner to facilitate grout injection into the annular space using these couplings. Grouting is best accomplished in lifts. Grout injection should begin at the ports located on the liner invert, then progress to subsequently higher ports on the liner. When grout return is observed at the 45-degree ports above the invert on each side of the invert, then the grout injection proceeds at these ports until return is observed at both springline ports. Next, grout is injected at the ports located at the springline until return is observed at the ports 45 degrees above the springline. Then grouting commences at the ports 45 degrees above the springlines and continues until return is observed at the pipe crown. Finally, the grout is injected into the port on the pipe crown. For each lift, grouting should begin at the downstream end of the conduit and proceed upstream. After grouting operations have been completed, a pipe plug can be installed in the threaded coupling. For steel liners, the plug may be welded and ground flush with the liner surface.

(2) Grout should be pumped at pressures not exceeding 10 psi at the injection ports. All valves and ports should remain in the open position such that any trapped air or water is expelled. Rings of grout ports should be spaced at approximately 40-ft intervals. The designer should determine the required number and location of all ports and actual grouting pressures to be used. The procedure above results in five levels of injection: invert, 45 degrees above invert, springline, 45 degrees above springline, and crown. For very large conduits, more frequent injections may be required, and conversely, for smaller conduits, a minimum of three injection points is recommenced (invert, springline, crown). The pipe liner position, circularity, and shape should be monitored during grouting operations.

(3) Slick Line Method. In this method, grout pipes (typically 1–1.5 in. in diameter) are fixed to the crown of the liner before installation. This can be accomplished using extrusion welding techniques and HDPE grout pipe and liner pipe. For steel liners, steel injection pipe welded to the steel liner can be used. For other systems, a suitable pipe mounting system must be selected such that slick lines are not displaced and damaged during liner insertion. Figure 21-2 shows an example of grout lines extrusion welded to an HDPE liner. The designer will need to determine the required number and length of each grout pipe. The number of grout pipes required for a particular lining application is a function of the diameter of the pipe and the expected length of grout travel after it leaves the end of the grout slick line. A rule of thumb assumes about 25–30 ft of grout travel from the end of the grout pipe. For example, if a conduit 150 ft in length is to be lined, four grout pipes (120, 90, 60 and 30 ft in length) would be needed to grout the annular space. The first injection would take place at the downstream bulkhead, so five injections would be necessary.

(4) Bulkheads are placed around the annular space of the slip liner at both ends of the conduit to contain the grout (Figure 21-3). The bulkheads must be secured in place and sealed so that no grout will leak during grout injection. Air vent bleeder systems are installed through the bulkheads near the crown of the existing conduit at both ends. This is to prevent air, bleed water, and any displaced water from being trapped within the annular space. Air vents should be a minimum of 150% larger than the injection slick lines. If the air vent becomes obstructed, grout injection must be stopped because the liner may collapse.

(Courtesy of FEMA)

Figure 21-2. Grout pipe slick lines extrusion welded at the crown of an HDPE liner.

(Courtesy of Gannett Fleming)

Figure 21-3. Downstream bulkhead with slick lines and air vents at a slip lining project.

(5) Pressure injection of grout into the annular space starts at the downstream end of the HDPE slip liner at the downstream bulkhead port and progresses upstream. As grout is injected, air will be forced out the vent hole at the downstream bulkhead. After thick grout is observed at this location, the vent should be plugged. Air will then vent through the slick lines and upstream bulkhead vent hole. After the theoretical volume required to fill the annular space up to the first slick line is injected, plus a safety margin of approximately 10%, the downstream bulkhead injection port is plugged and injection then continues at the next slick line. Another clear indication that grout has reached the next slick line is when air is no longer displaced from that slick line. This process is repeated until thick grout is observed at the upstream bulkhead vent hole, which indicates that the annulus has been filled.

f. Pressure Consideration. Control of the injection pressure and the pressure induced on the liner is particularly important in slip lining applications. If excessive pressures are used, the liner may collapse under the load. The liner manufacturer should be consulted regarding pressures on the liner.

21-3. Voids Adjacent to Conduits.

a. General. It is not uncommon for voids or areas of low density to develop adjacent to conduits. These conditions are typically the result of seepage flowing along the conduit or leaking conduit joints. In some cases, seepage and void development may be a result of poorly compacted backfill. If these conditions require remediation, grouting is generally the only method available short of complete conduit replacement.

b. Seepage Paths. In the case of seepage through joints or along the conduit, grouting can result in the formation of new seepage paths. If the structure is of critical nature, such as a penetration through a water-retaining structure, a filter is recommended at the downstream end of the conduit if one is not already present. The design and details of this filter are described in EM 1110-2-1913, *Design and Construction of Levees*; EM 1110-2-2902, *Conduits, Culverts and Pipes*; and EM 1110-2-2300, *Engineering and Design – General Design and Construction Considerations for Earth and Rock-Fill Dams*.

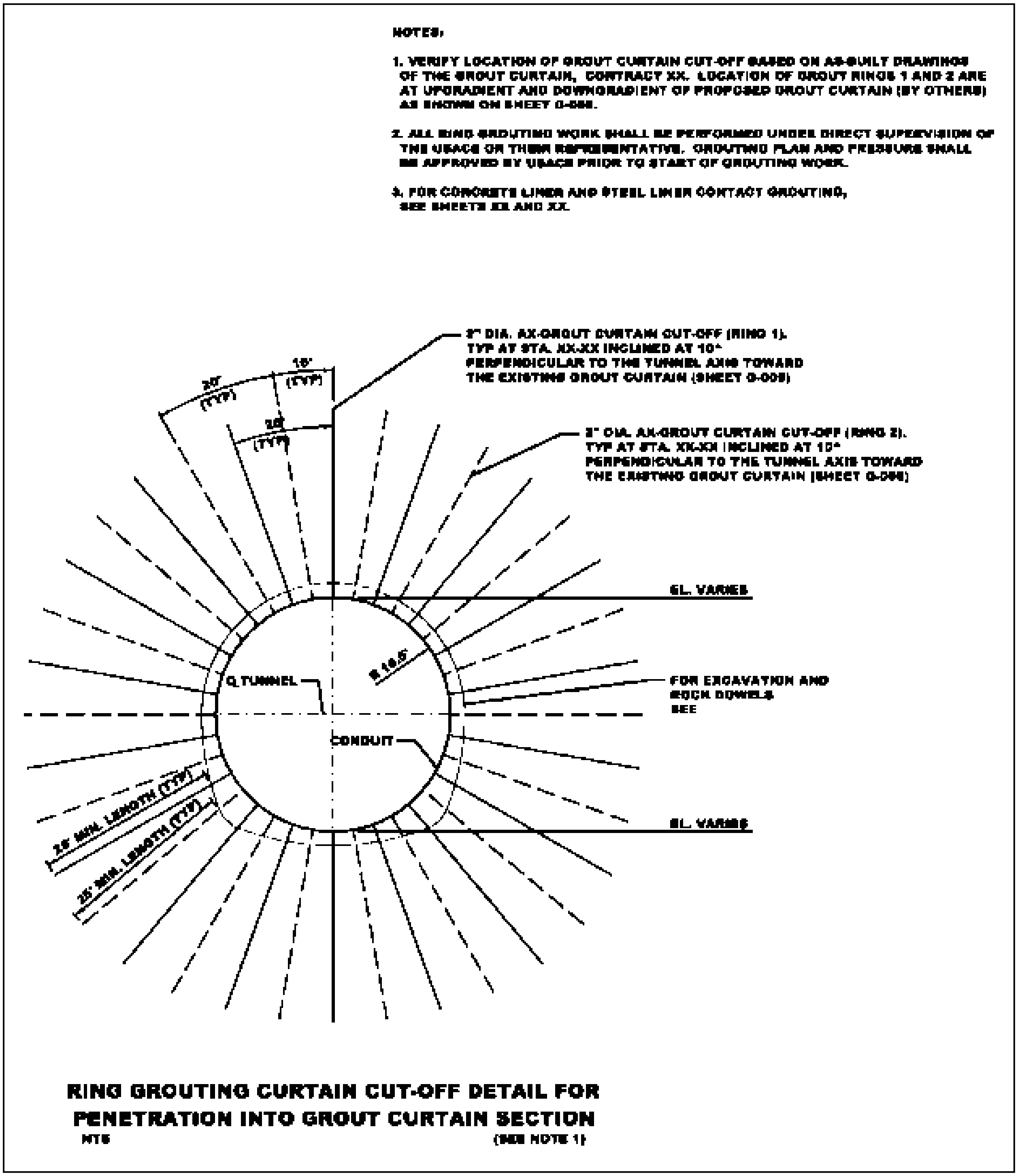
c. Grouting Voids. Grouting voids along a conduit is often not 100% effective. The best results are achieved if the conduit is accessible and grouting can be performed from inside the conduit. Figure 21-4 shows the drilling of a grout hole from the interior of a conduit. If the conduit is not accessible, grouting must be done from the embankment surface. Intercepting the voids becomes more problematic, and the filling of all the void spaces along the conduit cannot always be verified.

(Courtesy of FEMA)

Figure 21-4. Drilling a grout hole inside a conduit

d. Grout Mixes. High- and low-mobility cement grouts and chemical grouts are used depending on the conditions. HMGs will generally only permeate void areas or areas of sufficiently high permeability to accept cement grouts. Pressure must be controlled to prevent hydraulic fracturing of the embankment materials. If it is necessary to densify the materials surrounding the conduit, low-mobility grouting should be considered. The greatest concern with low-mobility grouts is the possibility that the conduit could be damaged by the high injection pressures. Vigilant observation of injection pressures is therefore necessary. It is also possible to jack or lift a conduit using low-mobility grouts, so monitoring of the structure may also be necessary. In some cases, this jacking action may be desirable to move the structure back to its original location. Chemical grouts have been used to treat voids adjacent to conduits; however, due to the high cost of materials, their use is often limited to treating cracks and leaking pipe joints.

e. Drill Hole Patterns. Holes drilled from inside the conduit may be located similar to hole patterns used for annular space grouting. Figure 21-5 shows a typical detail for ring grouting. Additional holes located between production holes can be used to verify the effectiveness of grouting. Grouting for filling of voids may require several series of holes to ensure complete filling. Drilling methods must be selected based on the materials to be drilled. Typically, small rotary percussion or diamond rotary drilling methods are appropriate. Diamond rotary methods allow for the cutting of steel reinforcement in concrete pipes. For above-ground work, holes are generally drilled parallel to the conduit alignment and offset some distance from the outside edge of the conduit. It is particularly important that the location of the conduit be known with certainty and that the layout of the drill holes and the alignment of the drill tooling are verified by survey or other appropriate means. Treating voids near the crown of the conduit requires that drill holes be stopped just above the crown of the pipe. This requires accurate conduit and drill hole collar elevation information, which necessitates the use of accurate surveying methods. Drilling methods appropriate for drilling through embankments are described in ER 1110-1-1807, *Procedures for Drilling in Earth Embankments*.



(Courtesy of Chicago District.)

Figure 21-5. Typical ring grouting detail from a contract drawing.

21-4. Grouting Cracks and Leaking Joints.

a. General. Chemical grouts include sodium silicate, acrylates, polyurethanes, lignins, and resins. Polyurethanes are most commonly used to repair leaking joints and cracks in conduits. Properties and applications of chemical grouts are described in EM 1110-1-3500, *Chemical Grouting*, but their use as it pertains to grouting conduits is briefly discussed here.

b. Sealing Cracks. Sealing of cracks and leaking joints in concrete conduits is a common application for chemical grouts. The concern with respect to these leaks is the loss of embankment materials through seepage into the conduit. If the problem goes unnoticed and the conduit is relatively shallow, sinkholes may develop above the conduit. Under the worst of scenarios, the seepage may result in the development of the preferential flow path along the conduit, which could result in an uncontrolled release from a water-retaining structure.

c. Procedure. The procedures for sealing joints and cracks in conduits with chemical grout are similar to methods for other crack repair. Generally holes are drilled at a close spacing and appropriate angle to intercept the crack or joint. Chemical grout is injected, sealing the crack. This method can generally only be applied to conduits that provide access to personnel.

21-5. Pipe Abandonment.

a. General. Pipes are abandoned for several reasons, including deterioration, obsolescence, and changes in loading conditions. Abandonment includes water supply wells, industrial pipelines, sewers, pipelines beneath levees, and spillway and outlet works conduits in embankment dams.

b. Abandonment of Wells. After a well has served its purpose and the decision has been made to abandon it, the well should be sealed for safety reasons, and to prevent pollution of groundwater resources in the area. Grouting the well from the bottom of the well screen to the top of the well casing is an effective means of sealing the well. The procedures for sealing a well by grouting are described in USACE EM 1110-3-161, *Water Supplies, Water Sources* (1984).

c. Abandonment of Industrial Pipelines and Sewers. Often when an industrial pipeline or sewer is abandoned, the pipe remains in place. If there are environmental or structural concerns, the pipeline should be filled with concrete or grout. Abandoned pipelines used to transport hazardous or contaminated materials at hazardous waste sites have been sealed using concrete and grout.

d. Abandonment of Conduits in Embankment Dams and Levees. Spillway and outlet works conduits in embankment dams and levees are sometimes abandoned due to deterioration or changes in loading conditions. If an embankment or levee is raised, an existing conduit may not have the strength to withstand the new loading condition. It may be technically feasible and cost effective to leave the conduit in place and to fill it with grout rather than to remove it. Removal likely would require lowering the reservoir, excavating the embankment fill, and then replacing the fill. The advantages of leaving the conduit in place must be weighed against the concerns of possible seepage paths, which could cause future problems, and continued conduit deterioration. Where a new levee is to be constructed, an existing pipeline in the foundation may be abandoned if the measures outlined in EM 1110-2-1913 are followed.

(1) Inspections and Condition Surveys. The decision to repair, strengthen, line, or abandon an existing conduit should be based on an evaluation using an inspection and condition survey. When the decision has been made to abandon the conduit and seal it in place, the results of the inspection and condition survey should be used in developing the scope of work. For example, if the inspection and condition survey results indicate the presence of voids outside the conduit, if seepage is detected flowing through joints or cracks in the conduit or along the outside of the conduit, or if a filter is not present at the downstream end of the conduit, the design must address these issues. If grouting the conduit must be done from the embankment surface, the exact location of the conduit must be known, and this information should be available from the survey. Chapter 9 of Federal Emergency Management Agency (FEMA) Technical Manual 484, *Conduits through Embankment Dams*, gives information about conducting inspections and condition surveys of both accessible and inaccessible conduits.

(2) Access for Grouting. Sealing the conduit with grout is most easily accomplished and achieves the best results if the conduit is accessible from the upstream or downstream end. If neither end of the conduit is accessible, it is likely that holes for grout injection will have to be drilled from the embankment surface. Additionally, if not directly accessible, a bulkhead must be constructed at the upstream end by drilling and grouting from the embankment surface. When this is necessary, a boring is advanced through the embankment directly above the conduit where the bulkhead will be constructed. The boring is drilled into the conduit and grout pipe is extended to the mid-height of the conduit. Low-slump materials such as concrete or low-mobility grout are then pumped into the conduit and the bulkhead is constructed. Bulkheads can be constructed from a wide variety of materials if direct access can be gained.

(3) Cleaning the Conduit Before Grouting. The existing concrete surfaces against which grout will be placed should be free of any deleterious materials, including roots, sediment, mineral deposits, dust, laitance, loose or defective concrete, curing compound, coatings, and any other foreign materials. Section 9.6 of FEMA Technical Manual 484, *Conduits through Embankment Dams*, provides guidance on cleaning conduits. Sediment and debris should be removed from the conduit invert. If the conduit is accessible, projections such as bolts should be cut off or ground smooth with the interior conduit wall.

(4) Grouting. Grouting can be conducted in a wide variety of ways. Applications such as slick lines mounted to the invert of the pipe may be appropriate. Bulk filling from the invert of the downstream bulkhead until grout is observed at the vent hole in the upstream bulkhead may also be appropriate. In some cases, complete filling of the pipe may not be necessary, in which case stiff, minimally flowable materials such as concrete or low-mobility grout may be used.

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CHAPTER 22

Void Filling

22-1. Introduction. This chapter addresses various types of void-filling applications such as mine voids, voids that develop beneath structure foundations, voids associated with abandoned tanks and similar structures, and other subsurface voids. Chapters 17, 21, and 24 of this manual contain other information specific to voids associated with those types of features.

22-2. Basic Considerations for Void-Filling Operations.

a. General. The design and execution of any void-filling operation must carefully consider the source that caused formation of the void, the purpose and requirements of the void filling, any secondary effects that might result from the void filling, and control of grouting pressures.

b. Source of Void Formation. Voids can form in response to a broad variety of subsurface conditions. Common conditions include underground mining, structure undermining related to seepage and piping, and limestone sinkhole subsidence. In some cases, grouting can arrest ongoing void formation and restore foundation support. However, in many other cases, particularly those associated with structure undermining related to seepage and piping, grouting of the voids might restore foundation support, but rarely will it remedy the cause of the void or prevent new voids from forming. In those circumstances, other measures are usually required to address the fundamental conditions that caused the void to form.

c. Purpose and Requirements for Void Filling. The intended purpose and the requirements for void filling must be clearly understood and established as part of the design. In particular, the completeness of the required filling is a basic consideration. For example, a purely structural application might require high-strength materials and adequate contact of the foundation with the grout, but it might or might not require perfect filling. Conversely, for hydraulic structures, complete filling is normally essential, and effective grouting might require multiple applications with different types of grout materials.

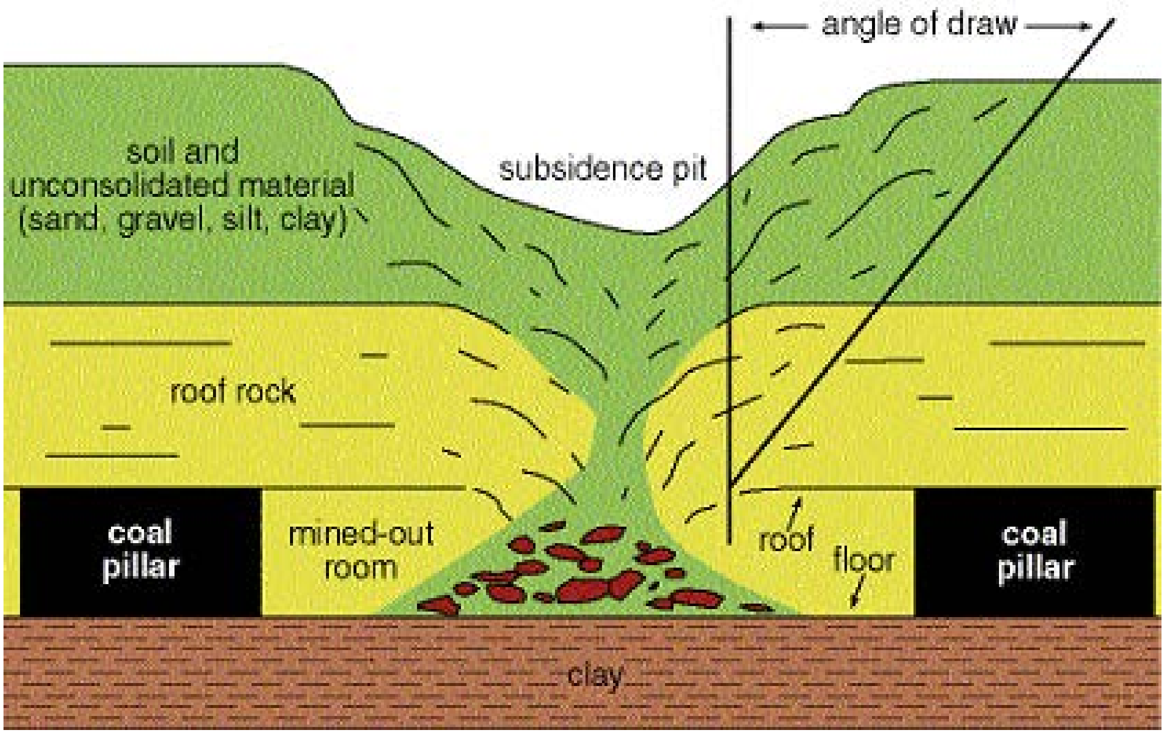
d. Secondary Effects. Void-filling programs must consider the potential secondary effects that might occur as a result of grouting. Specifically, grouting might increase the loads on the foundation materials beneath a structure, or in the case of pile-supported structures, it might increase loads carried by the piling. Void filling can also alter groundwater and seepage flow regimes, which might have unintended consequences in nearby areas. For example, where void filling is undertaken in limestone sinkhole areas, it is not uncommon for new sinkholes to form in adjacent areas. Grouting can also damage subsurface drainage systems and instrumentation.

e. Grouting Pressure Control. Extreme care must be taken in establishing the grouting pressures to be applied, particularly where grouting is being performed under slabs or other foundation elements. It is common that grouting pressures will be low, based on calculation of the pressures at which uplift of the structure would be expected to occur. Vent holes are often required, both for verification of grout filling and for limiting pressures.

22-3. Common Applications.

a. Mine Voids.

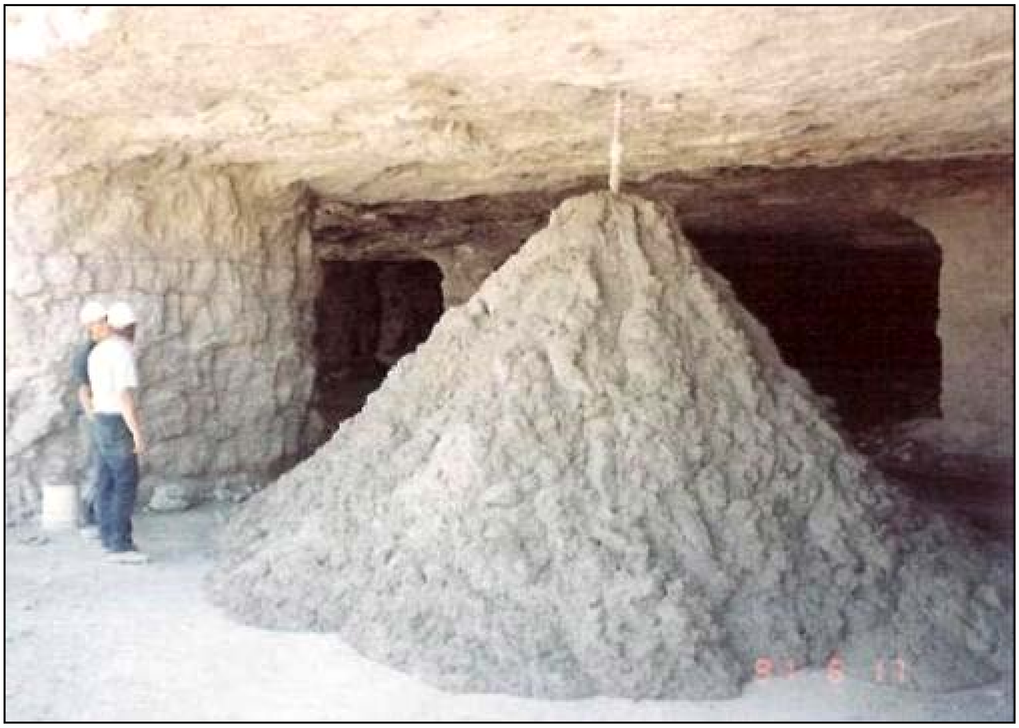
(1) Abandoned mines have a long history of causing subsidence and hydrologic problems ranging from acid mine drainage to alteration of surface water and groundwater flow regimes. The Wyoming communities of Rock Springs and Glenrock were built above abandoned mines. Subsidence necessitated grouting of the underlying mines (Holmquist et al. 2002). A similar situation occurred in Plum Borough, Allegheny County, PA. Coal mining beneath this community began before World War II using the room-and-pillar method. The abandoned mine was grouted when subsidence threatened over 200 homes (Pennsylvania Organization for Watersheds and Rivers 2002). In Guernsey County, OH, the eastbound lanes of I-70 collapsed as a result of an underlying abandoned coal mine. The repair included grouting the abandoned mine, which stabilized the area, and constructing a land bridge (Ruegsegger and Lefchik 2001). Figure 22-1 shows one mode of underground mine subsidence.



(From Crowell 2010, courtesy of Ohio Department of Natural Resources, Division of Geological Survey.)

Figure 22-1. Mechanics of underground abandoned mine subsidence.

(2) Abandoned mine tunnels often discharge water that is highly acidic or that contains pollutants. In Pennsylvania, for example, it is estimated that there are thousands of miles of streams having waters that are polluted by abandoned mine drainage. Grouting has been used to seal the tunnels with a complete filling or a substantial plug. As part of the current permitting process, many new mines are required to have an abandonment plan for the time when mining activities will cease. These plans often include grouting. Figures 22-2 and 22-3 show grouting in an abandoned mine.



(Courtesy of Hayward Baker, with permission from ASCE.)

Figure 22-2. LMG injection into a mine void.

(Courtesy of Hayward Baker.)

Figure 22-3. Time-lapse photos showing the advancement of LMG injected into a tunnel.

b. Structure Underpinning. During the life of a structure, voids may develop in the foundation as a result of flowing water, fluctuating groundwater levels, seismic activity, or other mechanisms. Whatever the cause, it is usually necessary to re-establish contact between the foundation of the structure and the underlying soil or rock. Large voids caused by flowing water have frequently occurred beneath concrete spillways and lock and dam facilities. Spillways and outlet works having inadequate terminal cutoff walls have been undermined. High-velocity flows adjacent to training walls have undercut wall foundations. In all of these cases, the voids required grouting. Figure 22-4 shows a thin-slab, chute-type spillway structure at an earth dam where the extent of undermining and damage had progressed to the point at which grouting was no longer possible.

(Courtesy of Gannett Fleming)

Figure 22-4. Extensive network of large voids beneath a spillway slab, which necessitated demolition and construction of new spillway.

c. Special Applications. There are several additional applications where grouting has been used to fill large voids, two of which are described here. The first example involves the case in which large voids have developed in earth embankment dams or their abutments. These voids can be the result of mechanisms such as seepage, flood discharge, and deteriorating timbers left in the dam. Filling of these voids may require special field techniques and materials. For example, if the voids are entirely within the embankment fill, the grout mix may need to exclude cement so that the fill is not brittle and is more compatible with the original embankment soil. Extreme care must be exercised in injecting the grout so that hydraulic fracturing does not occur within the embankment or on the slopes. The grout is often injected by gravity flow to prevent damage. When seepage is known to be the cause of the void, other measures must normally be implemented to prevent the void from re-forming. A second example of a special application is the grouting of abandoned large tanks at waste disposal sites. These tanks were used to collect waste solutions and sludge. Substantial volumes of grout, on the order of thousands of cubic meters, may be required. The purpose of the grouting is to provide structural stability and eliminate safety hazards. Typically these tanks still contain contaminants that must be stabilized and encapsulated. The grout mix must be designed to cure in the presence of these contaminants and have long-term chemical compatibility. Underwater placement is often required, and placement is typically accomplished over a long period of time.

22-4. Investigations.

a. General. Of primary importance in void filling is delineation of the void extent. In many situations the only evidence of the void is structure or ground subsidence. Records may be minimal or nonexistent, and direct observation of the void may not be possible. Various investigation techniques must therefore be employed.

b. Abandoned Mines. Mining operations in abandoned mines ended decades ago, and the available information about these mines is often limited, inaccurate, or nonexistent. The underground geometry must therefore be determined by investigation and exploration. All shafts and mine entry points should be determined. The mine floor and the ceiling elevations, slope, width, and direction of tunnels should be modeled. This is useful in planning field operations and in locating grout injection points. Often these mines are inaccessible or access is unsafe. Geophysical surveys and down-hole video cameras can be valuable resources for obtaining information. Abandoned mines often have large open areas and also rubble mounds of rock, soil, and other materials. These rubble mounds affect the grouting program, so their locations and geometry should be determined to the extent practical. The site investigation must include a study of the regional hydrogeology and hydrology. This is particularly important if one of the objectives of the grouting program is to address abandoned mine drainage. However, even if the only concern is subsidence, grouting an abandoned mine will affect groundwater flow regimes and also could affect surface water flow. Grouting will alter existing flow paths. Geophysical surveys may provide information about groundwater flow paths when access to the mine is not possible. Section 5 of the FHWA publication *Manual for Abandoned Underground Mine Inventory and Risk Assessment* provides detailed information on conducting site investigations. EM 1110-1-1802, *Geophysical Exploration for Engineering and Environmental Investigations,* should be used in planning a geophysical survey.

c. Voids Beneath Structures. As noted previously, sometimes the only evidence of a void is subsidence. With respect to structures, this can be evidenced by tilting, cracked, or displaced walls, slabs, foundations, or other structural elements. In the case of hydraulic structures, abnormal flow or head behavior such as locations where water is either being lost from the structure or entering the structure often indicate the presence of undisclosed voids. Investigation of a structure void should begin with mapping and a survey of the structure to determine the affected limits. Further investigations may be conducted by drilling holes through the structure and measuring the thickness of the void beneath. Drilling should continue in a radial pattern from the presumed center of the void until the outermost limits have been identified. Holes beyond the void limits may be appropriate to determine if other problematic and unforeseen conditions are present. Observation of the void is possible by using inexpensive pan-and-tilt down-hole cameras. Geophysical methods such as microgravity may also be useful in combination with drilling methods.

22-5. Design Considerations.

a. General. The design of the grouting program includes determining the appropriate mix for the void and the environment into which it will be injected, and for the required functional characteristics of the grout.

b. Mix design. The mix design depends on the feature being grouted. The variables are the proportions of water, cement, aggregates and additives and the desired properties of flowability, aggregate type, set time, and strength. Numerous materials ranging from HMGs to stiff concrete may be employed.

(1) High-Mobility Grout and Sanded Grout. Recommended HMG materials and mixes are discussed in detail in other chapters of this manual. HMG may be used to tighten certain

areas for seepage control in mine plugs or fill small voids beneath structures. The vast majority of mine void-filling operations use concrete, flowable fill, or low-mobility grout.

(2) Low-Mobility Grout. LMG is discussed in other chapters of this manual. The two greatest concerns with respect to LMG in void-filling applications are the flow characteristics and strength. The water-to-solids ratios may need to be varied to provide appropriate flow characteristics, and additional cement may be required, depending on the service load of the LMG column.

(3) Concrete. Discussion of concrete mix designs is beyond the scope of this manual. Lean concrete mixes typically provide adequate strength in most void-filling applications. The consistency of the material can range from highly flowable to stiff with zero slump. Stiffer concretes are typically employed when minimal travel is beneficial.

(4) Flowable Fill. Flowable fill is a controlled, low-strength material consisting of water, cement, fly ash, and aggregate. Much of the published information about flowable fill pertains to fills that may later be excavated, so the water/cement ratios are high and the corresponding compressive strengths are on the order of 100 psi. The main purpose of flowable fill for void filling is to provide adequate support for the overlying materials or structures. Care must be exercised in determining the required compressive strength and the appropriate water/cement ratio. The recommended slump range for flowable fill used for filling large voids is 2–8 in. Mix design guidelines are published by the American Concrete Institute in *Controlled Low Strength Materials (CLSM)*, Report No. 229, and the Portland Cement Association's *Cementitious Grouts and Grouting*, 1990.

c. Other Material Considerations. In selecting the physical properties of the injected materials, consideration must be given to the size of the voids to be grouted and the environment into which the material will be injected. Short set times may be appropriate. Penetrability of the injected mix must be considered. Dilution and segregation must be considered for underwater applications.

(1) Aggregates and Penetration. Manufactured aggregates contain angular particles, which decrease the flow characteristics and penetrability of a mix. Manufactured aggregates also tend to clog the injection pipes. Natural sand aggregates consist of rounded particles, which increases the flow properties and penetration of the mix. For penetration of rubble mounds in mines and small voids, only natural aggregates with a maximum particle size of 0.5 in. are recommended. Manufactured aggregates in concrete, flowable fill, and LMG are appropriate if the intention is to construct discrete column elements within the void.

(2) Set Time. Set times must be compatible with the volume of material to be injected and adjusted based on the pumping rate. In mine void-filling applications where large volumes are to be injected, typical pumping rates vary between 5 and 30 yd^3 per hour, depending on site conditions. If the intention is to inject several hundred cubic yards of material from a single location, the set time obviously must be compatible with the pumping rate.

(3) Underwater Grouting. In the case of waterfront and hydraulic structures, it may be necessary to place grout or concrete under water. Special precautions need to be taken in the mix

design and placement. If placed under water, the mix should be designed in accordance with the provisions of Chapter 10 of EM 1110-2-2000, *Standard Practice for Concrete for Civil Works Structures*, and EM 1110-2-2002, *Evaluation and Repair of Concrete Structures*. When placed under water, the grout mix must be highly flowable and must be resistant to water dilution and segregation. Water-reducing admixtures and an admixture to minimize the washout of cement and fines are often added. For underwater placement of concrete, it is desirable to have a mid-range slump. A high-slump grout increases the potential for water dilution. A low-slump mix is very stiff and has poor flow characteristics. Flowability is very important for consolidation since mechanical vibration may not be possible.

(4) Form and Reinforcement Design. Forms for bulkheads and for underpinning foundations must be designed to withstand the pressures that may be induced during the injection or placement of the grout or concrete. The forms also must be designed such that they do not move under the pressures induced by the grout or concrete once placement is complete. For underpinning foundations that are under water, forms require installation by a diver. Additional guidelines are given in EM 1110-2-2002. Since underpinning of foundations may result in significant load transfer to the injected materials, reinforcement of the grout mass using pre-embedded rebar may be necessary.

22-6. Field Methods.

a. General. The methods used to drill and inject grout or other materials for void-filling purposes are highly varied. In general, the only requirement is to economically and fully fill the void. While some methods are common between mine and structure voids, the materials chosen generally differ and therefore many of the field procedures also differ. The following paragraphs discuss field methods common to most void-filling applications, and those methods specific to mine voids and voids beneath structures.

b. Common Field Methods. The following methods are common to most void-filling operations.

