RECLAMATION
Managing Water in the West

Design Standards No. 13

Embankment Dams

Chapter 15: Foundation Grouting
Phase 4 Final

U.S. Department of the Interior
Bureau of Reclamation

September 2014

Mission Statements

The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Design Standards Signature Sheet

Design Standards No. 13

Embankment Dams

DS-13(15)-1: Phase 4 Final
September 2014

Chapter 15: Foundation Grouting

Foreword

Purpose

The Bureau of Reclamation (Reclamation) design standards present technical requirements and processes to enable design professionals to prepare design documents and reports necessary to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public. Compliance with these design standards assists in the development and improvement of Reclamation facilities in a way that protects the public's health, safety, and welfare; recognizes needs of all stakeholders; and achieves lasting value and functionality necessary for Reclamation facilities. Responsible designers accomplish this goal through compliance with these design standards and all other applicable technical codes, as well as incorporation of the stakeholders' vision and values, that are then reflected in the constructed facilities.

Application of Design Standards

Reclamation design activities, whether performed by Reclamation or by a non-Reclamation entity, must be performed in accordance with established Reclamation design criteria and standards, and approved national design standards, if applicable. Exceptions to this requirement shall be in accordance with provisions of *Reclamation Manual Policy*, Performing Design and Construction Activities, FAC P03.

In addition to these design standards, designers shall integrate sound engineering judgment, applicable national codes and design standards, site-specific technical considerations, and project-specific considerations to ensure suitable designs are produced that protect the public's investment and safety. Designers shall use the most current edition of national codes and design standards consistent with Reclamation design standards. Reclamation design standards may include exceptions to requirements of national codes and design standards.

Proposed Revisions

Reclamation designers should inform the Technical Service Center (TSC), via Reclamation's Design Standards Website notification procedure, of any recommended updates or changes to Reclamation design standards to meet current and/or improved design practices.

Chapter Signature Sheet
Bureau of Reclamation
Technical Service Center

Design Standards No. 13

Embankment Dams

Chapter 15: Foundation Grouting

DS-13(15) - Phase 4 Final
September 2014

Chapter 15, "Foundation Grouting," is a new chapter within *Design Standards No. 13 – Embankment Dams.*

This design standard chapter presents Bureau of Reclamation best practices for foundation grouting beneath embankment dams. The procedures discussed in this design standard are based on many years of shared foundation grouting experience within Reclamation gained throughout the arid and semi-arid 17 Western States. The best practices summarized in this design standard have been successfully implemented throughout Reclamation for decades.

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Chapter 15

Foundation Grouting

15.1 Introduction

15.1.1 Purpose

This design standard chapter presents Bureau of Reclamation (Reclamation) best practices for foundation grouting beneath embankment dams. The procedures discussed in this design standard are based on many years of shared foundation grouting experience within Reclamation gained throughout the arid and semi-arid 17 Western States. The best practices summarized in this design standard have been successfully implemented throughout Reclamation for decades. Seepage performance data at Reclamation dams that were grouted using these best practices indicate that they have been very effective in limiting underseepage and excessive uplift pressures beneath embankment dams.

15.1.2 Scope

Grouting is widely used in civil works projects for the purpose of foundation improvement. There are many applications for foundation grouting, such as water control, strengthening the foundation, or seepage reduction. The type of grout and equipment used to inject the grout for each application and foundation conditions varies substantially. This design standard chapter only discusses foundation grouting of rock formations beneath embankment dams using cement based grouts.

Foundation grouting for embankment dam foundation improvement is a process of injecting cementitious slurries under pressure into the underlying rock formations through specially drilled holes for the purpose of filling joints, fractures, fissures, bedding planes, cavities, or other openings. Grouting is generally used to reduce erosive leakage, excessive uplift pressure, and high water losses through the foundation rock. This use generally applies to the design of new dams, but grouting can also be used as a remedial measure to help control seepage at existing dams.

Foundation grouting for embankment dams includes curtain grouting, blanket grouting, and stitch grouting. Figure 15.1.2-1 illustrates curtain grouting, blanket grouting, and stitch grouting on a recent Reclamation project. Curtain grouting is probably the most common method of foundation seepage reduction used beneath new dams. This method consists of drilling holes into the foundation bedrock at some regular spacing along a line or lines parallel to the dam axis and normal to the seepage flow direction. In cases where fractured rock exists at the foundation contact, blanket grouting is often used to provide a firm foundation, to reduce seepage within the near-surface foundation bedrock, to reduce seepage from the embankment into the foundation, and to reduce the likelihood of internal erosion of the embankment materials into the foundation. Stitch grouting is used to seal isolated pervious discontinuities exposed on the foundation surface on an as-needed basis within or near the impervious core footprint.

Figure 15.1.2-1. Overview of grouting operations at Ridges Basin Dam.

Foundation grouting is an engineering process that must be designed and planned for in the office by an experienced designer, grouting specialist, and geologist. The design must then be executed by competent field personnel. Every foundation grouting project presents a unique set of conditions. For this reason, this design standard and all other grouting textbooks and manuals should only be used as a guide. Explicit rules pertaining to drilling and grouting methodology, including design, grout materials, and grouting equipment, may be inappropriate in some instances due to differing geological conditions. Adaptation to site-specific geologic conditions is always necessary. The grouting design and specifications must be flexible to adjust to the conditions observed in the field during grouting. The design of a foundation grouting program is not complete once the layout, depth, and spacing of the grout holes are laid out on a design drawing. A successful foundation grouting project requires that engineering decisions be made on a stage-by-stage and hole-by-hole basis. The design of a foundation grouting program is not complete until the last stage in the last grout hole has been grouted.

There are many references available on foundation grouting. This design standard provides limited references on the subject because this design standard focuses on presenting Reclamation's best practices. For a more basic understanding of grouting, Houlsby [1] provides a hands-on manual that explains grouting design, drilling, explorations, equipment, procedures, how to handle potential problems, and many other subjects.

It should be noted that foundation grouting should not be considered as a cure-all for all embankment dam foundation issues, particularly when selecting an alternative to address a dam safety issue for an existing embankment. Careful consideration should be given to other alternatives, or alternatives to be used in conjunction with grouting, that may provide a more reliable method for improving the geologic conditions beneath an embankment dam.

15.2 When Is Grouting Necessary

For a new embankment dam placed on a rock foundation, a grout curtain is almost always required. Preconstruction geologic site investigations are often limited and provide information on a very small percentage of the foundation. Even in circumstances where the foundation permeability is known to be low, a grout curtain should be incorporated in a design for a new embankment to provide high confidence that the actual foundation conditions observed during construction match the design assumptions and possibly limited preconstruction geologic information.

Reclamation experience indicates that the groutability of rock formation with a permeability less than 1.0×10^{-5} centimeters per second [cm/s] (10 feet per year [ft/yr] or 1.3 Lugeon Units) using cement-based grouts is typically very limited. Recommendations for grout hole spacing and depth discussed in this design standard can be altered if foundation conditions are believed to be ideal and of low permeability, such as tight fractures with limited connectivity and permeability, and if there is high confidence in the results of geologic investigations over the entire foundation.

The need for blanket grouting should be evaluated during design of a new embankment. In cases where fractured or jointed rock lie at the foundation contact, and they are not removed as part of the foundation cleanup, blanket grouting is used to provide a firm foundation and to reduce seepage from the embankment into the foundation and along the foundation contact. Subsequently, the likelihood of internal erosion of the impervious core into the foundation and along the embankment/foundation contact is reduced. In cases where high quality rock is believed to be present at the embankment/foundation contact, provisions for blanket grout holes (and/or stitch grouting) can be provided for in the specifications to seal any isolated fractures exposed during foundation cleanup or for conditions that deviate from the design assumptions. Additional surface treatment recommendations are provided in Chapter 3, "Foundation Surface Treatment [2]," of *Design Standards No. 13 – Embankment Dams*.

Water pressure tests of exploratory holes are the most commonly used method of measuring permeability and evaluating groutability of a foundation, even though the quantity of water injected into a stage is not necessarily indicative of the grout quantity that can be injected. For example, a porous sandstone formation may possess high permeability characteristics and accept water readily but refuse to accept a particulate grout (cement) due to the small size of the voids. Permeability testing in differing geologic conditions, such as one large crack in a stage or many fine cracks in another stage, may result in high water 'takes[1] in both cases; however, the subsequent grout 'takes' may vary considerably. The primary and secondary permeability of the rock mass also need to be considered. Primary permeability refers to the permeability of the interconnecting pore spaces within the rock mass. Secondary permeability refers to the permeability of the rock mass and is typically a measure of the permeability of the discontinuities such as open bedding planes, fractures, or joints within the rock mass. Consequently, water tests without consideration of geologic factors may be inaccurate in predicting grout 'takes.'

The groutability and amount of foundation grouting required can be greatly underestimated if erroneous permeability data is used to estimate the groutability of a foundation. Standard water testing procedures, which include pressures comparable to those used during grouting, water pumps, supply lines and packers capable of high volume injection, and standard hole sizes and stages, must be used to estimate the permeability of a formation. The drill holes used in permeability testing should be oriented to intercept the maximum number of discontinuities possible.

[1] This chapter uses the term 'takes' when referring to water or grout that is injected into rock during permeability testing or foundation grouting, respectively. The word takes is in single quotes to differentiate it from the normal usage of the word.

15.3 Design Considerations

It is often difficult to quantify the effects of foundation grouting within a seepage analysis. When laying out the design and closure criteria for a foundation grouting project for an embankment dam, some additional factors that should be taken into account include:

- A properly designed seepage collection system is always necessary within an embankment dam. Foundation grouting may not eliminate foundation seepage and may not eliminate the presence of high piezometric pressures beneath the embankment. Seepage beneath the embankment must be safely collected and monitored. The potential for high piezometric pressure beneath the embankment must be evaluated and accounted for during design. Small fractures that may not accept grout can still transmit water pressure downstream of the grout curtain.

- The extent of grouting may be related to the tolerable seepage losses for the project. In areas where water is a limited resource and almost no seepage can be tolerated, the costs for an extensive grouting program can easily be justified.

- The extent of grouting may be related to future operation and maintenance (O&M) issues for the project. If excessive seepage over time or high pore pressures are expected to result in the need for rock stabilization, increased monitoring, instrumentation, seepage collection systems, etc., the costs associated with a grouting program may be justified when considering the potential long-term O&M costs.

- An evaluation should be performed to determine the effects of the grouting program on the potential failure modes. Some considerations include the following:

  - What is the piezometric pressure within the foundation upstream and downstream of the grout curtain?

  - What effect will the grout curtain have on the piezometric surface within the core of the embankment?

  - What effect will the grouting have on seepage along the embankment/foundation contact? Is there a potential for grouting to result in increased seepage at the embankment/foundation contact?

  - What is needed to prevent the migration of the embankment materials into the foundation openings?

   - Will grouting increase the likelihood of any of the potential failure modes?

   - What would be the effect of any "window" in the grout curtain?

- When performing seepage analyses to evaluate foundation seepage beneath an embankment dam, the permeability of a well-built grout curtain(s) is likely to be about $1x10^{-5}$ cm/s or less. The width of the grout curtain used in the analysis should be determined by considering how many grout curtain rows are incorporated into the design, the geology, the closure criteria selected for the grout curtains, results of verification testing, and the suite of grout mixes used for construction. Water tests performed in grout verification holes are likely to be the best source of data for establishing a permeability value of the grouted rock mass.

15.4 Preconstruction Geologic Site Investigations

Understanding the geologic characteristics of a damsite is a key part of the successful design and construction of a foundation grouting program. The path(s) that grout follows when injected into rock is a direct function of the type of rock and its structural characteristics, such as joint openness, spacing, and continuity of features. Detailed geologic data of the rock or formation, such as age, origin, character, structure, properties, dips and strikes, size and location of faults, fractures, bedding planes, joints, seams and cavities, and the knowledge accumulated from past experience in similar geologic settings, all help to form the basis of a foundation grouting program design. Knowledge accumulated from past experience under similar geologic settings should also be used to interpret the exploratory data. Understanding the site geology gives the design staff and construction personnel the ability to plan for, recognize, and react to actual field conditions as construction progresses.

When determining the extent of grouting required for a project, the designer must work with the project geologist and a grouting specialist (engineer) to analyze the in situ properties of the foundation, particularly the competency and overall permeability of the foundation. It is equally important for the designer to understand the impacts a grouting program will have on local groundwater conditions. Successfully reducing and controlling seepage beneath an embankment can result in higher pressures or new seepage at other locations and should be a design consideration for any grout program.

The rock type can have a significant impact on grouting. A strong stable rock, such as a quartzite, will usually behave in a more predictable manner than a weak rock, like a shale or claystone. Some shales and claystones exhibit high shrink-swell characteristics and may close during permeability testing. Volcanic rocks, such as basalts, may have very complex fracture and jointing due to highly variable cooling and stress relief. Flow tubes in basalt may accept large quantities of grout. Likewise, karstic terrain in soluble rocks, such as limestone, could have extensive formation of secondary ground water paths. These consist of solution joints, vugs, sinkholes, caverns, and large-scale collapse features. Impounded reservoirs adjacent to karstic terrain could activate or reactivate dissolution of bedrock.

15.4.1 Surface Geology

Determining the grouting characteristics of a rock mass requires an understanding of the surface and subsurface geologic conditions. An investigation program should include detailed mapping of the surface geology at the damsite. Geologic mapping should include the footprint of the proposed dam, areas upstream and downstream of the damsite, adjacent ridges, and any other areas where seepage could potentially contact or bypass the embankment. This should include identifying rock units, which may not be exposed within the dam footprint but may daylight upstream or downstream of the dam in the valley floor or abutments. These units could also represent a potential seepage path under or around a dam. An understanding of the regional geologic structure can help identify potential weaknesses in rock that may not be visually apparent. Sometimes these weaknesses are not identified until testing is conducted or during grouting.

Exposed bedrock should be mapped to determine its extent, rock type, physical characteristics, and structural features including joints, bedding, foliation, and other discontinuities. Field mapping of joints and discontinuities for attributes that could affect grouting should include the trend (or strike), dips, spacing, continuity, openness, infillings, and weathering. Weathering of rock exposed at the surface often results in a significant reduction in the hardness and strength. This effect can be more pronounced along joints and discontinuities so that the grout requirements of weathered rock can be considerably different than less weathered rock in the subsurface.

15.4.2 Joints and Other Discontinuities

While rock porosity (primary permeability) may, in extreme cases, permit grout travel, it is generally not a factor in determining the permeability of a rock formation for a foundation grouting program. In most damsite foundations, seepage often follows joints and other discontinuities within the rock. Therefore, grout holes should be designed to intercept the maximum number of joints and other discontinuities as is possible per foot of hole drilled at an inclination less than or equal to 30 degrees from vertical. Grout holes should never be drilled flatter than 30 degrees from vertical, except fan holes at the end of the grout curtain. The strike and dip of the dominant joint sets play a primary role in determining the inclination and direction at which the grout holes are drilled. Inclined bedding or other rock structure should also be taken into consideration when determining the orientation of grout holes. When primary permeability is present, water tests must be carefully interpreted, taking primary permeability into consideration.

The degree to which a rock mass will accept grout is determined by the overall permeability of the rock formation. The permeability of a rock mass is related principally to the presence of rock discontinuities such as open joints, faults, shear zones, solution cavities, and interconnected fractures. Joint openness and continuity determine how easily and how far grout will travel in the rock mass, although continuity plays less of a role in grouting than with seepage due to comparatively lower travel distance of grout. Joint spacing combined with openness and continuity can give an indication of how difficult it will be to grout the rock formation at a damsite.

Reclamation's *Engineering Geology Field Manual* [3] contains detailed information for the mapping of discontinuity features and the engineering properties of rock.

15.4.3 Cored Holes

Reclamation typically uses diamond bit core drilling to investigate foundation suitability for grouting. Recovered rock core provides details on subsurface jointing and structure, allows more precise seating and placement of packers for permeability testing, provides a relatively smooth borehole wall for geophysical testing and camera inspection, and provides samples for physical properties testing. Prior to beginning a subsurface investigation program for grouting, a thorough review should be made of existing surface and subsurface data.

In the initial stages of a subsurface investigations program, the drill holes should extend to slightly deeper than 1.0 times the hydraulic head above the surface of the bedrock. Usually, a few holes are drilled to at least twice the maximum hydraulic head to investigate foundation bedrock directly beneath the grout curtain. Both vertical and inclined drill holes are needed to best determine foundation grouting needs. As bedrock foundation conditions are better understood, drilling is conducted at specific bearings, inclinations, and depths to test permeability along potential grout curtain hole orientations.

The number of cored holes necessary for a geologic site investigation for a grouting program will vary from project to project. Factors that influence the number of cored holes needed in preconstruction geologic investigation for a grouting program include:

- Complexity of the damsite foundation geology, such as extensively folded, faulted, or highly fractured rock; more than one rock type in the foundation; naturally weak rock; karstic rock; steeply dipping beds; adverse ground water chemistry; and artesian pressures.

- Length and height of the dam.

- The anticipated cost of the grouting program. The expense of additional cored holes can easily be justified for an extensive grouting program.

- Rock permeability and permeability variations along the damsite.

Each of these factors should be discussed with project team members to determine an appropriate number of core holes. Additional core holes will likely be necessary as the project progresses from the initial geology exploration phase to final design.

15.4.4 Permeability Testing and Interpretation

Water testing is necessary for evaluating seepage potential beneath the damsite and for determining how much grouting may be required. Water testing for designing a grout program is often secondary to the main purpose of the water testing program, which is to determine permeabilities for seepage evaluation or control. Design of an exploration program for grouting should consider the following:

(1) The geologic conditions at the site and the variations between areas or reaches must be understood. Generalizations based on other sites are usually inaccurate because geologic conditions depend on the interrelationship of the local depositional, tectonic, and erosional history that uniquely determine geologic conditions important to the permeability and groutability of a site. Damsite foundation permeabilities can vary over short distances because of lithology and fracture changes, faults, rebound of horizontal beds in the canyon bottom, or stress relief in the abutments. Proper evaluation of water test results requires that the values be correlated with geologic conditions. The permeability values should be noted and plotted on the drill logs, along with the water ‘takes’ and test pressures. The test interval should be indicated on the log so that the water test data can be related to fracture data.

(2) The level of seepage control desired. The allowable seepage quantities beneath a dam in the arid West compared to a dam located in an area with abundant rainfall are likely to be quite different based on the economic value of water.

(3) The potential for internal erosion failure modes. When an embankment contains or is founded on or adjacent to erodible or dispersive soils and rock, water testing and the effectiveness of the grouting program are critical in the design and construction of the dam.

(4) Rock mass permeability is not always an indicator of how a grout program should be conducted or how successful a grout program will be. A highly fractured rock mass with very small fracture aperture sizes may be very permeable but essentially ungroutable using standard cement grout. Water test data must be correlated with geologic data to properly assess groutability.

(5) The ability to cut off or effectively control seepage though the injection of grout (groutability) depends on the openness, continuity, joint infilling, and number of discontinuities. Connectivity may not be as important if discontinuity properties limit the travel of grout.

(6) Permeability depends on fracture openness, number, and connectivity. Highly fractured rock with low connectivity will have low permeability, and a slightly fractured rock with high connectivity can have high permeability.

(7) Exploratory drill hole orientations introduce a significant bias into water test results. The orientation of the drill hole relative to the fractures has a direct effect on the number of fractures intercepted by the hole. A vertical hole drilled in a material that has predominantly vertical fractures, like flat-bedded sediments, will likely not intercept the fractures that control the rock mass permeability. Drill holes should be oriented to cross the maximum amount of discontinuities per foot of hole drilled not only for more meaningful permeability tests, but also to obtain more meaningful rock mass design parameters.

(8) Water test calculation results can be very misleading. Water test calculations from a 10-foot interval with one 1/4-inch fracture taking water can have a significantly different seepage and grout potential than a 10-foot interval with dozens of relatively tight fractures taking the same amount of water. While the tighter fractures may readily convey water, grout 'takes' may be negligible. Each water test must be evaluated individually.