(1) Drilling. Nearly any hole drilling method is allowed. Air flush is specifically prohibited in permeation grouting applications due to the potential for clogging of fractures and hydrofracturing embankment materials. However, provided that no damage is anticipated as a result of air flush, there is no compelling reason for it not to be allowed in drilling holes for void-filling operations.

(2) Underwater Injection. It is important that the specification for injecting the grout under water require that the grout pipe initially be placed as low as possible. As grout begins to flow, the end of the grout pipe should remain in the freshly placed grout. With this procedure, the grout will displace the water in the void, minimizing the potential for the grout to be diluted or the fines separated.

(3) Refusal and Pressure. In grouting of abandoned mines, refusal is defined as no grout take at constant pressure for 5 minutes. The pressure is usually low, but it should be sufficient to provide complete grouting of the feature. When underpinning below-water structures where forming is required, it is best if the grout or concrete placement can be observed by a diver or an underwater camera. If the form is open, there is no refusal since grout will flow out of the form

when the void has been completely filled. Where forming is not required and the void space is confined, refusal is defined as stated previously. Injection pressures are typically low and should be selected conservatively. The greatest concern is “jacking” of the structure or overlying materials. Gravity grouting methods are commonly employed with higher-slump materials.

(4) Monitoring. Underpinning should be monitored visually or using a camera if possible. In the case of grouting confined void spaces, it will be necessary to rely on refusal data. Site monitoring should be conducted during the grouting of abandoned mines. Existing conditions may change, or new conditions may develop during construction. For example, grouting may unintentionally cause new subsidence. The development of a construction monitoring plan is discussed in more detail in Section 9 of the FHWA publication *Manual for Abandoned Underground Mine Inventory and Risk Assessment.*

(5) Verification Programs. In most applications, confirmation borings should be conducted to verify that the void is completely filled. These borings should be accurately located and logged. If any voids are encountered, the voids should be grouted. Depending on the extent of the void, a grid pattern similar to that used in consolidation grouting programs may be appropriate. Subsequent series of holes fundamentally become verification holes into which additional materials may be injected if conditions warrant.

c. Mine and Large Void-Filling Methods. The following methods and procedures are generally applicable to mine or large void-filling operations.

(1) Hole Spacing. Hole spacing for injection grouting mines is usually designed based on a pilot program. Published information about completed projects shows hole spacing for the production grouting typically ranging from 30 to 40 ft. In projects where flowable fill was used, the maximum hole spacing was approximately 300 ft; however, such a large spacing should only be considered if conditions are extremely favorable for complete filling. Hole spacings are considerably less on the majority of projects. Verification holes located between primary injection sites should be required, particularly where larger-than-anticipated quantities were injected or where observations indicate incomplete filling.

(2) Injection Methods. In large void-filling applications, the quantities of materials consumed may be enormous. The use of gravity grouting methods and stiff materials necessitates the drilling of large-diameter holes. Other conveyance systems include cased holes affixed to piston-type pumps.

(3) Slump Considerations. The mix for grouting abandoned mines can be highly variable, depending on the application, and multiple mix types may be used. For grouting large voids, a low-slump mix is typically used; however, voids within rubble mounds of soil and rock are substantially smaller, and a higher slump may be necessary to achieve the necessary penetration. The mix may have to be adjusted quickly whenever rubble mounds are encountered or when changing from grouting rubble mounds to grouting large voids.

(4) Spatial Considerations. To minimize the possibility of incomplete filling inside a mine or large void, it is recommended that the grouting begin on one edge of the mine and

proceed across the void area to the opposite side. If possible, areas of lowest elevation in the profiles of tunnels and shafts should be grouted first.

d. Structure Void-Filling Methods. The following methods and procedures are generally applicable to the filling of voids beneath structures.

(1) Hole Spacing. The hole spacing for injection grouting of voids beneath structures generally depends on the extent of the void to be treated, the geometry of the structure and void, and the flow characteristics of the injected medium. Voids beneath structures are generally smaller than voids encountered in mine works; however, this does not necessarily indicate fewer holes for proper treatment. Grid patterns similar to those used in consolidation grouting programs may be appropriate, and hole spacings on the order of several feet are common. This relatively close hole spacing is necessary to provide assurance that no voids remain beneath the structure. Hole sizes are typically on the order of 1–3 in. in diameter. For underpinning slabs or other relatively thin concrete, thin-walled diamond rotary core drills are typically used because they are capable of cutting through embedded reinforcing steel.

(2) Injection Methods. Injection is typically accomplished using the methods specified elsewhere in this manual for HMG. Both mechanical and pneumatic packers are appropriate. In some cases, a single injection hole may be used, with relief holes spaced at regular intervals (Figure 22-5). The relief holes are equipped with valves. When thick grout discharges from the hole, the valve is closed, which forces the grout to travel to the next relief location. Similar to grouting of other voids, it is recommended that grouting begin at the lowest part of the void and progress upward.

(Courtesy of Advanced Construction Techniques)

Figure 22-5. Grout holes for injection of HMG to fill a void beneath conduit.

CHAPTER 23

Pre-stressed Rock Anchor Grouting

23-1. Introduction.

a. General. This chapter provides an overview of the methods and materials used in grouting pre-stressed rock anchors. Since 1974, there have been five versions of the Post Tensioning Institute's (PTI's) anchor "Recommendations" document, which guides practice in all phases of rock and soil anchor technology on a national basis (Bruce and Wolfhope 2007). The contents of this chapter are consistent with the most recent version (2004) and may therefore be regarded as the current state of practice in the United States.

b. Anchor Construction. In the simplest terms, rock anchor construction involves five distinctly different processes: drilling the hole, waterproofing the hole, placing the tendon, grouting the tendon, and stressing and testing the anchor. This chapter discusses only the grouting-related processes that relate to pre-stressed rock anchors, namely waterproofing of the hole and grouting of the tendon.

c. Relevant References. Other relevant manuals related to other types of rock reinforcement, applications for pre-stressed anchors, and/or the design and construction of rock anchors of various types include:

(1) Engineer Manual 1110-1-2907, *Rock Reinforcement*, 15 February 1980.

(2) Engineer Manual 1110-1-2908, *Rock Foundations*, 30November 1994.

(3) Engineer Manual 1110-2-2200, *Gravity Dam Design*, 30 June 1995.

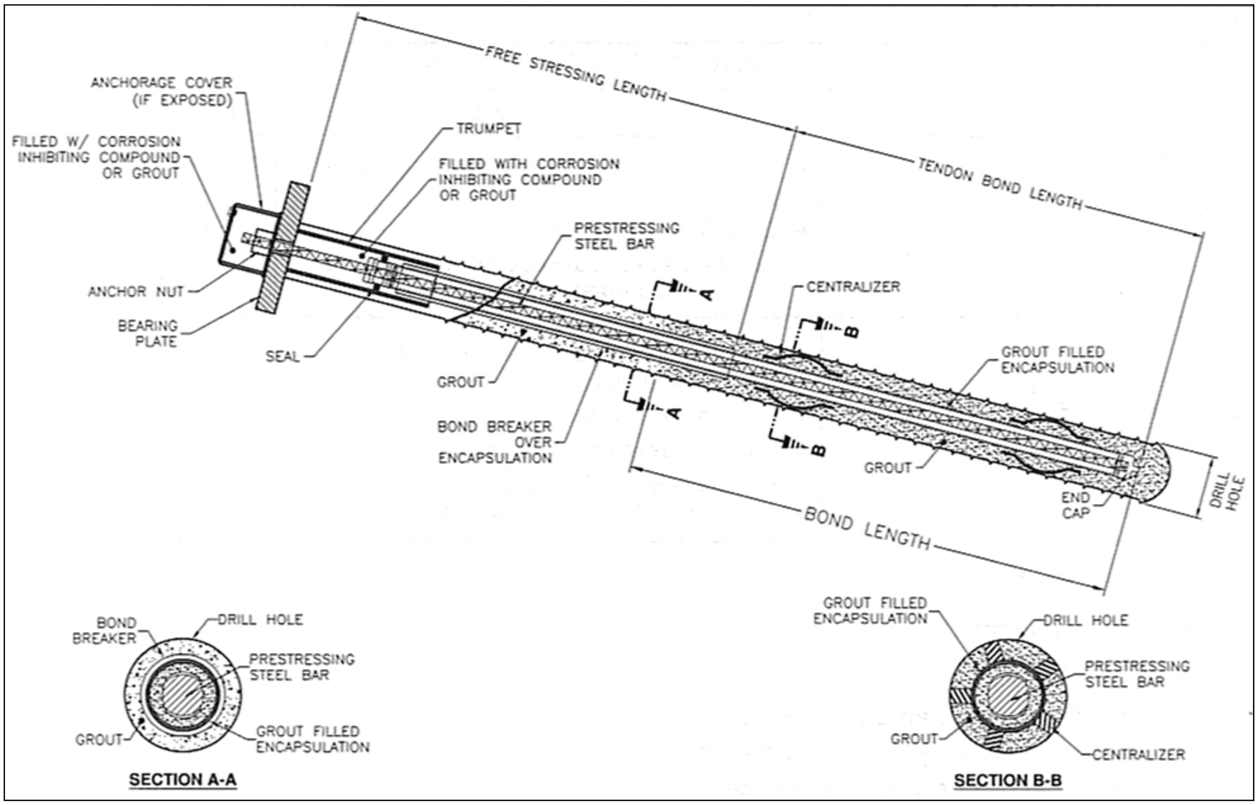
(4) Engineer Manual 1110-2-2901, *Tunnels and Shafts in Rock*, 30 May 1997.

(5) Engineer Manual 1110-2-2100, *Stability Analysis of Concrete Structures*, 1 December 2005.

Some of these documents reference earlier versions of the PTI anchor recommendations.

23-2. Basic Elements and Terminology.

a. Basic Elements. Figure 23-1 shows the typical elements of a pre-stressed rock anchor (for simplicity, the tendon to be composed of a single bar). In anchors with working loads in excess of 100 kips, the tendon will comprise multiples of seven-wire strands, the number of which may typically vary to over 60 per tendon, thereby being capable of providing working loads of over 2,000 kips, if required. Furthermore, as explained in Section 23-4, the level of the tendon corrosion protection (and hence the grouting processes) also varies with specific project requirements. Anchors regarded as "permanent," and having working loads of, say, 400 kips or more, will therefore feature sophisticated tendons and intricate grouting-related processes.



(From PTI 2004)

Figure 23-1. Typical components of an anchor.

b. Common Anchor Terminology. The following terms are specifically relevant to rock anchor grouting processes. Terms used elsewhere in this manual (e.g., additive, admixture) are not repeated here.

(1) Anchor: A system used to transfer tensile loads to the ground (soil or rock), which includes the pre-stressing steel, anchorage, corrosion protection, bondbreaker, spacers, centralizers, and grout.

(2) Anchorage: The combined system of anchor head, bearing plate, trumpet, and corrosion protection that is capable of transmitting the pre-stressing force from the pre-stressing steel to the surface of the ground or the supported structure.

(3) Apparent Free Tendon Length: The length of tendon that is apparently not bonded to the surrounding grout or ground, as calculated from the elastic load extension data during testing.

(4) Bond Length: The length of the grout body that transmits the applied tensile load to the surrounding soil or rock. (See also the definition of “tendon bond length.”)

(5) Bondbreaker: A sleeve placed over the anchor tendon in its free-stressing length to allow elongation of the tendon free-stressing length during stressing.

(6) Centralizer: A device to support and position the tendon inside the drill hole or the sheath so that a minimum grout cover is provided.

(7) Consolidation Grout: Portland-cement-based grout that is injected into the drill hole, before tendon grouting, to either reduce the permeability of the rock immediately surrounding the hole or otherwise improve the ground conditions (e.g., improve drill hole stability).

(8) Encapsulation: A corrugated or deformed tube protecting the pre-stressing steel against corrosion in the tendon bond length.

(9) Free-Stressing (Unbonded) Length: The designed length of the tendon that is not bonded to the surrounding ground or grout during stressing.

(10) Grout Sock: A geotextile encasement around all or part of the ground anchor length, used to control grout loss in certain highly permeable ground conditions.

(11) Fully Bonded Anchor: Anchor in which the free-stressing length without bondbreaker is surrounded by grout, after stressing, and so is bonded to the surrounding structure or ground.

(12) Horizontal Anchor: Any pre-stressed anchor that is placed at a slope within 0.1 rad (5 degrees) of the horizontal.

(13) Permanent Anchor: Any pre-stressed anchor for permanent use, generally defined as having at least a 24-month service life.

(14) Primary Grout: Portland-cement-based grout that is injected into the drill hole before or after the installation of the anchor tendon to allow the tendon to transfer load to the surrounding ground along the bond length of the tendon (also known as anchor grout). Polyester resins are also used in place of portland cement grouts in certain circumstances.

(15) Secondary Grout: A portland cement grout that is injected into the drill hole within the free-stressing length of the tendon for corrosion protection or load transfer.

(16) Sheath: A smooth or corrugated pipe or tube protecting the pre-stressing steel against corrosion.

(17) Spacer: A device to separate elements of a multiple-element tendon to ensure full bond development of each pre-stressing steel element.

(18) Temporary Anchor: Any pre-stressed anchor for temporary use, generally defined as having a service life less than 24 months.

(19) Tendon: The anchor assembly consisting of pre-stressing steel, spacers, anchorage, corrosion protection, bondbreaker, and centralizers.

(20) Tendon Bond Length: The length of the pre-stressing steel that is bonded to the grout.

(21) Transition Tube: A common sheath that is inserted into the top of the fluid grout and sealed to the trumpet.

(22) Unbonded Anchor: An anchor for which the free-stressing length remains permanently unbonded to the surrounding grout, ground, or structure.

(23) Upward-Sloped Anchor: Any pre-stressed anchor inclined greater than 0.1 rad (5 degrees) above the horizontal.

c. Bonded, Partially Bonded, and Unbonded Tendons. The designer determines if the free-stressing length shall be fully bonded, partially bonded, or unbonded to the surrounding ground or structure. Generally, the free length will remain unbonded after stressing, except to satisfy specific structural requirements. Fully bonded free-stressing lengths force the anchor to strain with the structure. Unbonded free-stressing lengths allow a more flexible performance of the anchor, and the averaging of structure strains results in less load change in individual anchors. Partially bonded free lengths provide redundant load transfer at the anchorage while leaving a certain amount of unbonded free length (Figure 23-2).

(1) Fully and partially bonded anchors require that grouting be accomplished in two stages, the first to grout the bond zone and the second to grout the free length after the anchor has been stressed, tested, and locked off.

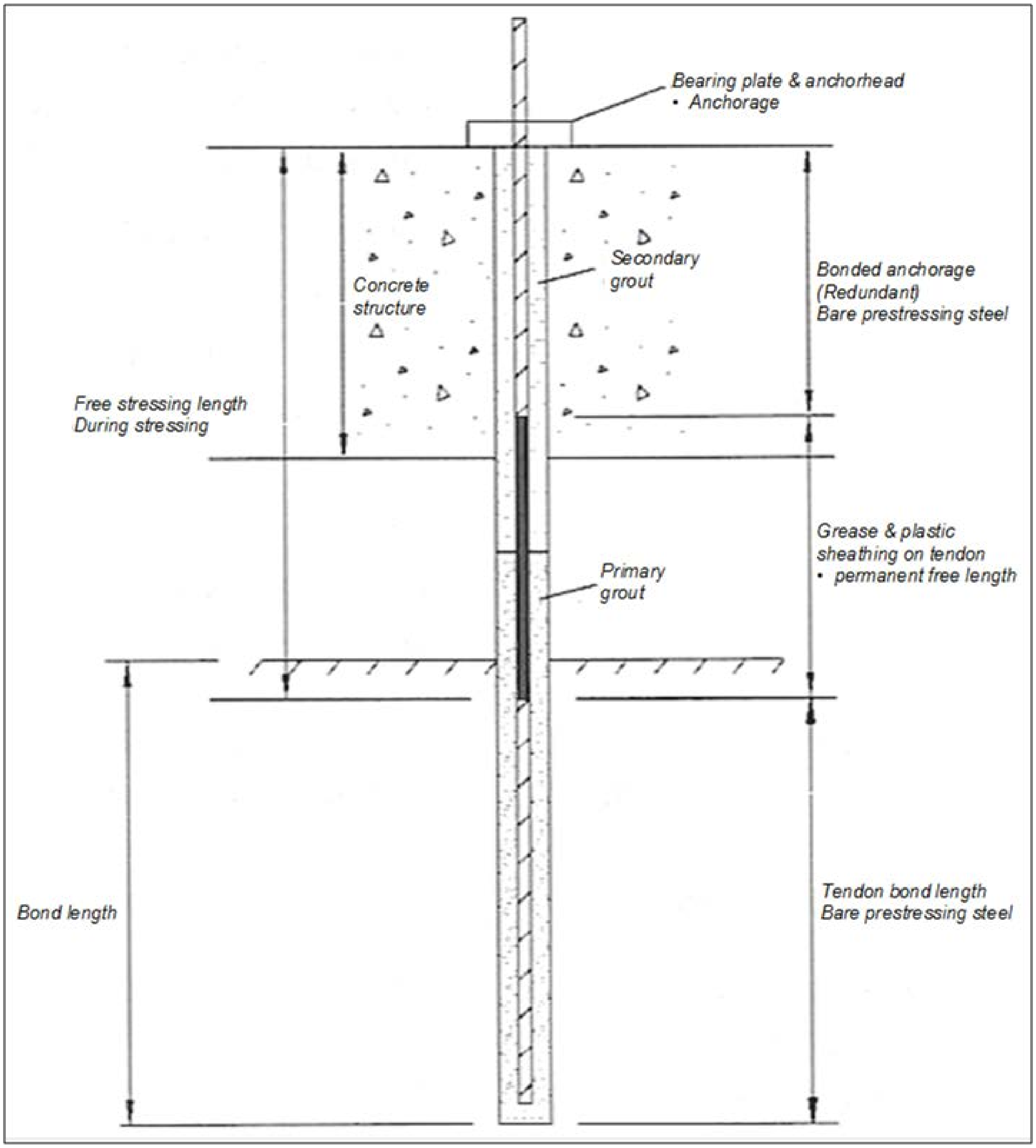
(2) Partially bonded free-stressing lengths can be created by terminating the bondbreaker at some depth below the anchor head and limiting the primary grout to a level below the top of bondbreaker. This upper bond length is then bonded to the structure by secondary grout.

23-3. Grouting for Anchor Construction.

a. General. Grouting is used at two times in anchor construction: to "waterproof" the anchor hole (more accurately referred to as "borehole pre-grouting") and to seal the borehole of the tendon assembly. The latter operation may also contain the subsidiary phase of "topping up" the borehole after the main grouting of the tendon assembly to compensate for any sagging of the grout into the rock mass and/or any minor bleed of the grout contained within the sheath.

b. Anchor Grouting. Rock anchors are invariably "gravity" grouted and as such correspond to Type A installations as defined by the FHWA (2000). Pressure grouting through the casing as it is withdrawn (Type B installation) is used in cohesionless soils, and Types C and D, which involve post-grouting through sleeve pipes (tubes á manchette), are not applicable in rock conditions and are not described further in this chapter. It might be noted, however, that Type D principles have been attempted in very weathered argillaceous rock masses as a means to enhance bond capacity, but these efforts have been concluded to be ineffective.

c. Borehole Pre-grouting. Permanent anchors that are bonded into rock must be first subjected to a water pressure test before the insertion and grouting of the tendon assembly. Derived from rock fissure grouting theory and based on the amenability of such fissures to Type I/II grouts, the criterion that triggers the need for pre-grouting is a water loss in excess of 10.4 L (2.75 gal) (for the entire hole or zone of a hole) at an excess hydrostatic pressure of 5 psi over a period of 10 minutes. It is held that a water loss in excess of this criterion indicates a situation in the tested zone whereby the tendon grout could be lost during placement. This would reduce the rock/grout surface contact area (and hence the bond capacity) and compromise the integrity of the corrosion protection as it relates to the integrity of the grout coverage.



(From PTI 2004)

Figure 23-2. Partially bonded tendon.

(1) The U.S. criterion for bond zone permeability is stricter than the criteria employed in other countries (Bruce and Wolfhope 2007). However, since it may be argued that other aspects of corrosion protection in the United States have historically been inferior to those in other

countries—an example being that, until 1996, there was no requirement to have a protective sheath around the bond zone steel—then this conservatism has undoubtedly been a saving grace in terms of long-term anchor performance. On most projects, it is prudent and instructive to test (and pretreat if necessary) the anchor borehole in three distinct zones: the bond zone, the dam/rock interface, and the entire hole. The first two require the use of a down-hole packer. This apparent intricacy will, in fact, prove cost effective by potentially reducing pre-grouting, redrilling, and subsequent water testing activities, and by enhancing the understanding of the permeability of the whole system.

(2) Water pressure testing and pre-grouting in rock anchor boreholes are typically conducted only for permanent applications such as dam stabilization. For anchors installed in soils, or for anchors in rock that are purely temporary, pre-grouting would be needed only in extreme circumstances. Conversely, in any project where systematic borehole interconnection occurs, where there are artesian conditions, or where an especially voided rock mass is encountered, pre-treatment by grouting should be regarded as a standard defense, regardless of the design life.

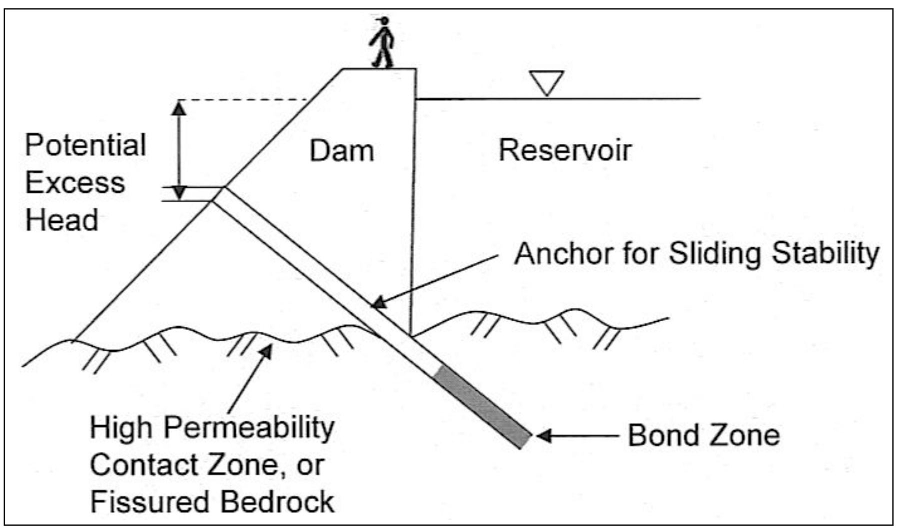
(3) Following pre-grouting, and before the grout has reached an in-situ strength in excess of the surrounding rock mass (typically 18–72 hrs after pre-grouting), the anchor hole is redrilled using an appropriate drilling method. The hole is then subjected to a second water test using the initial permeability criterion unless the initial pre-grouting volume take has been less than 120% of the theoretical hole volume. In most cases, one pre-grouting event is sufficient to seal the borehole effectively so that it will pass the subsequent water test. However, it is not unusual to encounter occasions when additional pre-grouting operations are required. In a minority of cases, a second, and even a third, pre-grouting operation will be necessary. The site-specific characteristics of the rock mass in the bond zone dictate the need for additional pre-grouting.

(4) Very careful analysis of the correlation between water test results and grout takes is essential to understanding the nature of the rock mass system and hence its ability to consume cement grout from the tendon grouting operation. If a "bad" failure on the water test is followed by a relatively large volume of pre-grout, one can confidently assume the presence of large fissures in the rock mass. The subsequent redrilling and water testing will thus provide a guide as to the efficiency of the pre-grouting and the need for a second phase of treatment. In these cases, it is common to pre-grout with a thicker mix, or even a sanded mix. Conversely, even a "bad" failure, followed by minimal grout consumption (i.e., less than 120%), will be indicative of a myriad of small fissures that, though collectively water permeable, will not permit cement grout particles to enter. In this case, the hole may or may not pass any subsequent water test, but further regrouting and redrilling would be futile and unnecessary; the rock mass is grout-tight, so no loss of tendon grout will subsequently occur.

(5) The following is a general guide to pre-grouting in various conditions:

(a) "Normal" Conditions. In this case, which is found in the majority of projects, the hole has simply failed the water test and the regional geology does not indicate the likelihood of massive voids. Pre-treatment is typically conducted with a neat cement grout (water-to-cement ratio [WCR] = 0.5–1.0 by weight, depending on the designer's "feel" for the fissure apertures). The pressure may be from gravity only (i.e., grout applied by tremie, full height) or via a down-hole packer sealed above the bond zone to, say, 0.5 psi/ft of depth. (In strict contradiction to the goals of curtain grouting, rock anchor pre-grouting is only intended to seal the area immediate to the borehole, not to infiltrate for relatively large distances radially away from the hole.) Grout volumes up to five times the theoretical borehole volume may be anticipated. Only one event is typically required, even though it may be anticipated that the subsequent water test may fail by a narrow margin.

(b) “Artesian” Conditions. It is unusual to encounter true artesian conditions in rock anchoring, especially under existing operating dams. However, it is common, and indeed predictable, to encounter water coming out of anchor holes when these holes are commenced at an elevation below that of the lake, and when drilling has reached the dam/rock contact or some highly permeable zone close below it (Figure 23-3). Drillers tend to refer to such emissions as “artesian.” This is a difficult situation that must be dealt with efficiently and on a “hole-by-hole” basis. A packer is placed above the zone encountering inflow. Using the typical neat cement grouts described for “normal” conditions, above, injection is conducted through the packer at a pump pressure equivalent to the “artesian” pressure plus the target effective pressure (e.g., 0.5 psi/ft). During injection, the mix may be thickened or otherwise abruptly modified (in its rheology or hydration) if there is no reduction in the Apparent Lugeon value within the first 20 minutes of pre-treatment. Upon refusal, the grout must be allowed to set with the injection pipe closed off and the packer in place. Clearly, site-specific selections must be made to facilitate this, such as the use of disposable, “drill-through” packers or a cap equipped with a shut-off valve that is closed on completion of grouting the hole.



(Courtesy of Dr. Donald Bruce.)

Figure 23-3. Conditions in which excess water pressure can be encountered.

(c) “Voided” Conditions. Certain geological terrains such as karstic limestone can contain voids or extremely weathered zones of great vertical and/or lateral extent. Pre-grouting of such conditions is essential for the proper placement and performance of anchors, and it may often have to be conducted two or more times in descending stages simply to allow the anchor borehole to be drilled to the target depth. The use of neat cement HMG may be uneconomical and wasteful in these circumstances. One alternative is to use a sand-cement “mortar” with injection under gravity head pressure. However, contractors dislike using such mixes in their typical HMG plants due to the greatly increased rate of wear they generate. Despite their good rheological (i.e., high internal friction) and strength qualities, sand-cement grouts are now rarely used, but are still considered an option. Another alternative is to use true LMGs in such extreme conditions. As discussed in Chapter 7 of this manual, these grouts are of concrete-like consistency with low slumps (typically < 4 in.) and very high internal friction. They require the use of concrete (as opposed to grout) pumps and can be delivered in Ready-Mix trucks, not requiring "colloidal" mixing. They are also easy to dose with admixtures and other materials, including accelerators, anti-washout agents, or polypropylene fibers.

d. Tendon Grouting. The grouting means, methods, and materials involved in sealing the tendon assembly into the pre-sealed borehole are relatively straightforward in principle, even if the geometric complexity of contemporary high-capacity tendons demands extra attention to detail in practice. The performance criteria for the grouts and the grouting processes are simple and logical:

(1) The grout must be pumpable, stable, durable, penetrative, and of sufficient strength to transmit bond stresses.

(2) The injection process must ensure complete filling of all target spaces (inside and outside any corrugated protection), the effect of this must be measurable and verifiable, and the operation must not cause distortion or distress to any component (e.g., the closed-end corrugated sheath) of the tendon assembly. Quantification of these parametric requirements is provided in Section 23-4.

(3) For tendons of relatively low capacity and short length, it may prove advantageous to pre-grout, under factory conditions, the tendon into the protective corrugated sheath. This operation, which also must be done with special care, is described in Section 23-5.

23-4. Anchor Grouts and Their Properties.

a. Materials. Grouts used for pre-grouting anchor boreholes and grouting the tendon assembly for high-capacity pre-stressed rock anchors are invariably cement based. Furthermore, as described in Section 23-4b below, they tend to be simpler in composition than the suites of high-performance grouts now used for routine rock mass grouting. The most common component materials are:

(1) Cement.

(a) Cement grout is made from Type I, II, III, or V portland cement conforming to ASTM C150 (2012c). Blended cements (conforming to ASTM C595 [2013e]) and oil well cements are typically neither necessary nor used for anchors. Also, mineral additives (fly ash, ground granulated blast furnace slag, and silica fume) have not been used in anchor grouts to date, even though they have the potential to provide certain technical benefits in the fluid and set phases. They are more commonly used in grouts for post-tensioning ducts.

(b) The ground is considered aggressive to Type I portland cement if the water-soluble sulfate (SO_4) content of the soil exceeds 0.10% by weight. Sulfates can attack portland cement grout. The intensity of sulfate attack on cement grout is assumed to be the same as for concrete. Type II portland cement is used if the sulfate content is between 0.1 and 0.2%, and Type V cement is used if the sulfate content exceeds 0.2% or if nearby concrete structures have experienced sulfate attack. However, there have been no recorded anchor failures resulting from chemical attack on portland cement grout.

(2) Water. The water used in the grout must be potable (suitable for public consumption) and free of injurious quantities of substances known to be harmful to portland cement or pre-stressing steel. If potable water is not readily available, local water can be used provided the water is tested to assure that it is not detrimental to the tendon or grout. It must meet the requirements of ASTM C1602 (2012d).

(3) Aggregates. Aggregates, if used, must meet all of the requirements of ASTM C33 (2007) except for gradation. Aggregates, if used inside a sheath, should have a maximum size of 1 mm (0.04 in.). Historically, aggregates have been used only for drill hole pre-grouting and for grouts in large-diameter anchors formed by augering. The shape and gradation of the sand particles strongly influence the rheology of the grout and therefore its pumpability and homogeneity. Aggregates can improve the overall performance of grouts for certain purposes, but their impact on mixing, pumping, and placing, as well as their effect on equipment, should be carefully considered from an operational viewpoint when considering their overall cost-effectiveness.

(4) Admixtures. Admixtures must be in accordance with ASTM C494 (2013d) and PTI (2004) and can be used to modify rheology, improve stability, vary hydration rates in fluid grouts, and increase strength and durability in set grouts. Their use must be approved by the Government, based on successful prior use and/or specific tests. Admixtures are not routinely needed, but may prove beneficial or even essential in certain circumstances.

(a) In any multi-component grout mix formulation, all additives and/or admixtures should be supplied by the same manufacturer to avoid potential problems with chemical incompatibility between the components. Furthermore, all admixtures must be compatible with the cement and other admixtures, if used, and must not cause any short- or long-term damage to the grout or any tendon component. Admixtures must be used in strict conformance with the manufacturer's recommendations.

(b) Expansive admixtures may be used only in grouts that fill sealed encapsulations, trumpets, anchor covers, or, in some applications, for secondary grouting and inside a sheath. Expansive admixtures should not be used in the bond length. For inert-gas-forming materials, the level of vertical height change must be no greater than 2% in up to 3 hrs (ASTM C940 [2010b]). Expansive admixtures are only effective when used in a confined space (i.e., a sealed encapsulation or trumpets) and are not necessary to achieve bond capacity in rock. Expansive admixtures achieve expansion by generating gas. In an open anchor drill hole, the expansion occurs upwards, and the resulting grout and its corrosion protection potential may actually be weakened.

(c) Accelerators should not be permitted in tendon grouting operations, but they may be considered for borehole pre-grouting under certain exceptional conditions (e.g., highly permeable ground or rock masses or dynamic water flows), as noted in Section 23-3.

(5) Pre-Packaged Grout. Pre-packaged grouts must conform to the requirements in PTI (2004). Their use is very rare in large anchor projects.

b. Properties.

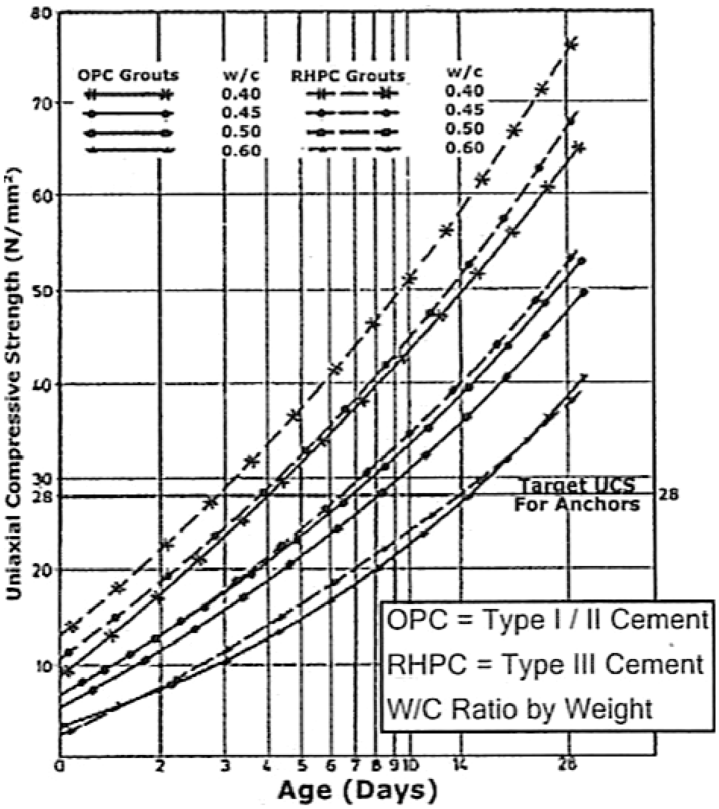
(1) Grouts for Pre-Grouting. Grouts used for pre-grouting are designed to block off fissures and other permeable features intersected by the borehole in the quickest and most cost effective manner. They may also be relied on to impart mechanical strength to the borehole wall and the rock mass immediately surrounding it. As noted in Section 23-3, the performance goal is therefore radically different from that of contemporary HMGs used for rock mass grouting: the latter grouts are designed to penetrate as far as practical away from the point of injection, and a ground strengthening contribution is rarely required. In contrast, pumpability (< 100 seconds marsh), setting time (< 4 hrs), and short-term strength (say 2,000 psi at 24 hrs) are the prime properties of grouts used for pre-grouting. Depending on the actual conditions encountered, anti-washout or accelerating admixtures may be required, while sand may also be advantageous. Therefore, a low-pressure filtration coefficient (< 0.040 min.$^{-1/2}$) and minimal bleed (< 5%) are not as important, especially given that the inherent instability of such grouts will result in water being expressed from the grout even under gravity head conditions, thereby facilitating the grout's ability to satisfy the in-situ goal of pre-grouting by limiting penetration.