(9) Hydraulic fracturing occurs when the rock mass is fractured due to excessive grout pressures. Different rock types, geologic structures, or in situ stresses have different hydraulic jacking and hydraulic fracture potential; therefore, they have different maximum acceptable water testing and grouting pressures. During the injection of fluid grout, the grout pump can act as a hydraulic jack and cause damage by the use of excessive pressures. Hydraulic jacking occurs when excessive grout pressures enlarge existing discontinuities. Dam foundations are more sensitive to hydraulic jacking and hydraulic fracturing than tunnels. Dam foundations can be seriously damaged by hydraulic jacking and hydraulic fracturing. A dam foundation in interbedded sedimentary rock with high horizontal in situ stresses is very sensitive to hydraulic jacking. Hydraulic jacking a dam foundation in hard, massive rock is extremely unlikely.

(10) Rock mass permeability or groutability information collected within drill holes should be supplemented with geologic mapping and an analysis of the fractures. Geologic mapping and analysis of the fractures are necessary factors in determining seepage potential and groutability. Judgments made solely on drill hole data may not provide a realistic characterization of fracture orientations and connectivity. All data should be integrated to determine rock mass permeability and groutability.

(11) "Rules of thumb" are not good substitutes for using data and judgment when making grouting decisions unless they are specifically developed for the site conditions.

(12) Hydraulic models can be used as a tool to evaluate seepage potential and groutability, but they depend on realistic design and data input parameters. Water test-derived permeability and groutability are important parameters for hydraulic models. Water tests must be carefully evaluated to ensure that bad test data are not used in models. Models large enough to approach characterizing a site are usually very large and expensive. The tendency is to build models that are small and economical and, therefore, have a limited connection with reality. Realistic parameters are often difficult to obtain in quality or quantity. Few exploration programs provide a statistically significant sample size to fully characterize a site. Model input parameters and design should be part of any modeling report so the output can be properly evaluated. Sensitivity studies should also be performed to evaluate the effect of the model input parameters and may provide justification for additional investigations to reduce the uncertainty within the model.

15.4.4.1 Procedure Calculations

Permeabilities can be calculated in Lugeons, feet per year, centimeters per second, or other units from the same basic field data. Lugeons are commonly used in the grouting industry; however, because the Lugeon test could represent permeability of the rock mass and/or the influence of joints in the rockmass, Reclamation typically reports permeabilities (k-values) in feet per year and the volume of water loss for an interval of time. It is important to note that the water tests performed in the field should measure the flow rate after flows have stabilized and should be run for at least 5 to 10 minutes at each pressure step.

One Lugeon unit equals:

1 liter per minute per meter of test length at a pressure of 10 bars

0.01076 cubic feet per minute per foot of test length at a pressure of 142 pounds per square inch (lb/in^2)

$1.3 * 10^{-5}$ cm/s

10 ft/yr

To calculate the Lugeon value at any test pressure, the following equation can be used:

$$\textbf{Lugeon units} = (Q/L) * (1801/P_{eff})$$

where Q equals the flow rate in gallons per minute (gal/min), L equals the stage length in feet, 1801 is a conversion factor, and P_{eff} equals the effective pressure applied to the test stage in pounds per square inch.

The effective pressure can be calculated by accounting for the gauge pressure at the surface during testing, the depth of the water column in the drill hole, groundwater depth at the time of drilling, line losses between the packer and the gauge, and the height of the gauge above or below the ground surface. See figure 15.6.3-1 (located later in this chapter) for additional guidance on calculating the effective pressure.

When using the Lugeon unit, it is important to consider the crack sizes and the spacing of the cracks within the test interval. For example, over a 10-foot interval, the same Lugeon value can be calculated for 1 large crack versus many small cracks. In the case of a large crack, if the length of the test interval was reduced from 10 feet to 2 feet and centered at the location of the large crack, the water ‘takes’ would be the same as a 10-foot test interval; however, the Lugeon value would be 5 times higher. For these reasons, additional geologic factors must be considered when using water test results during grouting. Borehole televiewers, which are discussed below, can provide valuable information to assist in the interpretation of water test results.

15.4.4.2 Geologic Data

The geologic data should be examined to determine the optimum drill hole orientations and locations necessary to intercept the maximum number of discontinuities per foot of drill hole at an inclination less than or equal to 30 degrees from vertical. Geologic structure, such as bedding and rock type, can be used to set the initial maximum water test pressures. Easily jacked or hydraulically fractured rock should initially be water tested at a pressure of 0.5 lb/in^2 per foot of overburden and increased pressures based on stepped pressure tests or jacking tests.

15.4.4.3 Stepped Pressure Tests

Stepped pressure tests are the best method of conducting water tests. Pressures are stepped up to the maximum pressure and then stepped down through the original pressures. Comparison of the calculated permeability values and the pressure versus flow curves for the steps can help indicate whether the flow is laminar, if jacking or hydraulic fracturing is occurring, and if fractures are being washed out or plugged. Single pressure tests can be misleading because of all the unknown pressure and flow variables affecting the test. The permeability or Lugeon value for the test interval should be determined by analysis of the individual test values and not necessarily by an average. The individual tests are used to determine the response of the rock mass, and one value from the five tests is usually the appropriate value to use.

Figures 15.4.4.3-1 through 15.4.4.3-5 are bar chart plots showing the relationship of pressure to Lugeon values for the more common types of water test results. Figure 15.4.4.3-1 is a plot of laminar flow in the fractures. The permeability is essentially the same regardless of the pressure and resultant water ‘take’. Figure 15.4.4.3-2 is a plot of turbulent flow in the fractures. Permeability decreases as the pressure and resultant flow increases because of the turbulent flow in the fractures. Figure 15.4.4.3-3 is a plot of flow in fractures that increase in size as the water washes material out of the openings. Permeability increases because fractures are enlarged by the test. Figure 15.4.4.3-4 is a plot of flow in fractures that are being filled and partially blocked as water flows, or the fractures are in swelling rock, which closes fractures over time because of the introduction of water by the test. Figure 15.4.4.3-5 is a plot of testing in rock that is being jacked along existing fractures or rock that is being fractured by the highest water test pressure. Flow is laminar at the lower pressures.

Combinations of these types of flow can occur and require careful analysis. If the pressures are increased to where jacking or hydraulic fracturing is occurring, the design grout pressures can be set as high as possible to obtain effective grout injection, yet preclude fracturing (or, if appropriate, induce fracturing). Hydraulic fracture tests are easier to analyze if a continuous pressure and flow recording are obtained. The resolution of a step test may not be adequate to separate hydraulic fracturing from hydraulic jacking. Figure 15.4.4.3-6 is a plot of a continuously recorded hydraulic fracture/jacking test.

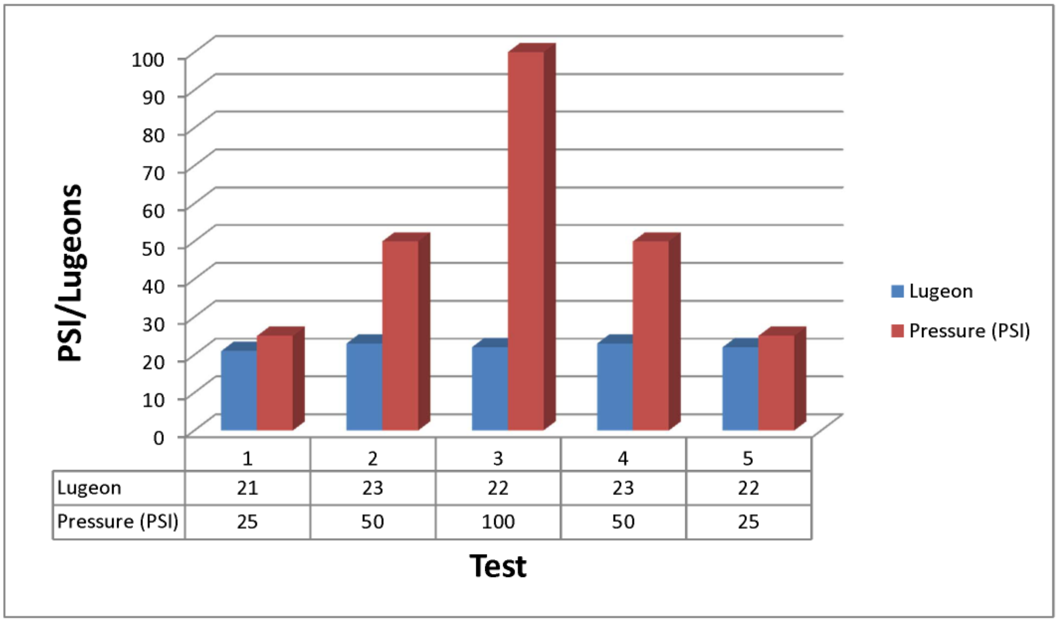

Figure 15.4.4.3-1. Bar chart showing relationship of test pressure and Lugeons in laminar flow.

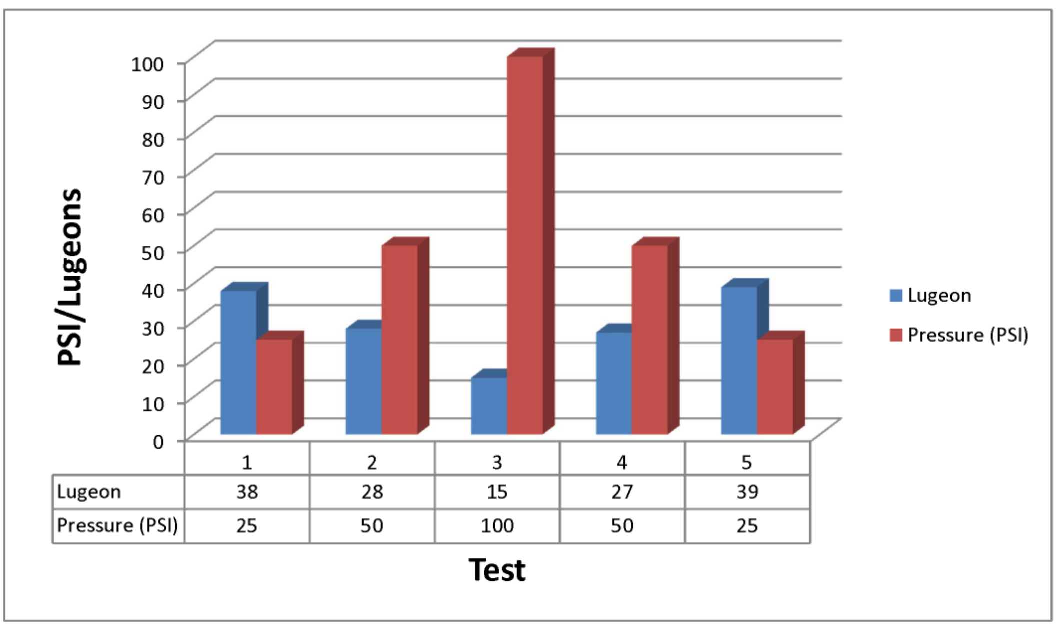

Figure 15.4.4.3-2. Bar chart showing relationship of test pressure and Lugeons in turbulent flow.

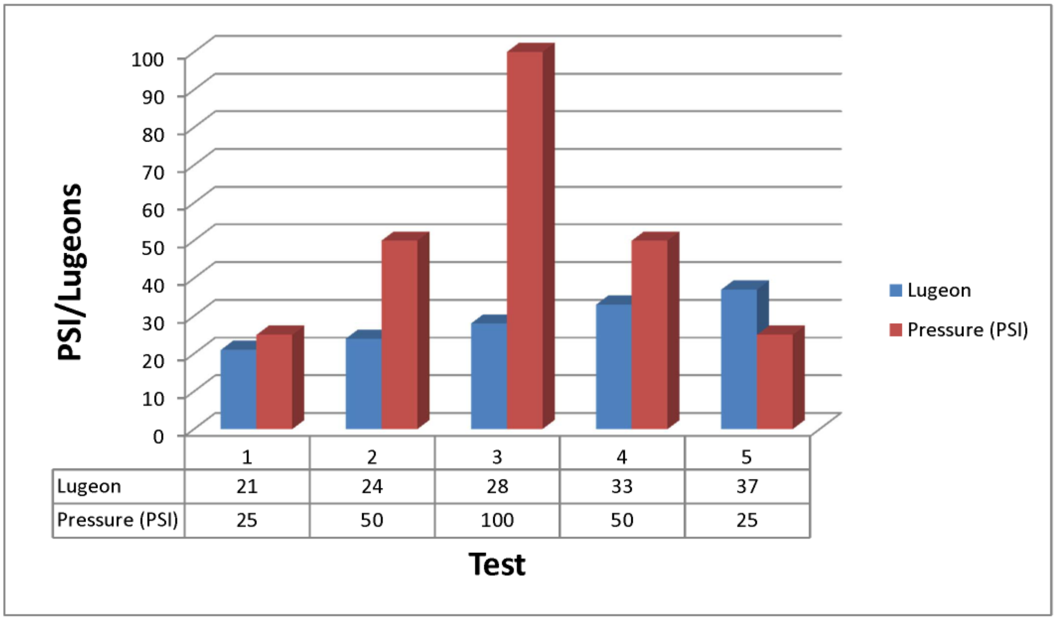

Figure 15.4.4.3-3. Bar chart showing relationship of test pressure and Lugeons when fractures are washed out.

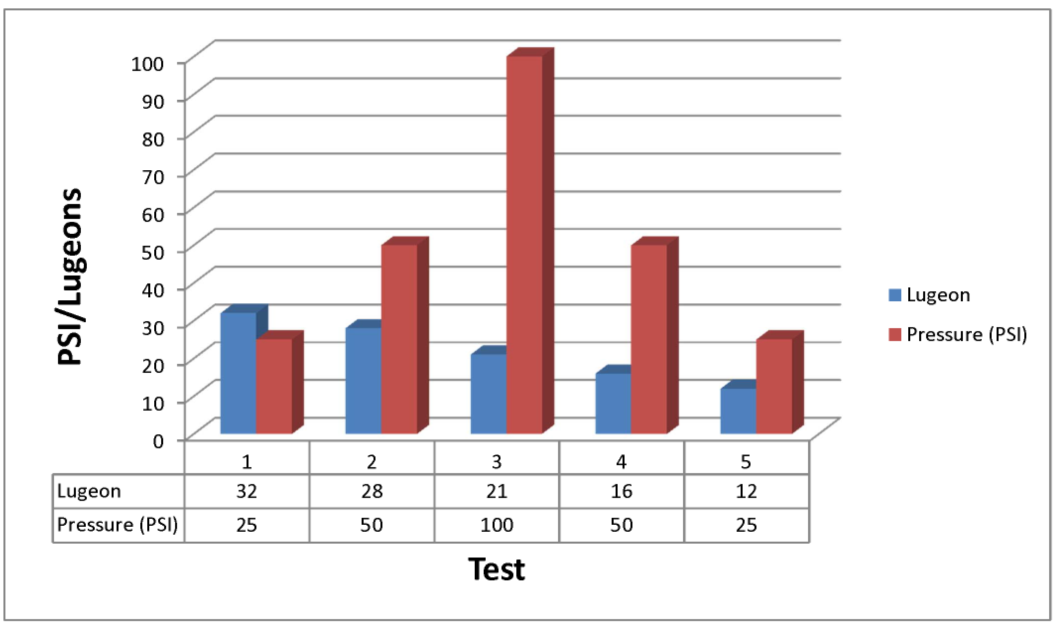

Figure 15.4.4.3-4. Bar chart showing relationship of test pressure and Lugeons when fractures are filling and swelling.

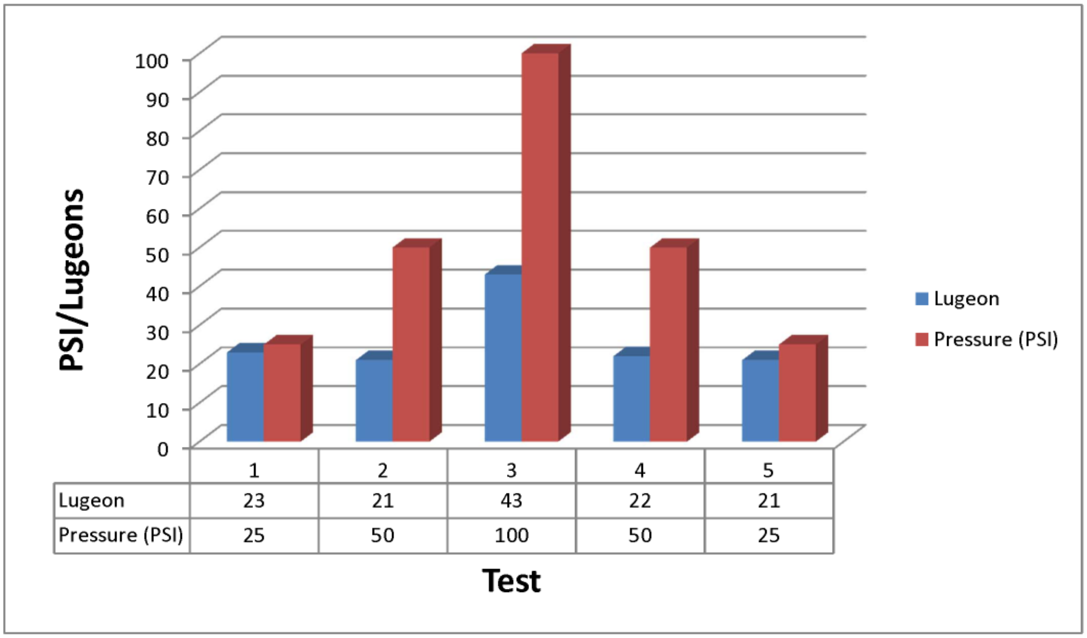

Figure 15.4.4.3-5. Bar chart showing relationship of test pressure and Lugeons when rock is hydraulically fractured or joints are jacked open.

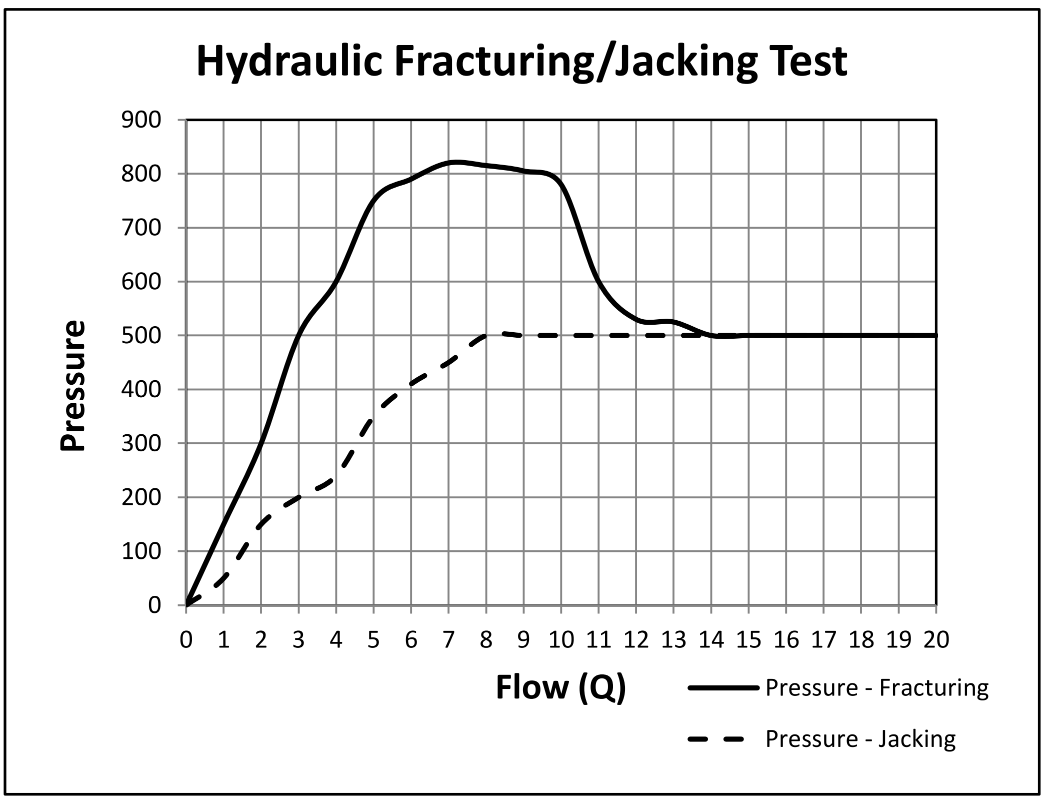

Figure 15.4.4.3-6. Continuously recorded plot of pressure and flow in a hydraulic fracturing and hydraulic jacking test.

15.4.4.4 Back Pressure and Holding Pressure

During water testing and grouting, it is important to determine whether or not there is back pressure or holding pressure. Back pressure and holding pressure should be measured after every test. Differentiating between back pressure and holding pressure takes place at the manifold. (Section 15.15.3.2 discusses the procedure for checking for back pressure and holding pressure). The cause for back pressure within a hole should be carefully evaluated. Back pressure often occurs when: (1) there is artesian flow into the grout hole; (2) the formation being water tested is lifted or squeezed, and the weight of the rock tends to force the water back out of the hole; and (3) there is compressed methane gas or other gases within the foundation. Back pressure could also occur if water fills voids above the point of injection. The presence of artesian flow and methane gas within the foundation should be well documented in the preconstruction geologic investigations.

15.4.4.5 Test Equipment

Test equipment can affect the test results. At moderate to high flows, the friction loss caused by the piping and the packer should be considered. Significant loss of pressure could occur between the gauge and the packer. At high flows, the plumbing system "permeability" can be the controlling factor and not the permeability of the test interval. If meters and gauges are located in optimum relation to each other and close to the hole, the arrangement of pipe, hose, etc., will not seriously influence shallow tests, although sharp bends in hose, 90-degree fittings on pipes, and unnecessary changes in pipe and hose diameters should be avoided. It is a good idea to lay the system out on the ground and pump water through the plumbing to determine the capacity of the system, especially if small-diameter piping or wireline packers are being used.

15.4.5 Geophysical Borehole Logging

Geophysical borehole logging can be used to supplement drill hole data to improve the understanding of the damsite geology and interpret packer test data. Geophysical borehole logging can also be used to evaluate the effectiveness of grouting programs during and after construction by imaging the areas that have already been grouted. Grouted intervals by design contain fewer open fractures and cavities than ungrouted intervals, and they should exhibit a lower overall formation porosity compared to ungrouted formations. Table 15.4.5-1 lists borehole geophysical log types that are applicable for grouting evaluation. Note that some of the logs discussed here use radioactive sources and require licensed operators. Logs are discussed according to their applicability below.

Table 15.4.5-1. Borehole geophysical log types

Source type	Geophysical log type	Physical parameter measured or inspected	Requires water in the borehole [Yes] [No]	Application in grouting evaluation/remarks
Nonradioactive	Caliper	Borehole diameter	No	Detect variations in borehole diameter possibly related to presence of fractures or cavities. Caliper should be run first to assess borehole condition before running more costly logging tools.
	Acoustic Borehole Image	Variations in borehole wall acoustic reflectivity	Yes	Examine borehole wall for signs of open cracks, joints, or cavities. This test may be run in cloudy, opaque water or in drill fluid.
	Optical Borehole Image	Variations in borehole wall image	No	Examine borehole wall for signs of open cracks, joints, or cavities. This log requires clear water or no water.
	Full Wave Sonics	Compression [P-] and Shear [S-] wave velocity	Yes	Variations in P- and S-wave velocity may be indicative of incomplete grouting, open fracturing, or open voids. Acoustic porosity log can indicate variations in bulk porosity.
Radioactive	Gamma-gamma	Bulk density	No	Detect variations in bulk density possibly related to presence of fractures or cavities. This tool uses a radioactive source. Caliper log should be run first to assess borehole condition.
	Neutron	Hydrogen (water) content	No	Detect variations in hydrogen (water) content possibly related to presence of fractures or cavities. This tool uses a radioactive source. Caliper log should be run first to assess borehole condition.

15.4.5.1 Geophysical Logs Used to Evaluate Fracturing

Primary logs used to assess fracturing include caliper (Cal), Optical Borehole Image (OBI), Acoustic Borehole Image (ABI), and Full Wave Sonics (FWS). Other logs that may be run for fracture delineation include gamma-gamma density (GG) and Neutron (Neu), which both use radioactive sources.

15.4.5.2 Geophysical Logs Used to Evaluate Formation and Density

Formation density should be expected to increase after grouting. Formation bulk porosity should be expected to decrease. The primary log used to assess density is the GG (radioactive). Primary logs used to assess porosity are the Neu (radioactive) and FWS.

15.4.5.3 Caliper Log

The caliper log tool generally uses three or four arms that extend from the tool and scrape against the borehole wall as the tool is being run up the borehole. This tool is typically run first in any borehole logging operation because a more expensive tool could become stuck in the hole due to poor borehole wall conditions. The caliper log data indicate borehole diameter, as well as variations in diameter that may be related to the presence of open fractures and voids.

15.4.5.4 Acoustic Borehole Image and Optical Borehole Image

The ABI log provides an image based on borehole wall acoustic (sound) wave reflectivity. This log is sensitive to fractures, jointing, and cavities in the borehole wall and may indicate poor grout ‘takes’ or ungrouted intervals in the borehole. The ABI log requires water to operate, and it may be run in clear or opaque water or in drill fluid.

The OBI log provides a visual image of the borehole wall. It can be run in dry holes or holes filled with clear water. Similar to the ABI, the OBI log images can show fractures, jointing, and cavities in the borehole wall and may indicate poor grout ‘takes’ or ungrouted intervals in the borehole. Generally ABI and OBI are run together, if possible, because the visual image can complement and enhance the interpretation from ABI alone. This log requires clear or no fluid in the borehole.

ABI and OBI logs are usually run and interpreted together because they give complementary images of the borehole wall. Figure 15.4.5.4-1 is an illustration of an OBI. Ungrouted voids or open joints intercepted by the borehole would appear as variations in the ABI or OBI images.

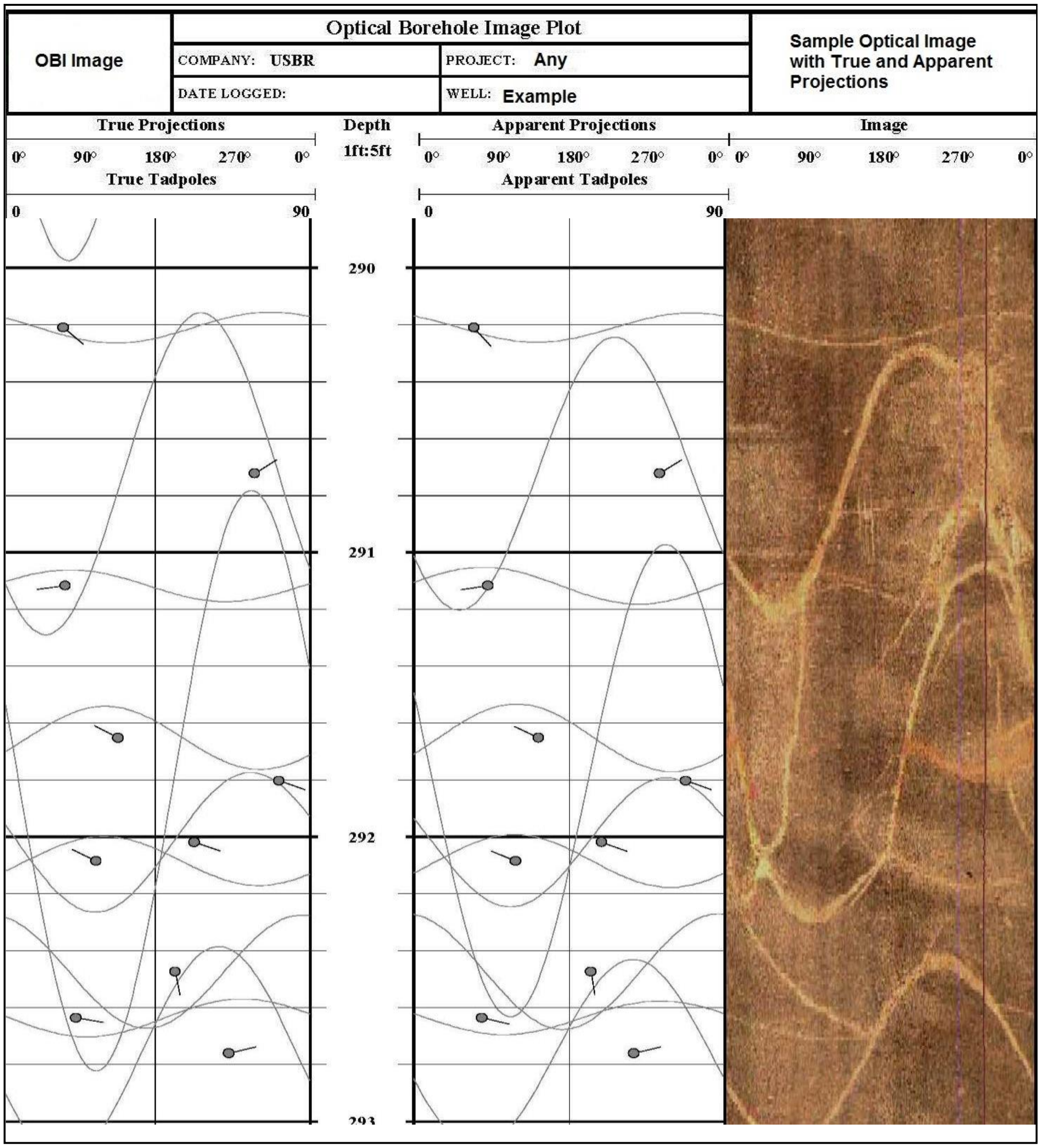

Figure 15.4.5.4-1. Optical borehole image with true and apparent dip poles.

15.4.5.5 Full Wave Sonic Log

An FWS log shows sonic wave signals that have traveled through the borehole wall and back to the receiver in the tool. It provides an image based on seismic velocities in and immediately behind the borehole wall. Variations in this log can show locations of poor grout 'takes,' voids, joints, and cavities. When used with the GG density log in pregrout and postgrout evaluations, the formation modulus values and modulus improvement values may be derived.

15.4.5.6 Gamma-Gamma Density Log

A GG density log uses a radioactive gamma ray source to bombard the borehole wall, as well as a gamma ray count detector in the logging tool to record how many rays have traveled through the formation and back to the detector. In a calibrated tool, this gamma ray count is then proportional to the bulk density of materials surrounding the borehole. When used in conjunction with the FWS log, formation modulus values can be obtained.

15.4.5.7 Neutron Log

The Neu log uses a radioactive neutron source to bombard the borehole wall, as well as a neutron count detector in the logging tool. The neutron count is proportional to the amount of hydrogen atoms (i.e., water molecules) in the borehole wall vicinity. Variations in this log may indicate intervals with very high or very low grout 'takes,' which can be used with ABI, OBI, and modulus data to interpret overall grouting effectiveness.

15.4.6 Exploratory Shafts and Adits

Exploratory shafts and adits can provide an excellent view of the foundation or abutment rock mass. Reclamation does not normally use shafts and adits for the sole purpose of investigating rock permeability and suitability for grouting. Typically, excavations of this type serve a multipurpose role, and grout design data collection should take place with a consideration of any exploratory shaft or adit. Detailed mapping can be made of the discontinuities and used to assess the suitability of the rock to grouting. Exploratory drilling and testing from within the tunnel allow the specific targeting of defects within the rock mass for testing.

15.4.7 Grout Testing Programs

Although water tests of exploratory holes and other geologic investigation methods can indicate the relative permeability of a foundation, there is no definitive or predictable correlation between the results of the geologic investigations and the quantity of grout that is needed to obtain closure at the damsite. Test grouting programs during preconstruction investigations may furnish useful design information to reduce some of the uncertainty in the effectiveness of a grouting program and provide higher confidence in the cost estimates. Reclamation does not typically perform grout test programs because a test program gives little indication as to how the total foundation will behave. Reclamation's experience is that no two grout holes behave the same way, nor can the results of a test program be extrapolated to how the total grouting program will behave. On most dam grouting projects, there are few similarities between either of the abutments and the foundation bottom.

15.5 Optimum Mix

The most important single factor for foundation grouting is the use of the best or optimum grout mix. Effective filling of fine joints and cracks may be significantly reduced by the use of grout that is too thick to penetrate into them. Conversely, injection of a thin grout into large voids may promote detrimental shrinkage/bleed, as well as excessive grout travel beyond the target grouting area. The guidelines discussed throughout this design standard chapter are based on the use of the optimum mix technique. Reclamation defines the “optimum mix” as the thickest grout mix which is readily accepted by the rock mass without prematurely closing off passageways. Determination of the optimum mix should be made in the field based on the actual grout ‘takes.’ The ultimate objective when grouting any stage is to establish an optimum grout mix whereby a stage can be completed to refusal without thickening the grout mix to a point that might slug passageways prematurely. Proper grouting of a stage to refusal should eliminate the need to reinject grout into the stage unless intermittent grouting is planned for large voids or severe leaks, or if rock movement occurs. The optimum mix is established on a stage-by-stage basis because every grout stage may behave differently.

Reclamation grouting projects typically use a starting grout mix of 5:1 (water:cement ratio, by volume) [2] with an appropriate dosage of super plasticizer for each stage unless large voids are discovered during drilling or the primary focus of grouting is to fill only large voids. The initial mix is gradually thickened until the optimum mix is reached and the stage achieves the desired closure criteria without being prematurely slugged with thick grout. Reclamation grouting practice uses initial grout mixes that are thinner than the expected optimum mix to avoid losing a stage if thinner grout is needed. The grout mix can be gradually thickened, as needed, but if the stage is prematurely thickened at the start of grouting, a new drill hole is necessary. Section 15.15.2 discusses the guidelines Reclamation follows for thickening the grout mix. The optimum mix technique maximizes the amount of cement pumped into the formation and reduces the amount of drilling required for the project.

If the primary purpose of a grout hole is to fill large voids, or when large voids are discovered during drilling, the initial grout mix should be adjusted to minimize bleed and contain thixotropic admixtures to increase cohesion as the grout enters the void. Section 15.10 of this chapter contains suggestions for grout mixes that minimize bleed, additives that modify the viscosity, and additives that alter the “set” time.

Some government agencies and engineering firms in the United States and around the world currently prefer the use of “stable” (consequently, thicker) initial grout

[2] All water:cement ratios discussed in this design standard chapter are by volume unless otherwise noted.

mixes relative to Reclamation's grouting practices. A "stable" grout mix is defined as a blend of water and cement combined with selected additives and admixtures that produce a grout with zero bleed, low cohesion, and good resistance to pressure filtration. The use of stable grout mixes provides the benefits of near zero water bleed from the grout mix and optimal pressure filtration performance. The initial grout mixes used on a Reclamation grouting project do not meet the criteria for zero bleed and optimal pressure filtration performance and are referred to as "unstable" grout mixes. It should be noted that if stable grout mixes are judged to be necessary by the grouting engineer and the designer, Reclamation is not opposed to their use. However, Reclamation grouting practice would still recommend starting with a 5:1 mix and gradually thickening from unstable grout mixes to the desired stable mixes only if the rock mass will readily accept the stable mix.

Bruce and Weaver [4] discuss in detail the potential benefits of using a stable grout mix. The main arguments in the grouting industry for eliminating unstable (or thinner) grout mixes are listed below, followed by a brief comment from Reclamation based on many years of grouting experience and performance data at Reclamation facilities.

Critique:

The potential for excessive bleed from unstable grouts results in partial crack filling.

Reclamation Position:

The seepage performance observed after decades of operation of an embankment provides the best argument for or against this comment. The seepage performance has been excellent at Reclamation embankments that were constructed using the best practices for foundation grouting described in this design standard. Appendix A includes five case histories that illustrate the use of Reclamation's grouting practices. Each of the grouting programs summarized in appendix A was generally constructed using the best practices discussed in this design standard. The seepage performance data of these dams, along with many other Reclamation dams, support Reclamation's position.

In Reclamation's experience, when using the optimum mix technique, excessive bleed, resulting in partially unfilled voids, has not been observed. The area being grouted is typically tested by drilling verification holes in the vicinity of the grouted area to check the quality of the in-place grout. The recovered samples from Reclamation grouting projects indicate the rock discontinuities are completely filled with high quality grout.

Since the failure of Teton Dam in 1976, all new Reclamation dams have been independently peer reviewed by noted experts in the field of embankment dam engineering and construction. After viewing Reclamation's grouting results, consultant review boards have not expressed any remaining concerns related to partial crack filling resulting from excessive bleed water generated during grouting.

In Reclamation's experience, as grout is injected into openings within bedrock under pressure, the cement particles will settle out at some distance from the injection point, be filtered at choke points in the discontinuity network, and stick to the rock opening being grouted. As grouting progresses over a period of time while under pressure, water is continuously pushed out of the grout mixture. Reclamation's combination of pressure, time, and grout 'take' requirements for stage refusal minimize bleed water left within discontinuities, and high quality grout is left in the bedrock opening.

Figure 15.5-1 illustrates what is typically observed in a grout check hole after grouting is complete. The grout observed in the bedding planes was hard, with no partial crack filling observed.

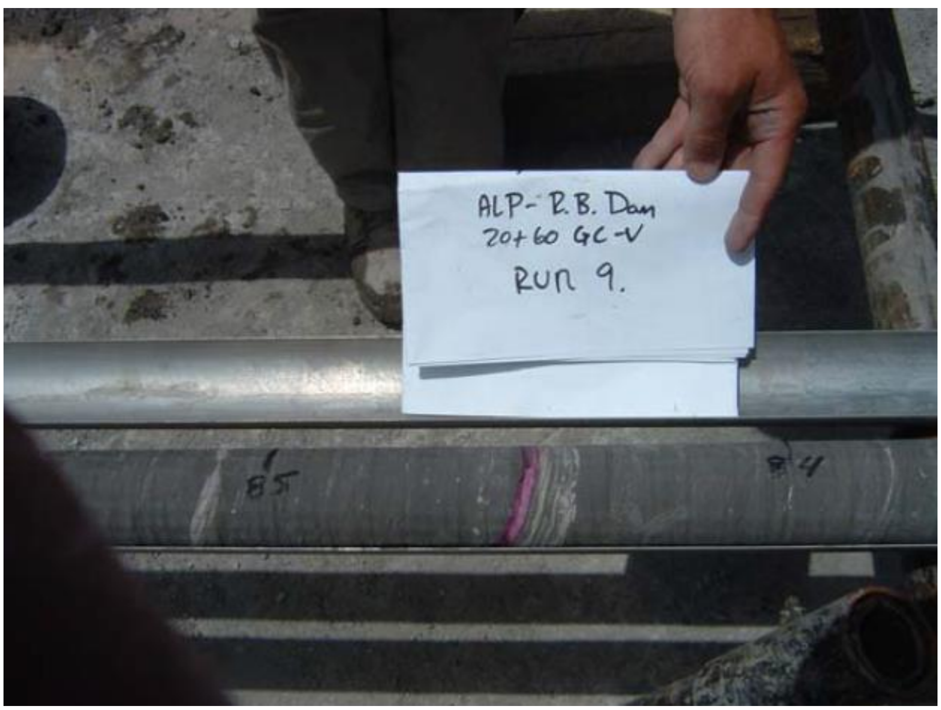

Figure 15.5-1. Photo illustrates grout infilling (shown in purple) of joint within a grout check hole

Critique:

There is perceived to be little benefit to using grout mixes that are thinner than 3:1.

Reclamation Position:

In the arid and semi-arid Western States of the United States, where Reclamation has constructed the majority of its dams, seepage losses must be minimized given the high economic value of water in these dry regions. In many cases, thin grout mixes were needed to penetrate finer openings in bedrock to reduce seepage losses to an acceptable level. The remedial grouting performed at Hoover Dam is an excellent case history [5] that supports the effectiveness of thin grouts in the Western United States. Uplift pressures and leakage into drainage galleries at Hoover Dam were judged to be excessive during initial filling of the reservoir in 1937 and 1938. It was believed that the original grout curtain was not deep enough to provide an adequate cutoff given the extremely large head induced by the reservoir; and that rapid setting of cement occurred due to high alkaline groundwater, resulting in limited grout travel. Installation of additional drain holes and remedial grouting was performed between 1938 and 1947 to reduce uplift pressures and seepage. Water:cement ratios generally ranged from 15:1 to 5:1 during the remedial grouting although some 20:1 mixes were used. At the beginning of the program, the water:cement ratios varied from 7:1 to 3:1 but thinner mixes were used more frequently as work progressed. Thinner grouts were found to be more successful for reducing seepage. As grouting progressed, grout thicker than 7:1 was rarely used unless surface leaks developed. The combination of additional drain holes and grouting was effective in reducing seepage and uplift pressures. In the late 1980s, an elevator shaft was excavated within the downstream right abutment of Hoover Dam. There was little to no seepage within this elevator shaft excavation that extended almost 600 feet vertically, which further supports the effectiveness over time of the thin grout mixes used at Hoover Dam.