(a) Neat pre-grouts range in WCR (by weight) from 0.5 to 1.0, and when sand is used, its ratio to cement is typically up to 1 (by weight).

(b) Admixtures are used in the standard proportions of HMGs (see Chapter 7 of this manual), while bentonite is never used, since it reduces the initial rate of strength gain and improves pressure filtration stability, neither of which is desirable in this application.

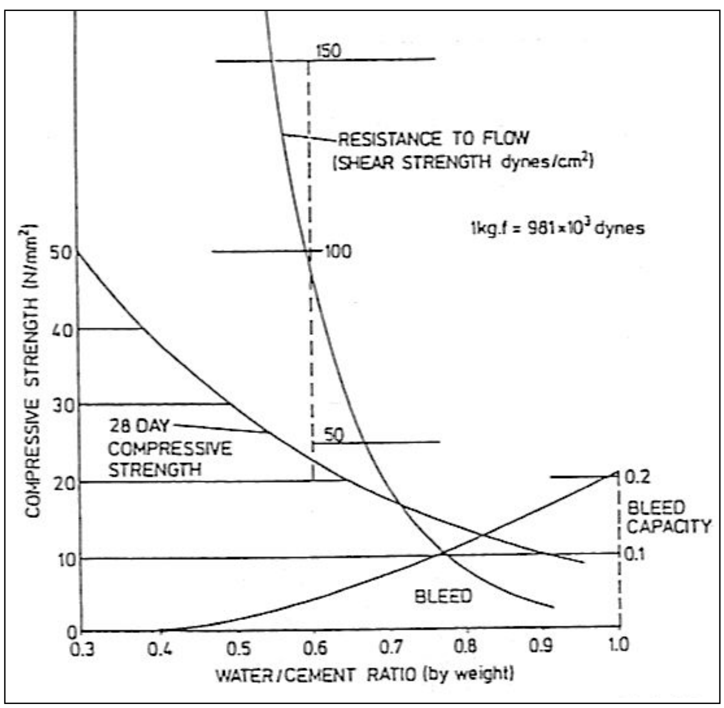
(2) Grouts for Tendon Assembly Grouting. In all cases, the tendon assembly should be placed and grouted in the borehole as soon as practical after the borehole has been proved, by water testing, to be of acceptably low permeability. The grout placed *around* the tendon assembly (such as the corrugated ducting used as corrosion protection, and the individual epoxy-coated strands) must have minimal bleed (preferably zero), good fluidity (marsh cone < 60 seconds), reasonable working time (< 4 hrs), and strength compatible with the design assumptions and project needs. These grouts have the combined roles of transferring the load from the tendon assembly to the rock and acting as an integral part of the corrosion protection system. In this regard it is common to specify a minimum unconfined compressive strength of 3,000 psi at stressing and/or a 28-day strength of at least 6,000 psi. To satisfy these criteria, tendon grouts are predominantly neat cement grouts with a WCR (by weight) not exceeding 0.45. Figure 23-4 shows design guidance, while the data shown in Figure 23-5 provide further justification for the use of relatively low WCR grouts (i.e., minimal bleed and high strength).

(3) Only in special conditions such as for very high ambient temperatures or long pumping distances is an admixture included to modify rheological or hydration characteristics. These grouts are also very durable, as they must be, since they are an integral part of the overall corrosion protection system.



(From Littlejohn and Bruce 1977. Courtesy of Dr. Donald Bruce.)

Figure 23-4. Strengths for various water-cement ratios.



(From Littlejohn and Bruce 1977, Courtesy of Dr. Donald Bruce)

Figure 23-5. Effect of water content on cement grout properties.

(4) Grout placed *inside* the encapsulation generally has the same minimum performance requirements. The contractor must demonstrate, before commencing work on site, that the grout design, batching, and placing techniques will ensure a stable, flowable grout that will not segregate when placed in a watertight duct. Recent research indicates that bleed and segregation can occur in long tendons in grout placed in a hydraulically closed system. It may prove necessary to use multi-component grouts inside such encapsulations to ensure that appropriate grout performance parameters can be met. Pre-grouting of encapsulated tendons must be conducted on an inclined, rigid frame or bed by injecting the grout from the low end of the tendon.

(5) Care must be taken to ensure that excess differential grout pressures acting on the encapsulation do not damage, distort, or displace it. Cutting of "windows" in the sheath or omission of the end cap to allow equalization of interior and exterior grout levels must not be permitted. This applies regardless of whether the duct is pre-placed or placed simultaneously with the tendon.

(6) Multiple lifts of grouting may be required in cases where the encapsulation is grouted in the hole before tendon placement. This will typically involve the use of numerous grout tubes placed around the encapsulation and terminating at different elevations. This procedure applies typically only in long, multi-strand tendons where the encapsulation could be damaged during installation by the weight of the strands (Figures 23-6 and 23-7).

(Courtesy of Dr. Donald Bruce)

Figure 23-6. Corrugated ducts being prepared before insertion into pre-grouted and waterproofed anchor holes at Gilboa Dam, NY. Note the multiple tremie tubes attached to the outside of the ducts to allow external grouting to be accomplished in several stages.

Figure 23-7. Corrugated duct being inserted into the prepared anchor hole at Bluestone Dam, WV (Huntington District). The numerous external tremie tubes are clearly visible.

(7) The in-situ grout can be weakened if diluted with groundwater before setting. The rate of strength gain of grouts cured at very low in-situ temperatures will be reduced. Special grouts are available for low-temperature applications, even for installations in permafrost. When aggregates or non-potable water are to be used and will be in direct contact with pre-stressing steel, the constituent materials used must be such that the acid-soluble chloride ion content of the grout does not exceed 0.08% by weight of portland cement as measured by ASTM C1152 (2013a), *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete*. Neat cement grout mixed with ASTM C150 (2012c) cement and potable water does not need to be tested for chemistry.

(8) Since materials and material combinations vary from location to location, the chloride ion content should be determined by independent analysis of the combined materials used at the site before grouting operations begin.

(9) Table 23-1 refers to two classes of corrosion protection and emphasizes the critical role played by the grouting in this regard.

Table 23-1. Corrosion protection requirements.

Class*	Corrosion Protection Requirements		
	Anchorage	Free-Stressing Length	Tendon Bond Length
I Encapsulated Tendon	Trumpet Cover if exposed	• Corrosion-inhibiting, compound-filled sheath encased in grout, or • Grout-filled sheath, or • Grout-encased, epoxy-coated strand in a successfully water-pressure-tested drill hole	• Grout-filled encapsulation, or • Epoxy-coated strand tendon in a successfully water-pressure-tested drill hole
II Grout-Protected Tendon	Trumpet Cover if exposed	• Corrosion-inhibiting, compound-filled sheath encased in grout, or • Heat shrink sleeve, or • Grout-encased, epoxy-coated bar tendon, or • Polyester resin for fully bonded bar tendons in sound rock with non-aggressive groundwater	• Grout • Polyester resin in sound rock with non-aggressive groundwater

*After PTI (2004).

(a) Class I Protection. A Class I protection system encases the pre-stressing steel inside a plastic encapsulation filled with either grout or a corrosion-inhibiting compound. Class I protection is often referred to as an “encapsulated tendon” or “double-corrosion-protected tendon.” An epoxy-coated strand tendon grouted into a drill hole that successfully passes the water pressure test satisfies the requirements for a Class I protection system.

(b) Class II Protection. A Class II protection system encases the pre-stressing steel over the free length and relies on the cement grout to protect the pre-stressing steel along the bond length. Class II protection is often referred to as a “grout-protected tendon” or a “single-corrosion-protected tendon.”

23-5. Grouting Equipment and Operations.

a. General. The details of grouting equipment and operations for rock anchors must be addressed with care, since the result of the process on the construction of such vital structural elements is not directly observable in place.

b. Grout Equipment and Mixing. Mixers, storage tanks, and pumps must have adequate capacity and must be sized to allow continuous grouting for an individual anchor in less than 1 hour. As a minimum, grouting equipment consists of a mixer, a storage tank, and a pump, along with the associated pressure gauges, flow meters, and hoses. On large projects, cement is provided in bulk in storage silos and weigh batched into the mixer (Figure 23-8).

(Courtesy of Dr. Donald Bruce.)

Figure 23-8. Typical grout batching plant being fed by bulk cement from a silo at Gilboa Dam, NY.

c. Neat Cement Grouts. Neat cement grouts, with or without admixtures, must be mixed on site with colloidal mixers. The use of paddle mixers should be prohibited. Colloidal (i.e., high-speed, high-shear) mixers provide superior cement hydration efficiency and uniformity relative to other types. They permit grouts of lower water-cement ratio to be quickly hydrated, and they provide grouts of superior stability, rheology, and set properties. Water and admixture measuring devices (batchers) are used to ensure accurate proportioning of grout ingredients. The accuracy of batching must be sufficiently controlled to assure that the water-cement ratio of the grout is within 5% of the target value. The accuracy of the dosage rate for admixtures should be within 10% of the target value.

(1) The grout must be continuously agitated until pumped, and it must be used within 30 minutes after the mixing begins unless hydration control admixtures are used.

(2) The storage tank must be kept at least partially full at all times during injection.

d. Site-Mixed Sanded Grouts. Sanded grouts require mixers and pumps built for grouts containing fine aggregates. When heavily sanded grouts are to be used (e.g., for pre-grouting), then paddle- or drum-mixed grout may be acceptable, provided adequate fluid and set performance has been previously demonstrated. Non-colloidal mixers should contain a screen having clear openings of 6 mm (0.25 in.) maximum size to screen the grout before its introduction into the storage tank. The screen must be inspected periodically during grouting operations. The presence of lumps of cement on the screen may indicate incomplete mixing and may result in blockages during injection.

e. Ready-Mixed Sanded Grouts. Sanded grouts for pre-grouting may be mixed at a Ready-Mix plant and delivered to the site by truck. Special attention must be paid to ensure that unmixed lumps are not permitted to be injected.

f. Temperature Considerations. During grouting operations in high ambient temperature (above 100 °F [38 °C]), the temperature of the grout should not exceed 90 °F (32 °C). If it is unavoidable that the temperature of the grout exceeds 90 °F (32 °C), then special precautions such as the use of suitable admixtures should be taken to control flash set. Grout for anchorage covers and trumpets must be placed when the temperature of the anchorage is expected to remain above freezing for 48 hrs. Grouts compatible with the pre-stressing steel and formulated for below-freezing applications can be used with the designer’s approval.

(1) Several techniques can be employed to reduce the temperature of the components quite effectively at the job site. Careful shading to keep the dry materials out of the hot sun has been found to be effective. The temperature of the dry materials should be taken with a probe thermometer and recorded, as the dry components are largely responsible for the temperature of the mixed grout due to the volume of the mixed ingredients. Ice added to the mixing water can also reduce the temperature of the freshly mixed grout to a level low enough to avoid flash set.

(2) Freezing of the grout must be prevented for a period of 48 hrs or until the grout has reached an unconfined compressive strength of 800 psi. Freezing of grout in drill holes is rarely an issue; grout used to fill trumpets and stressing pockets could be exposed to freezing temperatures, however.

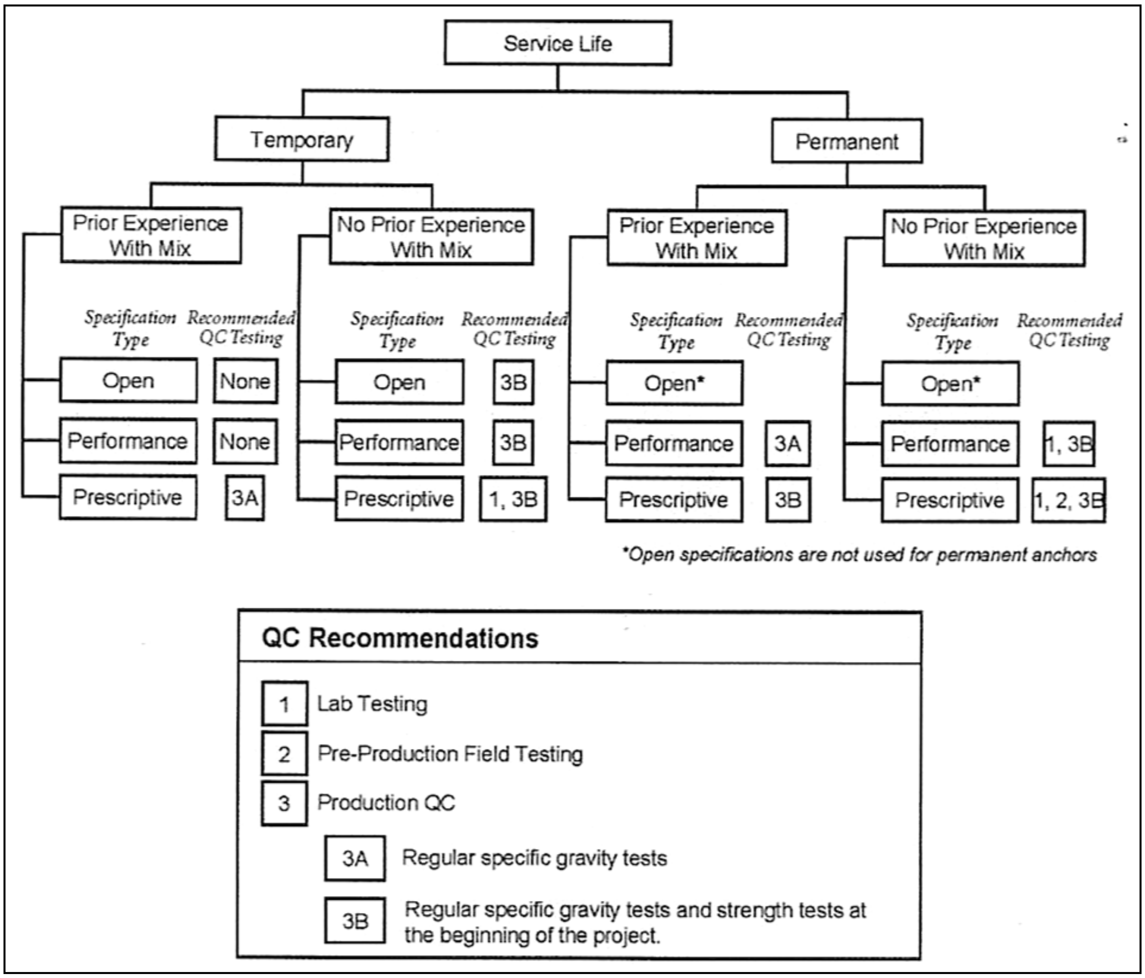
g. Finishing Work. For two-stage grouted anchors, the free-stressing length must also be filled by tremie grouting, and the transition tube, if required, must also be filled completely with grout.

(1) Pre-packaged grouts, neat cement grout with expansive admixtures, or multistage grouting operations may be required to ensure that the anchorage cover or trumpet is completely filled with grout (or corrosion-inhibiting compound). The importance of efficiently grouting the zone immediately under the top anchorage cannot be overstated. Of the few corrosion-induced anchor failures actually recorded, the majority have occurred on tendons otherwise unprotected at this location (FIP 1986), generally caused by insufficient attention to detail during design and/or construction.

(2) Filling the stressing pocket, if applicable, with non-shrink grout completes the anchor construction work.

23-6. <u>QA/QC for the Mixed Grout</u>.

a. General. The level of grout testing depends on the service life of the anchor, prior experience with the grout mix, and the type of specification for the project. Figure 23-9 depicts the minimum QC programs requirements for grouts, the level of which are determined by the designer. The designer may offer to waive the need for project-specific tests if the contractor can document the acceptability of the proposed mix designs via relevant and comparable studies conducted for prior projects using the same materials. Such preliminary acceptance should be conditional upon successful results from the pre-production or production field tests.



(From PTI 2004)

Figure 23-9. Minimum recommended levels of grout QC programs.

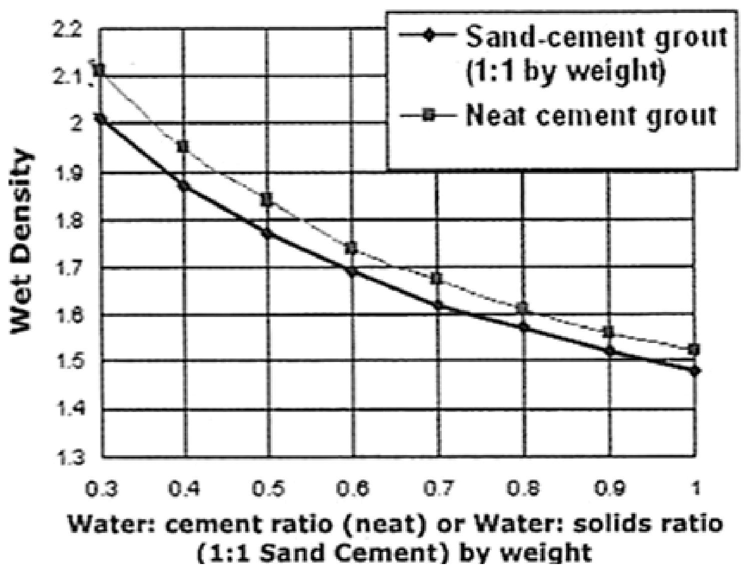
b. Pre-Production Lab Tests. Pre-production lab tests are conducted in a testing laboratory. Compressive strength tests are performed in accordance with ASTM C942 (2010C) for grout cubes or ASTM C39 (2010a) for grout cylinders. The wet density of the grout is determined using the API Mud Balance Test (API 13B-1) or ASTM C138 (2013b). Bleed tests are performed in accordance with ASTM C940 (2010b). As far as possible, the mechanical means of test sample mixing, the materials, and the curing environments must be the same as for the production mixing. The program should be conducted and analyzed within a time frame that does not adversely impact the contract schedule.

c. Pre-Production Field Tests. Pre-production field tests are conducted using the equipment and materials proposed for production work under typical ambient conditions. The location of these field tests may not actually be the job site. The purpose of these tests is to provide a quality and parameter “baseline” against which to judge the results from production testing. Compressive strength, wet density, and bleed tests are performed using the standard methods listed above. In particular, the rate of strength gain over a period of at least 7 days must be demonstrated. The contractor may proceed with the construction only after the designer approves the results of the tests.

d. Routine QA/QC During Production. Figure 23-9 shows the minimum requirements for routine QC testing during production. Compressive strength testing, when required, need not be repeated after the initial phase of testing, provided:

(1) There is no significant change thereafter in materials, equipment, or conditions.

(2) A regular program of fluid grout testing is conducted to verify the correct specific gravity (and hence water-cement ratio), as shown in Figure 23-10.



(From PTI 2004.)

Figure 23-10. Wet density vs. water-cement ratio.

(3) Measurement of the wet density (specific gravity) of the mixed grout permits the water-cement ratio to be determined, thereby confirming that the grout has been correctly batched. The strength can then be predicted with accuracy. (See Figure 23-4, which refers to neat cement grout mixed in a high-speed, high-shear colloidal mixer.)

(4) Maintaining records of all grouting activities (including volumes, pressures, components, and mix designs for every hole) and test results is typically the responsibility of the contractor.

23-7. Submittals, Records, and Reports.

a. General. Comprehensive submittals, records, and reports are essential and must be archived for any future reference. Certificates of conformance for all materials and their relevant properties should also be retained. Although all parties should share the responsibility for recordkeeping, it is typical that the contractor will maintain adequate and satisfactory “as-built” data to assure compliance with the preconstruction submittals.

b. Preconstruction Submittals. These must be in conformance with the specification and must cross-reference the relevant clauses. In the simplest cases, information is provided only in the form of shop drawings. It is more typical, however, to find that a separate, bound document is required, since such submittals often require the presentation of catalog cuts of equipment and materials, and data from previous lab and/or field test projects. Relevant information must be provided on all materials (including certifications), mix designs, and mixing and placement techniques and equipment. A full QA/QC plan must also be submitted, which will include the type and frequency of grout testing (fluid and set properties), equipment calibration (e.g., load cells and pumps) and procedures for real-time instrumentation and recordkeeping (e.g., pressure, rate of injection, and volume).

c. Records and Reports. These should contain the results of all construction-related measurements and readings and of all the activities of the QA/QC Plan. Non-conformances must be highlighted, and their significance, or otherwise, to the project clearly rationalized. Strength data should include the development of rate of gain of strength curves for each mix design.

d. Special Attention. Special attention must be paid to the results of the pre-grouting so that the path to grout tightness of every hole can be clearly understood. Whereas the stressing and testing of every anchor verifies its load-holding ability, the long-term durability of the anchor can be predicted accurately by carefully analyzing the details of each successive construction step, in particular the pre-grouting/water testing operations.

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CHAPTER 24

Tunnel Grouting

24-1. Introduction. Grouting is used in tunnel applications for one or more of the following purposes: reducing groundwater inflows, strengthening the surrounding soil/rock mass to facilitate tunneling, controlling ground subsidence, and enhancing soil/rock-structure interaction. This chapter provides a limited overview of the various grouting techniques commonly applied to tunneling. Henn's (1996) *Practical Guide to Grouting of Underground Structures* and Warner's (2004) *Practical Handbook of Grouting Soil, Rock, and Structures* are recommended resources for supplemental technical information and guidance. Chapter 25 of this manual discusses remedial grouting to reduce water inflows into existing tunnel structures.

24-2. Applications.

a. General. Grouting in tunnel applications may be performed before, during, or after completion of tunnel excavation. Depending on its intended purpose and access issues, grout holes may be drilled from the ground surface, from within the tunnel, or both. A wide variety of grouting techniques can be applied to soil, rock, or mixed face conditions, depending on the site conditions and outcome desired. Table 24-1 lists grouting techniques that are commonly employed along with their general application to tunneling in both soil and rock.

b. Soil Grouting Techniques.

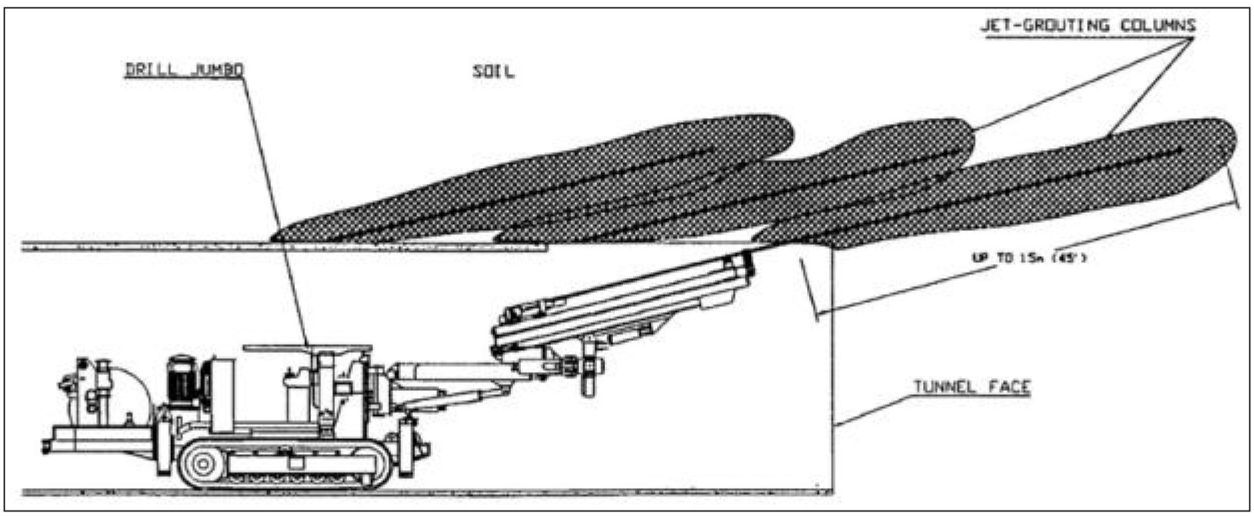
(1) Jet grouting involves the use of high-pressure jets of cement grout discharging perpendicular to the direction of hole advance. The high-velocity jets simultaneously excavate and mix the grout with the in-situ soil to create soilcrete columns. Jet grouting can be used either from within the tunnel or from the ground surface, as shown in Figures 24-1 and 24-2.

(2) Compaction grouting, as discussed in Chapter 27 of this manual, is the process of injecting very stiff, mortar-like grout at high pressure and in a controlled manner for the purpose of densifying the soil around the injected mass. In soft ground tunneling applications, compaction grouting is commonly used from the ground surface for settlement control (Figure 24-3).

(3) Permeation grouting in soil involves the injection of either cementitious or chemical grout into the pore spaces of soils to reduce seepage and/or improve the soil. Permeation grouting is typically performed before tunnel excavation. Injection holes can be horizontal or inclined from within the tunnel or from the ground surface (Figures 24-4 and 24-5). The soil particle size (grain size distribution) of the host formation will dictate the type of grout used. Figure 24-6 shows a "soil groutability" chart indicating the general suitability of cement and chemical grouts for various soil gradation bands.

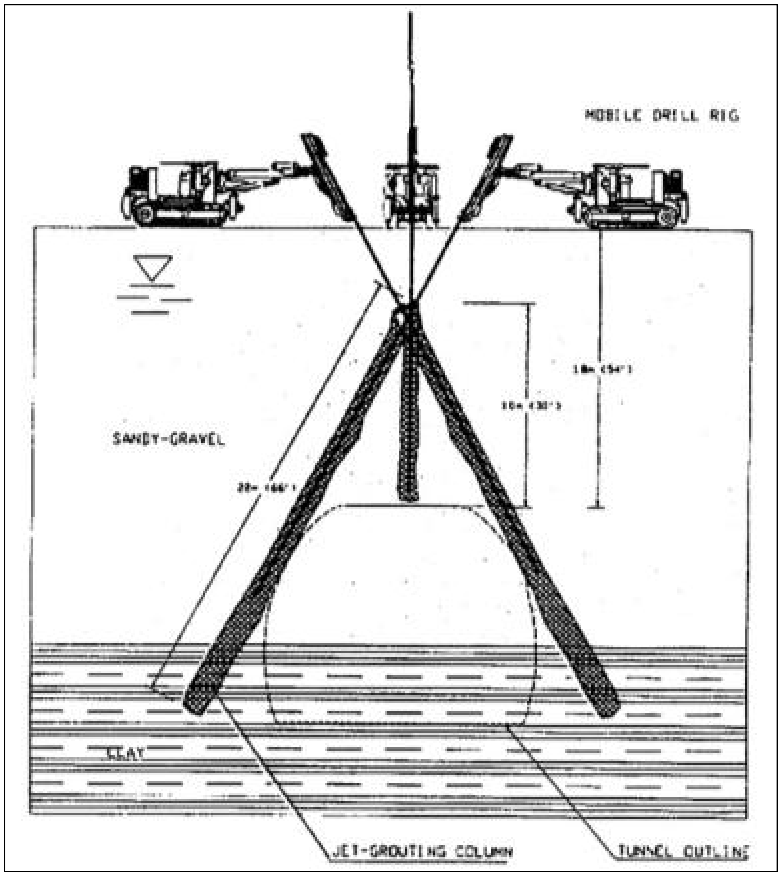
Table 24-1. Grouting techniques and their general application to tunneling.

	Grouting Techniques	General Applications
Soil	Jet grouting	Reduce seepage Improve strength Settlement control
	Compaction grouting	Settlement control
	Permeation grouting	Reduce seepage Improve strength
	Hydrofracture grouting	Improve strength Settlement control
Rock	Consolidation grouting	Reduce seepage Improve strength
	Permeation grouting	Reduce seepage
	Contact grouting	Void filling-structure interaction



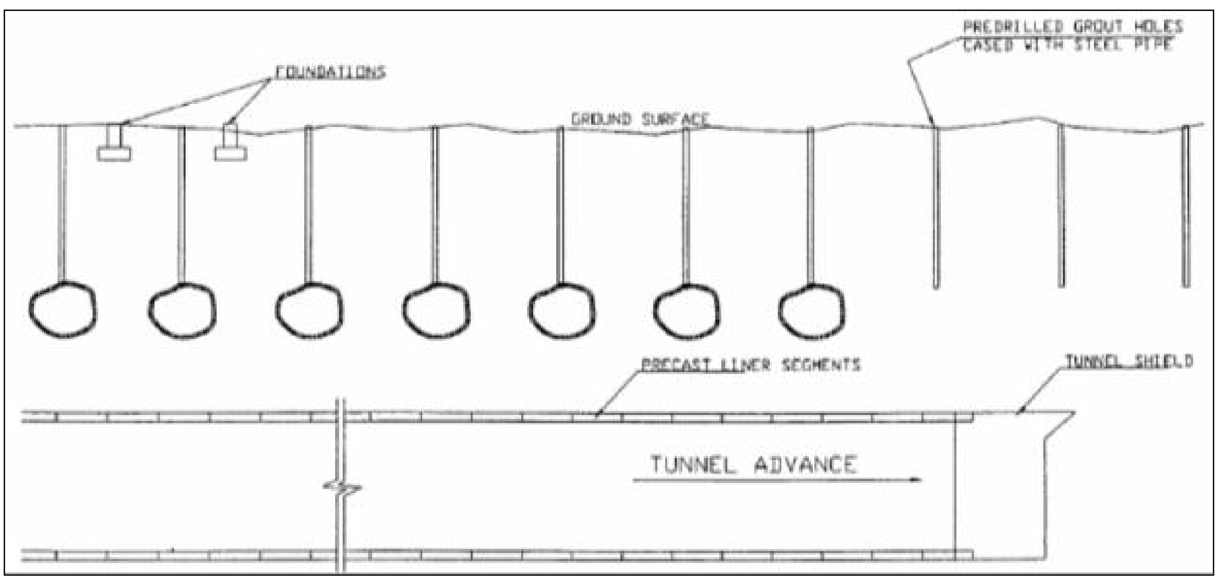
(From Henn 1996, with permission from ASCE.)

Figure 24-1. Jet grouting within a soft ground tunnel.



(From Henn 1996, with permission from ASCE.)

Figure 24-2. Jet grouting from the ground surface for a soft ground tunnel.



(From Henn 1996, with permission from ASCE.)

Figure 24-3. Compaction grouting from the ground surface for a soft ground tunnel.

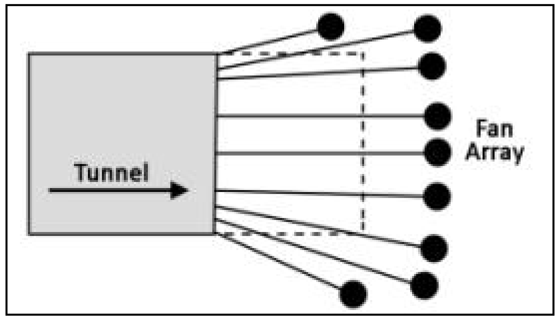


Figure 24-4. Permeation grout fan array from a tunnel heading.

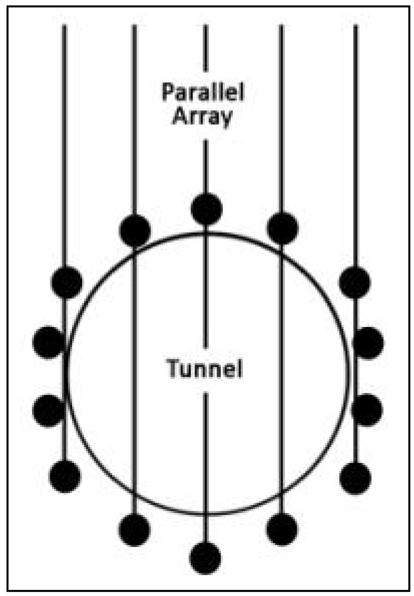
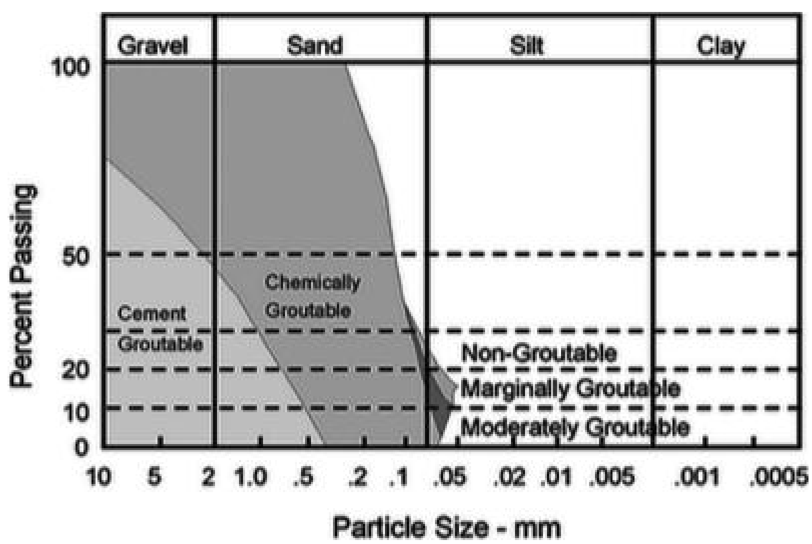


Figure 24-5. Permeation grout parallel array from the ground surface.



(Courtesy of Hayward Baker)

Figure 24-6. Soil groutability based on soil particle size.

(4) Hydrofracture grouting, which is covered in Chapter 28 of this manual, involves the injection of a high-mobility stabilized cement grout at a pressure sufficient to induce tensile failure of the soil, resulting in the formation of grout lenses within the soil. These pressurized lenses of grout densify the soil by plastic deformation in the vicinity of the lenses and also reinforce the soil mass due to the higher strength of the grout in comparison to the soil. Hydrofracture grouting is used to compensate or offset settlements experienced during soft ground tunneling. Hydrofracture grouting is typically conducted from the ground surface along the tunnel alignment similar to the compaction grouting process shown in Figure 24-3.

c. Rock Grouting Techniques.

(1) Consolidation grouting strengthens rock masses and reduces permeability by filling open fractures and other discontinuities within the rock mass. Consolidation grouting may be performed before tunnel excavation to improve the in-situ rock mass conditions (Figure 24-7) or after tunnel excavation in areas of poor rock quality or to remediate excessive seepage (Figure 24-8). A combination of pre- and post-grouting can also be employed, as shown in Figure 24-9.

(2) Permeation grouting for water control in rock tunneling operations is most often performed in advance of tunneling through probe holes performed from the tunnel heading (Figure 24-10). Probe holes are drilled ahead of the tunnel during advancement. If the probe holes detect excessive groundwater inflows, additional drilling and permeation grouting is conducted ahead of the excavation. As with grout curtain construction, secondary holes are drilled following the initial series of drilling and grouting to verify the effectiveness. If the secondary holes again detect excessive groundwater inflows, a second phase of permeation grouting is performed. The process is then continued until an acceptable level of inflow is achieved.

(3) Contact grouting is employed to fill the annular space between tunnel liner materials and the excavated rock surface. This is also commonly referred to as “backpack” grouting. Contact grouting is performed to provide uniform contact and structure interaction with the formation, to fill voids that might otherwise result in relaxation or raveling of the host formation, and/or to reduce seepage into the tunnel or eliminate seepage paths along tunnels.

(a) Selection of methods and materials used in contact grouting depend on the size of the annular space to be filled, on the size and frequency of planned grout holes, on the presence of water, and on the purpose (i.e., need for strength, seepage reduction, or simply bulk fill). Typical grout materials may consist of: cement grout with or without aggregates, fly ash, bentonite, or other additives to facilitate placement; cellular (foamed) grout; flowable fill; or chemical grouts.

(b) Grout hole patterns and sequencing vary, but normally begin with grout placement in the lowest portion of the tunnel. Grout is placed under pressure and forced upward, with grout holes at higher elevations used as vents and for verification of complete filling. As grout emerges from pre-drilled vent/grout holes at higher elevations, packers can be used to seal those holes, allowing grouting to continue from lower elevations until grout emerges from vent/grout holes in the uppermost portion of the tunnel. Figure 24-11 shows the grouting sequence used for contact grouting in the rehabilitation of an existing horseshoe-shaped tunnel that contained large void spaces behind the original tunnel lining and shoring system.

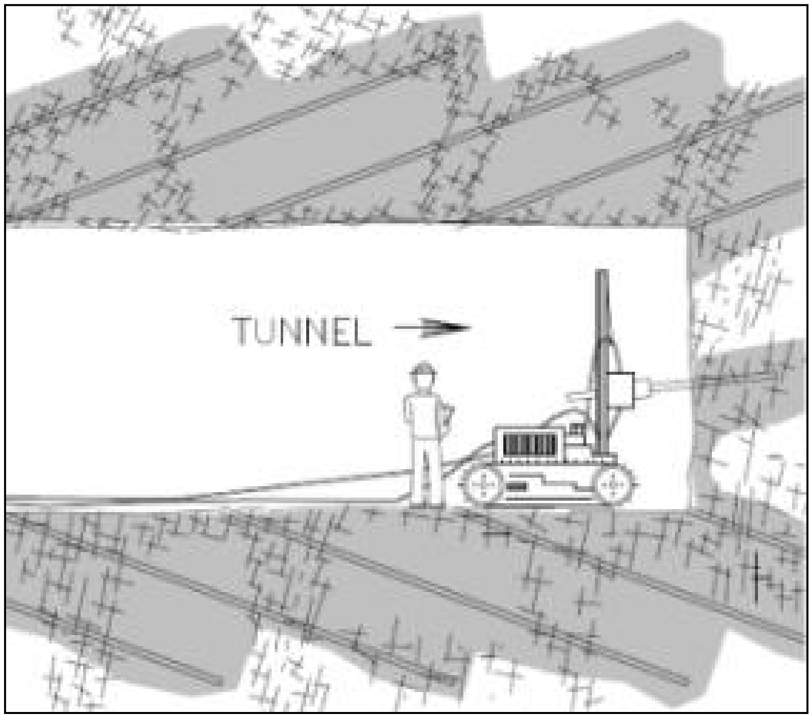


Figure 24-7. Pre-consolidation grouting ahead of tunnel excavation.

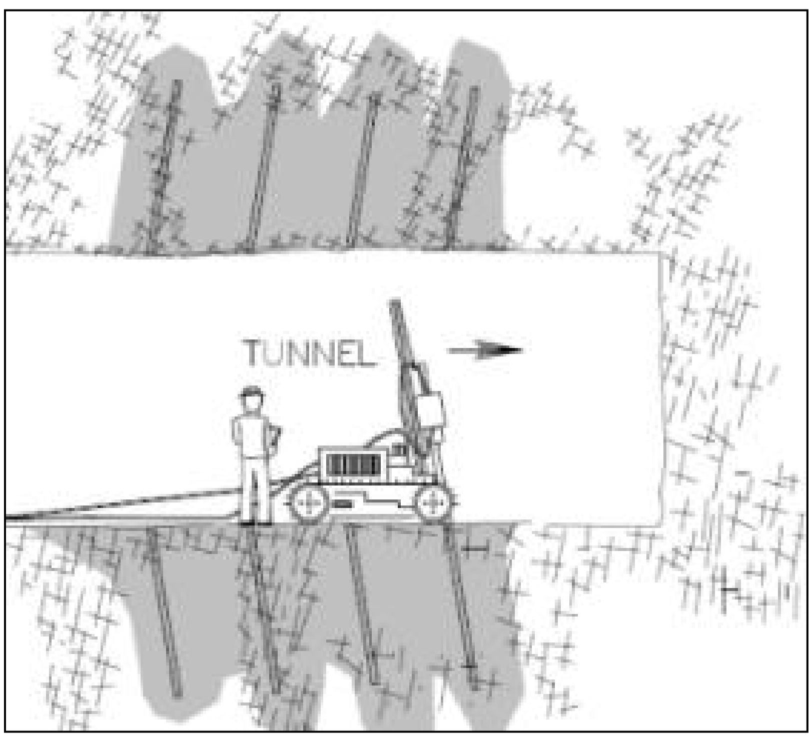


Figure 24-8. Post-consolidation grouting behind tunnel excavation.

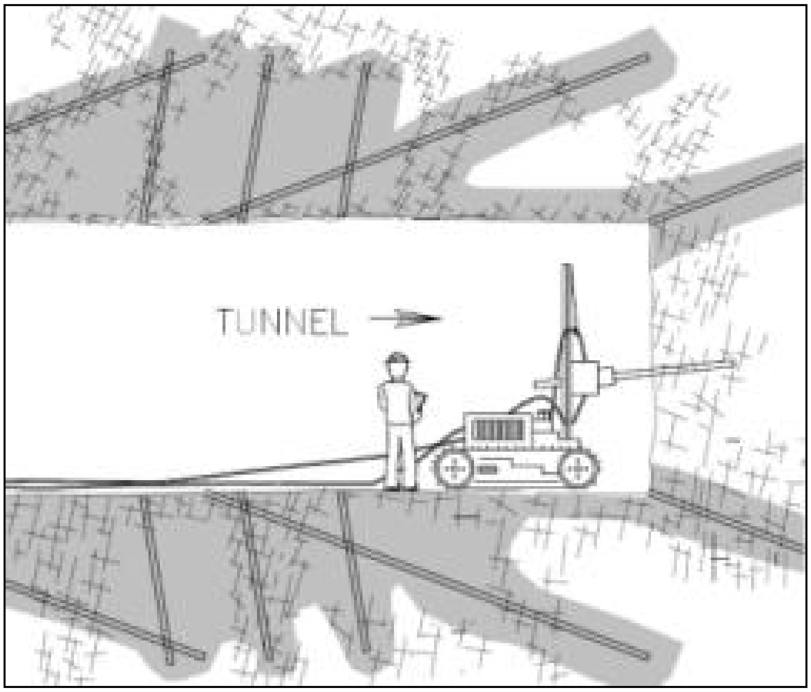
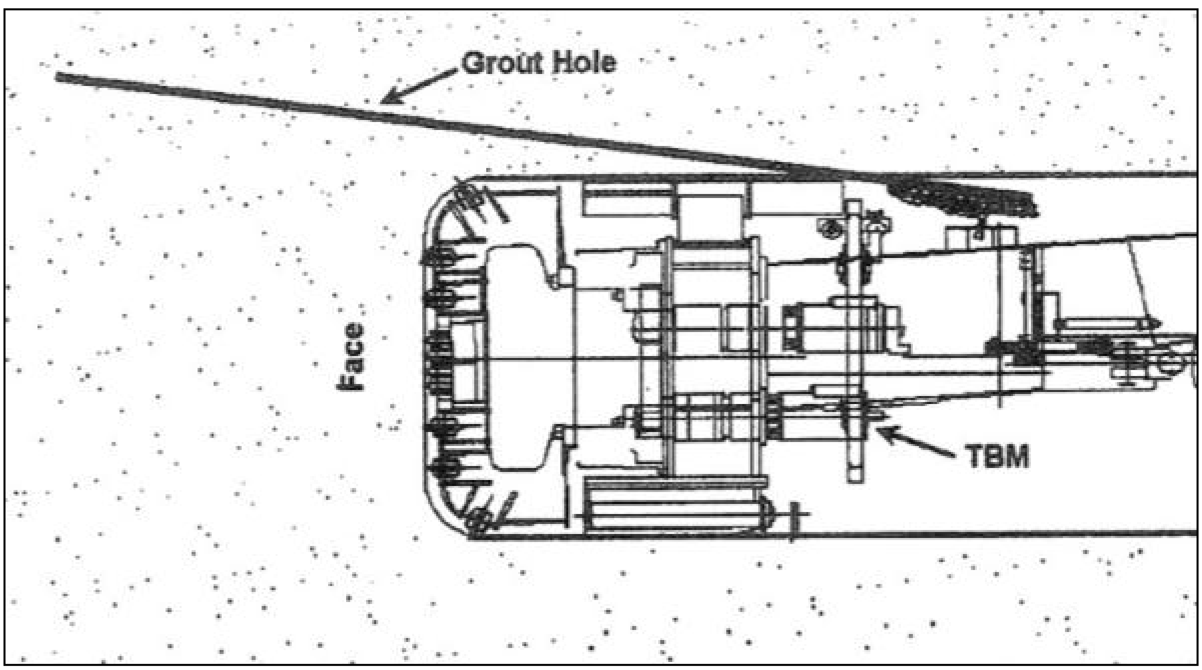


Figure 24-9. Combination of pre- and post-consolidation grouting.



(From Warner 2004. This material is reproduced with permission of John Wiley & Sons, Inc.)

Figure 24-10. Probe hole/grout hole drilled in advance of tunneling.

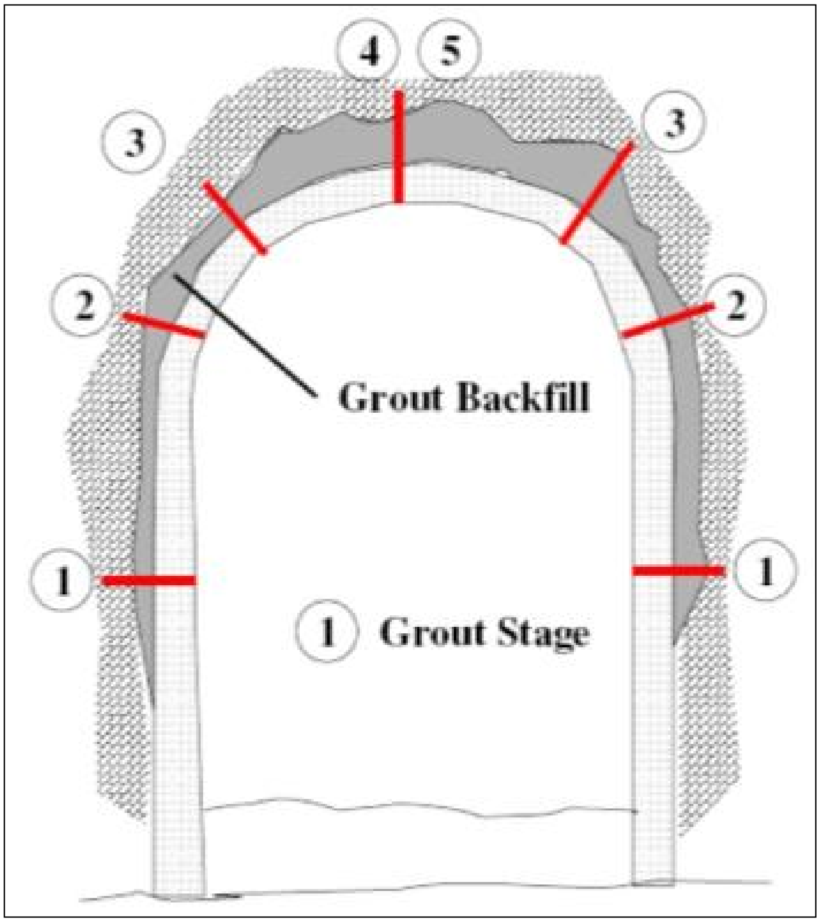


Figure 24-11. Sequence for contact grouting a horseshoe-shaped tunnel.

CHAPTER 25

Grouting of Concrete Structures

25-1. Introduction. Grouting with cementitious or chemical grouts can be employed to repair and/or fill cracks, joints, and voids. Cementitious grout is used when grouting between first and second stage placements of concrete and contraction joints, which are provided in the construction of concrete gravity dams and lock walls on USACE projects. This chapter provides a limited overview of the materials and general procedures. Recommended resources for supplemental technical information and guidance include: EM 1110-2-2002, *Evaluation and Repair of Concrete Structures* (1995), *Practical Handbook of Grouting: Soil, Rock, and Structures* (Warner 2004), *Practical Guide to Grouting of Underground Structures* (Henn 1996), and ACI 224.1R-93, *Causes, Evaluation and Repair of Crack in Concrete Structures, Design of Gravity Dams* (USDOI 1976), *Grouting – Construction Inspector Training* (USDOI 1991).

25-2. Materials.

a. General. Crack or joint repair and void filling can be performed by gravity filling or injection under pressure. Materials and methods for repairing cracks or joints and filling voids are highly dependent on the size of the opening(s) and the purpose of the repair. Openings less about 1/8 in. can be repaired with neat cement grout. Both portland cement and ultrafine cement have been used. For larger openings, sand can be incorporated into the mix. Chemical grouts can be used for small openings and have been used to repair cracks in concrete as narrow as 0.002 in.

b. Cement Grouting. Portland cement grouting of cracks and joints is effective in treating water infiltration and as structural filler. However, it is not used to structurally bond cracked sections. The general principles of grouting discussed elsewhere in this manual are applicable to repairing cracks with cementitious grouts. Cementitious grout mixes can include cement, water, and sand (depending on the size of the opening being repaired or filled). Water-cement ratios should be minimized to increase strength and reduce shrinkage. Water-reducing agents and other admixtures are commonly used to modify the properties of the grout mix, as discussed in Chapter 7 of this manual. Balanced stable grout mixes can be used to eliminate bleed potential.

c. Chemical Grouting. Chemical grout formulations are effective in most crack and joint repair applications and environments. Chemical grouts can be used to treat water infiltration, structurally bond cracked sections, or provide structural filler. Chemical grouts are discussed in detail in EM 1110-1-3500, *Chemical Grouting*. For the general purpose of crack repair in concrete structures, chemical grouts typically include epoxies, urethanes, and acrylates, depending on the specific application and environment. Epoxy resins are often used to repair structural cracks due to their high bond strength. Urethanes and acrylic-based grouts are primarily used to control water infiltration or to provide filler material.

25-3. General Procedures. The following general procedures are typically employed for both cementitious and chemical grouting for crack and joint repair in concrete structures. The above-listed references provide more detailed.

a. Cleaning the Crack and Concrete. Vacuuming is recommended to remove debris, because cleaning by blowing with compressed air may force loose material deeper into the crack. Flushing with water, acid solutions, or solvents has been used. However, any potential detrimental effects of using these liquids, including potential incompatibility with the intended grout material, must be thoroughly investigated. Some chemical grouts require pre-flushing with water to activate the product.

b. Sealing the Exposed Crack. In some applications, it can be beneficial to seal the surface expression of the crack before grouting. Depending on the crack size and anticipated grouting pressures, surface sealing may be accomplished with water-resistant tape, special sealants, cement paint, batten bars and plates, or epoxy grout.

c. Installing Grout Ports. Grout ports are typically installed at intervals adjacent to and on both sides of the crack or joint to be repaired, as shown in Figure 25-1. Holes are typically angled to intersect the crack or void at depth. All grout ports should be installed before grout injection. Some types of ports can be used for monitoring grout travel during grout injection (Figure 25-2).

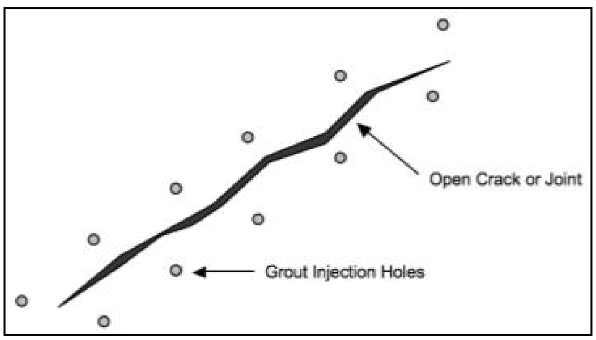


Figure 25-1. Diagram of crack/joint repair hole spacing.

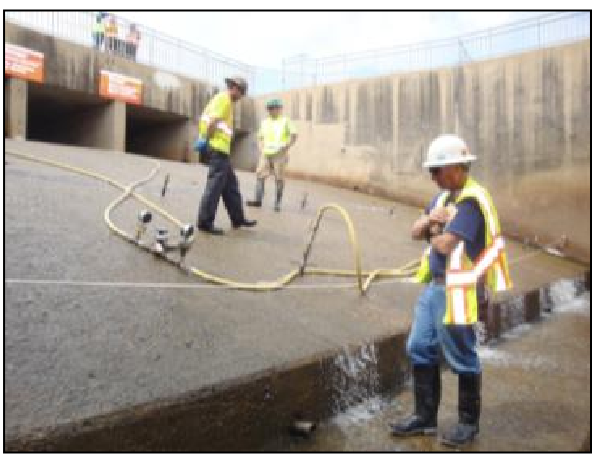


Figure 25-2. Backfill grouting operations to fill voids under the downstream parabolic spillway slab at Barker Dam outside of Houston, TX (Galveston District). Holes were drilled on 10-ft centers, and injection pressures were controlled by the manifold at the injection point. A circulating system was used to verify the volume of voids being filled with cementitious grout.

d. Mixing and Injecting the Grout Material. For crack or joint repair on vertical surfaces, grout injection should begin at the lowest elevation and continue until grout appears at the next entry port above or exiting the crack adjacent to the next higher port. The lower injection port can then be capped, and grouting can proceed at successively higher ports until the crack is completely sealed. For crack or joint repair on horizontal surfaces, grout injection should proceed in the same manner, starting at one end and proceeding continuously to the other.

25-4. Grouting Contraction Joints or Between First and Second Stage Concrete Placements. Mass concrete structures such as concrete gravity dams, high arch dams and lock walls are typically constructed in blocks separated by contraction joints. In dam structures, the contraction joints are vertical, extend from the foundation to the crest of the dam, and are normal to the axis while in lock walls, the monolith joints run perpendicular to the length of the wall. Contraction joints are used to facilitate construction, accommodate embedded metals and machinery, reduce damage and control cracking caused by shrinkage stresses, and permit secondary backfilling or placement of concrete. Contraction joints are grouted in concrete gravity and high arch dams to bind the individual blocks together so that the structure acts as one monolithic mass. This same grouting procedure, which is performed on lock walls between the gate monoliths and the adjacent downstream monoliths to transfer the stresses imposed by the water load on the gates, is intended to alleviate cracking of the concrete around the gate machinery.

a. Methodology.

(1) During construction of the first stage concrete placement, or the first block, a metal seal is in embedded in the joint face. The metal seal is then extended into the second stage concrete placement in the adjacent block and will serve to contain the grout within the joint later when grouting is performed.

(2) A system consisting of supply pipes, return pipes, and vent pipes are installed within the area of the seals. The system configuration depends on the number of lifts that will be performed in the grouting process and the design of the structure (Figures 25-3 and 25-4). Special care should be taken to ensure that the piping system is clear, leak-free, and properly anchored before the secondary placement or the placement of the adjacent monolith block. Monolith joint grouting of lock walls is typically conducted for the entire cross-sectional area of the wall, whereas for high dams, it is desirable to grout from the foundation up in 50- to 60-ft lifts and preferably uniformly across the length of the dam. Before the start of grouting, the system is checked for any leakages by running water through the system so that leaks may be treated or caulked. However, it is not unusual for additional leakages to occur during grouting and provisions should be in place to address and treat any breakouts. Usually, a grout mix of 2 to 1 w:c by volume is used to start the grouting and is thickened to a 1 to 1w:c by volume until the return exhibits the same consistency. When grouting has progressed to a point of approximately 75% complete, the mix may be thickened to a 0.8:1 or 0.75:1 w:c by volume mix. Pumping is continued until the desired pressure is reached and the supply header valve is closed. Grouting is considered to be complete when the maximum pressure is held for 30 minutes. Contraction joint grouting should be performed only after the mass concrete of the monoliths has adequately cured and has cooled to the desired temperature so that the joint space has reached its maximum.

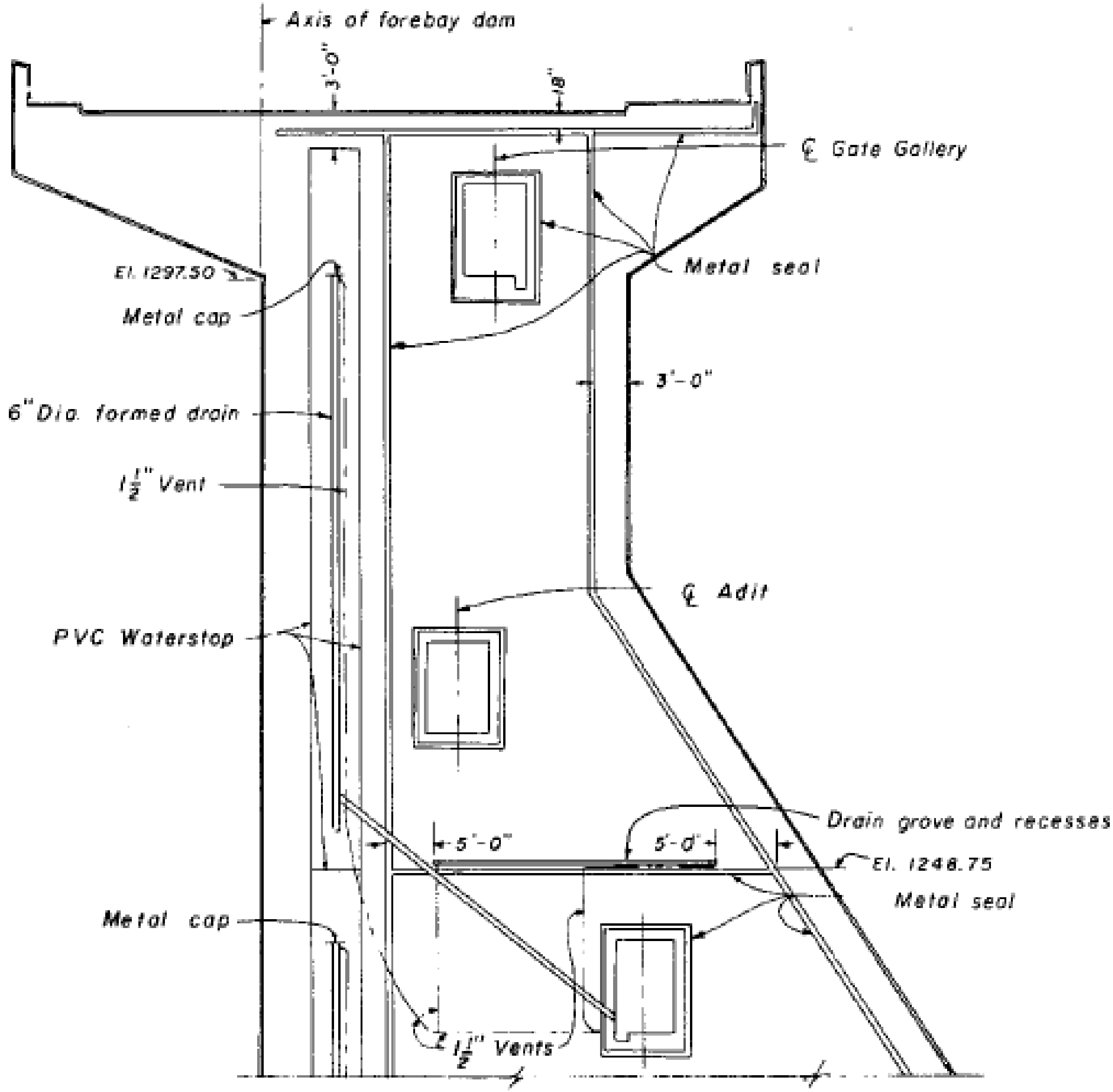


Figure 25-3. Metal seals inserted in the face of the first concrete placement and extended into the adjacent monolith, or second concrete placement to contain grout within area to be grouted.

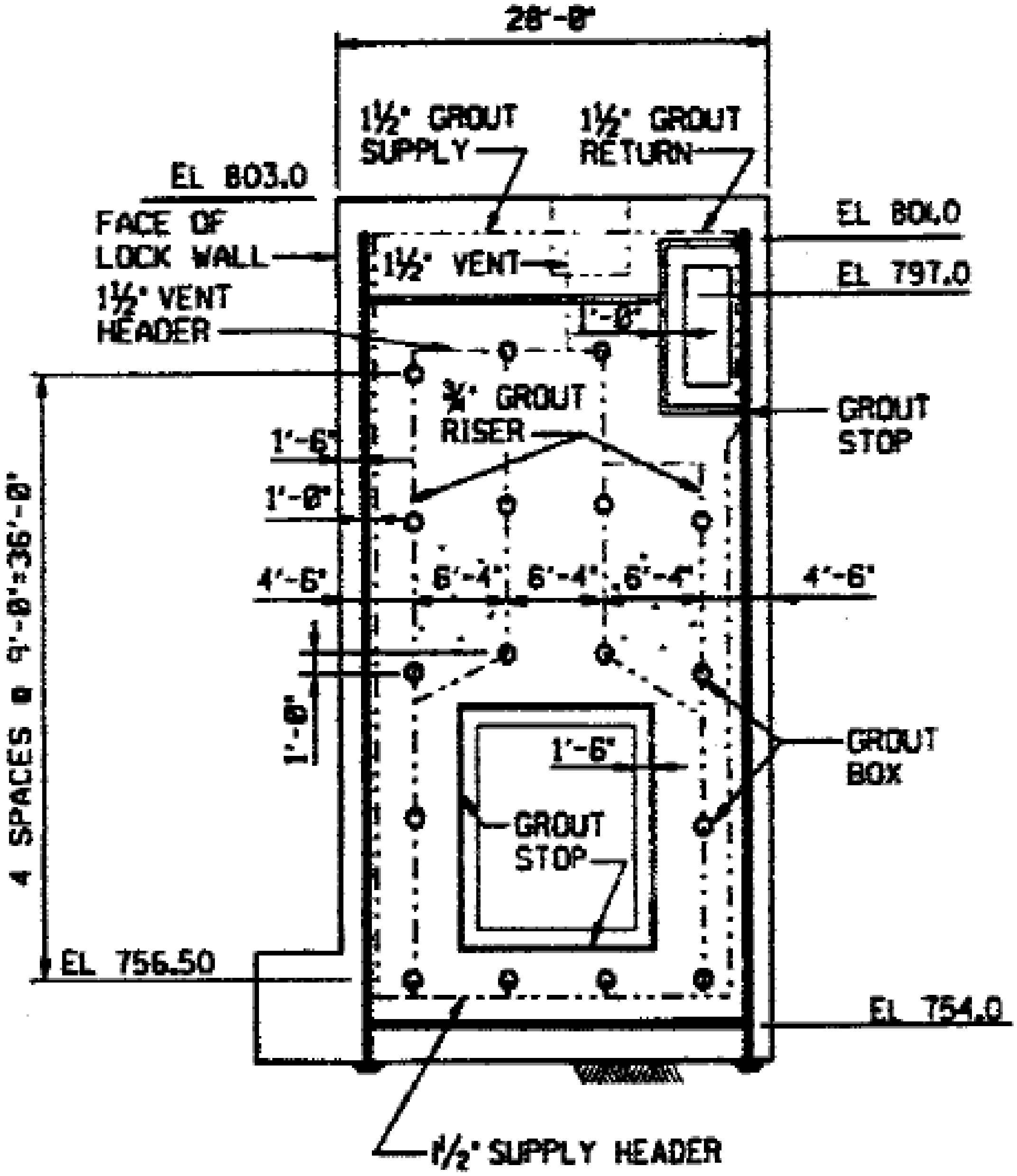


Figure 25-4. Typical grout piping system for a lock wall monolith joint.

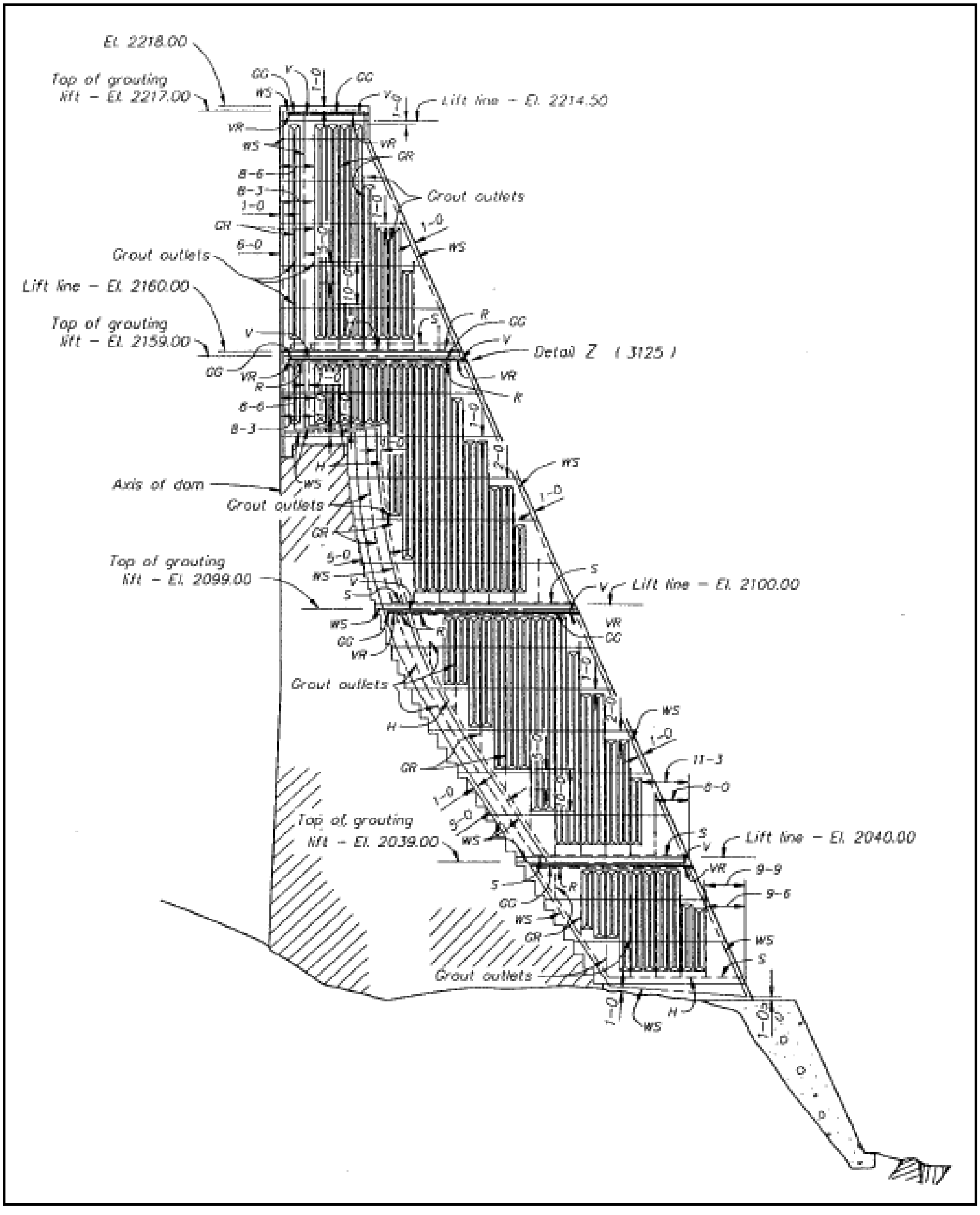


Figure 25-5. Contraction joint piping system at Theodore Roosevelt Dam, AZ.