In dry conditions, grout travel may be increased by using thinner initial grout mixes. In-place rock usually has both primary and secondary permeability. As grout is being injected into the rock formation, the pore space of a dry rock mass will absorb some of the water from the grout mixture. The use of thinner grout will result in greater penetration and travel since bleed water is available for lubrication.

Critique:

Excessive bleed from thin grouts results in weaker/less durable grout.

Reclamation Position:

In Reclamation's inventory of dams, this issue has not been observed when using the best practices described in this chapter. Performance of grout curtains on Reclamation projects show no indications of grout breakdown over time. Five case histories have been summarized in this chapter. Each of these case histories indicate that there is little to no change in the quantity of seepage collected over time.

Some additional benefits for using unstable (thin) initial grout include:

- The optimum mix technique maximizes the amount of cement pumped into the ground.

- The need for a multiple-row curtain is greatly reduced because the optimum mix technique results in relatively larger grout travel distances upstream and downstream of the single-line grout curtain.

- The likelihood of excessive drilling is reduced because no stage is prematurely “slugged” with grout that is not readily accepted by the rock mass.

- Progressive thickening of the grout mix and stage refusal criteria using the optimum mix technique are based on the grouting results, rather than having stage refusal being based on time allowed for each stage that can result in premature closure of the hole due to overthickening of the grout mix.

15.6 Grout Curtain Design

15.6.1 Grout Curtain Location

For a zoned embankment dam with a central impervious core, the grout curtain is typically located slightly upstream of the midpoint of the base of the impervious core material. The final location of the grout curtain should be based on the type of dam, the configuration of the impervious portion of the dam, seepage gradients, and the geologic conditions below the dam. Some general guidelines for locating a grout curtain, which should be consistent for any embankment dam design, include:

- If the grout curtain is placed closer to the upstream toe of the core, then high gradients may exist from the embankment core into the foundation.

- If the grout curtain is placed closer to the downstream toe of the core, then high gradients may exist from the foundation into the core.

- The location of the grout curtain should be at or upstream of the dam centerline.

Many examples of grout curtain location can be found in Reclamation's *Maximum Sections and Earthwork Control Statistics* [6]. In general, most of the grout curtains for more recent Reclamation embankment dams (designed after 1970) are located at or just upstream of the midpoint of the base of the impervious core of the embankment.

When locating the grout curtain, one of the most important considerations is the potential for high gradients at the embankment/foundation contact if the rock is permeable. Designers must consider the implications of high gradients at the base of the core and the potential for scour of the embankment core materials in this area. For permeable rock foundations, blanket grouting is highly recommended to reduce the potential for high gradients within the embankment core over the grout curtain.

Once the location, orientation, and depth of grout curtain holes are established, sufficient overlap distances need to be provided in areas where there is a sharp change in the grout curtain alignment, a reversal of the grout hole orientations, and beneath underground structures to avoid windows within the grout curtain. "Windows" in the grout curtain can result in areas of high seepage gradients that could cause scour of the joint infillings or of the embankment core materials. Figure 15.6.1-1 is a simplified illustration showing how reversing the grout hole orientation results in a large window within the grout curtain.

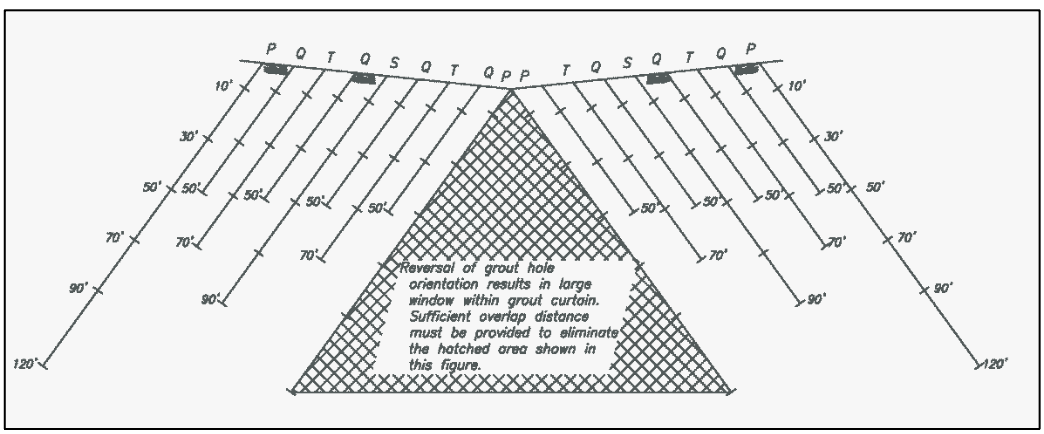

Figure 15.6.1-1. Illustration of large window in grout curtain resulting from reversal of grout hole orientation.

15.6.2 Curtain Depth

Unless special geologic conditions dictate otherwise, general practice for modern Reclamation dams is to extend the primary grout holes to a depth below the surface of the rock equal to about 0.5 to 1.0 times the reservoir head, which lies above the surface of the rock. The selected design depth for the grout curtain should be based on the geology, regional groundwater conditions, permeability test results, potential failure modes, O&M needs, and seepage analyses. In many cases, the curtain depth may vary across the damsite, depending on the damsite's geologic conditions.

Once the depth for the primary holes is selected, Reclamation typically decreases the specified minimum depths of the remaining intermediate grout holes by a stage length or two relative to the preceding series of holes (primary, secondary, etc.). Progressively diminishing the minimum depths of intermediate holes is justifiable because higher pressures are used on deeper stages, thus forcing grout to travel farther and cover a larger area.

It should be noted that the final depth of the secondary, tertiary, and quaternary grout holes will be determined in the field based on the actual grout 'takes' from the preceding series of grout holes. Section 15.6.7 discusses this issue in greater detail.

15.6.3 Injection Pressure

Reclamation practice is to inject the highest quantity of grout at the allowable injection pressure without causing uplift, horizontal movement, fracturing, rupturing, and excessive grout travel. Reclamation typically uses a maximum injection pressure of 1.0 pounds per square inch per foot ($lb/in^2/ft$) of depth **measured from the surface to the packer,** plus back pressure due to artesian waterflows. Adjustments are made to the maximum injection pressure, if necessary, based on the results of the preconstruction geologic investigations, engineering analyses, the behavior of the rock mass during water testing and grouting, and the results of the grouting program. Reclamation typically does not account for the static weight of the grout, frictional head losses, or groundwater levels when determining the maximum injection pressure of a grout stage. Back pressure is measured at the pressure gauge on the manifold. Reclamation specifies the use of a short tail hose length attached to the manifold; therefore, no adjustment is made for head losses or gains from the manifold to the grout hole standpipe. This maximum injection pressure is recommended for new dams only. Remedial grouting below an existing embankment dam requires special attention, and this situation is discussed separately in section 15.9.1 of this chapter.

Injection pressures must be monitored and controlled at all times to prevent damage to the foundation or structure. The injection of fluid grout can result in hydraulic jacking or hydraulic fracturing caused by the use of excessive injection pressures. Grout pressures must be adjusted, as needed, to eliminate hydraulic jacking or hydraulic fracturing. The maximum injection pressure is measured at the pressure gauge located between the control valve and grout hole valve on the grout manifold (see figure 15.6.3-3). The gauge pressure on the manifold does not represent the actual grout pressure at the bottom of the packer within the hole, which is often referred to as the effective grouting pressure (P_{eff}). The effective grouting pressure at the bottom of the packer must factor in the static head of the grout column, elevation of the manifold pressure gauge above or below the top of the rock surface, groundwater levels, and frictional head losses from the pressure gauge to the packer. Figure 15.6.3-1 is an illustration for calculating the effective pressure at the bottom of the packer.

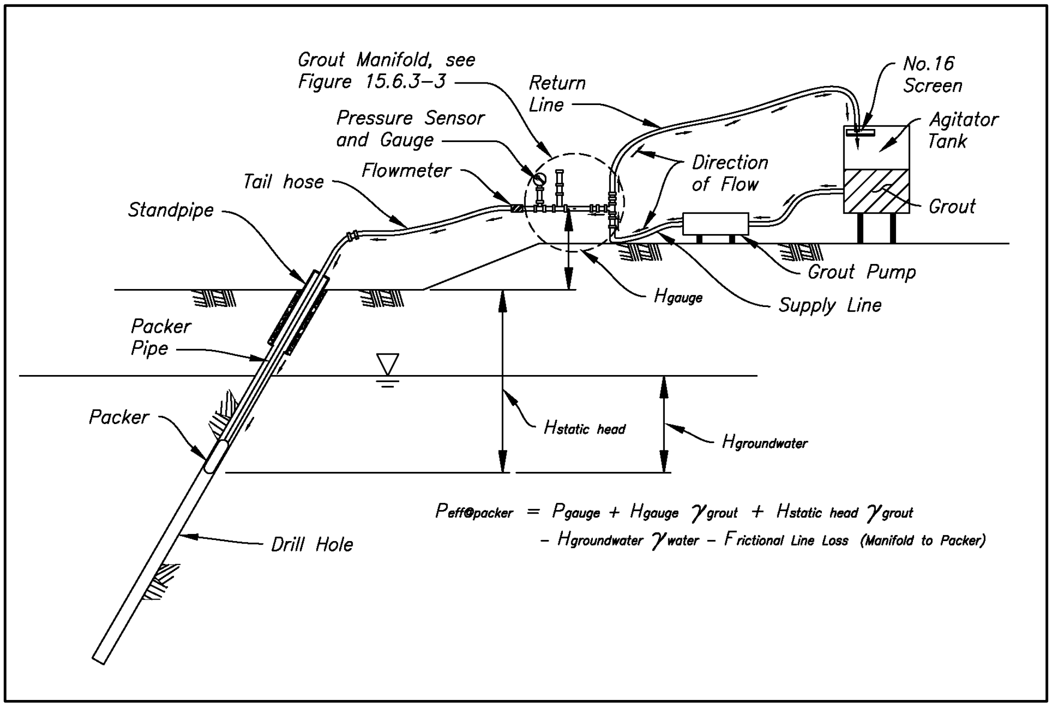

Figure 15.6.3-1. Illustration for calculating effective grout pressure at the bottom of the packer.

In the 1980s, Reclamation evaluated the need to attach a pressure gauge just below the packer within the grout hole [7]. The field testing results indicated that the effective pressure at the packer could be closely estimated by fairly simple calculations; however, thin grout mixes were used for this test, which limited frictional head losses in the packer pipe. Given the expense of the instrumentation necessary to make this measurement, the potential for damage of these instruments and the number of instruments that could be required during a large grouting project, Reclamation currently does not require a down-the-hole pressure gauge.

The maximum injection pressure may vary, depending on many factors such as ground water, distance to the nearest free surface, purpose of the grouting, orientation of the joints and bedding planes, rock quality, etc. For example, foundations with horizontal bedding planes tend to be more susceptible to uplift than steeply dipping formations. In hard, massive rock, Reclamation has often increased the maximum injection pressure by up to 50 lb/in^2 above the 1 lb/in^2/ft of depth. In bedded sedimentary rock units, Reclamation has often decreased the injection pressures to avoid uplift and fracturing. The tensile strength and three-dimensional effects in the rock mass are relied upon to avoid heave when the injection pressure exceeds the overburden pressure. An analysis of the rock mass stress relationships and tensile strengths along bedding planes may be necessary to evaluate whether or not the 1.0 $lb/in^2/ft$ injection pressure would result in damage to the rock mass. The grout injection pressures used in abutment fan holes should also be evaluated to reduce the likelihood of uplift or horizontal rock movement near the top of the abutments.

Reclamation requires the use of a circulating grout line for a foundation grouting project that includes a grout manifold (synonymous with "header"). Figure 15.6.3-2 illustrates a circulating line system. Figure 15.6.3-3 shows the standard Reclamation grout manifold design. This standard drawing number for the grout manifold is currently 40-D-7100 and is included in appendix E of this chapter. A grout manifold allows for monitoring and regulating grout flows and pressures. Figure 15.6.3-4 illustrates a grout manifold used at a Reclamation project.

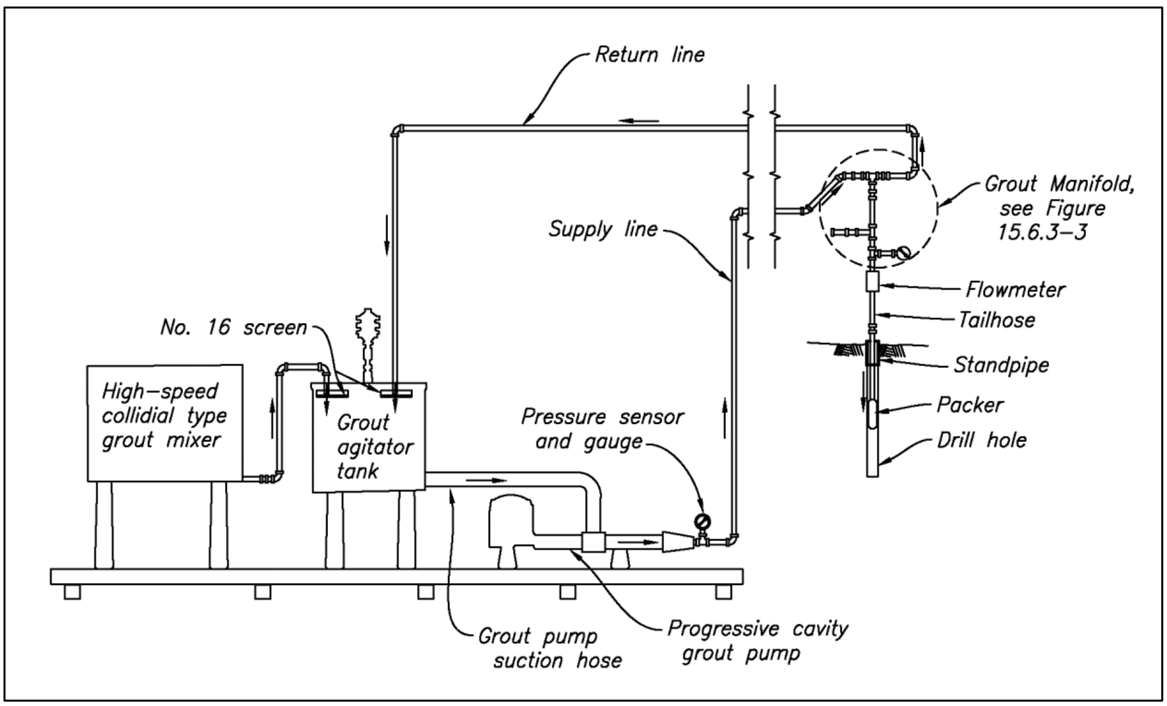

Figure 15.6.3-2. Circulating line system.

During grouting, it is important to ascertain if a stage has back pressure or holding pressure. Differentiating between back pressure and holding pressure takes place at the manifold (discussed in section 15.15.3.2). During a foundation grouting project, back pressure often occurs when the formation is being lifted or squeezed, and the deformation of the rockmass tends to compress and force the grout back out of the hole. Back pressure can also occur when grout flows above the injection point, such as in an abutment. Holding pressure is normal and occurs gradually as the stage approaches refusal. When the holding pressure equals the pumping pressure, grout is not being injected because equilibrium has been reached.

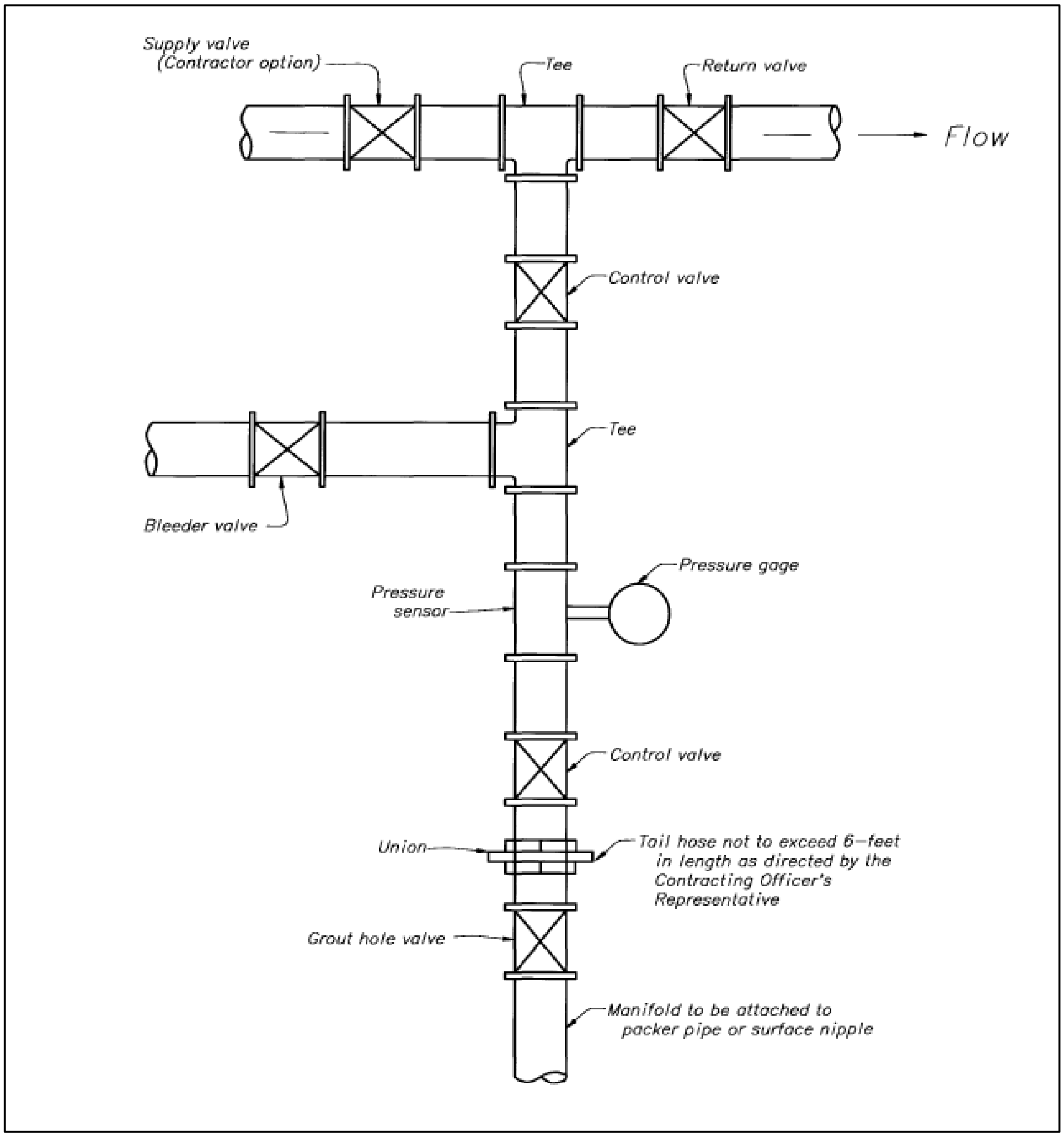

Figure 15.6.3-3. Reclamation's standard grout manifold drawing (flowmeter not shown).

Two pressure gauges, both protected by pressure sensors, should be installed in the circulating grout line. As discussed above, one gauge is installed on the manifold, and a second gauge should be installed in the outlet to the supply line at the pump. The gauge near the grout plant allows the operator to monitor and regulate the pressure to match the needs of the stage being grouted.

Figure 15.6.3-4. Grout manifold used at a Reclamation grouting project.

15.6.4 Grouting Methods

Methods of grouting commonly used by Reclamation include single-stage, upstage, or downstage grouting.

15.6.4.1 Single-Stage Grouting

Single-stage grouting consists of drilling the hole to full depth, washing the hole, water testing, and grouting the hole in one stage. This method should be limited to grouting shallow holes (± 20 feet) in relatively sound rock with no major surface leaks. Figure 15.6.4.1-1 illustrates single-stage grouting.