25-5. Instrumentation for Detecting Structural Movement. When repairing cracks or joints or filling voids in existing structures, it is important to use pressures that are low so as to not cause any movement or uplift of the structure. During grouting operations, the structure should be instrumented and monitored so that any potential movement can be readily detected (Figures 25-2 and 25-6). Should any movement be identified, the grouting operations should be terminated and the procedures re-evaluated to avoid any damage. The type of instrumentation used and the complexity of the monitoring system should be based on site-specific details such as the grouting methods being used, the location of the repair work with respect to the structure, and the type of structure being repaired. Crack displacement and uplift devices can be installed with a minimum of effort and can be monitored in real time as the grouting is performed.

Figure 25-6. Crack displacement devices installed at the spillway wall and parabolic slab at Barker Dam west of Houston, TX (Galveston District). The instruments were monitored in real time during grouting to ensure that there was no movement or displacement of the walls and slab.

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CHAPTER 26

Vibration Control Near Grouting Operations

26-1. Introduction. Vibrations near grouting applications can be a consideration on certain projects. It is not uncommon that vibration-inducing operations such as excavation occur concurrently with grouting or after grouting has been conducted. While little information is available regarding tolerable vibration levels for grout, a significant amount of information, including guidelines, is available for tolerable vibrations in the vicinity of concrete and various types of structures. Included in this chapter is a brief discussion of the types of vibrations induced during construction, typical vibration thresholds used for structures and concrete, and recommended vibration thresholds for grouting projects. While it is recognized that other construction activities might induce potentially damaging vibrations on a local scale, this chapter concentrates on vibrations associated with blasting activities.

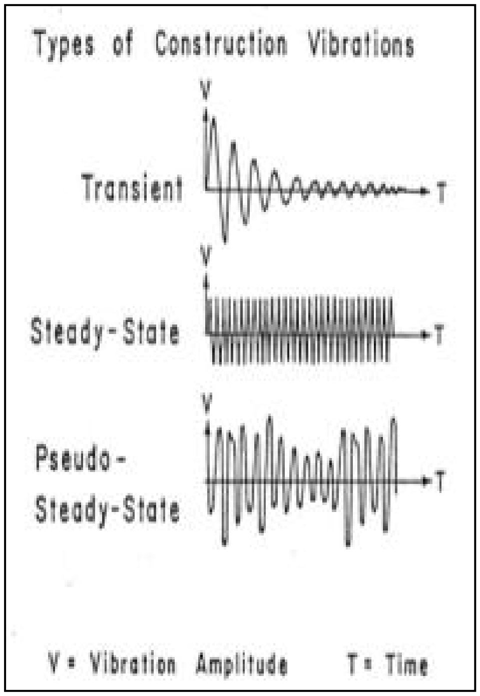
26-2. Other Information Sources. The information contained in this chapter is in no way comprehensive with respect to vibrations. The topic is complex, and USACE has made considerable efforts to publicize guidelines for blasting-induced vibrations. That information is not repeated here. The following USACE publications provide more detail for grouting practitioners who have concerns about blasting-induced vibrations: EM 1110-2-3800, *Systematic Drilling and Blasting for Surface Excavations*; EM 1110-2-2901, *Tunnels and Shafts in Rock*; and EM 385-1-1, *Safety and Health Requirements*. Additional publications of value include the *Blasters' Handbook* and *Four Major Methods of Controlled Blasting* by E.I. du Pont de Nemours and Company, Inc., *Rock Blasting and Overbreak Control* by Calvin J. Konya, and *Rock Blasting* by U. Langerfors and B. Kihlström.

26-3. Types of Vibrations. Vibrations can be divided into three subcategories: Transient, Steady-State, and Pseudo-Steady-State. See Figure 26-1 (Wiss 1981).

a. Transient. Transient vibrations are single events. The vibration decreases rapidly with time. Examples include vibrations induced by impact pile driving and blasting.

b. Steady-State. Steady-state vibrations occur continuously. The magnitude of the vibration is generally constant with time. Examples include vibrations induced by vibratory pile driving equipment and large, stationary machinery.

c. Pseudo-Steady-State. Pseudo-steady-state vibrations are relatively random in nature. The magnitude of the vibration varies with time. Examples include vibrations induced by impact hammers and large excavation equipment.



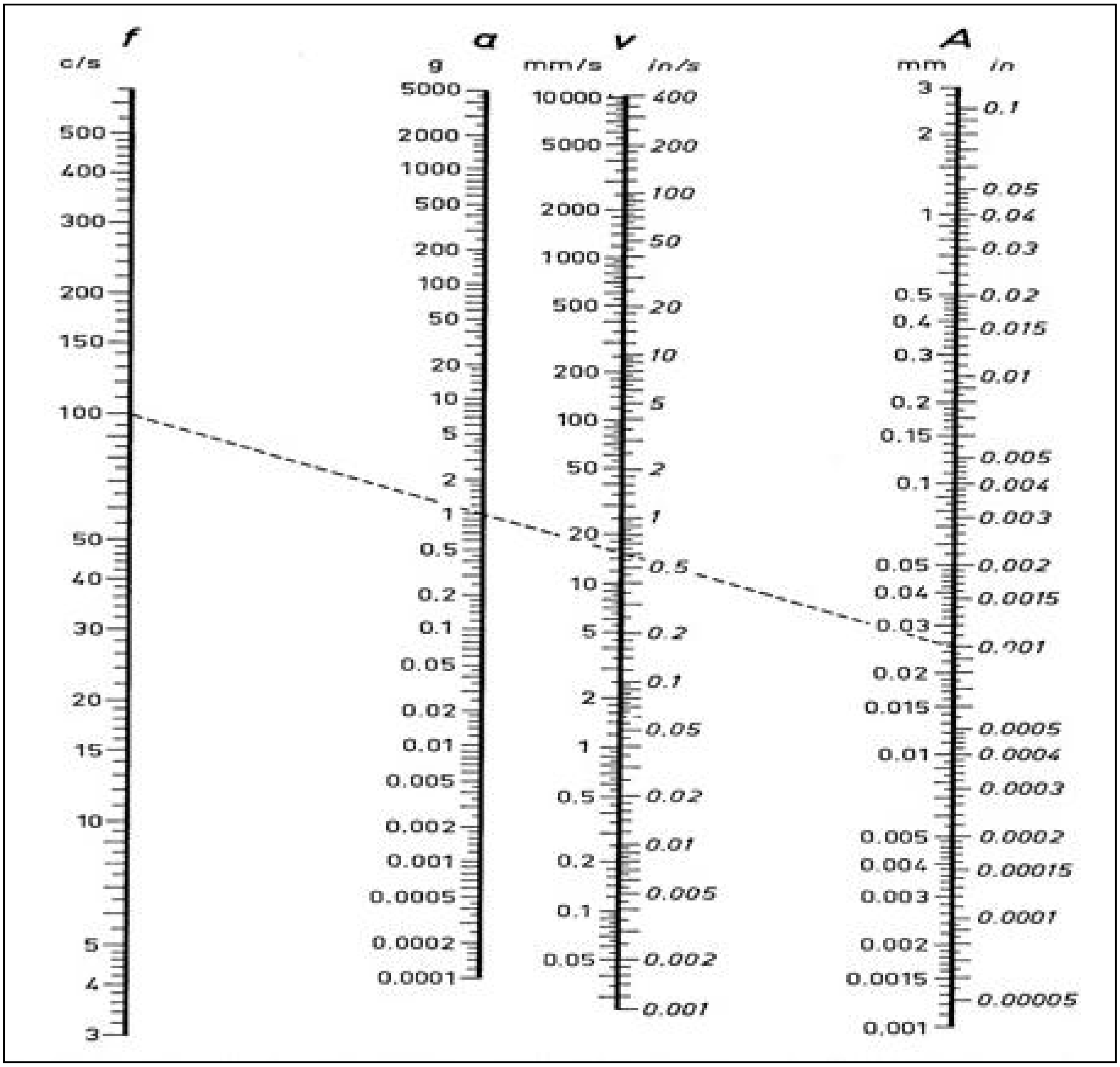
(Wiss 1981, with permission from ASCE)

Figure 26-1. Types of construction vibrations.

26-4. Variables in Vibration Problems.

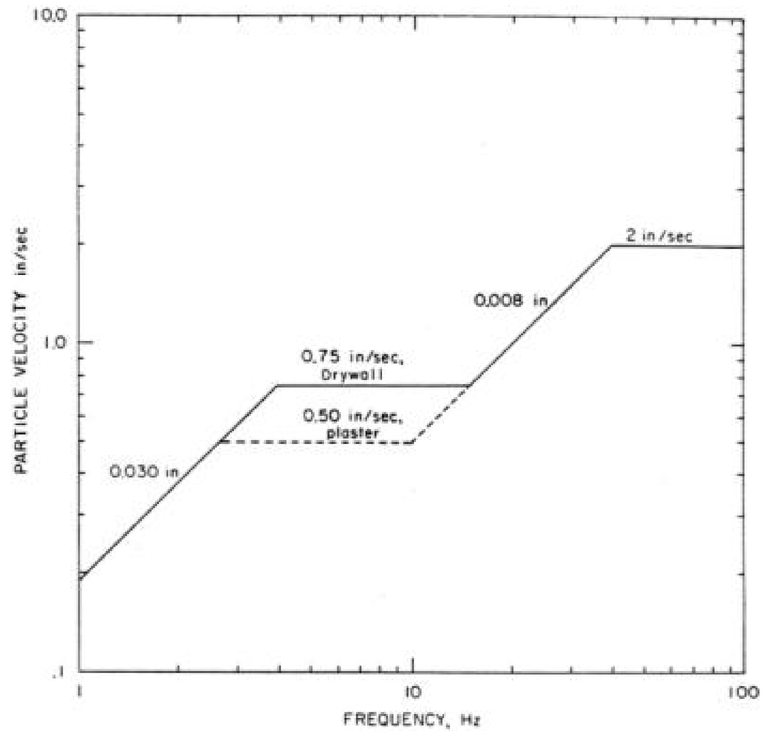
a. Numerous variables in vibrations contribute to the complexity of the problem. The variables of interest in evaluating vibration problems (these are discussed in various texts) include: (1) the amount of energy causing the vibration, (2) the range or distance from the source to the point of interest, (3) the attenuation properties of the ground conditions, (4) the frequencies of the vibration, (5) the natural frequency of the structure at the point of interest, and (6) the corresponding acceleration, velocity, amplitude, and strain induced by the vibration at the point of interest. Many of these variables are derivatives of others, such as velocity, which is the integral of acceleration with respect to time, or strain, which is proportional to velocity. Figure 26-2 shows the relationship between many of these variables.

b. Since many of the vibration variables are related, and for simplification purposes, a maximum velocity is typically specified as a threshold for limiting damage. The velocity generally specified is the peak particle velocity (PPV), which is the vector sum of the velocities measured along three axes (radial, vertical, and tangential). The frequency of the vibration is an additional consideration. As vibration frequencies approach the natural frequency of a structure, resonance can occur, greatly amplifying the strain induced on the structure. Additionally, studies have indicated that lower-frequency vibrations may induce damage at relatively low PPVs, whereas structures may tolerate higher PPVs at higher frequencies. For more information regarding vibration frequencies and the natural frequencies of structures, refer to U.S. Department of the Interior RI 8507, *Structure Response and Damage Produced by Ground Vibration from Surface Blasting*. Figure 26-3, which defines the safe blasting vibration criteria for houses from RI 8507, illustrates these effects.



(From Langerfors and Kihlström 1963, this material is reproduced with permission of John Wiley & Sons, Inc.)

Figure 26-2. Nomogram showing the relationships between frequency (*f*), acceleration (*a*), vibration velocity (*v*), and amplitude (*A*); e.g., if *f* = 100 c/s and *A* = 0.025 mm, then *v* = 15 mm/s and *a* = 1 g.



(From U.S. Department of the Interior RI 8507)

Figure 26-3. Recommended safe levels of blasting vibration for houses using a combination of velocity and displacement.

26-5. Established Thresholds.

a. General. Much information is available with respect to blast-induced vibration thresholds for damage to structures, which is commonly taken as a PPV of 2.0 in./s by many organizations. In many cases, significant research has been published from damage surveys conducted at residential and commercial structures, which includes pre-event and post-event surveys in conjunction with vibration measurements. There is also significant available information regarding tolerable vibration levels for concrete. However, human perceptions of ground vibration, whether damaging to structures or not, have influenced some agencies to set vibration thresholds significantly below 2.0 in./s to minimize both the probability of structure damage and the probability of objections from nearby residents. In some cases, agencies consider not only the PPV, but also the frequency of the vibration and the condition of the structure.

b. Buildings.

(1) EM 1110-2-2901 notes that “It is generally recognized that a PPV of 50 mm/s (2 in./s) will not damage residential structures or other buildings and facilities.” The U.S. Department of the Interior Office of Surface Mining (OSM) publication 30 CFR Sec. 715.19 limits the maximum PPV based on the distance from a blast to a structure as follows: 0–300 ft, PPV is 1.25 in./s; 301–5,000 ft, PPV is 1.00 in./s; 5,001 ft and beyond, PPV is 0.75 in./s. USACE recognizes the various criteria for PPV ranging from 0.5 to 2.0 in./s from various organizations and defers to EM 1110-2-3800 for a safe PPV of 2.0 in./s.

(2) Still other organizations classify tolerable PPVs based on the frequency of the vibration (as noted in Figure 26-3), the vibration source, and the classification of the structure. Table 26-1 lists allowable PPVs based on the type of building; the vibration source, where machines and traffic would be considered steady-state and pseudo-steady-state vibrations, respectively; and particular frequency ranges. Table 26-2 lists allowable PPVs based on the condition and type of building. Note that the allowable PPV varies from 0.08 to 4.0 in./s in these tables.

Table 26-1. Swiss standard for vibrations in buildings.

Building class[*]	Vibration source	Range of frequency (Hz)	PPV (mm/s)	PPV (in./s)
I[†]	Machines, traffic	10–30	12	0.5
		30–60	12–18	0.5–0.7
	Blasting	10–60	30	1.2
		60–90	30–40	1.2–1.6
II[‡]	Machines, traffic	10–30	8	0.3
		30–60	8–12	0.3–0.5
	Blasting	10–60	18	0.7
		60–90	18–25	0.7–1.0
III[§]	Machines, traffic	10–30	5	0.2
		30–60	5–8	0.2–0.3
	Blasting	10–60	12	0.5
		60–90	12–18	0.5–0.7
IV[‖]	Machines, traffic	10–30	3	0.12
		30–60	3–5	0.12–0.2
	Blasting	10–60	8	0.3
		60–90	8–12	0.3–0.5

* After Swiss Association of Standardization, Effects of Vibration on Construction.)

† Buildings in steel or reinforced concrete, like factories, retaining walls, bridges, steel towers, open channels; underground chambers and tunnels with and without concrete alignment.

‡ Buildings with foundation walls and floors in concrete, walls in concrete or masonry; stone masonry retaining walls; underground chambers and tunnels with masonry alignments; conduits in loose material.

§ Buildings as mentioned previously, but with wooden ceilings and walls in masonry.

‖ Construction very sensitive to vibrations, objects of historic interest.

Table 26-2. Summary of commonly used criteria for vibration-induced building damage.

Category*	Source	Particle velocity mm/s (in./s)
Industrial buildings	Wiss (1981)	100 (4)
Buildings of substantial construction	Chae	100 (4)
Residential	Nichols et al., Wiss (1981)	50 (2)
Residential, new construction	Chae	50 (2)
Residential, poor condition	Chae	25 (1)
Residential, very poor condition	Chae	12.5 (0.5)
Buildings visibly damaged	DIN 4150	4 (0.16)
Historic buildings	Swiss standard	3 (0.12)
Historic and ancient buildings	DIN 4150	2 (0.08)
*After Amick and Gendreau 2000.		

c. Concrete. EM 1110-2-2901 provides some guidance on allowable PPVs for concrete of various ages. The following excerpt is from this manual: “Both structural concrete and mass concrete are relatively insensitive to damage when cured. Concrete over 10 days old can withstand particle velocities up to 10.0 in/s or more. Very fresh concrete that has not set can withstand 2.0 in/s or more. On the other hand, young concrete that has set is subject to damage. The PPV in this case may have to be controlled to under 0.25 in/s, and particle velocities should not exceed 2.0 in/s until the concrete is at least 3 days old.” Another source of information on blasting near fresh concrete is *Rock Blasting and Overbreak Control* by Calvin J. Konya.

(1) The TVA criteria are less conservative than those recommended in EM 1110-2-3800, allowing for PPVs as high as 9.0 in./s for concrete less than 3 days old and 4.0 in./s for fresh concrete. The TVA criteria were developed through field testing and are supported by other authors. Note that the concrete applicable to the criteria include mass concrete free from planes of weakness (i.e., joints and penetrations) on rock foundations and having a base-to-height ratio greater than two. This applies to most mass concrete pours, such as dental concrete, footings, and slabs. Oriard and Coulson, authors of *TVA Blast Vibration Criteria for Mass Concrete*, noted “Fill or mass concrete appears to act like a continuation of the rock mass. It is usually of better quality and resists blast damage better than the adjacent rock, even if it has a lower compressive strength. Most rock masses contain discontinuities such as joints, bedding planes, foliation planes, microfractures, etc., while properly placed mass concrete is relatively free of such well-defined weak planes. A rock mass often has tensile and shear strengths lower than that of the adjacent concrete because of these discontinuities.”

(2) The equation from Chapter 7-3a(11) of EM 1110-2-3800 relates the tensile stress of concrete to the maximum PPV that will induce cracking:

$$v = \sigma/\rho c$$

where:

v	=	particle velocity for failure, fps.
σ	=	failure tensile strength, psi.
ρ	=	mass density, lb sec2/ft4.
c	=	propagation velocity, fps.

can also be defined as:

$$\sigma_t = \rho \times c \times u$$

where:

σ_t	=	allowable tensile strength (taken as 10% of the unconfined compressive strength), lb/ft2.
ρ	=	mass density = 150 lb/ft3.
c	=	seismic velocity of concrete, taken as 10,000 ft/s.
u	=	maximum particle velocity, ft/s.

26-6. Grouting Considerations.

a. General. Very limited information exists with regard to tolerable vibration limits for grout or grouted structures. However, given the ever-increasing prevalence of grouting in construction and new methods using grouting materials (jet grouting, soil mixing, etc.), it is an important consideration. Additional research and case studies on the subject will greatly aid in the determination of safe vibration levels that allow for economical excavation by blasting adjacent to grouted structures and foundations. It might be found that neither the vibration criteria for buildings and structures nor the criteria used by various organizations for concrete are directly applicable to grouting.

b. Grouting Application Considerations. The allowable extent of vibration depends on the purpose of the grouting application (e.g., seepage control, structural support, reduction of discontinuity volume), the ability or lack thereof to physically inspect areas where damage may occur, and the costs of remediation if damage were to occur. In applications such as dam foundation grouting, the grout is placed with the intent of eliminating all discontinuities. Cracking and displacement of grout placed into discontinuities are obviously undesirable in seepage control applications. If extensive vibration damage were to occur, the level of effort required to fully investigate and remediate the damage may equal the original cost of the curtain itself. In contrast, drilling and blasting techniques are common in the tunneling industry, and in many cases, grouting in advance of blasting is conducted at the face. Given the proximity of the grouted zone to the blasting, it is obvious that large vibrations are induced in locations where grout has been recently injected. However, in this application, the consequences of vibration damage may be minimal in that one of the primary functions of the grout is to fill spaces between rock blocks to prevent movement, and cracking of the grout would not impair that function.

Further, if the tunnel grouting also has a flow control function, any leakage into the tunnel as a result of blast damage is directly observable and can be remedied at minimal cost and effort. Potential problems in tunnel applications can also be reduced by using high-strength grouts and accelerators and by grouting multiple excavation rounds forward of the face.

c. Grout Strength Considerations. Assuming that tolerable vibrations for grout are proportional to those for concrete with respect to unconfined compressive strength, it is apparent that the strength of the grout may be an important consideration when specifying maximum PPVs. When grouting techniques induce mixing of grout with native materials, such as in soil mixing and jet grouting, the strength of the resulting mixed mass is likely to be more important than the strength of the grout itself. Table 26-3 shows a range of unconfined compressive strengths from balanced stable grouts for a recently constructed grout curtain. Table 26-4 lists the calculated range of u values (maximum particle velocities) obtained by using the equations given in Section(2) 26-5.c(2). As can be seen, a wide range of PPVs are calculated based on the strength of the grout, which changes with time, and the mix type, since thicker grouts have lower WCRs and therefore higher strengths. Even at 28-day strengths, the thinnest grouts may crack when subjected to PPVs in excess of 2 in./s using the above criteria. Note that this calculation was intended for use in evaluating the potential for cracking in massive concrete structures, and therefore it may not be entirely applicable to grout. Also note that the data in Table 26-4 do not include a safety factor, so the calculated values would be threshold values that, when exceeded, would lead to a likelihood of cracking.

d. Previous Study. On one USACE project, the proximity of blasting and the induced vibrations on the grout curtain were evaluated; excavation operations for the core at the Tioga Dam left abutment on the Tioga-Hammond Lakes Project (Baltimore District) were not completed until after grouting of the foundation. Mechanical excavation and pre-split blasting were used to remove rock ledges and weathered bedrock and to achieve the finished foundation grade in the core after completion of the grouting operations. This prompted concern that the vibrations induced by these operations, particularly blasting, had damaged the curtain. A verification program was initiated that included redrilling and regrouting 11 previously drilled and grouted holes. Because rapid refusal was encountered upon regrouting of the holes, the conclusion was that the curtain was intact. The magnitude of vibrations induced by the blasting are not noted in the Review of Abutment Treatment report.

Table 26-3. Range of unconfined compressive strengths for various balanced stable grouts from a recent USACE project.

	Compressive Strength (psi)							
Mix type	Mix A	Mix B	Mix C	Mix D	Mix E	Mix F	Mix G+	Mix G
3-day	160	195	240	375	475	750	810	915
7-day	190	265	430	595	725	1005	1100	1175
14-day	365	380	510	820	1060	1285	1410	1455
28-day	420	535	735	1070	1490	2560	2395	2755

Table 26-4. Range of calculated maximum particle velocities for grout using the above equation derived from EM 1110-2-3800.

Mix type	Calculated Max. Particle Velocity (PPV) in in./s from EM 1110-2-3800							
	Mix A	Mix B	Mix C	Mix D	Mix E	Mix F	Mix G+	Mix G
3-day	0.9	1.1	1.3	2.1	2.6	4.2	4.5	5.1
7-day	1.1	1.5	2.4	3.3	4.0	5.6	6.1	6.5
14-day	2.0	2.1	2.8	4.6	5.9	7.1	7.8	8.1
28-day	2.3	3.0	4.1	6.0	8.3	14.2	13.3	15.3

26-7. Recommendations Specific to Vibrations and Grouting.

a. General. While little information exists regarding vibration-induced damage to grouting applications, it is possible to observe certain criteria that will minimize, if not eliminate, the potential for damage to a grouted structure.

b. Unset Grout. Whenever possible, it is preferable that no blasting be allowed until at least 3 days after any injected grout has taken initial set. The reason for this recommendation is twofold. First, blast waves may induce high pore-water pressures in groundwater. These pore pressures may result in movement and shifting of the unset grout, causing windows and defects in the curtain. Second, it minimizes the potential damage to the grout, since the grout will have obtained some nominal minimum strength before the blast.

c. Peak Particle Velocities. In the absence of other information, a maximum allowable PPV of 0.5 in./s as measured at the point of the grouting application is recommended. This recommendation is based on:

(1) RI 8057, of the U.S. Department of the Interior Bureau of Mines, which recommends this as a maximum safe limit for plaster, which is not unlike a weak grout.

(2) This value corresponds well with the conservative values noted in the Swiss Standards in Table 26-1.

(3) For the grout strength tests, Table 26-3, and the calculation results presented in Table 26-4, the findings represent a safety factor of approximately two against development of cracks in weaker grouts.

d. Site-Specific Data. If blasting is of critical importance with respect to a particular grouting project, and if the two activities cannot be sequenced such that grouting is conducted after blasting, the project may warrant a detailed on-site investigation and testing program to develop site-specific vibration criteria higher than those noted above. If such an investigation is conducted, it is recommended that vibrations be monitored both at the ground surface and below the ground surface to the full depth of grouting. This can be accomplished by installing monitoring devices into several boreholes within the curtain alignment. Several test boreholes should be drilled and grouted at locations and in conditions similar to the production curtain. Pressure testing of the redrilled boreholes with accurate flowmeters and gauges should be conducted after subsequently more intensive blasts. The threshold for damage would be evidenced by the PPV above, in which increased borehole permeability was noted. It is recommended that a safety factor be applied to any site-specific threshold.

CHAPTER 27

Compaction Grouting

27-1. Introduction.

a. General. Compaction grouting is the process of injecting very stiff mortar-like grout at high pressure and in a controlled manner for the purpose of densifying the soil around the injected mass. Compaction grouting has been in use for nearly 50 years and is currently widely used throughout North America and other countries. The *Practical Handbook of Grouting Soil, Rock, and Structures* (Warner 2004), one of the most comprehensive references on this topic, provides a history of the technology and practical guidelines for the use of compaction grouting. Numerous case histories are also presented in this reference. This chapter presents some of the critical aspects to be considered in compaction grouting design and construction. Warner (2004) is a recommended resource for supplemental technical information and guidance.

b. Function. When proper materials, equipment, and procedures are used, the injected grout mass displaces and compacts the surrounding soil, as shown in Figure 27-1. Functionally, compaction grouting is essentially a static compaction process. When properly executed, compaction grouting results in the formation of a columnar mass. Figure 27-2 shows a properly injected compaction grout mass that was extracted for verification.

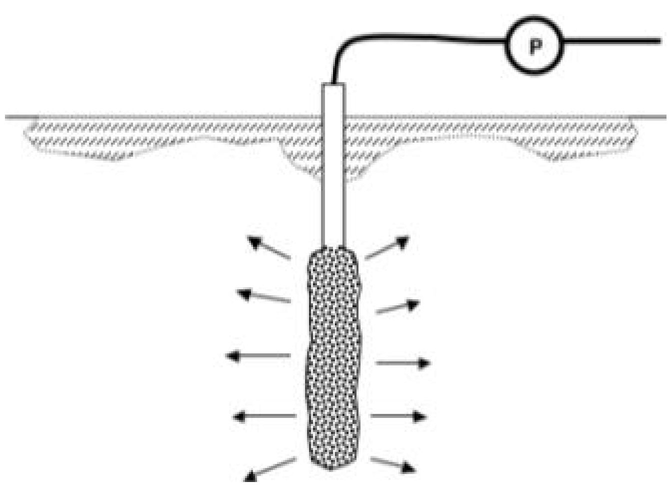


Figure 27-1. Injected grout mass displacing and compacting surrounding soil.

(Courtesy of Gannett Fleming)

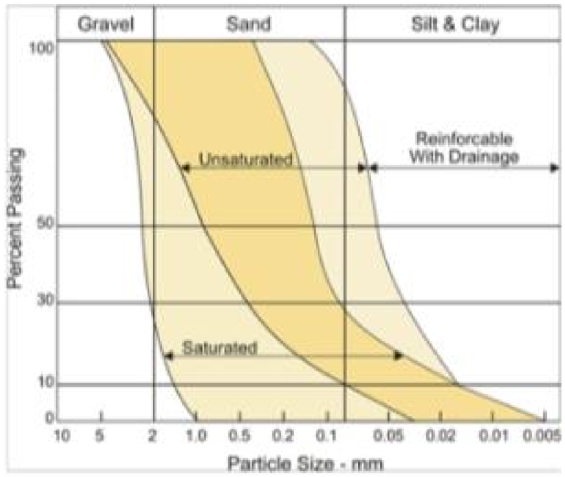
Figure 27-2. Extracted columnar-shaped compaction grout mass.

27-2. Applications. The primary benefit of compaction grouting is densification of loose, in-situ soils or man-made fills. Therefore, it can be considered for geotechnical applications requiring increased bearing capacity and/or improvement of settlement characteristics for both preconstruction site improvement and mitigation of problems at existing facilities. Compaction grouting has also been applied as a ground improvement technique to mitigate liquefaction potential and control surface settlement during construction of soft ground tunnels. Compaction grouting is widely used to improve soils adjacent to or underlying existing structures because it can be accomplished in areas of limited access with small drilling rigs and remotely located mixers and pumps. It is therefore often performed in close proximity or even inside existing structures requiring foundation treatment. This technique is used for improving subsurface conditions underlying both shallow and deep foundations to improve bearing capacity, mitigate settlements, or reduce liquefaction potential. In some cases, it has been used to increase the lateral resistance of soils adjacent to deep foundations.

27-3. Design Considerations.

a. Subsurface Conditions. As with all geotechnical ground improvement techniques, a thorough understanding of the subsurface conditions is required. The in-situ vertical stress must be sufficient for densification to occur without uncontrolled heave at the ground surface. Insufficient confining pressure will limit horizontal displacement and therefore limit densification. Compaction grouting is most effective in free-draining granular soils. In saturated soils, pore pressures increase during grout injection as a result of ground displacement, which in turn limits densification. In these conditions, the applied injection rate must be slow enough to allow excess pore pressure to dissipate and densification to occur. Saturated fine-grained soils and sensitive clays are also prone to the loss of shear strength due to remolding. Compaction grouting is therefore not typically performed in saturated and near-saturated low-permeability soils because the slow pumping rate and/or time required for pore pressure dissipation is not economical. Figure 27-3 shows a typical range of soils considered suitable for compaction grouting. Large rocks, boulders, or other obstructions that inhibit the installation of casings for compaction grouting are also problematic.

b. Treatment Zone. The depth and lateral limits of compaction grouting treatment should be established based on appropriate stress distribution theory under the loaded area for which improvement is desired. Consideration must also be given to the post-treatment condition, because the compaction grout material may substantially increase the weight of the treated zone and the density of the surrounding soils. The strata below the proposed zone of treatment should be analyzed to confirm that they can support the increased load of the treated zone. Improvement to greater depths may be required.



(Courtesy of Hayward Baker)

Figure 27-3. Typical range of soils suitable for compaction grouting.

c. Hole Angles. Holes should generally be vertical, as inclined holes create greater heave forces at lower pressures and therefore limit grout quantities. However, due to access constraints, holes are often inclined to improve soils underlying existing facilities. If inclined holes are necessary, the angles should not exceed 20 degrees from vertical to limit the surface area over which uplift is applied at the surface.

d. Hole Spacing. Compaction grout hole spacing is typically on a grid pattern of 6–12 ft. According to Warner (2004), "Although grout hole spacing is soil specific, 10–12 ft has proven to be an optimal spacing when the pumping rate is within a range of 1 to 2 ft^3 per minute." Primary holes are installed initially, with split-spaced secondary holes installed to measure the degree of improvement and provide final tightening.

e. Basis of Design. Property improvement arises from an increase in density. A number of design parameters, such as shear strength, settlement parameters, and liquefaction potential, are functions of soil type and either total density or relative density. Therefore, the basis for design can be consideration of the total volume change required to produce the desired total density (unit weight) or relative density of the improved mass. Clearly, the compaction grout elements themselves also interact with the soils in other beneficial ways due to vertical reinforcement effects. However, in the absence of full-scale field tests, detailed analyses, and extraordinary field control and verification programs, these secondary benefits are normally not considered in the design. When total unit weight is the basis for correlation of design parameters, the required percent displacement volume within a given soil mass can be calculated using the following equation (Byle 2003):

$$V_d = \frac{\gamma_f - \gamma_o}{\gamma_f} \times 100$$

where:

V_d = percent displacement volume required

γ_f = final desired total unit weight

γ_o = initial in-situ soil unit weight.

The volume of compaction grout required for the mass to be treated can then be estimated based on the percent volume required times the volume of material to be treated.

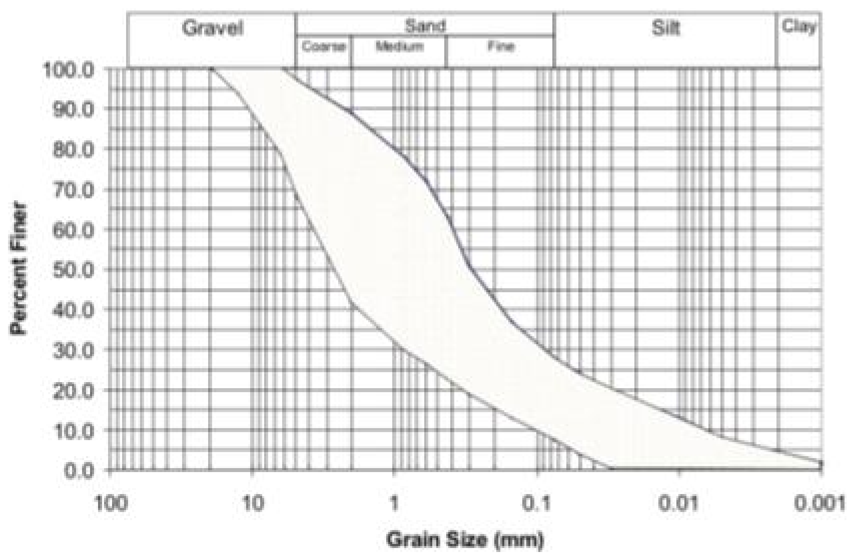
f. Sequence and Staging. Compaction grouting should begin at the perimeter of area to be treated and should work progressively toward the interior. In this manner, the outer treated area provides the benefit of confinement and hence enhances densification. Grouting should be performed on primary holes first, followed by a secondary grid pattern, and tertiary if needed. Compaction grout injection is done in incremental stages typically 1–2 ft in length and may be performed working from the bottom of the treatment zone upward (upstage) or from the top down (downstage). Upstage grouting is the most economical; however, downstage grouting may be required at shallow depths. At shallow depths (e.g., less than 15 ft), there will likely be insufficient vertical stress to provide confinement and avoid heave at the ground surface. Using the downstage method, compaction grout injection is first accomplished at shallow depths. Subsequent stages are then injected below, after drilling through the hardened upper stages. In this manner, the completed upper stages provide additional vertical stress and confinement for lower stages, thereby allowing higher pressures and greater densification. Warner (2004) gives further practical details on the procedures of upstage and downstage compaction grouting and their applications.

g. Pumping Rate. A pumping rate of about 1–2 ft^3 per minute is normally considered to be optimum. Greater amounts of compaction grout can be injected at lower pumping rates, thereby increasing the densification. High pumping rates should be avoided, as pore pressure builds up and requires tighter hole spacings to achieve the same result as that achieved by lower pumping rates and wider hole spacings.

27-4. Materials. Compaction grout typically consists of aggregate containing non-plastic fines, cement, and sufficient water to provide a stiff, but pumpable mix. Figure 27-4 shows extruded compaction grout material. Figure 27-5 shows a preferred aggregate gradation band. As the chart shown in Figure 27-5 indicates, considerable fines are required for pumpability. However, the inclusion of clay-sized particles should be avoided. Experience and demonstrations have shown that the inclusion of clay-sized particles or other additives may improve pumpability, but they can cause the compaction grout material to behave as a fluid, thereby increasing the potential for hydrofracturing of the soil and loss of control of the grout mass. Coarse-grained particles should be rounded, as angular particles will negatively impact pumpability. Cement content typically ranges between 6 and 12%. Since the purpose of the compaction grout is to densify adjacent soils, grout strength is typically not an important mix parameter.

(Courtesy of Gannett Fleming)

Figure 27-4. Stiff mortar-like compaction grout material being extruded through casing.



(After Warner 2004)

Figure 27-5. Preferred aggregate gradation band for compaction grout.

27-5. Equipment.

a. General. Equipment for drilling, mixing, and pumping compaction grout material are discussed in Chapter 11 of this manual. The following paragraphs discuss some important special aspects relative to compaction grouting.

b. Casing. Flush joint casing with a diameter of 2 in. is commonly used. Casing lengths of less than 5 ft are common and facilitate handling during withdrawal. With standard stage lengths of 2 ft, casings longer than 5 ft result in unmanageable stick-up during upstage grouting. The casing and joints (threaded or welded) must be strong enough to withstand the substantial forces of installation, grouting pressure, and withdrawal. Installation may be by drilling or driving. However, a tight fit in the hole is required to develop enough friction to resist upward movement during grout injection. Withdrawal is typically accomplished with extraction jacks (Figure 27-6).

(Courtesy of Gannett Fleming)

Figure 27-6. Casing withdrawal using extraction jacks.

c. Pumps. Pumps for compaction grouting should be positive displacement piston types capable of pumping at various rates on the order of 0.5–3 ft^3 per minute at continuous pressures of up to approximately 1,000 psi.

d. Mixers. It is preferable to mix compaction grout material on site, as commercial batch plants do not normally maintain stockpiles of aggregate meeting the gradation band depicted in Figure 27-5. Mixing on site may be accomplished with any commonly available equipment that will uniformly blend the materials.

e. Delivery Lines, Valves and Fittings. Delivery lines should consist of high-pressure hose or rigid pipe. Gauge savers should be supplied, and wide sweep bends should be used instead of standard pipe fittings to facilitate flow of the stiff mix. Figure 27-7 shows the configuration of the delivery lines at the connection to a compaction grout casing.

27-6. Procedures.

a. General. As previously discussed, compaction grouting may be performed upstage or downstage. Upstage grouting is typically more economical and is used for most common applications. Downstage grouting may be required in areas of shallow overburden depth where confinement or additional vertical stress is needed. The following paragraphs discuss the basic procedures for upstage and downstage grouting.

b. Procedure for Upstage Compaction Grouting.

(1) Install the compaction grout casing to the bottom of the treatment zone by drilling or driving. If installed by driving, a sacrificial knock-off point is required.

(2) Raise the casing 1–2 ft and inject grout to the predetermined stage refusal criteria.

(Courtesy of Gannett Fleming)

Figure 27-7. Large-bend radius at a connection to a compaction grout casing.

(3) Continue incremental casing withdrawal and compaction grout injection to the top of the treatment zone.

c. Procedure for Downstage Compaction Grouting.

(1) Drill an oversized hole to the top of the treatment zone and install the compaction grout casing in the oversized hole.

(2) Grout the casing in place by filling the annular space with neat cement grout.

(3) Drill through the casing, advancing the hole for the first stage of compaction grouting.

(4) Inject compaction grout to the predetermined stage refusal criteria.

(5) After the grout has set, drill through the grouted zone, advancing the hole to the next stage. Continue compaction grouting and drilling through the grouted zone until the last stage is completed at the bottom of the treatment zone.

d. Refusal Criteria. Stage refusal criteria for compaction grouting projects must be established based on site-specific conditions and improvement goals. Ideally, refusal criteria will be established based on a review of the results of a test section or during the early stages of production grouting. Typical refusal criteria are:

(1) Heave/Movement. Ground heave or structure movement is reportedly the most common refusal criterion, and grouting is typically stopped at the first observation of movement at the ground surface. Surface heave as a result of compaction grouting can damage the site and existing structures. Furthermore, little additional densification occurs once surface heave is initiated. Surface heave or structure movements of 1/32 in. have been specified as the refusal criterion for compaction grouting.

(2) Injected Volume. In cases where the soil to be improved is relatively uniform, attainment of a specified grout injection volume per stage may be used as a refusal criterion. The specified grout volume may be calculated (or estimated) based on the improvement desired as noted in Section 27.4.

(3) Pressure Criterion. A maximum or sustained pressure criterion can be established, but should be based on data collected from a test program or during the initial stages of compaction grouting. Maximum pressures on the order of 1,000 psi for a period of 3 minutes have been specified. Warner (2004) also reports that most soils are efficiently compacted at sustained pressures on the order of 350–500 psi. Because pressure is directly related to the pumping rate, when a maximum or sustained pressure criterion is used, it is important to ensure that a constant pumping rate is observed for the pressure criterion to be valid.

27-7. Monitoring.

a. Movement. Monitoring of ground surface heave and structure movement is essential to prevent site and/or structure damage. Figure 27-8 shows surface heave and cracking during compaction grout injection. Movement monitoring should extend to a perimeter of about 30 ft around the point of injection and may be accomplished by various means, from simple observations to automated data collection and alarm systems, depending on the sensitivity of the site and structures that may be impacted.

(Courtesy of Gannett Fleming)

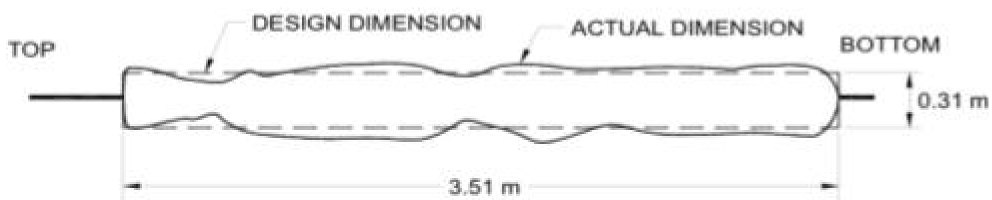
Figure 27-8. Ground surface heave and cracking during compaction grout injection.

b. Measurements. Monitoring and evaluating the grout quantities, injection rate, and pressure during the progress of the work are key components of the success of a compaction grouting project. Ideally, data are collected automatically by instruments with direct input to a computer system. This data acquisition and manipulation system allows for real-time evaluation of the work in progress so that modifications can positively impact both the cost and quality of the work. Measurements should include the quantities injected for the various stages in each hole, the rate of injection, and the injection pressure during each stage. Monitoring and maintaining the grout consistency is of utmost importance to prevent the grout from becoming too fluid.

27-8. Verification.

a. General. The improvement achieved by compaction grouting can be assessed by reviewing the grouting records, including injection rate, pressure, and volume. Secondary grouting locations should show less grout injected at higher pressures due to the higher density achieved by the primary grouting locations. Also, observations and records of drilling or driving compaction grout casings may also be used to assess increases in density.

b. Excavation and Extraction. Excavation and extraction of compaction grout columns at test locations have been used to compare the “design” volume of the compaction grout column to the actual volume achieved. Figure 27-9 shows the actual dimensions of an extracted compaction grout column compared to the design dimensions.



(Courtesy of Gannett Fleming)

Figure 27-9. Actual dimensions of an extracted compaction grout column compared to the design dimensions.

c. Other. Other methods of verifying the improvement achieved by compaction grouting include performing pre- and post-improvement density testing using standard penetration, cone penetrometer, dilatometer, or pressure meter testing.

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CHAPTER 28

Hydrofracture Grouting

28-1. Introduction. Hydrofracture grouting is the process by which grout is injected at pressures sufficiently high to induce intentional tensile failure of the ground. The intended result may be densification of the formation, permeability reduction, foundation strengthening, and/or mitigation or prevention of settlement. This chapter discusses applications and mechanisms of hydrofracture grouting, the means and methods that have been successful on previous projects, and several case histories.

28-2. Complexity of Terms.

a. General. The term "hydrofracture grouting" is often used to describe various methods of grouting both soil and rock. For purposes of this manual, hydrofracture grouting is defined as the application of low- to medium-viscosity grouts at high pressures, typically in fine-grained soils, for the purpose of hydraulically inducing tensile failure and corresponding grout propagation. The same process can be applied to rock, which is a common practice in Europe. The general theory in rock suggests that the application of high pressures opens existing fractures that previously were not accessible to the grout while simultaneously closing some non-injected fractures. High pressures also create new fractures and maximize the potential for fully grouting all discontinuities. While many authors, particularly those in Europe, are proponents of hydrofracture grouting in rock, this practice has generally not been adopted or accepted in the United States. Historically, the U.S. grouting community has viewed hydrofracturing in rock during grouting, which is normally described as "uplift," as an event that must be prevented out of concern that it will damage areas where grouting has previously been completed or create new potential flow paths where none previously existed. A limited recent exception to this perspective that is gaining some acceptance is the concept of allowing or actively promoting hydrofracturing in soil-filled zones within rock or in mixed soil-rock materials, particularly when grouting in karst environments, as discussed in Chapter 18 of this manual.

b. Compensation Grouting. The term "compensation grouting" is often confused with hydrofracture grouting. Compensation grouting is a general term used to describe the procedure of injecting grout materials beneath or adjacent to structures to compensate for settlement. This settlement is often a result of strain caused by material loss or soil deformations associated with excavations or nearby tunneling. In nearly all cases, compensation grouting is performed in soils. Hydrofracture grouting is a method sometimes used to perform compensation grouting. Table 28-1 lists several methods that can be used in compensation grouting practice (Rawlings et al. 2000).

c. Discussion. All of the above methods, with the exception of permeation grouting, are typically applied reactively when movement in a structure is observed. The injections are meant to maintain the structure at its present location. However, these applications may be used after settlement has occurred to re-level or "jack" foundations or structures back to their original, pre-disturbance locations. Permeation grouting is covered in detail elsewhere in this manual. Compaction and intrusion grouting are similar applications with different materials. Intrusion grouting uses thick, HMG, whereas compaction grouting uses mortar-like, low-mobility grout. Chapter 27 of this manual provides additional information on compaction grouting.

Table 28-1. Applications of compensation grouting.

Term	Method	Application in Compensation
Permeation grouting.	Injection of low- to medium-viscosity grouts at low to moderate pressure for the purpose of permeation.	This method is generally limited to conditioning or solidifying ground conditions before application of other compensation methods.
Hydrofracture grouting.	Injection of low- to medium-viscosity grouts at high pressures for the purpose of hydraulically inducing tensile failure and corresponding grout propagation.	Injected materials compensate for materials lost as a result of local excavations.
Intrusion grouting.	Injection of high-viscosity grouts at high pressures for the purpose of displacement, while also inducing limited hydrofracturing in soils.	Injected materials compensate for materials lost as a result of local excavations.
Compaction grouting.	Injection of mortar-like grout at high pressures to displace soils.	Injected materials compensate for materials lost as a result of local excavations.

28-3. Principle Theories and Terms.

a. General. Proper application of hydrofracture grouting requires an understanding of the principles related to fracture development and propagation in the subsurface. While not understood in great detail due to the difficulties associated with inspecting the completed work and inherent heterogeneities in ground conditions at different sites, the principles outlined in the following paragraphs are generally agreed upon by the grouting community.

b. Fracture Propagation. The propagation of the fractures depends on the state of stress within the ground. In an underconsolidated or normally consolidated soil, the vertical effective stress is greater than the horizontal effective stress. In theory, when hydrofracture grouting these types of materials, the initial fracture that propagates is vertical. In subsequent re-injections at the same location, the stress state of the soil is progressively altered, and the angle of the fractures moves from vertical toward horizontal. The process continues until the fracture is horizontal and available deformation within the soil mass has been maximized. At this point, heave or lifting of the mass may begin. In reality, ground conditions are often not homogeneous to the extent that initial vertical propagation is guaranteed. The presence of localized zones of differing stress states and incipient cracks, fissures, or other planes of weakness will all alter the orientation of the fracture planes.

c. Rawlings et al. (2000) note the following regarding fracture propagation. “The fluid pressure causes tensile failure within the ground and a new fracture is initiated or an existing closed discontinuity opened. Grout fills the discontinuity and the fissure propagates in the direction of minimum resistance. In theory, this is the direction of major principal stress, but in practice it is usually controlled by anisotropic factors in the ground (e.g., bedding in soils, fracture orientation in rocks).” The authors further note that in overconsolidated soil conditions, the initial fracture propagates in a nearly horizontal direction. Bernander (2004) notes that grout propagation typically follows the “interfaces between stiff and softer structures,” such as along the soil/rock interface or the interface between soft and stiff strata. He also notes that large, hard obstructions in the ground, such as boulders or foundations, are commonly encapsulated by a layer of grout. For additional information of fracture propagation, see *Grouting in Sedimentary and Igneous Rocks with Special Reference to Pressure Induced Deformations* by Bernander (2004).

d. Pressure.

(1) For hydrofracture grouting to be effective, tensile failure of the soil is necessary. In contrast to permeation grouting, where pressures are intentionally kept low to prevent hydrofracturing, higher pressures are necessary to intentionally induce hydrofracturing. In general terms, the stronger the medium and the deeper the point of injection (i.e., high overburden pressure), the more pressure will be required to induce hydrofracturing. The pressure required to induce hydrofracturing is often referred to as the “frac” or “claquage” pressure. Claquage roughly translates in English as an artificial fracture induced by hydraulic pressure. Hence, the terms “frac grouting” and “claquage grouting” are normally interchangeable with hydrofracture grouting.

(2) Sometimes the frac pressure also depends on the method by which the grouting operation is performed and may exceed the pressure required to cause tensile failure of the ground. In many circumstances, sleeve-port pipes or tube-á-manchette pipes are used. These casings are installed in the ground, and the annulus between the casing and the borehole sidewall is backfilled with grout. The pressure necessary to open the sleeves and break through the casing grout may exceed the pressure necessary to induce tensile failure of the soil. In these cases, a short-duration application of very high pressure is necessary to open the sleeves and break through the grout, followed by a reduced-pressure application to propagate the hydrofracture.

e. Fracture Width. The fracture width generally depends on the viscosity of the injected grout. Thin grouts typically result in a network of fine, grout-filled fractures. Thicker grouts typically result in thicker grout lenses, but at less frequent intervals. The fracture width also depends on the ground conditions and may vary as multiple injections are conducted in the same zone.

f. Penetration. The penetration distance of a hydrofracture-induced grout lens depends on the ground conditions and the applied pressure. Bernander (2004) divides the ground conditions into confined and unconfined conditions. Confined conditions exist at depth in both soil and rock. Bernander states that penetration in confined conditions depends on the local deformations in the mass as a result of the grout pressure applied. Unconfined conditions exist at shallow depth and extend into rock to a depth approximately equal to the depth of soil cover above the top of rock (Bernander 2004). Provided that the applied grout pressure is appreciably greater than the overburden pressure, there is no practical limit to the penetration distance in unconfined conditions. For the purposes of this manual, the grouting of soil is assumed to be an unconfined condition.

28-4. Ground Improvement Mechanisms and Applications.

a. General. Because hydrofracture grouting improves ground conditions, it lends itself to many applications. In addition to the uses in compensation grouting noted above, the following paragraphs discuss other beneficial properties imparted by the method.

b. Densification. Similar to compaction grouting, the application of hydrofracture grouting can densify subsurface strata. The principle is similar to compaction grouting in that hydrofracture grouting induces high local stresses near the point of grout penetration. Both methods rely on plastic deformation of the soil to achieve the densification. The difference between the two methods is that hydrofracture grouting induces multiple fractures in varying directions with small incremental plastic deformations along each fracture, while compaction grouting induces larger plastic deformations locally around each injection point. In both techniques, the maximum possible degree of densification is controlled by surface heave, which occurs when the existing overburden pressure is exceeded by stresses created in the soil during the application. The maximum densification possible varies in the two methods because of the differing stress conditions created in the soil mass due to the differences in the techniques. Densification is only effective in soils that are free draining in response to the grout pressure applied.

c. Soil Strengthening. The premise of hydrofracture grouting is to inject grout under pressure, causing tensile failure of the soil or opening of pre-existing fractures, resulting in the injection of grout lenses within the soil. These pressurized lenses of grout densify the soil by plastic deformation in the vicinity of the lens and also reinforce the soil mass due to the higher strength of the grout in comparison to the soil (soil strength is commonly measured in pounds per square foot; grout strength is measured in pounds per square inch, an increase of two orders of magnitude). The result of the multiple injection process is a network of grout lenses in all directions, each of which has caused densification; the network leaves in place a skeleton of stronger material, which strengthens the entire soil mass. Quantifying this benefit can, however, be problematic due to uncertainty about what has occurred beneath the ground and due to strain compatibility issues for the grout versus the soil.

d. Chemical Changes. In addition to strengthening as a result of a network of grout lenses, hydrofracture grout has also been reported to result in chemical changes to clay soils. Zuomei and Pinshou (2004) report that hydrated lime, which is liberated as a result of the reaction of cement with water, reacts with carbon dioxide in the clay. This reaction forms calcium carbonate, which enters the clay structure under osmotic action, increasing the resistance to pipe and slaking.

e. Permeability Reduction. One clear advantage of hydrofracture grouting is its ability to interconnect with void areas or areas of sufficiently high permeability that the grout can permeate the open strata. While hydrofracture grouting is rarely used in remediating dam embankments because of the potential for damage to the embankment, one notable dam project that did use this method is the remediation of Mud Mountain Dam discussed later in this chapter. On this project, loss of materials due to soil arching and piping resulted in both a local low-density condition and possibly a hydraulic “pipe” within the embankment of high permeability. In addition to permeating open materials, the network of grout lenses densified the embankment and provided additional resistance to further hydrofracturing from slurry wall construction.

While there is no design methodology currently available for this type of application, the benefits of a network of high-strength, low-permeability material are conceptually easy to envision.

28-5. Field Methods.

a. General. The following paragraphs discuss various field procedures and equipment typical of hydrofracture grouting projects. These methods are not the only suitable means for performing hydrofracture grouting, but they have proven suitable on past projects.

b. Methods. Two methods of injection are common in the industry. The use of sleeve-port pipe systems is the most common method and is discussed in more detail in the following paragraphs. Grouting through the bottom of a casing, often referred to as “end-of-casing” grouting, is another method used to perform hydrofracture grouting. This method is similar to compaction grouting in that the grout materials are injected through a casing that is drilled, jacked, or driven to depth. As with compaction grouting, the casing is raised to inject grout at different zones. This method has several inherent disadvantages. First, after injection at the deepest stage, the grout must be allowed to set before injection at shallower stages. This is necessary to ensure that grout is penetrating the desired zone and is not continuing to propagate from the deeper, previously grouted stage. Second, rather than hydrofracturing the ground, the grout may flow along the casing and emerge at the ground surface. Third, using multiple grouting applications in a particular stage is more difficult in comparison to sleeve-port pipe methods.

c. Drill Holes. Drill holes for hydrofracture grout injection are typically spaced at intervals similar to those used for permeation grouting. Hole spacings on the order of 5–10 ft are generally appropriate. Holes are commonly drilled vertically since there is no inherent benefit to angled holes unless obstructions must be avoided. In confined conditions, such as grouting from tunnels or shafts, fan arrays may be necessary. Drilling methods must be selected based on the subsurface conditions and intended injection method (end-of-casing or sleeve-port pipe). Cased drilling methods such as sonic or duplex rotary are most appropriate for sleeve-port pipe methods since the holes can be advanced with minimal flushing fluid and a temporarily cased borehole allows easy installation of the sleeve-port pipe.

d. Sleeve-Port Pipe. Hydrofracture grouting is typically conducted using sleeve-port pipe or tube-á-manchette. The sleeve-port pipe is inserted inside the temporary drill casing. The drill casing is removed, and the annulus between the sleeve-port pipe and the borehole sidewall is grouted. This is normally accomplished by inflating a single injection packer just above the lower-most sleeve. Grout pressure opens the sleeve, and the annulus is filled. After grout is observed at the surface, the injection is terminated and the packer is released. Hydrostatic pressure from the annulus grout column then forces the sleeve to the closed position. Casing grout is typically specified to be a weak cement-bentonite grout so that minimal frac pressure is necessary to open the sleeve. Depending on the applied pressures, sleeve-port pipe casing can be constructed of either plastic or steel. Steel is most appropriate for very high pressure applications, while plastic is often selected for economic reasons or for situations where other drilling or excavation activities would be adversely impacted by a steel obstruction. Sleeves can be spaced at any interval; however, sleeve spacings of 2–5 ft are common in practice.

e. Grout Materials. Most hydrofracture projects are conducted using high-mobility cementitious grouts. Often accelerators are added to enhance set times for subsequent reapplication after a brief set time. Chemical grouts have been used in hydrofracture projects, but their use for this purpose is relatively uncommon.

f. Pumps and Injection Equipment. Grout pumps of any type can be used provided they can develop the necessary pressures. In high-pressure applications, piston and ram-type pumps are most appropriate. Other injection equipment, such as mixers, grout conveyance means, and packers, are generally conventional and are described elsewhere in this manual. Packers, in particular, must be selected appropriately because the injection pressures in hydrofracture grouting can be high. The greatest concern with packers is slippage under the applied injection pressure. Double packer systems with packers spaced at close intervals are typically used to isolate individual sleeves for treatment. After injection, the casing may be flushed with water to facilitate subsequent reinsertion of the packer system.

g. Pressure, Flow Rate, and Refusal.

(1) Grout moving along the hydrofracture is subject to head loss due to friction. However, when the grout front encounters an obstruction, the flow rate is essentially zero, and the corresponding head loss is therefore also zero. Therefore, the full injection pressure at the borehole is also present at the grout front, which induces additional hydrofracturing and penetration. In unconfined conditions, then, a pressure refusal criterion is generally not appropriate. After fracture propagation is initiated, grout propagation will continue as long as the injection pressure exceeds the overburden pressure. Volumetric refusal is typically a more successful method to ensure that the treatment does not extend beyond the intended treatment area. However, it is recommended that a maximum pressure limit be specified and not exceeded on a hydrofracture program. (This pressure is not appropriate as a refusal criterion.)

(2) Injection rates are difficult to specify. Best results have been achieved using relatively low rates of injection. Experimentation is recommended to determine an appropriate injection rate for any particular site. Injection rates may vary throughout a project, depending on the extent of improvement gained from previous injections. If possible, a grouting test section should be built at the site and at the same depths using the production equipment. Appropriate injection rates and volumetric refusal criteria are best determined using this method.

28-6. Gauging Improvement. Since pressure refusal criteria are generally not appropriate, and since volumetric refusal gives no clear indication that the ground will not accept additional grout, it is difficult to determine the extent of improvement in a hydrofracture grouting program. Verification of hydrofracture grouting to determine if the design intent has been met therefore must rely on other methods. In the case of permeability reduction, permeability tests such as pressure tests or falling/constant head tests may be appropriate. Piezometer data may also be a valuable resource. Densification and strengthening applications require different methods, such as cone penetrometer and pressure meter tests. Laboratory testing of recovered specimens may be appropriate. One clear indication of improvement is generally an increase in the required frac pressure with subsequent grouting applications in the same zone.

28-7. Case Histories.

a. General. While detailed case histories of U.S. applications are rare, there are a few notable hydrofracture grouting projects that deserve mention. The following paragraphs describe two such cases.

b. U.S. Postal Service Building, Clarksburg, WV. The U.S. Postal Service Building in Clarksburg, WV, was constructed on a reclaimed strip mining site. Uncontrolled fill was placed during reclamation. Shallow controlled fill was subsequently placed to facilitate building construction. After completion, the building experienced excessive and differential settlement, resulting in significant distress to the structure. The facility houses automated sorting equipment that is sensitive to movement, so a remedial grouting program was conducted. Grouting was conducted from shafts excavated around the perimeter of the structure. Casings were jacked to the appropriate location beneath the structure in a fan array. Hydrofracture grouting was then conducted. The intent of the grouting was to strengthen the ground and fill presumed voids within the uncontrolled fill, both of which were intended to minimize future settlements. After completion, settlements were negligible. Heenan et al. (2003) provides a complete description of this project.

c. Mud Mountain Dam. Mud Mountain Dam was constructed in 1941by USACE Seattle District. The dam is located within a very narrow valley with vertical or nearly vertical sidewalls over much of both abutments. A zoned embankment consists of upstream and downstream rock-fill shells with an impervious core. The shells were not designed to filter the core materials. In the early 1980s, piezometric data indicated a problem with the structure. Further investigation and evaluation determined that the core was failing due to either piping or washout of fines caused by fluctuating pool levels. Clean gravel zones were encountered during subsequent investigative drilling programs. The intended remedial method was to construct a concrete cutoff wall. During construction of the wall, rapid and significant slurry losses were experienced as the panels were advanced through the core. It was determined that the low overburden stress in the soil that occurred as a result of soil arching in the narrow valley and between the core and rock shells resulted in hydrofracturing of the embankment by the excavation slurry. Ironically, the proposed remedial treatment used hydrofracture grouting, which was chosen because the method would fill cracks in the core, permeate and fill zones of high-permeability materials, and densify the core materials. Sleeve-port pipe methods were chosen as the preferred alternative. Injections were limited to 200 L per sleeve in the core, while higher volumes were permitted at the core/rock interface to treat presumed defects in the rock surface. After completion of the grouting program, the panels were advanced to their design depth with minimal slurry losses. Note that the slurry wall construction at Mud Mountain Dam extended to depths of up to approximately 120 m and that hydrofracture grouting was successful in minimizing slurry losses in the core at these depths. Monitoring while drilling, also known as drilling parameter recording, was conducted during the grout hole drilling; it accurately defined zones of low stress in the embankment that subsequently took significant volumes of grout. Refer to Davidson et al. (1992) for additional information on this project.

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CHAPTER 29

Grouting Contract Considerations

29-1. Fundamental Considerations for Contracting.

a. General. The method of contracting, and the content and structure of the contract documents and payment items, can all dramatically impact quality, schedule, the potential for cost growth, and the inspection forces required. The development of an Acquisition Strategy Plan, which defines the methods to be used, is the primary responsibility of the Contracting Officer, but should be done in close coordination with the entire Project Delivery Team. Selecting the appropriate contracting methods requires consideration of the project's size, the complexity of the work, the intent and criticality of the grouting to the overall project performance, the degree of uncertainty in the work, the knowledge and experience required of the grouting contractor, and the experience and number of organizational personnel available for oversight. As such, the acquisition strategy should be integral to and concurrent with the development of the technical specifications.

b. Grouting Specialty Contractors. Without exception, grouting should be performed by specialty contractors who are regularly and successfully completing grouting projects. Grouting results can be highly dependent on the skill and knowledge of the contractor. An adequate number of specialty contractors are available to service both small and large grouting projects. In defining and/or evaluating the qualifications of grouting contractors, the following items and issues are normally considered.

(1) Contractors must be able to mobilize sufficient equipment of the type required for the project. Prospective contractors would normally be asked to submit the details of all equipment that will be used (manufacturer, model, age).

(2) Contractors must be able to provide sufficient competent, experienced personnel for all of the work items required. Corporate experience per se may have little relevance. Prospective contractors are often required to submit detailed resumes of all of their key personnel and superintendents. Resumes should identify all experience and specific duties directly applicable to the proposed project. Project listings for individuals should state their roles in the project and the duration of their activities.

(3) Project descriptions for similar work should include only those projects in which one or more of the proposed staff were in a position of responsibility for the work. Project information should include references, the initial cost at the time of contract issuance, and the final cost at completion of the project. Similarly, initial and final contract completion dates for projects should be provided.

c. Separate Grouting Contracts vs. Grouting Subcontracts. The choice of using a separate grouting contract vs. including grouting work as an element of a larger contract is one of the fundamental contracting decisions. A separate grouting contract can offer advantages, including: (1) Government forces can have direct communication and interaction with the grouting contractor, making it easier to administer, control, and modify the grouting operation, (2) a separate contract eliminates sometimes problematic site coordination issues between a subcontractor and a general contractor, (3) a separate contract can provide project schedule advantages if grouting is one of the early operations to be performed, and (4) perhaps most importantly, it can prevent required modifications in the scope of the grouting program from creating a ripple effect into other work, thereby preventing complex requests for equitable adjustments related to compounding impacts. Conversely, some projects are not suitable for using separate contracts due to required sequencing of the project work or the amount of grouting to be performed relative to the overall project size. In addition, when the grouting precedes work such as cutoff wall construction, the wall contractor cannot fully endorse the adequacy of the grouting efficacy as it relates to their means and methods. One of the benefits of having grouting included as a part of larger contracts is that the contractor derives direct benefit from the quality of the work performed. Regardless of whether grouting is performed as a separate contract or as a subcontract, the information obtained from grouting is important data that should be available to follow-on contractors performing subsurface work in the vicinity of the grout program.

29-2. Source Selection and Contract Types.

a. General. In general, there are two types of source selections: negotiated selection (i.e., Best Value Selections) and non-negotiated selections (e.g., Sealed Bid, Technically Qualified – Low Price). Contract types included fixed-priced and cost-reimbursable contracts. Typically, fixed-priced contracts are appropriate for most grouting contracts. A variety of other contract methods are available, depending on the specific project's needs. For instance, depending on the criticality of time or other factors, incentives and/or a successively pricing method such as Early Contractor Involvement (FAR Part 16.403) may be considered. The procedures and rules for using the various selection methods and contract types are contained in the FARs. Applicable FAR parts include Part 14, *Sealed Bidding*; Part 15, *Contracting by Negotiation*; Part 16, *Types of Contracts*; Part 17, *Special Contracting Methods*; and Part 37, *Service Contracting*.

b. Recent Trends.

(1) In recent years, negotiated Best Value Selections (FAR, Part 15) have frequently been used to acquire standalone grouting contracts for large and/or complex grouting projects where access to the latest technology and/or comprehensive services is desired. This acquisition strategy allows USACE to solicit and negotiate technical proposals based on weighted technical and cost factors (i.e., tradeoffs). The negotiated Best Value selection methodology was used by the Louisville District for the Patoka Lake Project, the Chicago District for the Chicago McCook Reservoir grouting, the Little Rock District for grouting at Clearwater Dam, and the Nashville District at Center Hill Dam. In each of those projects, the tradeoff process as described in FAR 15.101-1 was used to determine the technical and cost proposal that was in the best interest of USACE. Additional information on Best Value Source Selection is contained in the Army Material Command publication AMC-P 715-3, *Contracting for Best Value – A Best Practices Guide to Source Selection* (1 January 1998). Appendix C includes copies of the documents describing the evaluation factors recently used by the Chicago and Little Rock Districts, along with the corresponding unit price schedules, measurement and payment provisions, and project technical specifications.

(2) Another source selection strategy that might be employed on large or small projects where it is not necessary to evaluate technical proposals, but where the technical ability and/or experience of the contractor is essential to the successful outcome of the project is the two-step Technically Acceptable, Low Price selection method. In this method, the lowest priced proposal is selected from offerors deemed technically qualified based on an evaluation of the qualifications specified in the solicitation.

(3) An innovative contracting method that could be particularly useful on projects where the grouting is part of a larger foundation treatment program is the Early Contractor Involvement (ECI) method. Under this method, the standard Fixed-Priced contract is supplemented with FAR Clause 16.403, which allows for successively pricing all or part of the work at designated target points. Within the context of a large foundation treatment or foundation remediation program, a grouting and/or test program might be carried out under the Fixed-Priced Base Contract with the follow-on remediation and treatment structured as an Unpriced Contract Option that is subject to Clause 16.403 to be negotiated and awarded after the results of the grouting program or other test are evaluated and the final design is completed. In addition to the obvious advantages of incorporating the construction contractor's expertise and perspective into the final design, it provides actual on-site production data to be used in the cost estimate and negotiation of the final design, and it also enables the project to begin using Construction General Funds while still completing the design. Generally, an ECI contract is awarded with the design at approximately 35%. Hence the decision to use this method must be made early in project development. ECI contracting using FAR Clause 16.403 was used by the Kansas City District on the Tuttle Creek Foundation Modification Project in which both a test program and production testing of the final remediation technology were carried out under one contract before completion of the final design. The methodology was also used by the New Orleans District on much of the reconstruction work after Hurricane Katrina to accelerate the design and construction schedule.

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CHAPTER 30

Measurement and Payment

30-1. General Considerations. Chapter 29 of this manual describes available contracting methods and considerations for contracting. Chapter 31 of this manual covers estimating grouting quantities, and provides general guidance for establishing the appropriate unit price schedule items and commentary on measurement and payment provisions.