15.6.4.2 Upstage Grouting

Packers permit grouting of predetermined stages at any depth. Upstage grouting consists of drilling the hole to full depth, washing the hole, setting the packer within the drill hole at the top of the lowest stage level, water testing, and grouting the stage. Grouting of additional stages in the drill hole is performed upward using packers. Once refusal is reached on a stage, Reclamation practice is to proceed to water testing and grouting the next stage immediately unless back pressure is encountered. Water should be flushed through the packer to clean it between stages. If back pressure is encountered, water testing and grouting of the next stage is delayed until the grout in the previous stage has reached initial set (2 to 6 hours). When practical, Reclamation prefers to use upstage grouting because the drill may only need to be set up once, which leads to considerable cost and time savings compared to downstage grouting. Figure 15.6.4.2-1 illustrates upstage grouting.

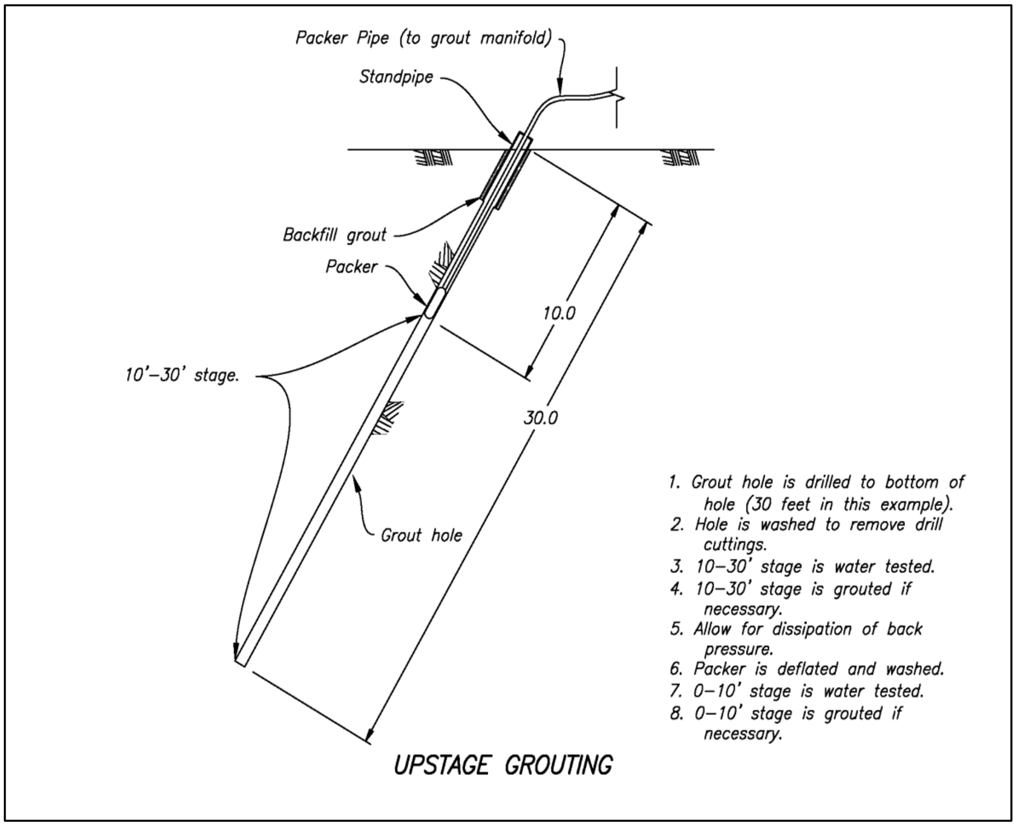

Figure 15.6.4.1-1. Single-stage grouting.

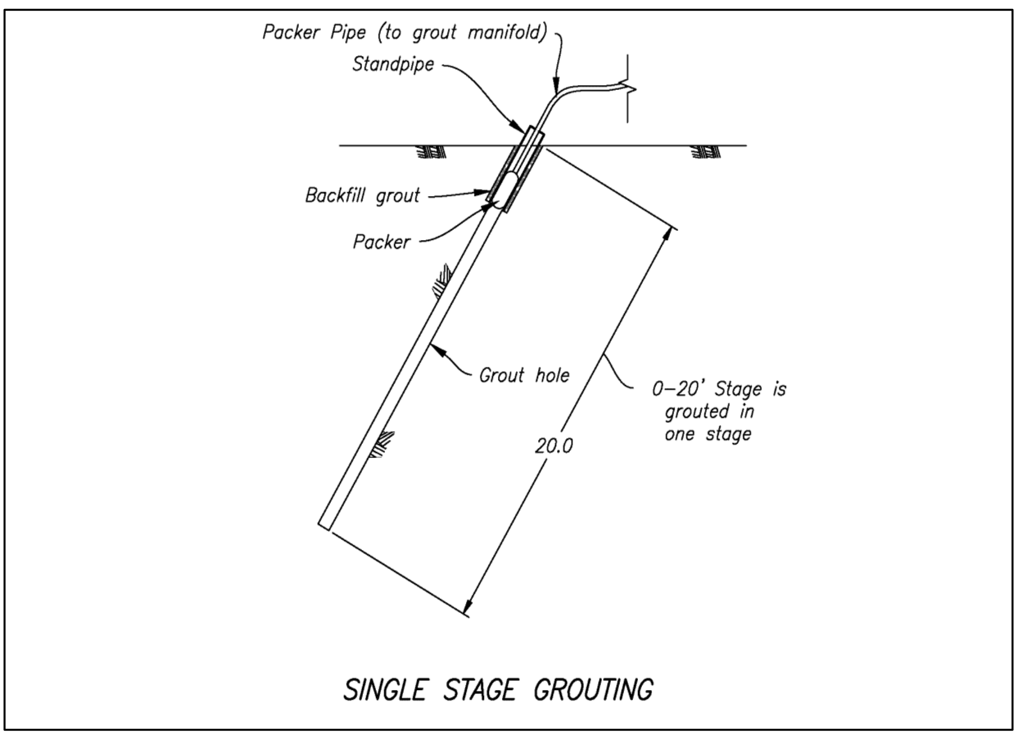

Figure 15.6.4.2-1. Upstage grouting.

When drilling an upstage grout hole, the contractor and inspector must closely monitor the rate of drilling water injection and return from the grout hole. If there is a drill water loss of 50 percent or more, or if artesian flows are encountered when drilling a grout hole for upstage grouting, Reclamation practice is to drill several additional feet below the pervious feature. The pervious feature would then be cleaned, water tested, and grouted in accordance with the specifications. Once the grout has set, the remaining length of the grout hole can be drilled. Figure 15.6.4.2-2 illustrates upstage grouting when water losses are encountered.

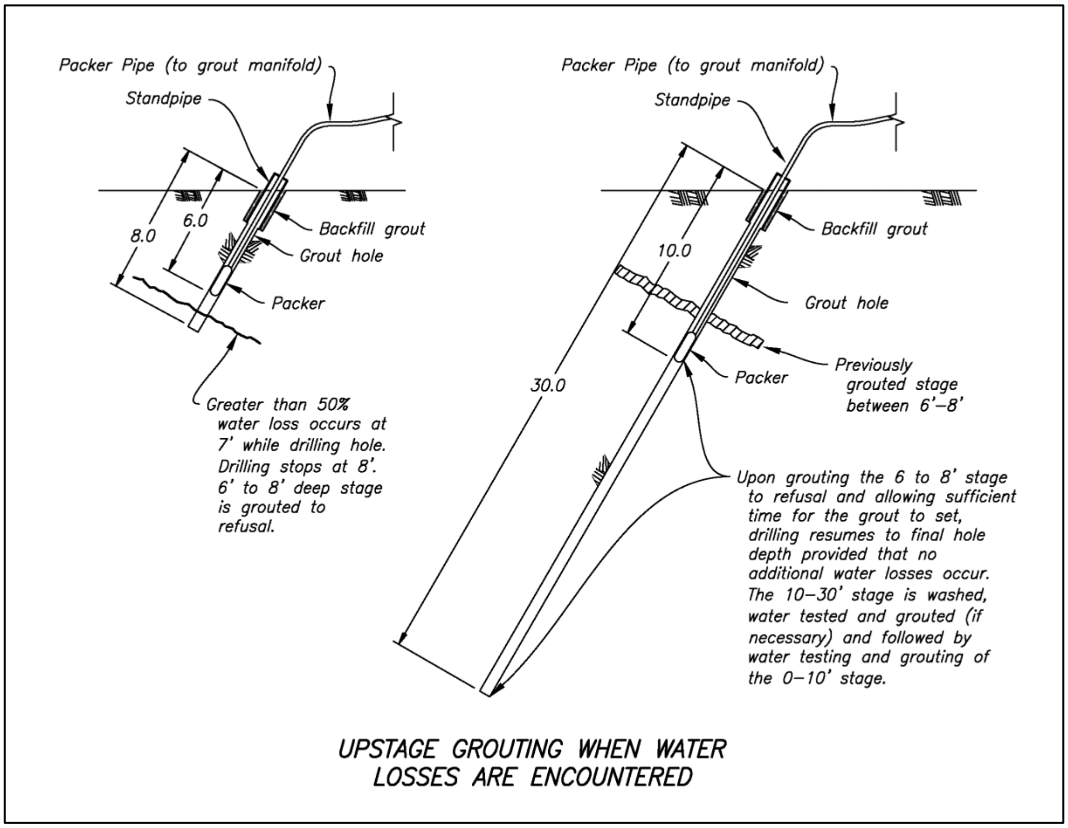

Figure 15.6.4.2-2. Upstage grouting when water loss occurs.

15.6.4.3 Downstage Grouting

In descending stages, the hole is staged from the top downward by drilling one stage at a time. Once the first stage is drilled, washed, water tested, and grouted, sufficient set time must be provided before redrilling to prevent washout of partially set grout in the stage. Once the grout has sufficiently set, an additional stage length is drilled. The packer is set at the top of the new stage, followed by water testing and grouting. Subsequent lower stages are drilled and grouted in a similar manner. Because drills have to be set up multiple times on one hole, downstage grouting is more expensive than upstage grouting and is only used by

Reclamation if poor conditions (such as caving, squeezing, or packer leakage) within the grout hole are encountered. Figure 15.6.4.3-1 illustrates downstage grouting of a grout hole.

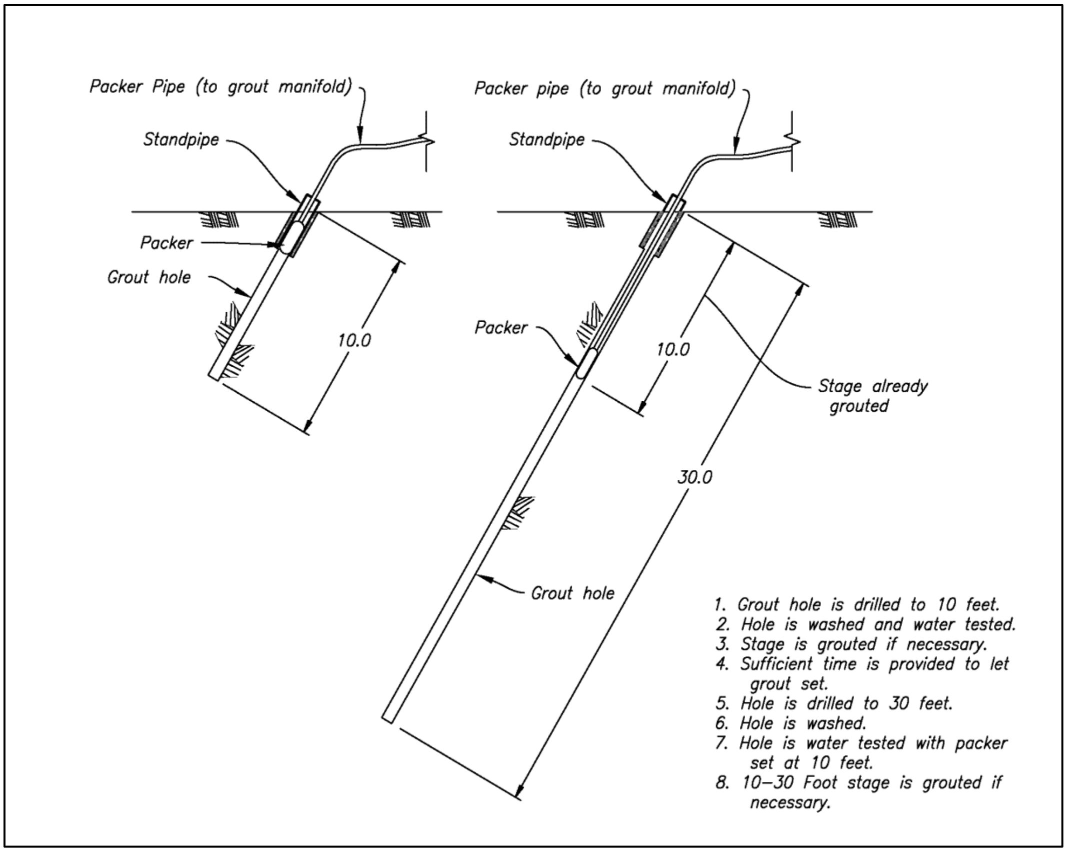

Figure 15.6.4.3-1. Downstage grouting.

If there is a drill water loss of 50 percent or more, or if artesian flows are encountered when drilling a grout hole for downstage grouting, Reclamation practice is to drill several additional feet below the pervious feature. The pervious feature would then be cleaned, water tested, and grouted in accordance with the specifications. Once the grout has set, the remaining length of the grout hole can be drilled.

If no blanket grouting is included in the design, and excessive surface leaks are occurring that delay the project, consideration should be given to grouting the shallowest stage of a curtain hole first to seal near surface leaks, followed by drilling to the lowest stage level and using upstage grouting procedures to grout the remaining stages in the drill hole. Sealing near-surface leaks at a lower injection pressure in shallow holes can save considerable time if delays are occurring from surface leaks while deeper stages are grouted under larger injection pressures.

Staging downward is recommended when the foundation formation is relatively soft and the drill water tends to wash the hole larger, which could prevent proper seating of a packer in ascending stages. Staging downward is also helpful for grouting jointed rock that allows grout to bypass the packer or stabilizing a caving formation by grouting and allowing the grout to set before redrilling.

15.6.4.4 Multiple Port Sleeved Pipe

If grouting is necessary in extremely poor rock conditions, and the methods discussed above will not work (such as in friable rock or collapsing borehole walls) multiple port sleeved pipe (MPSP) casing provides grouting ports at regular intervals along the casing. Reclamation has no dams in its inventory for which MPSPs were necessary for grouting. If extremely difficult rock is encountered, Bruce and Weaver [4], the U.S. Army Corps of Engineers (USACE) [8], and Houlsby [1] provide some guidance on MPSP systems.

15.6.5 Stage Length

Reclamation typically specifies shorter stage lengths near the rock surface and longer stage lengths at greater depths within the grout hole. As the injection pressure increases with depth, longer stage lengths are used. The shallowest stage in a grout curtain hole is often limited to 0 to 10 feet. Between depths of 10 feet and 90 feet, each stage length is often limited to 20 feet. Between depths of 90 feet and 220 feet, each stage length is typically limited to 30-foot stage lengths. At depths exceeding 220 feet, stage lengths of 40 feet can be used. In poor rock conditions, the stage lengths at depth may need to be reduced as needed.

15.6.6 Single Row Versus Multiple Row Grout Curtain

When using Reclamation’s optimum mix technique, a single row of grout holes has commonly been used by Reclamation to form the grout curtain. For dams where geologic conditions are difficult, such as highly fractured rock, soluble rock, presence of large voids, or high permeability rock, multiple rows of grout holes may be necessary to form the grout curtain. When three rows of holes are planned, the upstream and downstream rows are grouted ahead of the middle row. The two outer rows serve as barriers to limit grout travel from the middle row (or closure row).

If stable grout mixes are exclusively used for a grouting project, current grouting industry practice would require the use of a multiple row grout curtain.

15.6.7 Grout Hole Spacing and Closure Criteria

15.6.7.1 Split Spacing Method

Reclamation uses the split spacing method to obtain closure during foundation grouting. Curtain grouting is initially divided into “pattern” lengths. A typical Reclamation closure pattern is 80 feet long with grout holes on 10-foot centers measured perpendicular from one hole to the next, and consists of primary (P), secondary (S), tertiary (T), and quaternary (Q) grout holes. Using the optimum mix technique, grout travel can be extensive in some cases. The 80-foot spacing of the primary holes limits the likelihood of communication to adjacent primary grout holes. Figure 15.6.7.1-1 illustrates the layout of primary, secondary, tertiary and quaternary grout holes within an 80-foot closure pattern.

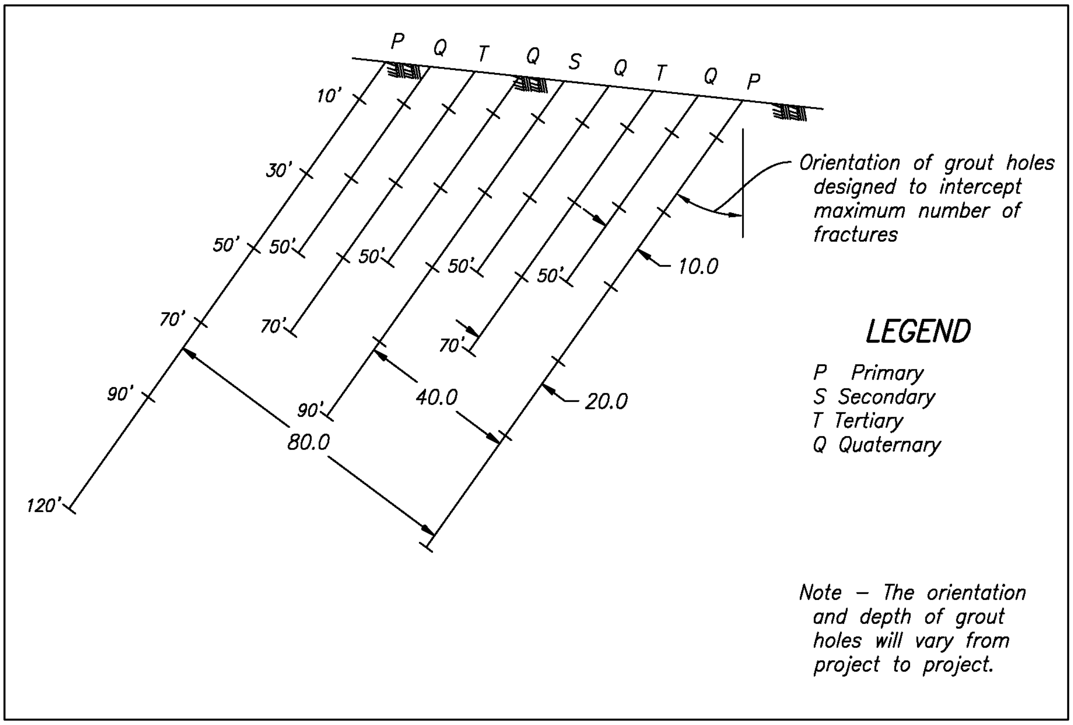

Figure 15.6.7.1-1. Typical Reclamation 80-foot grout hole pattern using the split spacing method.

The split spacing method begins with drilling and grouting the primary holes. Upon completing grouting of the primary holes within a closure pattern, the secondary hole is drilled halfway between the primary holes and grouted. Upon completing grouting of the secondary grout hole within a closure pattern, the tertiary holes are drilled halfway between the secondary and primary holes and grouted. The quaternary holes are then drilled halfway between the primary, secondary, and tertiary grout holes and grouted. For a single row grout curtain or the closure row in a multiple row grout curtain, additional split spacing may be necessary, depending on the actual grout ‘takes’ in the quaternary holes and the closure criteria. Section 15.6.7.4 discusses the method of determining the need for additional grout holes based on the grout 'takes' and closure criteria. When grouting the upstream and downstream rows in a multiple row grout curtain, no additional split spacing is performed upon completing the scheduled grout holes in each grout pattern.

Ideally, the grout 'takes' and water test results should decrease as closure progresses. The optimum mix will likely be somewhat thinner in the closure tertiary and quaternary holes than in the primary holes. Reclamation experience has shown that the primary grout holes may take 55 to 70 percent of the total volume of cement, the secondary holes may take 10 to 30 percent of the total volume of cement, the tertiary holes may take 5 to 20 percent of the total volume of cement, and the quaternary holes may take 0 to 5 percent of the total volume of cement. Additional closure holes usually account for less than 1 percent of the total volume of cement 'take.' If unusually high 'takes' occur in the closure holes, an assessment must be made to determine the cause.