30-2. Unit Price Schedule Items.

a. General. Experience of the Corps of Engineers indicates that the items discussed in the following paragraphs should be considered for inclusion in any estimate or unit price schedule for a drilling and grouting program. Selected items must be appropriate for the needs of the particular project. When establishing line items for a project, the work must be broken down in such a manner that the Government has flexibility to modify the program based on the conditions encountered, and that the contractor is not required to assume undue risk for the conditions or flexibility required by the Government. Otherwise, if the contractor is required to accept risk for the conditions that may be encountered, the cost for assuming that risk will be reflected in higher prices in the unit price items.

b. Technical Requirements. All requirements regarding hole locations, hole angles, drilling depths, stage lengths, upstage or downstage methods, injection pressures, pump capacities, number of hole series, material properties, or any other item of work that has specific requirements must be clearly described in the technical specifications or drawings. Technical specification sections for other ancillary items of work, such as installation of instrumentation, drain holes, or other secondary work activities, should contain content specific to those work items. Measurement and payment provisions should only describe how the work will be measured and paid and should not include work performance requirements or procedures.

c. Mobilization and Demobilization. Drilling and grouting equipment and all supporting project infrastructure must be assembled at the job site before a grouting program can be started and must be removed from the site when the work is completed regardless of the amount of work actually performed. A separate pay item (or pay items) for these operations should therefore be included in the specifications. Payment for these two elements of work is commonly set up under one pay item with provision for a partial payment to the contractor upon completion of the mobilization (typically 60%) and for payment of the remainder after the grouting program is completed and the materials and equipment are removed from the site to the satisfaction of the Government.

d. Site Preparation and Site Restoration. The required extent of preparation for a site may vary substantially from project to project, but it is commonly a major item. In some cases, a substantial work platform must be constructed to accommodate the equipment and provide effective environmental controls (see Chapter 13 of this manual). Site preparation activities are typically measured on a lump sum basis, but in some instances unit prices for support of excavation, fill, and concrete paving might be appropriate. If it is anticipated that the limits of work might be extended, or if the project is divided by area and grouting might (or might not) be conducted in particular areas, provisions should be made for measuring site preparation in each area separately. It is also common to include a site restoration line item. For both site preparation and site restoration, the contract must clearly define the scope associated with those activities.

e. Environmental Protection. A separate measurement and payment item should be considered for environmental protection. A separate pay item is considered necessary for any project where unusual or special provisions are required, such as protection of a water intake or where there are rigorous environmental requirements for monitoring and protection of downstream areas or waters. Lump sum measurement and payment is appropriate for a project with standard requirements. A separate monthly lump sum or unit price might be appropriate for any special project specific environmental monitoring or protection requirement. The monthly payment may also be limited to only those months where grouting was performed for a substantial portion of the month.

f. Drill Setups. Drill setups are measured for payment each time the drilling equipment must mobilize to the hole. Provisions for drill setups for each different drilling method required per hole should be included. On upstage grouting programs, one drill setup should be measured for payment per hole. Downstage programs require one drill setup per stage.

g. Drilling Grout Holes. Measurement and payment for drilling is almost universally paid for on a per-foot basis. Different types of drilling should be measured and paid for separately. In addition, differentiation of angled holes versus vertical holes is standard practice. The list of drilling pay items might include:

(1) Overburden Drilling Vertical.

(2) Overburden Drilling Angled.

(3) Production Rock Drilling Vertical.

(4) Production Rock Drilling Angled.

(5) Exploratory Drilling Angled.

(6) Exploratory Drilling Vertical.

(7) Production Drilling Gallery Angled.

(8) Production Drilling Gallery Vertical.

h. On a project where the same classification of drilling has substantially different scope depending on the location, multiple pay items for the same drilling type might also be included. For example, if a project includes overburden drilling through an existing high embankment and overburden drilling on an abutment with shallow overburden, separate pay items should be established based on location or based on depth. If the embankment drilling will be performed on a horizontal platform while the abutment drilling will involve drilling on sloped terrain, then separate payment based on location and not depth would be more appropriate. Additionally, drilling that must be conducted in confined spaces, such as within galleries, should be measured for payment separately from above-ground work. If it becomes necessary to drill through grout in a hole after it has set, or re-drill an obstructed hole, a special payment provision for redrilling should be specified. Measurement for payment is contingent on the determination that redrilling is required through no fault of the contractor. Redrilling is sometimes specified as one-half of the cost of initially drilling the grout hole rather than allowing the contractor to provide a separate bid price for this item.

i. Temporary Casing for Grout Holes. If grouting of temporary casings is a requirement for the project, then quantities associated with these operations should be measured for payment. It is common on many existing embankment dams for curtains to be constructed on the upstream slope where shell materials are present. Often, when grouting the overburden casing, the shell materials will accept large quantities of grout before grout is observed at the ground surface in the casing annulus. Grouting the annulus through a pervious shell might not be appropriate or necessary and should be evaluated before injecting large volumes of grout. Additionally, the soil-rock interface or first several feet of rock may readily accept casing grout. Since the contractor has little control over the volumes of grout the ground will accept nor the time required to accomplish the grouting, it is recommended that the grouting time and grout materials consumed be measured for payment rather than using a lump sum price for each installation. The grout connection required for grouting of the casing can be incidental to the overburden drilling, or may be measured for payment in accordance with other grouting connections. Solid temporary casing can either be measured for payment as a separate item or it can be defined to be incidental to overburden drilling. The contract must be specific with respect to these matters. If sleeve-port pipe will be used, it is recommended that it be measured for payment under a separate line item, since the amount of sleeve-port pipe installed is typically at the direction of the Government based on the conditions encountered, and sleeve-port pipe is materially different from solid casing.

j. Hole Washing. Hole washing separate from washing through the drill tooling should be paid under a separate hole washing line item. On many past contracts, hole washing and pressure testing have been paid under one line item. This is not recommended since the equipment required for each operation is materially different. Measurement units can include lump sum per hole, or washing time. Either can be effective provided the ground conditions are appropriate. Rock formations of high quality and not subject to slaking will often result in near equal wash times per lineal foot from hole to hole. In these instances, lump sum units may be appropriate. When variable wash times are expected, such as in karst, measurement by unit time is more appropriate. The technical specifications must detail the required pressure and flow rate for hole washing.

k. Pressure Testing. As stated above, it is not recommended that hole washing and pressure testing be measured and paid under one line item. In addition to the equipment necessary for performing hole washing, pressure testing requires: packers, packer inflation system, flowmeters, pressure transducers, possibly a different pumping system, and possibly an additional operator. As with hole washing, pressure testing can be measured as either lump sum items for each test or as units of time. If the lump sum per stage unit is used, technical specifications must provide the basis for reasonably establishing the unit costs, such as the maximum or average test time per stage. Measurement of time units generally provides more flexibility to the Government since extension of pressure testing duration will not negatively impact the contractor.

l. When using time units as the basis for measurement and payment, the start time should be specified as when the pressure testing equipment is inserted into the top of the hole and the end time should be specified as when the equipment is removed from the hole. Time deductions should only be made for any interruptions in the operations resulting from equipment failure or other contractor controlled causes. Because the amount of time required to lower the equipment to the depth required and/or move the equipment from stage to stage is significant, including that time in the measurement is recommended provided that it is conducted as a continuous process. When using this method, however, it is also recommended that the technical specification require the use of continuous hose rather than permitting the use of pipe sections, which greatly increases the time of the operation.

m. Placing Grout. It has been common on many past projects to measure grout placement for payment on the basis of the volume placed. The practice places high risk on the contractor and is a disincentive to quality since injection rates and volumes are predominantly controlled by the geologic conditions. Since the contractor's cost of performing the work, not inclusive of materials, includes plant and labor time, the contractor is forced, under this method, to estimate costs with an assumed average injection rate of grout. During grouting, when flow rates are above that assumed average the contractor will be quite pleased. However, when flow rates drop below that assumed average, such as at the critical refusal period, it can be a contentious issue. The result may be inappropriate and premature thickening, inadequate refusal, and any other measures or actions (often unknown to the inspection staff) that allow the contractor to move on to more productive and more profitable operations. Premature refusal can result in a poorly constructed, low-quality grouting project.

(1) For these reasons and given the criticality of the grouting operations, placing grout should be measured on a unit time basis. Measurement should initiate at the start of injection and end upon the completion of refusal. If multiple injections are conducted simultaneously, each injection should be separately measured for payment. The contract documents should be specific with respect to the required injection flow rates, including minimum required injection rate when maximum injection pressure is not achieved. Pressures and flow rates should not be a payment consideration during the initial 5 to 10 minutes of grouting on a stage since the pressures are purposely increased slowly during this phase of the application. After the maximum permitted injection pressure is achieved, the ability of the formation to accept grout totally controls the rate of injection.

(2) There are instances in grouting when measurement of volume injected is appropriate for payment. These would include any and all instances where the volume of material to be placed is known with certainty. Void-filling operations and annular space grouting may warrant consideration of a volumetric payment provision. This is never the case in permeation grouting of rock or soil.

n. Cement. The units of measurement for cement should be tons. The contractor has little control over the volumes of material the ground will accept at the pressures and using the mixes directed by the Government. Cement in grout that is wasted due to no fault of the contractor, such as at the completion of grouting when a cleanout of the circulation loop is required, should be measured for payment.

o. Other Grout Constituents. All other grout ingredients, with the exception of water, may be individually measured for payment under separate line items using appropriate units, however this method may create additional effort. Typically dry products such as bentonite, sand, fly ash, silica fume, or diutan gum are measured on a weight basis. Liquid products such as superplasticizer, retarders, etc. may also be measured on a volumetric basis. Note that pre-packaged products are also available.

p. Connection to Grout Holes. Connections to grout holes should be measured and paid for as a lump sum price for each connection. A connection is defined as mobilization of the grouting equipment to the hole for purposes of injecting grout. A connection line item is provided to pay for all labor and equipment necessary for mobilizing equipment, including the header, packers, packer inflation system, flow meter, pressure transducer, and circulation lines to the hole, and also preparing the batch plant and mixing the first few batches of grout necessary to fill the circulation loop. On upstage grouting programs one connection should be measured for payment per hole. On downstage programs one connection is measured for payment per stage.

q. Computer Monitoring System. The computer monitoring system should be measured for payment on a time basis (hourly, weekly or monthly dependent on site specifics) and should include the equipment and operator costs. Measurement should begin 1 week before the start of grouting to allow for setup and calibration of the system on site to the satisfaction of the Government. Measurement should end at the completion of grouting. If additional use of the system is required beyond the completion of grouting, such as for purposes of generating information for a project completion report or otherwise required on site by the Government, then this time should also be measured for payment. Operation of the system is normally included with this pay item. The contractor determines the number of operators required based on the contract deliverable requirements and the staffing plan, and on a consideration of the capabilities of the system being provided. The cost of records, both digital and hard copy, generated by the system should be incidental to the system payment. The specifications should clearly define the frequency of submission and the format of the records required.

r. Geologists/Geotechnical Engineers for Recordkeeping and Drilling Observation. Payment for on-site technical staff required by the contract can either be measured and paid for on a per-shift basis or can be made incidental to the drilling. If this item is incidental to the drilling, the number of staff must be clearly identified in the contract. Typically, the number of staff required is specified based on the number of drills. For example, the specifications would state that one geologist is required for every exploratory drill and that, on average, one geologist is required for every two production overburden or rock drilling operations. Measurement and payment on a shift basis can also be used effectively. In this case, the number of on-site staff can be varied by the Government based on the nature of the work (critical or not critical), the level of detail in the records that is desired, and the intensity of the operations. Practical resource planning and cost considerations dictate that a minimum notification time of approximately 2 weeks be provided before the Government increase or decrease the on-site technical staff requirements.

s. Instrumentation Monitoring. Similar to computer monitoring, instrumentation monitoring should be measured for payment on a time basis. The specific instruments requiring monitoring, the required frequency of readings, and the format of records or reports to be provided must be clearly described in the contract. If the intent is to simply read the on-site instruments daily, then a daily rate unit is appropriate. If measurements will be obtained on a more frequent basis, then a shift, weekly, or monthly rate is appropriate. Long duration projects lend themselves to monthly units.

CHAPTER 31

Quantity Estimates

31-1. General Considerations.

a. General. After all aspects of the design, technical requirements, and specific procedures have been established for a grouting project, the drilling and grouting quantities must be estimated. Because grouting is an underground operation and is ultimately controlled by the geologic conditions, the work involved in a drilling and grouting program can only be approximated in advance of construction. In addition to natural variations in the subsurface conditions, the means and methods employed during construction can also impact the final quantities. For example, overly conservative injection pressures might require more holes to meet the project goals in comparison to using the maximum safe grout injection pressure.

b. Estimated Quantities. The contract specifications and bid items should be prepared so that the estimated quantities for each of the bid items can vary without affecting unit prices. A common practice to address the variation in quantities in grouting is to use split line items. ER 1180-1-8, *Contracts - Labor Relations* (2006) contains guidance in providing for variations in estimated quantities by using subdivided items. However, a concerted effort must be made to reasonably estimate the drilling and grouting quantities and to fairly represent the work required so that the appropriate level of funding can be requested.

c. Discussion. Chapter 29 of this manual describes available contracting methods in detail, and Chapter 30 addresses measurement and payment for grouting. This chapter provides guidance for estimating the quantities for inclusion in the contract. For most major projects, all of the methods described here should be considered when developing an estimate.

31-2. Test Grouting. For medium and large projects, the most reliable method for estimating drilling and grouting quantities is to perform a full-scale test section of substantial length. The site for the test section should be representative of the anticipated conditions across the project. The performance achieved in the test section and the quantities consumed can be extrapolated to estimate the total project quantities. Depending on the uniformity of conditions across the site, this extrapolation might not be linear at all locations and may warrant separate reaches to be designated as test sections.

31-3. Use of Historical Records. Another tool for estimating quantities is to refer to historical grouting records from the same site or from sites located in areas that have similar geology and rock types. Records from grouting projects in similar settings should give a general order of magnitude for the quantities and their interrelationship. Experience and knowledge of grouting on the part of the estimator are required to extrapolate the data from one site to another site. Differences in grouting means, methods, and results must be carefully considered when using historical data.

31-4. Site Characterization Information. Evaluating the cores from the exploration program, the results of the water pressure tests, and other available information is fundamental in the initial stages of preparing a grouting estimate. The number, strike, dip, spacing, and typical aperture of the predominant joint sets are essential information for establishing the estimated final hole spacing and the orientation of the holes. Joint count and aperture information, if available, can also be used in estimating grout quantities; this approach has rarely been accomplished, but the availability and quality of borehole imaging information might result in this type of estimate becoming more common.

31-5. Unit Take Method. A method frequently used during preparation of detailed estimates for drilling and grouting programs is called the “unit take” method. This method is most appropriate when historical data are available from the site or from a site with similar geologic conditions. In this method, the area to be grouted is divided into horizontal reaches and vertical zones of varying permeability based on site geology, pressure test results, and extrapolation from historical data. Estimates are made of the number of primary and split-spaced holes required to complete each area (reach) and zone. Grout take in volume per linear foot of grout hole is assigned, as is the estimated reduction in grout take for each split (higher-order hole) and zone. The amount of grout take in each series of split-spaced holes normally would be less than the preceding set of holes, and in multiple lines, the take in each line would generally be less than on previously grouted lines. Each zone of each hole is assigned an estimated take in cubic feet of grout per linear foot of grout hole. The data in Table 31-1 illustrate a typical estimate using this method.

Table 31-1. Example of estimated grout take in cubic feet/linear foot.

Reach*	Line	Zone	Depth (ft)*	Primary	Secondary	Tertiary	Quaternary
A	A	1	0–10	1.5	0.75	0.2	0.05
A	A	2	10–25	1.0	0.40	0.1	
A	A	3	25–50	0.4	0.25	0.1	
A	A	4	50–100	0.1	0.01		
* Note: The above figures are for illustration only and should not be used for purposes of estimating, criteria for split spacing, or completion of grouting.							

31-6. Estimating the Quantity of Individual Bid Items.

a. Guidance. The following paragraphs provide general guidance on establishing quantities and incorporating appropriate contingencies. Chapter 30 of this manual discusses the measurement and payment items that should be considered for inclusion in any estimate or unit price schedule for a drilling and grouting program. No guidance is required for the estimation of the quantity of lump sum items. However, estimating the cost of lump sum items does require a reasonable estimate of the anticipated project schedule, as do items paid for as monthly lump sums. Estimating project duration is included in the items below. For schedule estimating, the most reliable source of information is the average production rates achieved on recent past projects using similar equipment.

b. Drilling Quantities. After the horizontal and vertical limits of the curtain and the angles of individual holes are established, the length of each grout hole is easily established. The remaining variables required for estimating drill footage are the number of lines and the final hole spacing. Without information from a grouted test section that provides more definitive guidance on the hole spacing and achievable results, the following are recommended assumptions for estimating quantities.

(1) For a single-line grout curtain, it is recommended that the final hole spacing be established at 5 ft and that contingent drill footage equal to 25–50% of the quantity for a single line be added for problematic areas that require deepening of holes or adding of additional lines to achieve the desired residual permeability.

(2) For a three-line grout curtain, it is reasonable to assume a final hole spacing of 5 ft on each line with no contingency for additional holes. It might be possible to shorten the holes on the center line, but that footage would then be available for deepening holes or adding additional holes on the outside lines.

(3) For remedial grouting, drilling through an existing embankment is often required. Therefore, the drilling quantities must be separated by drilling type. Overburden holes are commonly installed several feet into rock to seat a temporary casing. This footage should be included in the overburden drill footage. If grouting is for pre-treatment in advance of a slurry wall, a two-line curtain will likely be used. In this situation, it is recommended that a final hole spacing of 5 ft be assumed and that 100% of the footage be included for both lines. An additional allocation of drill footage should be included equal to 25% of the footage for one of the lines to permit verification holes to be drilled and tested along the proposed centerline of the cutoff wall. For these projects, the quantity of overburden casing is equal to the overburden footage.

(4) A quantity of the production drilling should be set aside as exploration holes. The number of exploration holes required depends on the complexity of the site and the extent of any prior investigations. If a site has been extensively explored previously, exploratory drilling might not be necessary before beginning the grouting program. However, it is useful for the grouting contractor and the QC personnel on site to see several cores in advance of grouting. In the absence of any other guidance, it is recommended that one exploratory hole be allocated per 100 ft of curtain for pre-grouting exploration. A similar quantity should be set aside for verification holes to be performed after grouting. The pre-grouting exploration holes are commonly incorporated as production holes by drilling them at primary hole locations on the first line. The verification holes can be located as the quaternary holes of a single-line curtain, or on the final line of a multiple-line curtain. In these instances, the quantity of exploratory drilling should be deducted from the previously calculated production drilling quantity. For a cutoff wall pre-treatment program, the verification holes will commonly be added as additional footage and not deducted from the production drilling quantity.

(5) The number of geologists or geotechnical engineers needed for logging drill holes should be determined based on the specification requirements for the number of drills each geologist is to be responsible for logging, the anticipated number of drills, and the average assumed production rate. The best source for estimating the number of drills and production rates is a review of similar past projects.

c. Grouting Time. Grouting time depends on many variables, including grouting pressures, discontinuity aperture, mix design, stage length, and refusal behavior. On any given line, the primary holes generally take longer to reach refusal than the secondary holes. Similarly, each stage of the secondary hole would be expected to take longer to grout than the tertiary stages. If a test section has not been performed at the site and previous information on grouting times is not available at the site or at a similar site, then it is recommended that an overall average grouting time of 0.5–1.5 hrs per stage be assumed. Shorter times can be assumed for multiple-line curtains. One hour per stage would be a reasonable assumption in most cases. Grouting time in karst sites can vary widely, depending on the solution features within the rock. The presence of even a small cavity can greatly increase the amount of time required to adequately grout any given stage.

d. Grout Materials. The quantity of grouting materials is one of the more difficult items to estimate in the absence of a test grouting program or a recent relevant project in a similar setting. The actual injected grout volume depends on the compatibility of the grout mixes to the formation, the maximum injection pressure, the fracture aperture, the connectivity of fracture sets, and the refusal criteria. Fortunately, materials represent only a small percentage of the cost of a grouting program in comparison to grouting time and drill footage. The quantity of grout per foot of grout hole drilled will vary widely across a site, but it should steadily decrease as the work proceeds and split spacing is performed. Historically, grout consumption has been estimated and evaluated as the dry volume or weight of cement injected per foot of hole. A common example of this is the Grout Take Classification System proposed by Deere (1976). Now that stable grout mixes are used, it is more common to estimate grout material quantities by assuming the average grout take per foot of hole, and then to calculate the corresponding weight of cement. For comparison, Table 31-2 lists Deere's original criteria. To aid the user in estimating grout quantities when using stable grouts, the Deere criteria have been modified in the table to characterize takes in terms of grout volume, not bags or weight of cement.

(1) Although this classification system has commonly been used to characterize the grout consumption after grouting is complete, it can also be used in conjunction with the available site characterization information to grossly estimate grout volumes in advance of grouting. Estimating grout takes based solely on Lugeon values from site investigations is not recommended. Lugeon values are directly dependent on the stage length used in the testing, and the test does not account for the fracture apertures. Grout penetration into a fracture is directly proportional to the fracture aperture, and if radial penetration is assumed, the injected volume is proportional to the cube of the fracture aperture. This is the explanation for stages with similar Lugeon values having significantly different grout takes. A stage with one fracture in a 10-ft length with the same Lugeon value as a 10-ft stage with 10 fractures will take more grout at a given injection pressure. In addition, fractures finer than are penetrable with a given grout mix will still be permeated with water during testing. For a site with a low to moderately low average permeability (<30 Lu) before grouting and a typical fracture aperture of 0.5 mm, a grout take of 3–5 gal per foot of hole would be considered an appropriate assumption. Similarly, for an initial moderate pre-grouting permeability of 50 Lu with a typical fracture aperture of 1–2 mm, a reasonable assumption would be a grout take of 10 gal/ft. For highly permeable rock masses with fracture apertures of several millimeters, a take of 25–50 gal/ft would not be an unreasonable expectation.

Table 31-2. Deere's Grout Take Classification System.

Classification*	Symbol	Original Criteria (maximum bags/ft)	Modified Criteria (Stable Grouts) (maximum gal/ft)
Very Low	VL	0.09	1.0
Low	L	0.18	2.5
Moderately Low	ML	0.36	5.0
Moderate	M	0.71	10.0
Moderately High	MH	1.43	25.0
High	H	2.85	50.0
Very High	VH	>2.85	>50.0
*Source: Deere (1976).			

(2) For a multiple-line curtain, the assumed takes should be reduced for the second and subsequent lines. Although not supported by specific data, for a site with a high to very high initial grout take, it would be reasonable to assume a high take for the first line, a moderate take for the second line, and a low take for the interior line. The volume of grout required to backfill the drill holes must also be calculated. Typical hole diameters range from 3 to 4 in. Backfill volumes for hole sizes of 3, 3.5, and 4 in. are 3.7, 5, and 6.5 gal/ft, respectively.

(3) After the volume of grout has been estimated, it is recommended that the estimator assume an average WCR 1:1 by weight if no design information is available. Therefore, the cement quantity in bags of cement will be the estimated grout volume expressed in cubic feet divided by two. A mix of 1:1 by weight includes 1.5 $ft^3$of water and 1 ft^3 of cement. Since the absolute volume of a cubic foot of cement is 0.5 ft^3, the volume of a one-bag mix prepared at 1:1 by weight is 2 ft^3. After the number of cubic feet of dry cement or bags is known, the remaining grout constituents can easily be estimated using the typical proportioning guidelines based on weight of cement expressed in terms of percentage of weight of cement, as provided in Chapter 7 of this manual.

(4) Grout takes at karst sites can vary widely. Grouting at Wolf Creek Dam in the early 1970s resulted in an average grout take of approximately 1.5 bags per foot. Past grouting projects at Center Hill Dam, located in similar geology, consumed 10 bags of cement per foot. Major foundation grouting projects have been completed or are underway at Wolf Creek Dam, Center Hill Dam, and Clearwater Dam. Estimators of future grouting projects in karst would be well served to obtain the results from these projects to assist with estimating quantities in karst using modern grouting mixes and techniques.

e. Water Testing Time. The advent of computer monitoring in grouting has significantly increased the speed at which pressure testing can be performed. An average water testing time of 5–10 minutes per stage, including the time to pull the packer between stages, is normally adequate. If step pressure testing is to be performed, 15 minutes per stage should be assumed for those stages.

f. Special Washing. On projects where hole washing is required in addition to the hole flushing and washing that occurs during drilling, hole washing or special washing is measured for payment on the basis of time. Sites with infilled fractures will take longer to wash than sites with clean fractures. A typical hole washing time is 1 hour for each 100–200 ft of hole.

g. Project Duration. Although drilling is the most expensive procedure in a grouting project, grouting time generally controls the schedule. After accounting for site development time for any preparatory work in advance of drilling, plus any time required for installing overburden casings for remedial work, the duration should be estimated by applying efficiency to the grouting hours. Grouting efficiencies are typically on the order of 50–70% and rarely exceed 75%. The number of grouting operations must also be factored into the estimated time. For example, a grouting project with 5,000 grouting hours and two grouting operations will have a base grouting time of 2,500 hrs, assuming an average grouting time of 1 hour per stage. Applying an efficiency of 60% results in a duration of 4,167 hrs. This is equivalent to 520 8-hour workdays or 417 10-hr workdays. If the project will be worked 5 days per week, 10 hrs per day, the grouting duration will be 584 calendar days, or approximately 19.5 months. Adding a reasonable time for mobilization, site preparation, and demobilization would result in a total project duration of approximately 24 months. Sometimes the duration may be constrained by follow-on contracts or urgency based on dam safety concerns. These may require multiple shifts per day with a lower efficiency for night work factored in. Site constraints may limit the number of grouting operations that can be accommodated. In these cases, mobilization of additional equipment will not efficiently shorten the project duration.

h. Computer Monitoring. The efficiency of the grouting operations as identified above will also dictate the number of months required for monitoring the operations. In general, an experienced operator of a monitoring system can routinely monitor two operations, with occasional, but not continuous, observation of three simultaneous operations. If three or four water testing or grouting operations will routinely be performed, then a second operator should be included in the estimate.

APPENDIX A

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APPENDIX B

Pressure Calculations

B-1. Introduction. This appendix addresses the method of calculating effective pressure applied during water pressure testing and grouting stages. Proper calculation of effective pressure is an important consideration for dam safety, consistency in grouting practice, and proper grouting and evaluation technique. Noted here are several key concepts that pertain to calculating effective pressure, along with several examples of effective pressure calculations ranging from simplistic to complex, multi-variable problems.

B-2. Key Concepts.

a. General. The basic formula for determining effective pressure is simply the sum of all head losses and head gains in the injection system and the ground. The key considerations of the calculation include: geometry of the hole and injection system (hole angle, top and bottom stage depths, gauge height), injection fluid (grout or water), and groundwater conditions. The following paragraphs address these considerations, along with others.

b. Water Pressure. For purposes of grouting, and as appropriate for most civil engineering applications, the density of water is generally accepted as 62.4 lb/ft^3. The specific gravity is generally accepted as 1.00. While minor variations in the values are noted, particularly due to salinity and other dissolved solids, these values are appropriate in nearly all instances, and very minor variations in the values result in negligible changes to subsequent calculations. The pressure induced per unit foot of water head is calculated as:

$$62.4\ \frac{\text{lb}}{\text{ft}^3} \times \frac{1\ \text{ft}^2}{144\ \text{in}^2} = 0.433\ \text{lb/in}^2/\text{ft} = 0.433\ \frac{\text{psi}}{\text{ft}}$$

c. Grout Pressure. Since the specific gravity of a substance is simply a unitless comparison of the density to that of water, the pressure induced per unit foot of grout head is simply the unit water head pressure (calculated above) times the specific gravity of the grout. For example, the pressure induced per unit foot of grout head with a grout specific gravity of 1.35 is:

$$0.433\ \frac{\text{psi}}{\text{ft}} \times 1.35 = 0.584\ \frac{\text{psi}}{\text{ft}}$$

d. Hole Angles. Correction of the head induced by a fluid column is necessary for angled holes (holes drilled at some angle other than vertical). This is necessary because the unit head pressures noted above are only appropriate in the vertical direction. For hole angles referenced from the vertical direction, simply multiply the depth of interest measured along the length of the hole (stage midpoint, groundwater depth, etc.) by the cosine of the hole angle to determine the vertical component. For hole angles measured from the horizontal, multiply by the sine of the angle. Typically, holes are drilled at an angle to maximize the frequency of encountering vertical fractures, and depths are measured along the length of the hole rather than vertically (by elevation); as a result, this correction is common on most grouting projects.

e. Groundwater. The proximity of groundwater to the point of interest, typically the stage midpoint, is an important and often overlooked consideration. When the groundwater level is above the point of interest, it results in a net head loss that must be overcome. The head loss is a result of the buoyant forces on the grout or water injected. One can also consider this as the minimum injection pressure required to force water out of a fracture or to displace the water from within a fracture. The depth to groundwater is typically measured along the length of hole, so when angled holes are used, a correction for the hole angle is necessary.

f. Gauge Height. The height of the pressure sensing instrument above the hole collar must be considered in the effective pressure calculation, since this is the location where the gauge pressure is measured. This variable is not corrected for the hole angle since the value is simply the vertical height difference between the instrument and the hole collar.

g. Point of Interest. The point of interest is the location, origin, or reference at which all effective pressure calculations are based. It is common to use the midpoint of the stage as the point of interest. This is appropriate and recommended since it provides an average of the pressure over the length of the stage. However, in some cases, particularly the first stage in rock when grouting embankment dams, it may be appropriate to check the effective pressure at the top of the stage. This is an important dam safety issue in embankment dams, since allowable effective injection pressures in overburden are generally significantly less than those allowed in rock, and pressure will be applied near the overburden/rock interface. It is recommended that the stage midpoint be used in determining the effective pressure, but that the effective pressure at the top of stage be verified and the gauge pressure set such that the effective pressure at the top of rock does not exceed allowable values.

B-3. Head Losses and Head Gains. Head losses and head gains are determined as described in the following paragraphs, and as shown in Figure B-1.

a. Gauge Head. The pressure induced by the portion of the injection system downstream of the pressure sensor and upstream of the hole collar may either be a head gain or head loss, depending on the location of the instrument. If the instrument is located above the hole collar, it is considered a head gain. If the instrument is located below the hole collar, it is considered a head loss.

b. Column or Static Head. The pressure induced by the static head within the injection system, that is, from the hole collar to the point of interest (typically the stage midpoint), is almost always a head gain. This may include an angle correction. In the rare instance when grouting is conducted overhead, it would be considered a head loss (the head that must be overcome).

c. Groundwater Head. The pressure induced by the groundwater is either a head loss (if groundwater is located above the point of interest) or has no effect and is therefore not considered (if it is below the point of interest). Artesian pressures indicate that the groundwater level is above the hole collar and are considered a head loss.

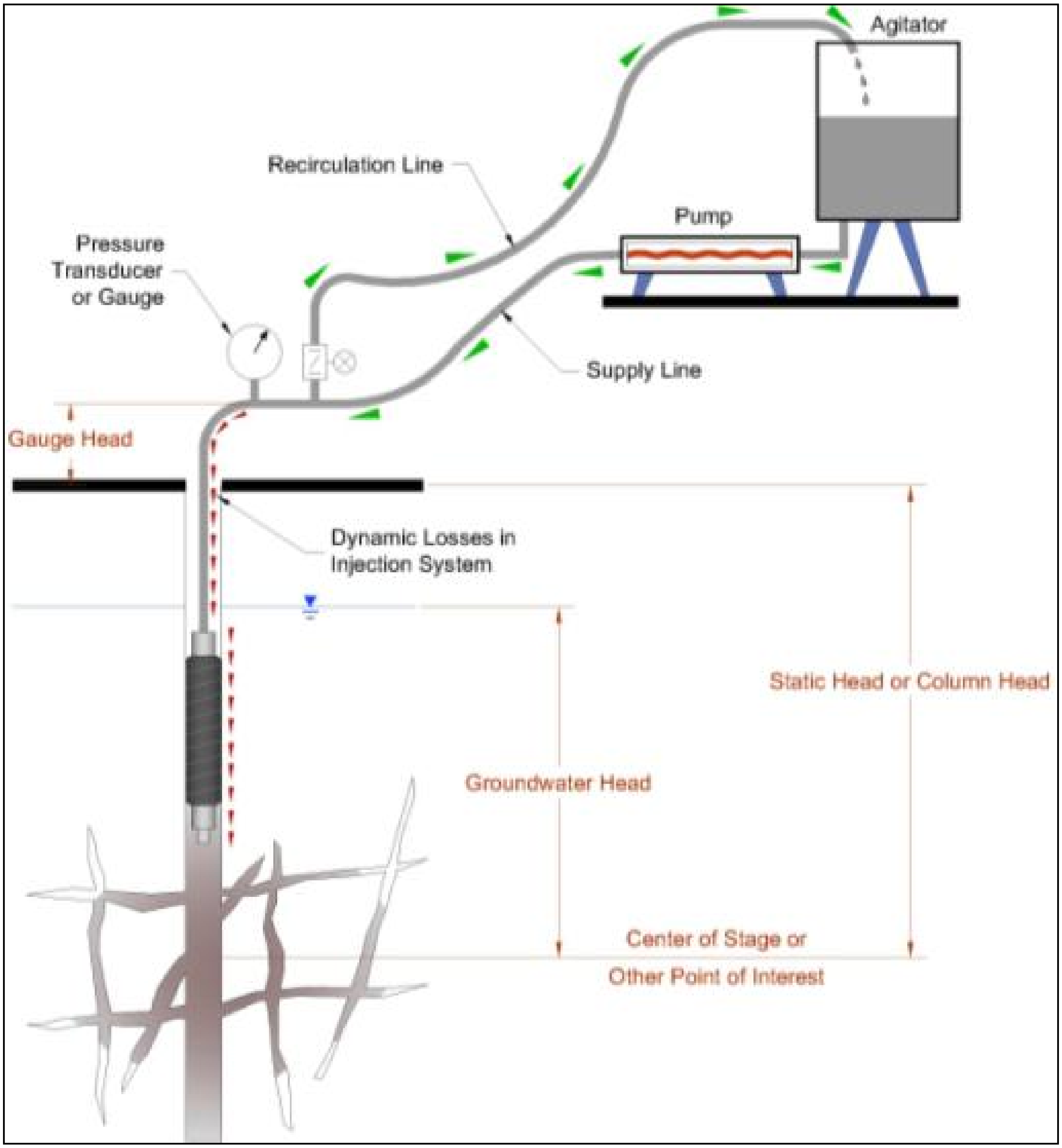

Figure B-1. Illustration of an injection system.

d. Field Procedures.

(1) Head Loss Curves. The head-loss properties of the fluid may or may not be considered in the calculations. If it is a static condition, the head loss for water is zero and is not considered. However, if it is a flowing (dynamic) condition, then pressure is lost due to friction between the fluid and the injection system, resulting in a head loss that must be considered in the calculation. A head-loss curve (the relationship of pressure lost in the injection system versus flow rate) is necessary to calculate the amount of pressure lost due to friction in the system. This also applies to grout, with

one exception. The fluid properties of the grout (in particular, the cohesion) results in some pressure required simply to initiate movement of the grout through the injection system. This is commonly referred to as the "head-loss" or the "cohesion" intercept and will vary from mix to mix (thicker mixes have higher cohesion and a corresponding higher head-loss intercept). This is evident on the head-loss curve in that the best-fit line does not intercept the Y-axis at zero. The result is a head loss that must be considered in the calculation even in the static condition.

(2) Calculating effective grouting and pressure testing pressures, and the corresponding gauge pressure required to achieve the effective pressure, is an everyday occurrence on a grouting project. Computer monitoring systems can be used to perform some or all of these calculations.

e. Calculation Methods. It is important that effective pressure calculations, such as those presented in the following paragraphs, can be rapidly reproduced in the field. Given the relative simplicity of the calculations, effective pressures may be calculated by hand using a calculator. However, on any given grouting shift upwards of a hundred calculations may be required to determine appropriate gauge pressures, so spreadsheets are typically more appropriate. If a spreadsheet is developed for setting maximum gauge pressures, which is typically the reason for calculating effective pressures separate from an automated data collection system, its accuracy should be verified by USACE. The greatest concern is that an error in the spreadsheet may result in the application of potentially damaging pressures to the ground. The inadvertent application of a pressure that is less than desired may result in minimal grout travel and may necessitate additional grout holes.

f. Calculating Gauge Pressures. The primary purpose for effective pressure calculations (separate from acquiring real-time pressure data during the grout injection or water pressure testing) is to set gauge pressures for the field header operator to maintain. Effective pressures are not measured, but calculated. The only pressure measured during an injection is the gauge pressure at the header. Since maximum effective injection pressures are commonly specified, it is necessary to calculate the gauge pressures required to achieve the target effective pressure. When testing water pressure at a single pressure, only one target effective pressure would be used, so only one gauge pressure calculation would be required. For stepped water pressure testing, a gauge pressure must be calculated for each effective pressure step in the test. Grouting may require that gauge pressures be modified throughout the course of the injection. While the geometric properties of the injection system do not change during a given injection, the injected mix may change, and the physical properties of the mixes (specific gravity and head loss intercept) may necessitate a gauge pressure change (typically gauge pressures are increased as the grout is thickened). In this situation, a gauge pressure is determined for each grout mix, and the system operator radios the new gauge pressure to the header operator whenever a mix change occurs. During injection, dynamic losses can be ignored at low-flow rates when determining the gauge pressure; otherwise the engineer would constantly be phoning the header operator with a new maximum gauge pressure. At higher flow rates, the dynamic losses might be large, and increasing the gauge pressure to offset these losses might be appropriate. As refusal is approached, gauge pressures should be calculated using the assumption that the required effective pressure is achieved under a no-flow condition.

B-4. Example Calculations. The following paragraphs give several examples of effective pressure calculations. In many cases, the intent of the calculation is to arrive at an allowable injection gauge pressure. In other cases, these same calculations may be used to calculate effective pressures and subsequently used in Lugeon value calculations.

a. Problem 1. Determine the gauge pressure required to achieve an effective pressure of 1 psi/ft in the static condition at the stage midpoint (Figure B-2). The injection fluid is water and the hole is dry.

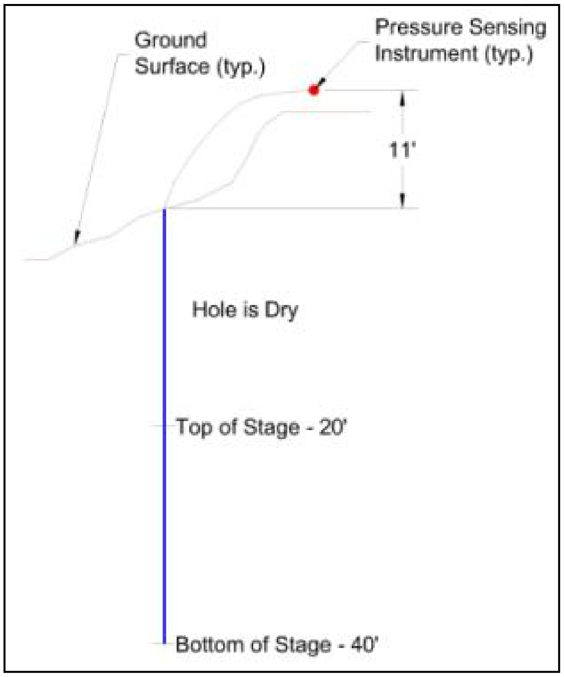

Figure B-2. Conditions for Problem 1.

(1) Gauge Head. The gauge is above the hole collar, so this is considered a head gain:

$$11 \text{ ft} \times 0.433 \ \frac{\text{psi}}{\text{ft}} = 4.8 \text{ psi}$$

(2) Column Head. The column head is a head gain and is equal to the vertical distance from the hole collar to the stage midpoint (30 ft) times the unit water pressure:

$$30 \text{ ft} \times 0.433 \ \frac{\text{psi}}{\text{ft}} = 13.0 \text{ psi}$$

(3) Required Effective Pressure. The required effective pressure is 1 psi/ft, or the vertical distance from the hole collar to the stage midpoint:

$$30 \text{ ft} \times 1 \ \frac{\text{psi}}{\text{ft}} = 30 \text{ psi}$$

(4) Required Gauge Pressure. The required gauge pressure is the required effective pressure minus all head gains and plus all head losses (none in this case):

$$30 \text{ psi} - 13.0 \text{ psi} - 4.8 \text{ psi} = 12.2 \text{ psi}$$

Rounded, the answer would be 12 psi.

b. Problem 2. Determine the gauge pressure required to achieve an effective pressure of 1 psi/ft in the static condition at the stage midpoint (Figure B-3). The injection fluid is grout with a specific gravity of 1.4.

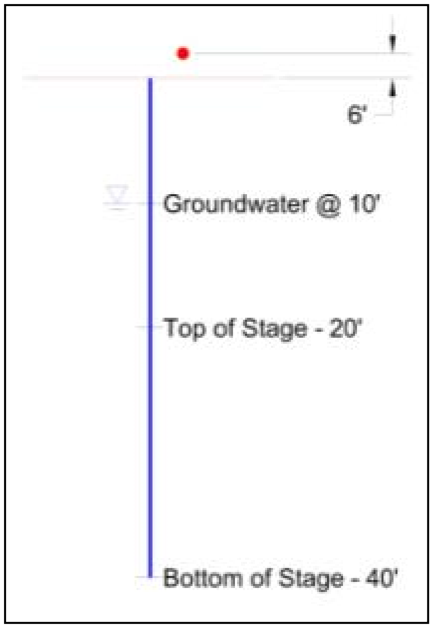

Figure B-3. Conditions for Problem 2.

(1) Gauge Head. The gauge is above the hole collar, so this is considered a head gain:

$$6\text{ ft} \times 0.433\,\frac{\text{psi}}{\text{ft}} \times 1.4 = 3.6\text{ psi}$$

(2) Column Head. The column head is a head gain and is equal to the vertical distance from the hole collar to the stage midpoint (30 ft) times the unit grout pressure:

$$30\text{ ft} \times 0.433\,\frac{\text{psi}}{\text{ft}} \times 1.4 = 18.2\text{ psi}$$

(3) Groundwater Head. Since groundwater is located above the stage midpoint, it is considered a head loss equal to the head of water above the stage midpoint times the unit water pressure:

$$20\text{ ft} \times 0.433\,\frac{\text{psi}}{\text{ft}} = 8.7\text{ psi}$$

(4) Required Effective Pressure. The required effective pressure is 1 psi/ft, or the vertical distance from the hole collar to the stage midpoint:

$$30\text{ ft} \times 1\,\frac{\text{psi}}{\text{ft}} = 30\text{ psi}$$

(5) Required Gauge Pressure. The required gauge pressure is the required effective pressure minus all head gains and plus all head losses:

$$30\text{ psi} - 3.6\text{ psi} - 18.2\text{ psi} + 8.7\text{ psi} = 16.9\text{ psi}$$

Rounded, the answer would be 17 psi.

c. Problem 3. Determine the gauge pressure required to achieve an effective pressure of 1 psi/ft in the static condition at the stage midpoint (Figure B-4). The injection fluid is grout with a specific gravity of 1.4.

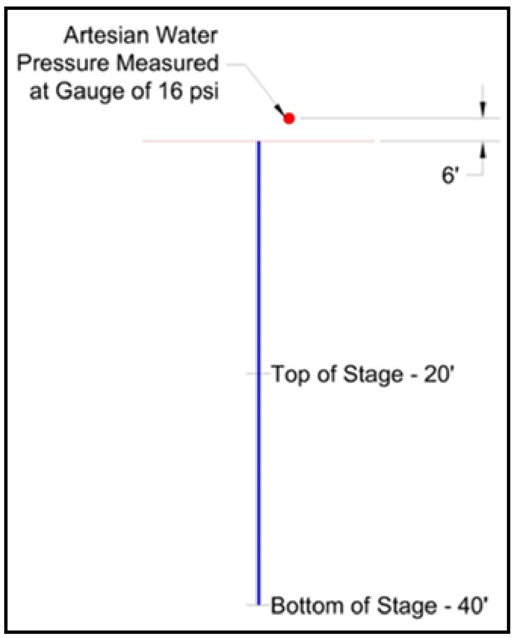

Figure B-4. Conditions for Problem 3.

(1) Gauge Head. The gauge is above the hole collar, so this is considered a head gain:

$$6\text{ ft} \times 0.433\,\frac{\text{psi}}{\text{ft}} \times 1.4 = 3.6\text{ psi}$$

(2) Column Head. The column head is a head gain and is equal to the vertical distance from the hole collar to the stage midpoint (30 ft) times the unit grout pressure:

$$30\text{ ft} \times 0.433\,\frac{\text{psi}}{\text{ft}} \times 1.4 = 18.2\text{ psi}$$

(3) Groundwater Head. An artesian condition is present. The groundwater head that must be overcome (the head loss) is equal to the distance from the stage midpoint to the gauge times the unit water pressure plus the artesian pressure measured at the gauge:

$$(30\text{ ft} + 6\text{ ft}) \times 0.433\,\frac{\text{psi}}{\text{ft}} + 16\text{ psi} = 31.6\text{ psi}$$

(4) Required Effective Pressure. The required effective pressure is 1 psi/ft, or the vertical distance from the hole collar to the stage midpoint:

$$30\text{ ft} \times 1\,\frac{\text{psi}}{\text{ft}} = 30\text{ psi}$$

(5) Required Gauge Pressure. The required gauge pressure is the required effective pressure minus all head gains and plus all head losses:

$$30\text{ psi} - 3.6\text{ psi} - 18.2\text{ psi} + 31.6\text{ psi} = 39.8\text{ psi}$$

Rounded, the answer would be 40 psi.

d. Problem 4. Determine the gauge pressure required to achieve an effective pressure of 1.5 psi/ft in the static condition at the stage midpoint (Figure B-5). The injection fluid is water.

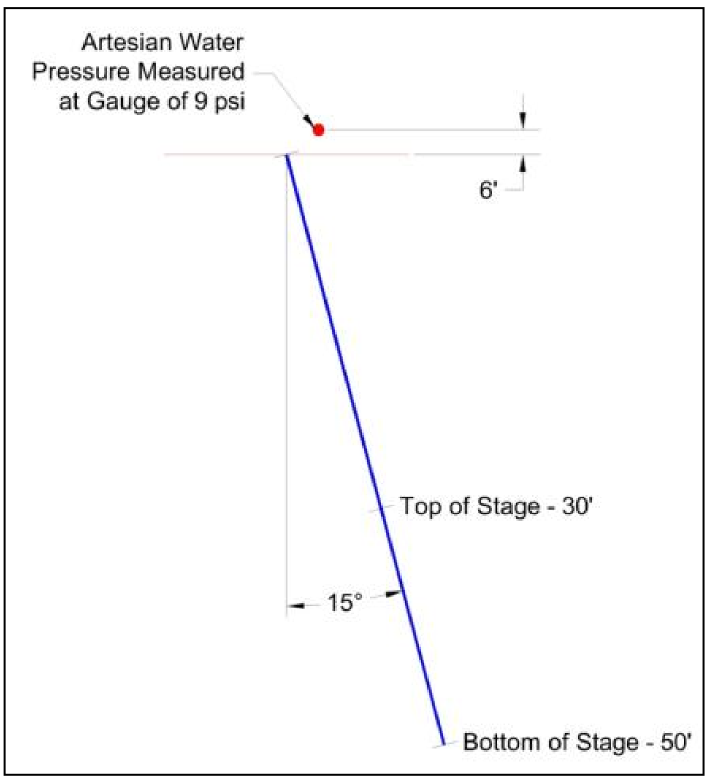

Figure B-5. Conditions for Problem 4.

(1) Gauge Head. The gauge is above the hole collar, so this is considered a head gain:

$$6\text{ ft} \times 0.433\,\frac{\text{psi}}{\text{ft}} = 2.6\text{ psi}$$

(2) Column Head. The column head is a head gain and is equal to the vertical component of the distance from the hole collar to the stage midpoint times the unit water pressure:

$$40\text{ ft} \times \cos 15° \times 0.433\,\frac{\text{psi}}{\text{ft}} = 16.7\text{ psi}$$

(3) Groundwater Head. An artesian condition is present. The groundwater head that must be overcome (the head loss) is equal to the vertical distance from the stage midpoint to the gauge times the unit water pressure plus the artesian pressure measured at the gauge:

$$40\text{ ft} \times \cos 15° \times 0.433\,\frac{\text{psi}}{\text{ft}} \times 6\text{ ft} \times 0.433\,\frac{\text{psi}}{\text{ft}} + 9\text{ psi} = 28.3\text{ psi}$$

(4) Required Effective Pressure. The required effective pressure is 1.5 psi/ft, or the vertical distance from the hole collar to the stage midpoint:

$$40\text{ ft} \times \cos 15° \times 1.5\,\frac{\text{psi}}{\text{ft}} = 58.0\text{ psi}$$

(5) Required Gauge Pressure. The required gauge pressure is the required effective pressure minus all head gains and plus all head losses:

$$58.0\text{ psi} - 2.6\text{ psi} - 16.7\text{ psi} + 28.3\text{ psi} = 67.0\text{ psi}$$

Rounded, the answer would be 67 psi.

e. Problem 5. Determine the gauge pressure required to achieve an effective pressure of 1 psi/ft in the static condition (at refusal) at the stage midpoint (Figure B-6). The injection fluid is B-mix grout with a specific gravity of 1.32 (Figure B-7).

(1) Gauge Head. The gauge is below the hole collar, so this is considered a head loss:

$$13\text{ ft} \times 0.433\,\frac{\text{psi}}{\text{ft}} \times 1.32 = 7.4\text{ psi}$$

(2) Column Head. The column head is a head gain and is equal to the vertical component of the distance from the hole collar to the stage midpoint times the unit water pressure:

$$42.5\text{ ft} \times \cos 15° \times 0.433\,\frac{\text{psi}}{\text{ft}} \times 1.32 = 23.5\text{ psi}$$

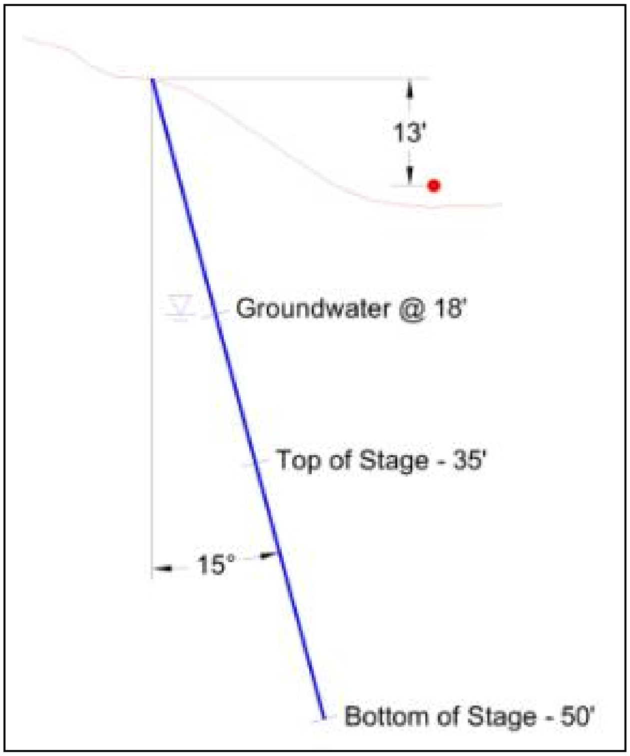

Figure B-6. Conditions for Problem 5.

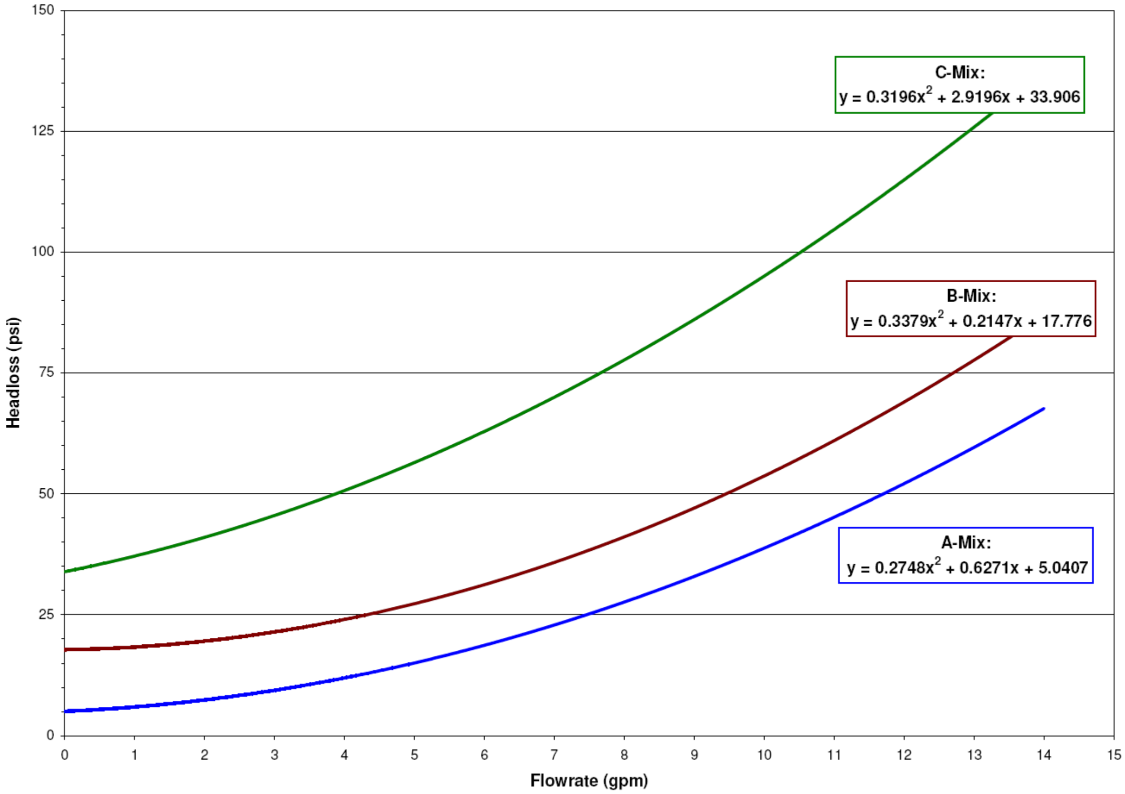

Figure B-7. Head-loss summary for grout.

(3) Groundwater Head. The groundwater head that must be overcome (the head loss) is equal to the vertical distance from the stage midpoint to the observed groundwater level times the unit water pressure:

$$(42.5\text{ ft} - 18\text{ ft}) \times \cos 15° \times 0.433\,\frac{\text{psi}}{\text{ft}} = 10.2\text{ psi}$$

(4) Head-Loss Intercept. The head-loss intercept is a head-loss. The value is equivalent to the location where the B-mix best-fit curve in Figure B-7. intercepts the Y-axis. From the equation from best-fit curve, this is 17.8 psi.

(5) Required Effective Pressure. The required effective pressure is 1 psi/ft, or the vertical distance from the hole collar to the stage midpoint:

$$42.5\text{ ft} \times \cos 15° \times 1\,\frac{\text{psi}}{\text{ft}} = 41.0\text{ psi}$$

(6) Required Gauge Pressure. The required gauge pressure is the required effective pressure minus all head gains and plus all head losses:

$$41.0\text{ psi} + 7.4\text{ psi} - 23.5\text{ psi} + 10.2\text{ psi} + 17.8\text{ psi} = 52.9\text{ psi}$$

Rounded, the answer is 53 psi.

f. Problem 6. Determine the gauge pressure required to achieve an effective pressure of 1 psi/ft in the dynamic condition with a flow rate of 10 gpm at the stage midpoint (Figure B-8). The injection fluid is B-mix grout with a specific gravity of 1.32.

(1) Gauge Head. The gauge is below the hole collar, so this is considered a head loss:

$$13\text{ ft} \times 0.433\,\frac{\text{psi}}{\text{ft}} \times 1.32 = 7.4\text{ psi}$$

(2) Column Head. The column head is a head gain and is equal to the vertical component of the distance from the hole collar to the stage midpoint times the unit water pressure:

$$42.5\text{ ft} \times \cos 15° \times 0.433\,\frac{\text{psi}}{\text{ft}} \times 1.32 = 23.5\text{ psi}$$

(3) Groundwater Head. The groundwater head that must be overcome (the head loss) is equal to the vertical distance from the stage midpoint to the observed groundwater level times the unit water pressure:

$$(42.5\text{ ft} - 18\text{ ft}) \times \cos 15° \times 0.433\,\frac{\text{psi}}{\text{ft}} = 10.2\text{ psi}$$

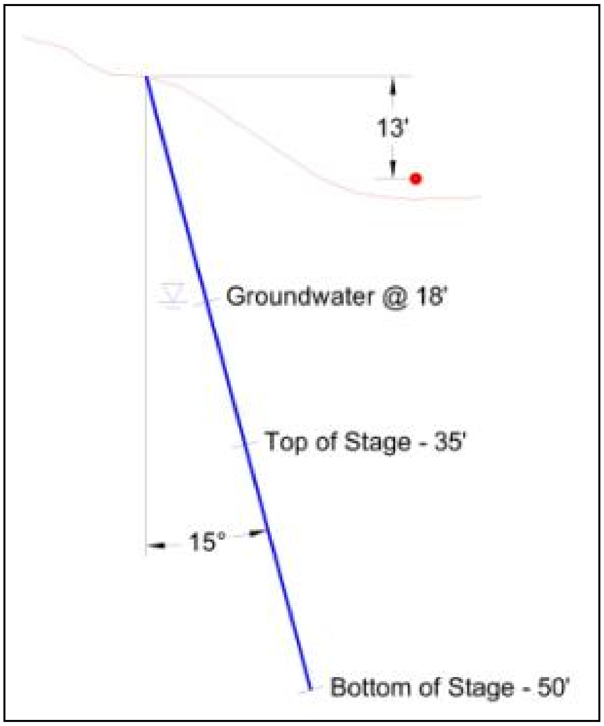

Figure B-8. Conditions for Problem 6.

(4) Dynamic Head Loss. The head loss is a function of the flow rate. The relationship is the equation of the best-fit curve for the head loss, as shown above. In the polynomial function used, the X value is the flow rate and the Y value is the corresponding head loss at that flow rate. Dynamic loss is calculated as:

$$\text{Head loss psi} = 0.3379 \times (10\text{ gpm})^2 + 0.2147 \times (10\text{ gpm}) + 17.776\text{ psi} = 53.7$$

(5) Required Effective Pressure. The required effective pressure is 1 psi/ft, or the vertical distance from the hole collar to the stage midpoint:

$$42.5\text{ ft} \times \cos 15° \times 1\,\frac{\text{psi}}{\text{ft}} = 41.0\text{ psi}$$

(6) Required Gauge Pressure. The required gauge pressure is the required effective pressure minus all head gains and plus all head losses:

$$41.0\text{ psi} + 7.4\text{ psi} - 23.5\text{ psi} + 10.2\text{ psi} + 53.7\text{ psi} = 88.8\text{ psi}$$

Rounded, the answer would be 89 psi.

APPENDIX C

Glossary of Terms and Abbreviations

Alkali–Aggregate Reaction
Chemical reaction in grout between alkalies (sodium and potassium) from portland cement or other sources and certain constituents of some aggregates. Under certain conditions, deleterious expansion of the grout may result.

Aquiclude
A body of relatively impermeable rock or soil that is capable of absorbing water slowly, but functions as an upper or lower boundary of an aquifer and does not transmit groundwater rapidly enough to supply a well or spring.

Aquifer
A stratum or zone below the surface of the earth capable of producing water as from a well.

Aquitard
A confining bed that retards, but does not prevent, the flow of water to or from an adjacent aquifer; a leaky confining bed.

Area Grouting
Grouting of a shallow zone in a particular area that uses holes arranged in a pattern or grid. This type of grouting is often referred to as “blanket” or “consolidation” grouting.

Bentonite
A clay composed principally of minerals of the montmorillonite group, characterized by high adsorption and very large volume change with wetting.

Blanket Grouting
See Area Grouting.

Bursting Pressure (Grouting Equipment)
The pressure at which equipment becomes inoperative.

Cement Factor
Quantity of cement contained in a unit volume of grout, expressed as weight or volume.

Cementitious Factor
Quantity of cement and other cementitious materials contained in a unit volume of concrete, grout, or mortar, expressed as weight or volume.

Circuit Grouting
Grouting in a continuous manner with a grout circulating from the pump to the bottom of the zone to be treated and back to the pump.

Coefficient of Permeability (Laboratory) (to Water)
The rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions, usually 68 °F (20 °C).

Colloidal Grout
A grout that has an artificially induced cohesiveness, or the ability to retain the dispersed solid particles in suspension, i.e., a grout mixture that does not settle or bleed.

Consolidation Grouting
See Area Grouting.

Effective Pressure
The sum of all head losses and head gains in the injection system and the ground.

False Set
The rapid development of rigidity in a freshly mixed grout without the evolution of much heat. Such rigidity can be dispelled and plasticity can be regained by further mixing without the addition of water. Premature stiffening, hesitation set, early stiffening, and rubber set are other terms that refer to the same phenomenon.

Final Set
A degree of stiffening of a grout mixture indicating the time in hours and minutes required for cement paste to stiffen sufficiently to resist the penetration of a weighted test needle (Vicat needle).

Flash Set
The rapid development of rigidity in a freshly mixed grout, usually with the evolution of considerable heat, and the rigidity cannot be dispelled nor can plasticity be regained by further mixing without the addition of water, also referred to as “quick set” or “grab set.”

Free Water
Water that is free to move through a soil mass under the influence of gravity. Other terms are gravitational water, groundwater, and phreatic water.

Grout
A mixture of cementitious or non-cementitious material, with or without aggregate, to which sufficient water or other fluid is added to produce a flowing consistency.

Grout Placement
The introduction of grout by gravity or pressure, usually accomplished by grouting through pipes placed in the medium to be grouted or through drilled open holes penetrating the medium.

Grout Take
The volume of grout placed.

Heat of Hydration
Heat generated by chemical reactions of cementitious materials with water, such as that generated during the setting and hardening of portland cement.

Hydrofracturing
The fracturing of an embankment or underground stratum by pumping water under a pressure in excess of the tensile strength and minor principal stress.

Hydrostatic Head
The pressure produced by the height of a fluid above a given point.

Initial Set
A degree of stiffening of a grout mixture indicating the time in hours and minutes required for cement paste to stiffen sufficiently to limit penetration of a weighted test needle (Vicat needle) to 25 mm.

Neat Cement Grout
A fluid mixture of cement and water or the hardened equivalent of such mixtures. Also called neat slurry.

Packer
An expandable mechanical or pneumatic device used to seal a hole or isolate portions of a hole.

Perched Groundwater
Any groundwater separated by unsaturated rock from an underlying body of groundwater.

Perched Water Table
The water table above an impermeable bed underlain by unsaturated rock or soil of sufficient permeability to allow movement of groundwater.

Permeability (Laboratory) (Coefficient of, to Water)
See Coefficient of Permeability.

Pore Pressure
Stress transmitted through the pore water (water filling voids). Also called neutral stress and pore-water pressure.

Pressure Testing
A test performed to measure the rate at which water can be forced into a hole under a specific pressure.

Pressure Washing
A process of washing between holes to remove mud and loose material from cracks and seams in the rock. In effect, it is a sluicing operation whereby water or air and water alternately are introduced under pressure into a hole and allowed to vent into adjacent cracks or escape from one or more adjacent holes.

Primary Holes
The first series of holes to be drilled and grouted, usually at the maximum allowable spacing.

Primary Permeability
The permeability of intact rock, rather than permeability due to fracturing.

Primary Porosity
The porosity that develops during final stages of sedimentation or that was present within the sedimentary particles at the time of deposition.

Refusal
The point during grout injection when little or no grout is accepted under the maximum allowable pressure or other specified conditions.

Rheology
The science of deformation and flow of matter.

Secondary Holes
The second series of holes to be drilled and grouted, spaced midway between primary holes.

Section
A linear or a real subdivision of the grout treatment pattern without regard to the depth of treatment.

Seep
An area where water oozes from the earth.

Series Grouting
Similar to stage grouting, except that each successively deeper zone is grouted by means of a newly drilled hole, eliminating the need for washing grout out before drilling the hole deeper.

Split Spacing
The procedure by which additional grout injection holes are located equidistant from previously grouted holes.

Stage
One complete operational cycle of drilling, cleaning, pressure washing, pressure testing, pressure grouting, and grout cleanout within a zone. The depths of stages in any hole depend on conditions encountered in drilling that dictate where drilling should stop and grouting commence.

Stage Grouting
The grouting of progressively deeper zones in stages. Previously emplaced grout is removed before hardening, the hole is drilled to a deeper depth, and another stage is emplaced. Where upstage grouting is used, the hole is drilled to full depth and grouted in stages starting at the bottom of the hole and working upward. For downstage grouting, the hole is drilled for a single

stage, which is then tested and grouted. After the grout in this stage has set, the hole is drilled to the next stage and grouted. This process is continued until the hole has reached its final depth.

Stop Grouting
The grouting of a hole beginning at the lowest zone (bottom) after the hole is drilled to total depth. Packers are used to isolate the zone to be grouted.

Sulfate Attack
Harmful or deleterious reactions between sulfates in soil or groundwater and grout.

Tertiary Hole
The third series of holes to be drilled and grouted, spaced midway between previously grouted primary and secondary holes.

Thixotropy
The property of a material that enables it to stiffen in a short period of standing and to regain its initial viscosity by mechanical agitation; the process is reversible.

Time of Setting
1. Final setting time. The time required for a freshly mixed grout to achieve final set (harden).
2. Initial setting time. The time required for a freshly mixed grout to achieve initial set.

Unit Weight
The weight of freshly mixed grout per unit volume, often expressed as pounds per cubic foot.

Viscosity
Friction within a liquid due to mutual adherence of its particles, i.e., the “thickness” of a mixture.

Void Ratio
The ratio of the volume of void space to the volume of solid particles in a given soil mass.

Washing
The physical act of cleaning a hole by circulating either water, water and air, acid washes, or water and dissolved chemical substances through drill rods or tremie pipe in an open hole.

Water-Cement Ratio (Cement Only)
The ratio of the amount of water to the amount of cement in a grout mixture, expressed by weight or volume.

Water-Cement Ratio (Total Cementitious Materials)
The ratio of the amount of water to the amount of total cementitious materials in a grout mixture, expressed by weight or volume.

Water Table
The upper surface of a saturation zone, except where that surface is formed by an impermeable body.

Working Pressure
The pressure determined best for any particular set of conditions encountered during grouting. Factors influencing the determination are size of voids to be filled, depth of zone to be grouted, lithology of the area to be grouted, grout viscosity, and resistance of the formation to fracture or uplift.

Zone
A predetermined subdivision of the overall depth of grout treatment. A single zone may make up the full depth of treatment, or the depth of treatment may be divided into several zones.

APPENDIX D

Acronyms

Term	Definition
API	American Petroleum Institute
ASCE	American Society of Civil Engineers
ATR	Agency Technical Review
BCOE	Biddability, Constructability, Operability, and Environmental Review
CADD	Computer-Aided Drafting and Design
COR	Contract Officer Representative
DA	Department of the Army
DC	District of Columbia
DQC	District Quality Control
EC	Engineer Circular
EM	Engineer Manual
EOP	Environmental Operating Principle
FHWA	Federal Highway Administration
GIS	Geographic Information System
GMS	Groundwater Modeling System
HMG	High-Mobility Grout
HQUSACE	Headquarters, U.S. Army Corps of Engineers
IEPR	Independent External Peer Review
LMG	Low- (or Limited-) Mobility Grout (or Grouting)
NRC	National Research Council
PED	Preconstruction Engineering and Design (Phase)
QA	Quality Assurance
QC	Quality Control
RQD	Rock Quality Designation
SDS	Strategic Decisions & Solutions (Corp.)
TVA	Tennessee Valley Authority
U.S.	United States
USACE	U.S. Army Corps of Engineers

Term	Definition
USDOI	U.S. Department of the Interior
USBR	U.S. Bureau of Reclamation
USCS	Unified Soils Classification System
USGS	U.S. Geological Survey