15.6.7.2 Water Testing of Grout Holes

Prior to pressure grouting in the grout curtain holes, each stage shall be tested with clean water under continuous pressure. The pressure used for each water test should be equal to the pressure anticipated for grouting. In upstage grouting, Reclamation does allow the contractor to water test and grout the lowest stage and then upstage with water testing and grouting of the remaining stages in the hole. The packer should be cleaned by flushing the hole with water between grouting and water testing.

Once waterflow into the drill hole has stabilized, the water pressure test performed prior to grouting is performed again for at least 5 minutes per stage. Longer periods may be needed if significant water 'takes' occur to verify the rate of 'take' or to wash out material from foundation discontinuities. Figure 16.6.7.2-1 illustrates a water test manifold configuration used by a contractor on a Reclamation grouting project.

Reclamation uses the results of the water test to determine if grouting of a stage is necessary. Reclamation experience has shown that if a formation 'takes' less than 1 cubic foot (ft^3) of water over a 5-minute water test duration, the stage is likely to accept little to no grout. The criteria for when to grout a stage (referred to as a "hookup" in grouting specification paragraphs) must be verified on every job for each location of the grout curtain alignment (left abutment, valley, and right abutment), as well as for variations with depth and geology. **Reclamation recommends using an initial criterion for hookups of 1 ft^3 of water or greater over 5 minutes**. If minimal grout 'takes' are occurring at somewhat low water test values (1.0 to 2.0 ft^3 over 5 minutes), the criterion for hookups could be increased to 2 ft^3 of water or greater over 5 minutes. If large grout 'takes' are occurring at somewhat low water test values (1.0 to 2.0 ft^3 over 5 minutes), the criterion for hookups could be reduced to 0.5 ft^3 of water or greater over

5 minutes. If, at any time, the grouting engineer or grout inspectors feel that a hookup is necessary on a particular stage, regardless of the water test result, this judgment should outweigh the result of the water test.

Figure 15.6.7.2-1. Typical manifold configuration used by contractors for water tests.

15.6.7.3 Stage Refusal

For a single row grout curtain and the closure row within a multiple row grout curtain, Reclamation recommends discontinuing grouting when the grout rate of flow is less than 1 ft^3 of grout in 20 minutes if pressures of 50 lb/in^2 or less are used, in 15 minutes if pressures between 50 and 100 lb/in^2 are used; or in

10 minutes if pressures between 100 and 250 lb/in^2 are used. Grouting of any stage could also be discontinued when less than two bags of cement per hour are being injected during continuous pumping over a period of 2 hours.

For the upstream and downstream grout curtain rows within a triple row grout curtain, Reclamation recommends doubling the stage refusal criteria established for the closure row.

15.6.7.4 Grout Pattern Closure Criteria

Reclamation uses the total quantity of cement injected into each stage to determine grout pattern closure criteria. Reclamation's current closure criterion for a single row grout curtain, or the closure row within a multiple row grout curtain, is 0.5 bags of cement per foot of linear drill hole length (bags/ft) within each split-spaced grout closure pattern for each stage. Reclamation dams that have been grouted to this closure criterion have shown exceptional performance. In addition, many of the dams built using this criterion have been located in areas where water is a precious resource and minimal seepage was desired. If constructing a dam in areas where some seepage can be tolerated, it may be acceptable to loosen the closure criterion to 1.0 bags/ft. Prior to 1970s, many Reclamation grout curtains used 1.0 bag/ft as the closure criterion, and their performance has been satisfactory.

For the upstream and downstream grout curtain rows within a multiple row grout curtain, Reclamation recommends doubling the closure criteria established for the closure row to determine the final depth of each grout hole.

For a single row grout curtain or the closure row within a multiple row grout curtain, the final hole spacing and depth of the grout holes are dependent on the actual grout 'takes.' The primary holes are always drilled to the design depth of the grout curtain. A minimum length is provided in the specifications for all other scheduled holes. Figure 15.6.7.1-1 provides an example of the initial layout of a 120-foot-deep grout curtain. The final maximum depths of all other grout holes in the 80-foot closure pattern are dependent upon the grout 'take' within each stage in the preceding series of holes (primary, secondary, etc.). If the closure criteria at the bottom stage in any hole is exceeded, the next hole series adjacent to this grout is usually extended another 20 feet deeper than the design depth.

The following information provides guidance for determining the depths of a secondary grout hole between two primary grout holes:

- Upon completing the adjacent primary holes, the secondary hole within a closure pattern should extend to the design depth or to the lowest stage in the primary grout hole at which grout 'takes' exceeded a ½ bag/ft plus 20 feet. In instances where the stage at the bottom of the grout curtain in the primary grout hole has grout 'takes' that exceed a ½ bag/ft, the designer and field staff may elect to extend the depth of the secondary hole below the depth of the primary hole to obtain closure at depth.

- The greater of the two depths is selected in the field for the secondary grout hole.

- Figure 15.6.7.4-1 shows an example for determining the depth of secondary grout holes.

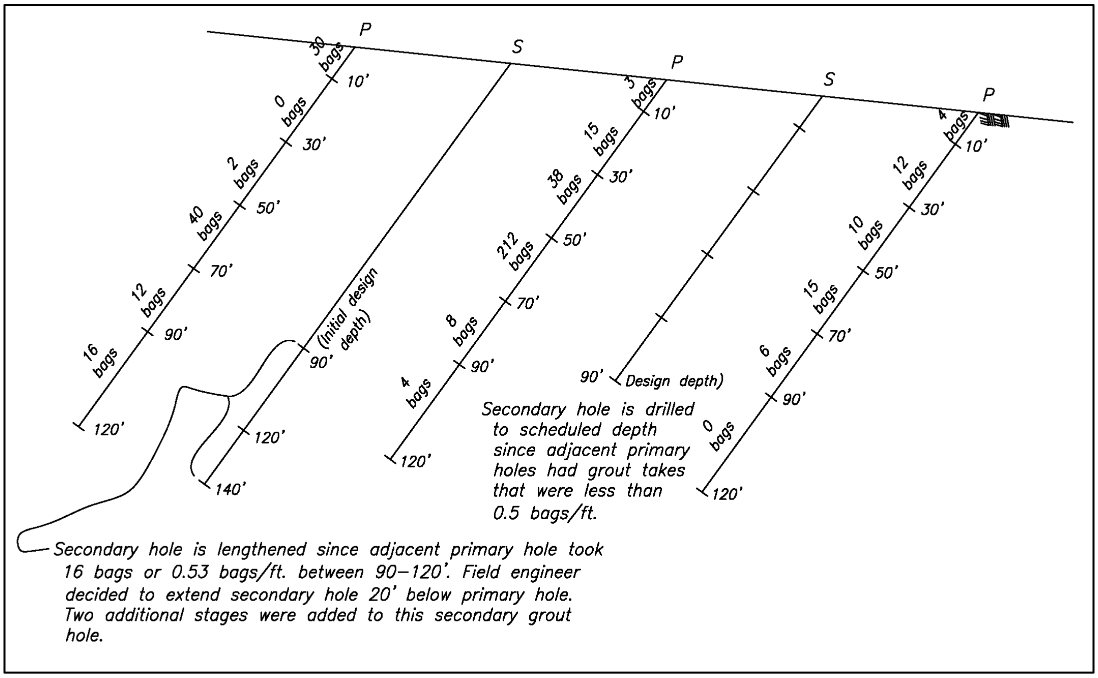

Figure 15.6.7.4-1. Evaluation of secondary hole depth within an 80-foot grout hole pattern.

The following information provides guidance for determining the depths of the tertiary grout holes.

- Upon completing the primary and secondary grout holes, the tertiary grout holes should extend to the design depth or to the lowest stage (or lower, depending on geology) at which an adjacent secondary grout hole had grout ‘takes’ that exceeded a ½ bag/ft plus 20 feet.

- The greater of the two depths is selected in the field for the tertiary grout hole.

The following information provides guidance for determining the depths of the quaternary grout holes.

- Upon completing the primary, secondary, and tertiary grout holes, the quaternary grout holes should extend to the design depth or to the lowest stage (or lower, depending on geology) at which an adjacent tertiary grout hole had grout 'takes' that exceeded a ½ bag/ft plus 20 feet.

- The greater of the two depths is selected in the field for the quaternary grout hole.

If any of the quaternary grout hole stages within a single row grout curtain or closure row in a multiple row grout curtain exceed ½ bag/ft, additional closure holes would be added using the split spacing method until closure is obtained to ½ bag/ft or less. If closure is not obtained on quinary holes, which would be the 5^{th} series of grout holes in a closure pattern and would reduce the grout hole spacing to 5-foot centers, consideration should be given to locating additional grout holes (senary and septenary) upstream or downstream of the grout curtain, rather than locating grout holes on 2.5-foot centers.

For the upstream and downstream rows in multiple row grout curtain, only the scheduled grout holes would be grouted. No additional split spacing outside of the scheduled grout holes is performed. The guidance discussed in this section for determining the final depth of each grout hole would not change for the upstream and downstream rows within a multiple row grout curtain.

15.6.8 Grout Standpipes and Grout Cap

Prior to drilling and grouting, grout standpipes (also known as nipples or grout collars) are installed into the rock formation to facilitate drilling and grouting. Grout standpipes consist of steel pipes of an appropriate diameter to accommodate the drill hole size necessary for grouting. On most Reclamation projects, the inside diameter of the standpipe is the same nominal diameter as the drill bit. When drill hole caving issues arise, the standpipe diameter is typically increased by ½ inch to accommodate casing pipe within the drill hole if it is necessary. The inside diameter of the standpipe should not be more than ½ inch larger than the anticipated drill bit size. The use of larger standpipes results in a decrease in velocity of the flushing fluids through the standpipe and results in issues related to removing drill cuttings from the hole.

Figure 15.6.8-1 shows a typical grout standpipe detail. The grout standpipes are installed at all scheduled hole locations within the 80-foot closure reach prior to drilling. The grout standpipes should extend at least 6 inches above the rock surface and are typically set at least 2 feet into bedrock to hold the pipe firmly. Additional depths may be needed to provide a firm base. The height of the grout standpipes above rock should not interfere with the drill rig specified for the project. The standpipes should be set within 1 degree (in both the horizontal and vertical direction) to the desired inclination to avoid "windows" in the grout curtain. Figure 15.6.8-2 illustrates a contractor drilling and setting standpipes along a single row grout curtain.

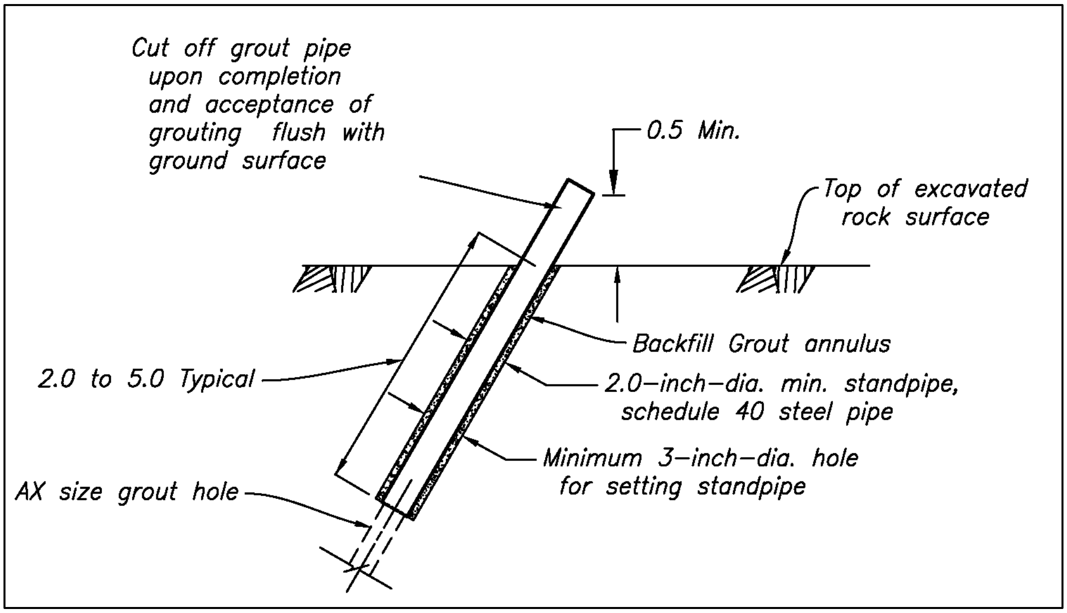

Figure 15.6.8-1. Typical grout hole standpipe detail.

Figure 15.6.8-2. Contractor setting standpipes along a single row grout curtain.

In cases where bedrock is badly jointed or broken below the surface, a grout cap may be necessary to establish a cutoff at the surface. Badly jointed or broken rock near the surface often leads to excessive surface leaks near surface in the vicinity of the grout standpipe. Surface leaks can limit grout pressure and, subsequently, grout travel. Caulking leaks under these conditions is sometimes difficult. The grout cap is usually about 3 feet wide and 3 to 8 feet deep. If blasting is needed for this excavation, a grout cap is probably unnecessary. Reclamation has typically used a rock saw and/or standard excavation equipment for the grout cap excavation. The grout cap is created by placing concrete into the excavation. The grout standpipes are embedded in the grout cap at the specified hole inclination and direction. Figure 15.6.8-3 shows a typical detail for a grout cap.

Table 15.6.8-1 summarizes the advantages and disadvantages of a grout cap. It should be noted that Reclamation practice is to avoid the use of grout caps under embankment dams, if possible, because of the difficulty in sealing the contact between the grout cap and the foundation rock. In most embankment dam foundations, it is difficult to keep the grout cap at the same elevation as the rock, which results in additional foundation treatment and special compaction in this area. Alternatives to grout caps may include extending the depth of the standpipe or leaving the foundation high and performing the final excavation after grouting is complete, provided that no blasting is required for the final foundation excavation. Blanket grouting, which is discussed in section 15.7, may also reduce the need for a grout cap.

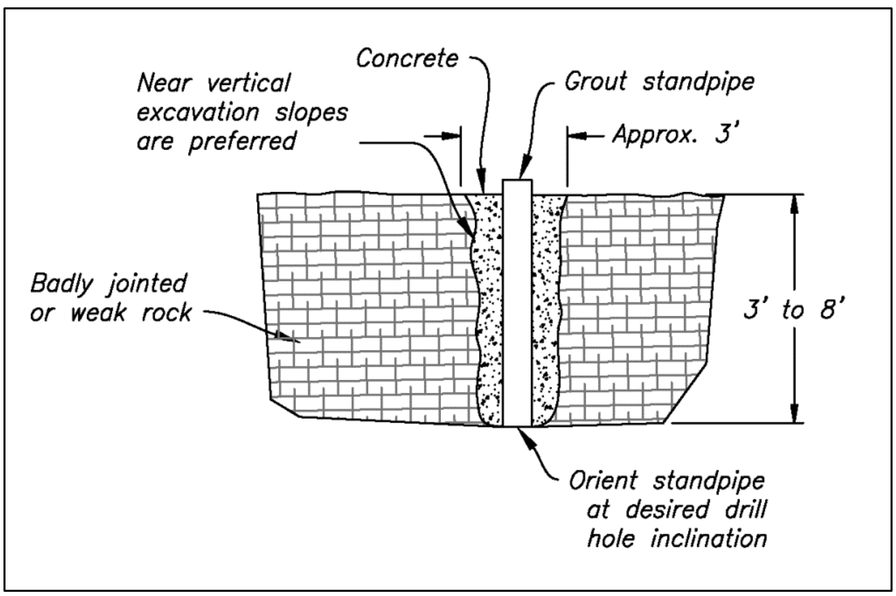

Figure 15.6.8-3. Grout cap detail.

Table 15.6.8-1. Advantages and disadvantages of a grout cap

Grout Cap	
Advantages	**Disadvantages**
Good anchorage for standpipes and may allow for the use of shorter standpipes	Increased costs associated with excavation and concrete
Forms near-surface seepage barrier of zone 1 contact	Potentially creates the need for special compaction, particularly where rock deteriorates next to the grout cap, or if the grout cap is not flush with the foundation surface
Provides good work platform for drilling and grouting	Excavation for the cap may disturb (damage) the foundation
Provides control for heave monitoring and inspection	

15.7 Blanket Grouting

15.7.1 Location

Blanket grouting (also known as "consolidation grouting") involves the use of low-pressure shallow grouting to seal near-surface bedrock beneath the impervious core of the embankment and provide a firmer foundation for the dam. For an embankment dam, blanket grouting is used in areas of highly fractured rock or high permeability, open jointed rock that may lead to seepage and internal erosion issues when the reservoir is filled. Blanket grouting is typically limited to the impervious core footprint beneath the embankment. In most cases, the foundation beneath the impervious core requires some blanket grouting. The extent of blanket grouting is initially laid out by the designer, grouting engineer, and project geologist. After the final excavation surface is exposed, the final extent of blanket grouting is reviewed and adjusted by the designer, grouting specialist, and field staff.

Blanket grout holes are scheduled grout holes that are shown in the design drawings. Blanket grouting is performed prior to curtain grouting to limit the amount of surface leakage during higher pressure curtain grouting.

15.7.2 Depth

Blanket grout holes are typically 20 to 30 feet deep. For embankments that are less than 100 feet high, 20-foot-deep blanket grout holes are typically sufficient.

For larger dams, Reclamation typically uses 30-foot-deep blanket grout holes. Geologic factors may influence the depth of the blanket grout holes.

15.7.3 Stage Length

Blanket grout holes can be grouted in one stage, or two stages, depending on the depth and the amount of surface leakage observed during grouting. Blanket grout holes that are 30 feet deep are typically grouted into two stages: 0 to 10 feet, and 10 to 30 feet. Blanket holes that are 20 feet deep are often grouted in one stage, unless extensive surface leaks occur. If surface leaks occur, the 20-foot-deep hole may be separated into two stages, with the shallow stage grouted first at lower pressure, in an effort to limit surface leakage from the higher pressure stage.

15.7.4 Pressure

The top stage, or only stage, for blanket grout holes is typically limited to 5 to 10 lb/in^2 at the manifold, plus back pressure due to artesian waterflows. Injection pressures are adjusted in the field, as necessary, to safeguard against foundation displacement. In hard, sound rock, Reclamation has used an injection pressure as high as 25 lb/in^2 for the top stage. If two stages are necessary, the maximum injection pressure for the lower stage is limited to 1 lb/in^2/ft of depth, plus back pressure measured from the surface to the packer.

15.7.5 Blanket Grouting Methods

When practical, Reclamation prefers to use upstage grouting procedures because the drill may only need to be set up once, which results in considerable cost and time savings, compared to downstage grouting, unless caving or a loss of water circulation occurs.

Section 15.6.4 discusses the procedures for upstage, downstage, and single-stage drilling.

15.7.6 Spacing and Closure Criteria

15.7.6.1 Split Spacing Method

Blanket grout holes are typically arranged in a grid and are spaced 10 to 20 feet apart, depending on the nature of the bedrock surface. Figure 15.7.6.1-1 shows a typical Reclamation blanket grout hole pattern with a grout hole spacing at roughly 10-foot centers. The primary grout holes are initially drilled and grouted on 40-foot centers within each row. The location of the primary grout holes are typically offset in adjacent rows. The secondary grout holes are then drilled and grouted at the midpoint between the primary grout holes within each row. Tertiary grout holes are then drilled and grouted at the midpoint between the primary and secondary grout holes within each row. The quaternary and quinary grout holes are located within a new row. The quaternary grout holes are drilled and grouted at the midpoint between the primary and secondary grout holes in adjacent rows. The quinary grout holes are drilled and grouted at the midpoint between the quaternary grout holes. If the closure criterion is not reached on the quinary grout holes, additional grout holes are then drilled and grouted around the quinary grout holes that exceeded the closure criteria. Figure 15.7.6.1-2 illustrates an array of blanket grout holes on a recent Reclamation grouting project.

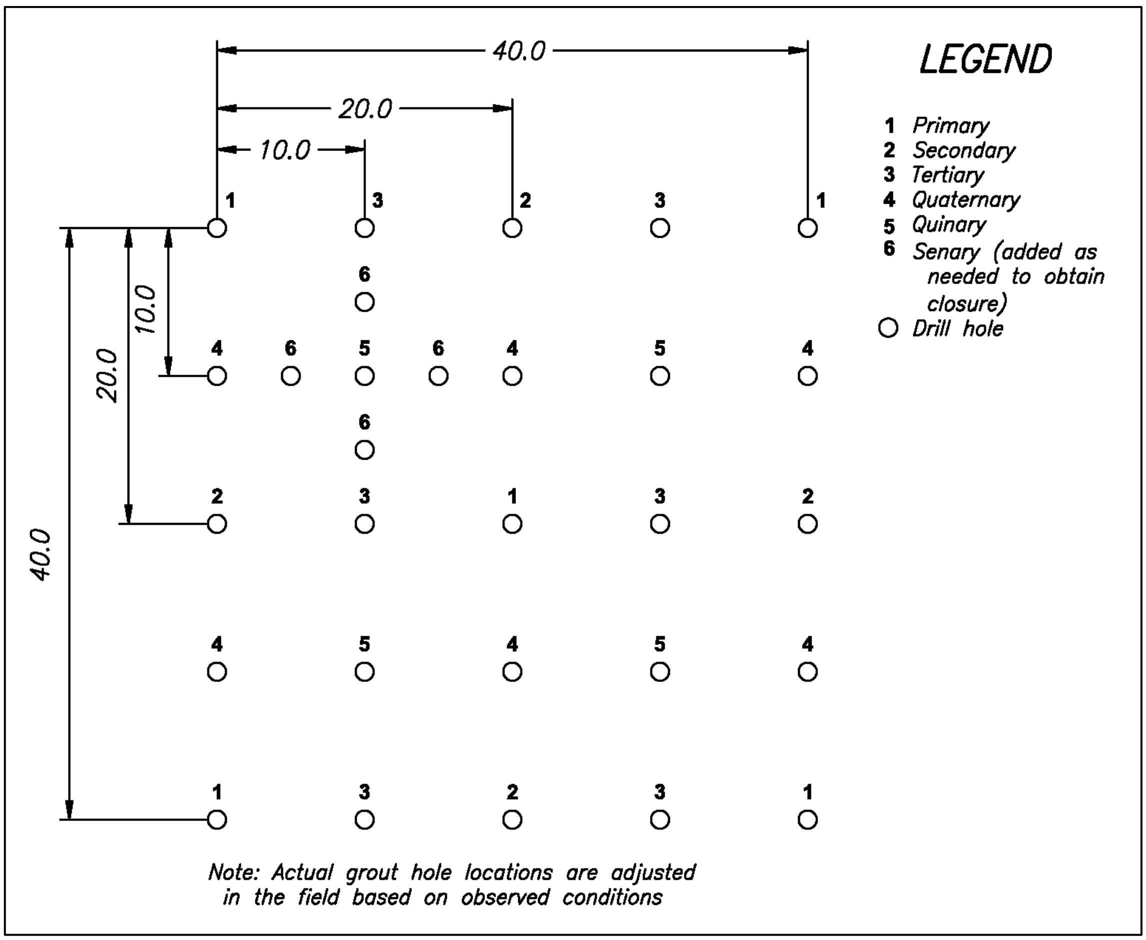

Figure 15.7.6.1-1. Plan view of typical Reclamation blanket hole layout.

Figure 15.7.6.1-2. Drilling and grouting of blanket grout holes on a Reclamation project.

15.7.6.2 Water Testing

Reclamation's best practices for water testing a grout hole stage and when to grout a stage, based on the water test result, were previously discussed for the grout curtain in section 15.6.7.2. Reclamation's water testing practices for water testing blanket grout holes are similar to the practices discussed for curtain grouting.

15.7.6.3 Stage Refusal Criteria

Reclamation's best practices stage refusal were previously discussed for the grout curtain in section 15.6.7.4. Reclamation's stage refusal criteria for blanket grout holes are similar to the criteria discussed for curtain grouting.

15.7.6.4 Grid Closure Criteria

The blanket grout holes often have the same closure criteria selected for the grout curtain holes. If any of the quinary grout holes exceed the closure criteria, additional quinary grout holes should be added, as needed, to obtain closure. Figure 15.7.6.4-1 shows how to evaluate the need for additional closure holes. As shown in the figure, one of the quinary holes had a stage that exceeded ½ bag/ft. For this reason, four senary grout holes were added around the quinary grout hole that exceeded that closure criteria.

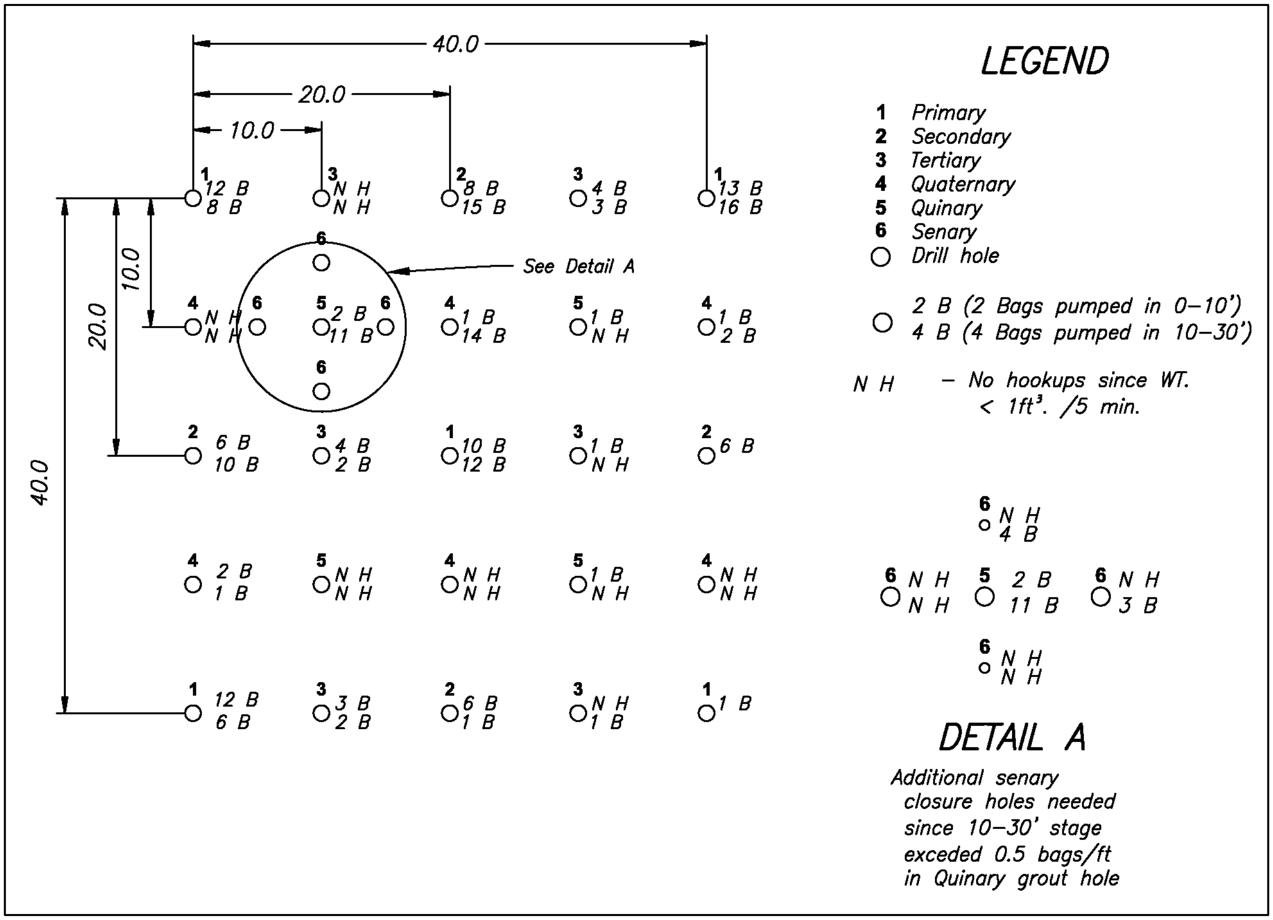

Figure 15.7.6.4-1. Evaluation of closure for blanket grout holes.

15.8 Stitch Grouting

Reclamation uses the term "stitch grouting" to refer to grouting that is performed to seal isolated pervious fractures, such as fault zones, that are exposed on the foundation surface that lie outside of the blanket grouting footprint and within the footprint of the impervious core. Provisions are made in the specifications that allow the field staff to add additional grout holes for stitch grouting.

The design for stitch grouting is very similar to the design for blanket grouting. The orientation of the fracture(s) needs to be known to allow for optimal grout hole inclination. Stitch grout holes are typically about 30 feet deep. Primary grout holes are usually laid out on a 20- or 40-foot pattern along very large fractures, and faults with closure holes are added in between them as needed.

15.9 Special Conditions

15.9.1 Remedial Grouting Through Embankments

If remedial grouting beneath an existing embankment dam is performed, care must be taken to avoid damaging the embankment during drilling, washing the hole, water testing, and grouting. A conservative "do no harm" approach must always be taken when grouting beneath an existing embankment dam. When designing a remedial grouting program through an existing embankment dam, the grout mix selected, allowable injection pressures, curtain location and depth, grouting method, etc., must include consideration for dam and foundation specific features such as the orientation of joints and bedding planes within the foundation rock, filters, drains, instrumentation, etc.

Some common issues that could occur during drilling and grouting include:

- Excessive grout pressures within the foundation bedrock may result in hydraulic fracturing of the embankment.

- Poor embankment/foundation contact seal, combined with damage from drilling and grouting fluids, result in erosion of the embankment materials.

- Poor embankment/foundation contact seal, combined with damage from drilling and grouting fluids, result in hydraulic fracturing of the embankment.

To avoid these issues, casing must be installed through the embankment or overburden materials prior to initiating drilling and grouting through the foundation bedrock, and the casing must extend a minimum of 2 feet into bedrock.

15.9.1.1 Drilling

Reclamation prefers the use of hollow stem augers and sonic drilling within embankment dams and prohibits the use of air and fluid based drilling methods because the use of air or drill fluid within an embankment dam can result in hydraulic fracturing. Compressed air drilling should never be used within an embankment dam. In some cases, rotary duplex drilling may be necessary and approved when drilling within an embankment dam or overburden materials for remedial grouting. The use of water within the inner drill string within a duplex drill may be approved in special cases provided that all of the drill cuttings and fluid are returned to the surface within the annular space between the inner and outer casing. The use of mud is not recommended because the casing will be embedded several feet in rock, and this mud could plug off fractures at the embankment/foundation contact that would accept grout. If hollow stem augers are specified, a duplex drill with an auger as the inner drill string would allow a flush joint casing to be installed with the outer drill string. The advantage in this scenario is that the augers can be retrieved, and the outer casing can be pulled, after the annulus between the outer casing and standpipe is grouted. Reclamation would permit the outer casing to be installed with rotation. Advancing the outer or inner casing via wet or dry percussion is not permitted. The initial drilling within the embankment must extend 2 to 5 feet into sound bedrock. The initial bedrock embedment depth of the casing is based on the quality of the bedrock at the embankment/foundation contact. Drill holes should be inclined to intercept the maximum number of fractures possible per foot of drilling.

15.9.1.2 Casing of Grout Holes

After the outer casing is installed, a standpipe is placed within the drill hole to allow for future drilling and grouting within bedrock, while providing a proper seal through the embankment. The size of the inner casing material is selected based on the drill hole size necessary for drilling and grouting within bedrock. Polyvinyl chloride (PVC) pipe is often used as the standpipe material.

Care must be taken to ensure that a proper seal is developed through the embankment and at the embankment/foundation contact. Once the standpipe and outer casing are installed, the annulus between the two pipes must be grouted. The procedure for grouting this area depends on whether or not the interface at the embankment/foundation contact needs to be grouted. For most Reclamation embankments where zone 1 materials were placed directly on bedrock, final foundation cleanup likely removed weathered material and poor rock at the bedrock surface, and high quality compacted fill was placed directly over bedrock. In this scenario, treatment at the interface is probably unnecessary, and the annulus between the inner and outer casing would be grouted in one stage. Tremieing grout between the two pipes at the surface is unlikely to result in complete filling of the annulus and should not be permitted in the specifications. Filling of the annulus should occur from the bottom up. As the annulus is being grouted, the outer casing can be removed; however, care needs to be taken to ensure that the grout level is kept above the bottom of the outer casing, while it is being withdrawn, to reduce the likelihood of collapse of the drill hole walls. After grout is flowing from the surface, sufficient time must be provided to allow the grout to set.

In cases where grouting is performed at natural soil/rock interface zones, special treatment of this area may be necessary. The following quotation, taken from USACE *Engineering Manual* (EM) 1110-2-3506 [8], discusses why it may be necessary to isolate the embankment/foundation contact area:

> 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 [8].

Stare et al. [9] recommend a procedure using a MPSP system to isolate the soil rock interface zone in these conditions. Reclamation supports this procedure. Figure 15.9.1.2.1, taken from Stare et al. [9], shows this procedure. The barrier bag noted in figure 15.9.1.2.1 is shown in figure 15.9.1.2-2. This procedure allows for proper grout mix selection to seal this contact area, rather than using the same grout mix that was used for filling the annular space between the casing. It also allows for monitoring of the amount of grout injected into the contact area. The maximum injection pressure for grouting the embankment/foundation contact should be limited to 5 lb/in^2 or less. Gravity grouting is preferred at this location.

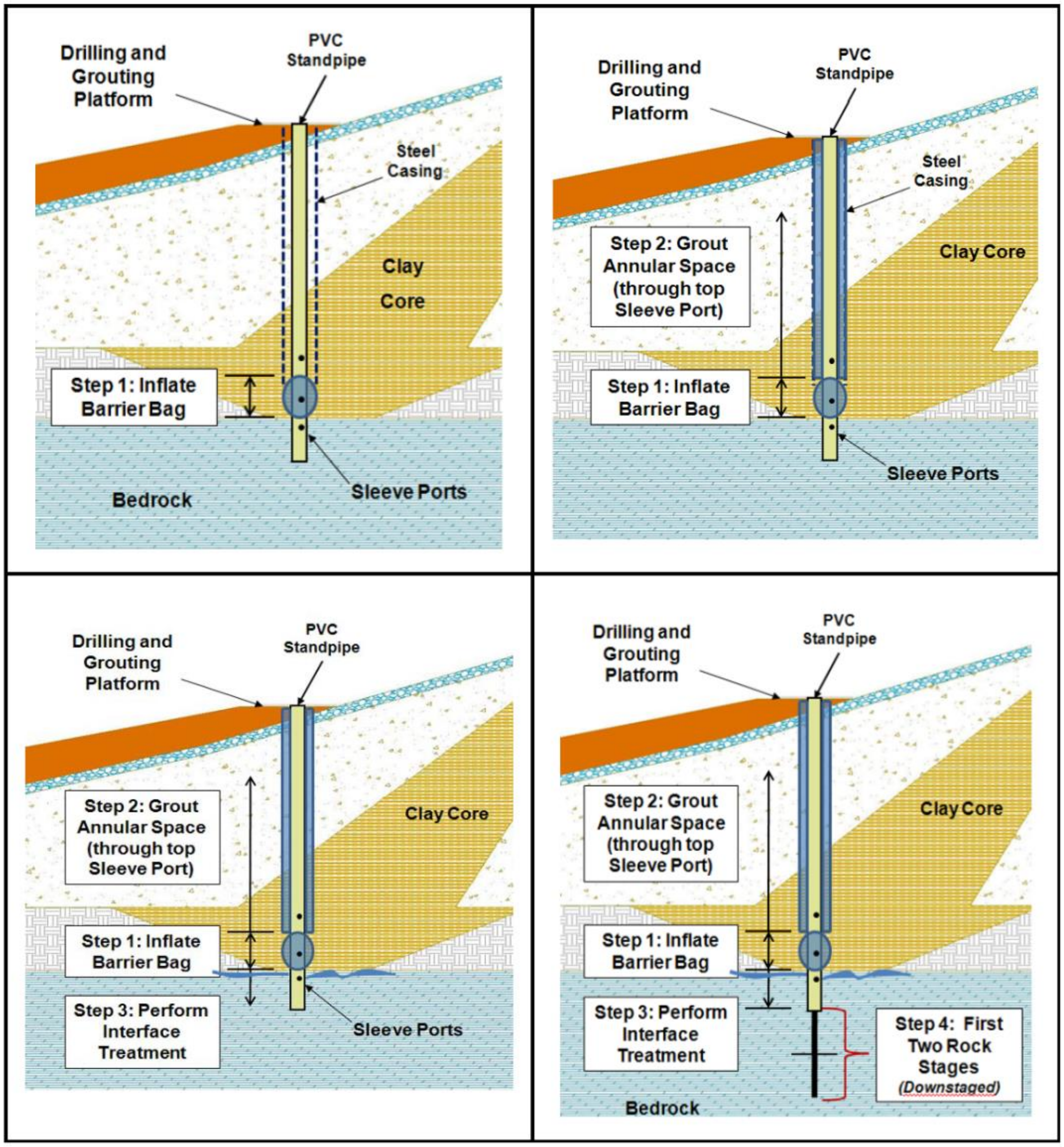

Figure 15.9.1.2-1. Illustration of standpipe and interface zone grouting using MPSP system (from Stare et al. [9], with permission from the American Society of Civil Engineers).

Multiple Packer Sleeved Pipe (MPSP) MPSP with Textile Barrier Bag

Figure 15.9.1.2-2. MPSP and MPSP with textile barrier bag (from Stare et al. [9], with permission from the American Society of Civil Engineers).

If the MPSP system is not used to isolate the soil/rock interface zone, the amount of grout necessary to fill the annular space between the standpipe and borehole wall should be closely examined. If the volume of grout that is used to fill the annular space exceeds the approximate volume of the annular space, grout is probably flowing into the interface zone. If this occurs, a MPSP system should be used to allow for proper grouting of the interface zone, as discussed below.

15.9.1.3 Grouting Method and Injection Pressure

After the casing is properly sealed, Reclamation recommends downstage grouting one or two 10-foot-long stages immediately below the embankment in all of the scheduled grout holes prior to initiating drilling and grouting the primary grout holes to full depth. The recommended gauge pressure at the manifold for this first stage should be the lower value of 0.5 lb/in^2 per vertical foot of overburden height or 10 lb/in^2. If a second stage is downstaged, the recommended gauge pressure at the manifold for this first stage should be the lower value of 0.5 lb/in^2 per vertical foot of overburden height or 20 lb/in^2. Adjustments to this gauge pressure should be made to factor in the phreatic surface within the embankment and foundation and the observed behavior during grouting in order to seal the foundation below the embankment. If high injection pressures from lower stages were to travel to the embankment, hydraulic fracturing could occur. This is illustrated in figure 15.9.1.3-1 (taken from Stare et al. [9]). The remaining stages could then be completed using upstage grouting methods.

For all other stages beneath an existing embankment dam, Reclamation limits the maximum injection pressure to the lower value of either: (1) the pressure equivalent to the maximum water surface at the top of rock, or (2) 0.5 lb/in^2 per vertical foot of overburden plus 1.0 lb/in2 per vertical foot of bedrock plus back pressure.

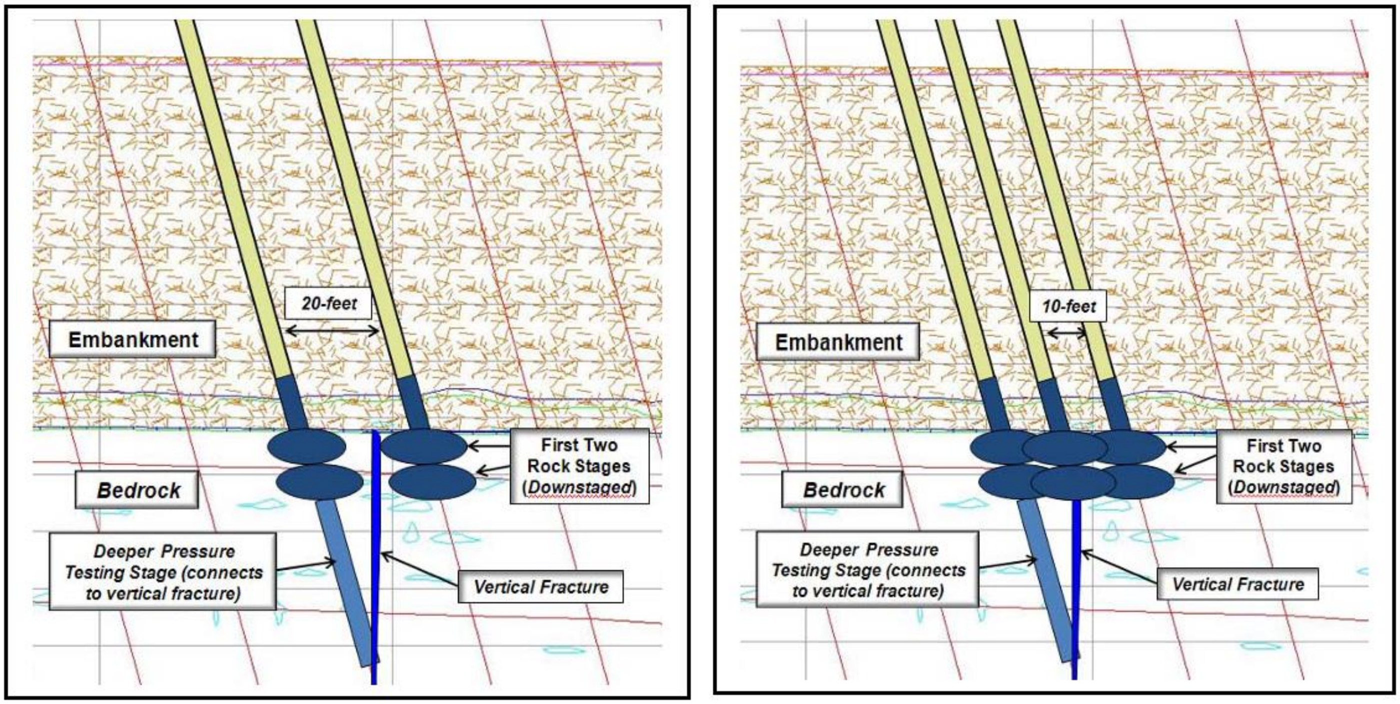

Figure 15.9.1.3-1. Illustration of downstage grouting immediately below an embankment dam (from Stare et al. [9], with permission from the American Society for Civil Engineers).

Adjustments to the maximum injection pressure are made, if necessary, based on the behavior of the rock mass, the embankment, and the results of the grouting program. When grouting below a high hazard facility, it may be necessary to evaluate the stresses to locate zones of low confinement stress and determine safe injection pressures. Extreme care must be taken to avoid hydraulic fracturing of the embankment.

As discussed in section 15.13, Reclamation requires the use of real-time computer monitoring when grouting below an embankment dam to assist in detecting hydraulic fracture within the embankment.

15.9.2 Artesian Flow

It is common to encounter artesian flow during a foundation grouting program, especially in the foundation bottom. If artesian flow is encountered, the effective grouting pressure on the grout manifold should be 1 lb/in^2/ft of drilling, plus back pressure from the artesian flow. If artesian flow is encountered, additional uplift monitoring is probably necessary, particularly if artesian flow is occurring during blanket grouting or stitch grouting. After a stage experiencing artesian flow reaches refusal, the packer must not be removed until the measured back pressure on the manifold dissipates.

15.9.3 Large Grout 'Takes'

Preconstruction geologic investigations and foundation grouting programs may expose zones of high permeability within the foundation. Unless large geologic features are identified, closure within zones of high permeability in bedrock resulting from open jointing, bedding planes, fractures, shear zones, voids, etc., are typically achieved by progressively thickening the grout mix. If there is no reduction in the bag 'take,' even when pumping the thickest grout mix, intermittent grouting can be beneficial. In these instances, stable grout mixes should be used. On rare occasions, Reclamation has added fine sand and calcium chloride to limit excessive grout travel and reach closure, while using the specified onsite grouting equipment. Some experience and/or laboratory testing is necessary when adding calcium chloride to avoid damaging the grouting equipment because set times may vary, based on many factors. Additional closely spaced closure holes would be necessary in these areas to verify that the high permeability zones have been sealed.

In addition to the karst features discussed below, geologic features that could result in extremely large grout 'takes' could be lava tubes, large voids in basalt flow tops, and excessive open jointing resulting from rapid cooling of volcanic deposits. These types of features must be identified during geologic investigations. Once identified, additional planning is needed during design to determine how to treat these portions of the foundation when progressive thickening of the grout mix will not work. For large features where grouting is judged to be ineffective, additional foundation treatment may include secant piles, concrete plugs, low mobility grouting (similar to compaction grout mixes), and hot bitumen grouting. For smaller features, multiple row grout curtains using high mobility grouts combined with some combination of fine sand, calcium chloride, bentonite, sawdust, cellulose, synthetic fibers, diatun gum, etc., may help create upstream and downstream plugs, which are followed up by a center closure row.

15.9.4 Karst Environments

Karst topography is a geologic landscape shaped by dissolution (or dissolving) of soluble bedrock such as limestone, dolomite and gypsum. The karst landscape is often characterized by underground drainage paths systems, caves, pinnacles, and sinkholes. Dissolution of bedrock beneath an embankment could result in many issues related to internal erosion and excessive seepage. Over time, seepage through karstic bedrock could enlarge existing features. Enlargement of existing fractures may lead to an unfiltered exit point for seepage and initiate internal erosion.

Reclamation has several embankment dams in its inventory that overlie karst formations, including Alcova Dam in Wyoming and Brantley Dam in New

Mexico. At both of these facilities, extensive grouting programs were performed during construction to limit seepage through the foundation. The USACE has recently experienced significant issues related to karst environments at Center Hill Dam in Tennessee, Wolf Creek Dam in Kentucky, and Clearwater Dam in Missouri [8]. Extensive foundation remediation, which included remedial grouting, was performed at each of these facilities to address dam safety issues related to solutioning. The Alabama Power Company has experienced significant issues related to karst features beneath Logan Martin Dam in Alabama. Grouting has been ongoing for many years at Logan Martin Dam to improve the embankment stability and minimize seepage through the dam foundation.

Prior to undertaking a grouting project in karst topography, the assistance of a noted expert in the field of foundation grouting is required to provide expert level assistance. A karst grouting project can become very expensive and can be ineffective if the design and construction staffs are not experienced with the potential issues that may arise. It should be noted that constructing an embankment dam on a karst formation should be avoided if possible.

The USACE has provided a detailed writeup on grouting in karst environments in EM 1110-2-3506 [8]. This guidance is applicable when designing a new embankment dam or remedial grouting beneath an existing embankment dam. For additional information, see EM1110-2-3506.

15.10 Grout Materials and Mixes

15.10.1 Grout Materials

The criteria pertaining to groutability of foundations in relation to permeability, grain size, void size, etc., are important in choosing a grout material. The grout material must have properties that are compatible with a foundation to effectively reduce permeability, increase strength, or both. Important properties to consider in determining the best grout for a job are viscosities, setting times, strength, stability, durability, impact of wet-dry cycles, and toxicity. In addition, cost is a very important factor, and estimates for comparison should include all facets involving materials, labor, equipment, and injection.

Grouts are divided into two principal types as follows:

1. Particulate grouts contain particles or solids in suspension, such as cement or clay, and often have a high viscosity. Reclamation practice is to use cement based grouts for foundation grouting projects beneath an embankment dam.

2. Chemical grouts are comprised typically of organic compounds that react in place and normally have a controllable gelling time. This design standard focuses on foundation grouting using cement-based grouts. Chemical grouts are not discussed, and their use for grouting beneath an embankment dam is strongly discouraged.

Based on volume, cement grout is typically more economical than most chemical grouts. However, in some instances where a cement grout is being carried away by flowing water, a chemical grout with a controllable set time may be more economical by requiring a smaller quantity of material to reduce the flow as a temporary measure.

15.10.2 Cement Grout

A grout mixture of cement and water, frequently used in combination with other materials, is the most widely used grout for improving foundations beneath embankment dams. Grout mixtures can contain supplemental cementitious (pozzolanic) materials, clays (bentonite), water reducing additives, stabilizers or gums, accelerators, extenders, and other additives. Except for relatively high viscosity and difficult to control setting time, the properties of cement grout are superior to most other grouts, particularly when comparing strength and economy.

15.10.2.1 Water

The water used in cement grout should be free from objectionable quantities of silt, organic matter, oil, alkali, salts, and other impurities. In addition, it must meet the appropriate water requirements for concrete (ASTM C1602 – Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete). In addition, the temperature of the water must be considered. Abnormally high or low temperatures can have significant impact on grout properties from delayed set to rapid hardening.

If bentonite is to be used, the chemical composition of the water should be tested to ensure that there are no impurities in the water that would affect the full hydration of the bentonite.

15.10.2.2 Cement

Hydraulic Portland cement is the most commonly used cement for foundation grouting. The cement should be fresh and free of lumps due to warehouse set, especially where filling fine cracks or small voids is involved. It should meet the requirements for cement used in concrete. Protection of cement until use is essential. Cement is usually furnished in bags weighing 94 pounds, where grout ‘takes’ are relatively small. However, where large quantities are used (as for solution channels and large voids), bulk cement is normally used for economy.

ASTM C150 classifies Portland cement into five types (I, II, III, IV, and V), according to limitations of certain compound composition and particle size.

- Type I cement is commonly used for grouting. However, the other types of cement having special properties may be used generally or for specific problems. Type I cements typically have blaine fineness in the 3,000 to 5,000 square centimeters per gram (cm^2/g) range.

- Type II cement has moderate resistance to sulfate attack and a lower heat of hydration. Type II cement is available and is required for grouting in much of the Western United States.

- Type III cement has characteristics to produce high, early strength; therefore, it is frequently ground finer than other types to attain that property. Type III cements typically have blaine fineness ranging from 5,000 to 7,000 cm^2/g. However, there is no fineness requirement for Type III. High, early strength may not always indicate a finer grind.

- Type IV cement has lower heat of hydration and slow strength development properties, and it costs more than other types of cement. It is seldom used for grouting.

- Type V cement has high sulfate resistance and would be of value where the formation or ground water has high sulfate content. Unless high sulfate resistance is required, Type II cement is typically used instead of Type V because of its lower cost.

Roughly 90 percent of the total weight of hydrated cement paste consists of the following four products:

1. Tricalcium silicate: $3CaO.SiO_2$
2. Dicalcium silicate: $2CaO.SiO_2$
3. Tricalcium aluminate: $3CaO.Al_2O_3$
4. Tetracalcium aluminoferrite: $4CaO.Al_2O_3.Fe_2O_3$

Chemical shorthand for these compounds is as follows:

$A = Al_2O_3$, $C = CaO$, $CH = Ca(OH)_2$, $F = Fe_2O_3$, $H = H_2O$, $S = SiO_2$, and $\overline{S} = SO_4$.

If there are 2CaO molecules, it is expressed as C_2.

The Portland cement phase hydration reactions are shown below in equations (1) through (6); however, these equations do not represent the complexities of the reactions.

(1) Tricalcium silicate + water = calcium silicate hydrate + calcium hydroxide

$$2C_3S + 11H \rightarrow C_3S_2H_8 + 3CH$$

(2) Dicalcium silicate + water = calcium silicate hydrate + calcium hydroxide

$$2C_2S + 9H \rightarrow C_3S_2H_8 + CH$$

(3) Tricalcium aluminate + gypsum + water = ettringite (calcium trisulfoaluminate hydrate)

$$C_3A + 3C\overline{S}H_2 + 26H \rightarrow C_6A\overline{S}_3H_{32}$$

(4) Tricalcium aluminate + ettringite + water = calcium monosulfoaluminate

$$2C_3A + C_6A\overline{S}_3H_{32} + 4H \rightarrow 3C_4A\overline{S}H_{12}$$

(5) Tricalcium aluminate + calcium hydroxide + water = tetracalcium aluminate hydrate

$$C_3A + CH + 12H \rightarrow C_4AH_{13}$$

(6) Tetracalcium aluminoferrite + calcium hydroxide + water = calcium aluminoferrite hydrate

$$C_4AF + 2CH + 10H \rightarrow C_6AFH_{12}$$

The hydrated cement particles are made up of a high-density calcium silicate hydrate (C-S-H) gel and act as solid particles in a continuous matrix. The outer hydration product is the continuous phase in the capillary pore space and consists of solid C-S-H gel, gel pores, calcium hydroxide, and calcium sulfoaluminate phases.

Cement types other than Type I or II are often more expensive. For this reason, Types I and II cement are frequently used for foundation grouting. Type III cements are typically not used in the Western United States due to high alkali soils. In addition, most cement now is made to meet more than one ASTM type (for instance, the cement could be made to meet both ASTM Type II and IV).

There are other specialty hydraulic cements, including Portland cements with air entraining additives, and expansive cements. High alumina cements, calcium sulfoaluminate cements, and various mixtures of these are also available. However, foundation grouting may require special compounds for specific grout properties, which are usually added during mixing of the grout. Preblended cements are not typically used for grouting.

In general, particulate grouts will penetrate fissures and voids larger than three times their largest particle size. The largest particle in ordinary cement is about

0.1 millimeter (mm) (0.004 inch), so joint openings less than 0.3 mm (0.012 inch) are generally not groutable using ordinary cement.

There are also micro-fine and ultra-fine cements, which are specially ground and processed so that the average particle size is much smaller. They can range in size from 0.01 to 0.001 mm and smaller. For example, the particle size distribution of micro-fine slag cement is 98 percent below 7 microns and 50 percent below 3 microns. The particle size distribution of ultra-fine Portland cement and ultra-fine blast furnace slag cement is 98 percent below 10 microns and 50 percent below 4 microns. Micro-fine and ultra-fine cements can have blaine fineness in the 8,000 to 10,000 cm^2/g range and higher. These cements can be used to grout masses with very small pore sizes or fracture sizes. However, the costs of these products can be three to four times (or higher) than the cost of ordinary Portland cements. Designers should carefully consider the need for micro-fine or ultra-fine cements. Reclamation has never used these products for a foundation grouting program in rock.

15.10.2.3 Pozzolans

Pozzolans are siliceous or siliceous and aluminous materials, which, in themselves, possess little or no cementitious value but will chemically react with calcium hydroxide in cement to form compounds possessing cementitious properties. Examples of pozzolans are fly ashes resulting from burning coal, calcined clays and shales, and volcanic tuffs and pumicites. For concrete, Reclamation typically only uses ASTM C618 Class F fly ash, which comes from burning anthracite or bituminous coal. Fly ash is roughly comparable to cement in fineness. Natural pozzolans such as volcanic tuffs or pumicites can be used if the quality is consistent, but they are not typically used. Shales and clays frequently need to be calcined at temperatures between 1,200 and 1,800 degrees Fahrenheit and ground prior to use, which can make their use more expensive than conventional fly ash. Pozzolans can improve fluidity of cement grouts and reduce bleeding, shrinkage, and permeability. They typically reduce early age strength gain, but later age strength can be higher than grouts made without fly ash.

Typical dosage is variable and can be quite high. Dosage can be from 10 to 30 percent Class F, by weight, of cement. For some void filling, 100 percent fly ash grouts can be used, which are typically mixtures of ASTM C618 Class F and C fly ashes. Cement can be added to increase strength of these high fly ash mixes.

15.10.2.4 Bentonite

Clays are a popular admixture to cement grouts. The small particle size of clays aids fluidity by keeping the cement particles in suspension and preventing settlement and bleeding of the grout. Injected cement-clay grout shows greater impermeability when compared to neat-cement grout or cement-sand grout.

Clays are usually stable in alkaline suspensions and tend to flocculate when the pH is changed to acidic.

Bentonite (sodium montmorillonite) clays are sometimes used for grouting or mixed with silicates or other chemicals. There are a wide variety of grades and types of bentonite. Because of this, bentonite from Wyoming is frequently used because of its consistency. Wyoming bentonite has a high proportion of sodium montmorillonite, which is often used for grouts where high fluidity is important.

When using bentonite in a grout, the sequence and quality of mixing is very important. Bentonite should be hydrated by mixing in water for at least 12 hours prior to being used in grout. However, that time can be reduced if tests show that equivalent hydration can be achieved with a high shear mixer. Grouts containing bentonite that has not been properly hydrated can experience many problems, including highly variable properties as the bentonite hydrates during grouting. After addition of dry cement to the bentonite slurry, a high-speed, high-shear mixer should be used, followed by constant agitation to keep the cement-bentonite mixture from clumping.

The addition of bentonite to cement grout reduces compressive strength, may increase shrinkage, and can increase a grout’s resistance to pressure filtration. It has seldom been used on Reclamation jobs. Bentonite is typically used at doses less than 2 percent, by weight, of cement.

15.10.2.5 Water Reducers and High Range Water Reducers

One of the most common practices now is to include a water reducing admixture (WRA) or a high range water reducing admixture (HRWRA) in the grout mix. These materials are also known as plasticizers or super plasticizers. The addition of a WRA or HRWRA will usually allow for less water in the grout mix but with the same penetrability and higher strengths than a comparison mix. A normal WRA can allow for a 5- to 10-percent water reduction in the grout, while maintaining similar flow properties, and an HRWRA can allow for a 15- to 30-percent or more reduction in water, while maintaining the same flow properties.

HRWRAs reduce water requirements by breaking up clumped cement particles. Breaking apart these clumps allows the grout to flow much easier (lower viscosity) and penetrate finer cracks than grouts without an HRWRA. However, the addition of excessive HRWRA to a grout mix can result in rapid settlement of cement grains in grout and cause weak zones. This can be especially true of newer generation HRWRAs. For cement and water mixes, HRWRAs should not be dosed higher than about 0.5 percent, by weight, of cement. Typically, HRWRA use is most effective after the cement and water have already been mixed. If very thick grouts are being mixed, it is sometimes necessary to add part of the HRWRA to the water before adding cement, and then adding the remainder of the HRWRA after initial mixing. If other additives are used, such as thixotropic additives, the dosage of the HRWRA may need to be higher, up to about 2 percent, by weight, of cement.

15.10.2.6 Thixotropic Agents

These are materials that, when added to grout, can lead to rapid changes in viscosity in response to changes in shear. This property results in fluids that readily flow but are capable of suspending or stabilizing components. These are high molecular weight gums (biopolymers) specifically formulated for use with cements. Earlier versions used welan gum, but more recent versions use diutan gum.

These materials can significantly enhance resistance to pressure filtration. They can also be used to impart anti-washout properties for grouts that are exposed to flowing water. The dosage is typically quite small (approximately 0.1 percent, by weight, of cement).

15.10.2.7 Hydration Controlling Admixtures

These admixtures can be classified into three distinct groups: (1) accelerators, (2) set retarders, and (3) hydration stabilizers. Accelerators reduce setting time. The most common accelerators are sodium silicate and calcium chloride. Set retarders extend the gel and set times of grouts. Set times of several days can be attained with some products. Finally, hydration stabilizers (set extenders) can be used as one- or two-component systems involving the use of a stabilizer and an activator. The stabilizer stops the hydration process for a specific time or until an activator is introduced to the grout. After that occurs, hydration begins.

15.10.2.7.1 Accelerators

Accelerators are used to speed the set time and achieve high early strength. The use of accelerators in cement grout is common when flowing water is a problem or when attempts are made to restrict travel in large voids below the surface. Cement grouts will usually set in about 4 to 6 hours after contact with water. Set times for cements are hard to adjust between about 10 minutes and 4 hours.

Solutions of sodium silicates can greatly accelerate the set of cement grout. For setting times under 10 minutes, sodium silicate solutions between 10 and 20 percent, by weight, of cement can be used to cause setting. Reclamation has performed tests to determine the accelerating effect, which have shown that grouts can set in a few seconds with these accelerators.

Calcium chloride is considered an economical accelerator, and has been used by Reclamation in concentrations up to 6-1/2 percent of the weight of cement with no detrimental effects. Calcium chloride is usually added to thick, neat water-cement grout mixes to accelerate cement hydration and limit grout travel or to accelerate the setting time. Care should be used with calcium chloride in grout that will be in contact with metal because rapid corrosion of metal could occur.

Some chemical grouts are compatible with cement grout and may be used as admixtures to accelerate the set time of the grout. This is useful in sealing off flowing water and springs. Sodium silicates and acrylamides each have been combined successfully with cement grout to control setting time in flowing water.

15.10.2.7.2 Retarders and Set Extenders

Although Reclamation seldom uses retarders and set extenders in grout mixes, these admixtures are sometimes used to lower the viscosity, reduce the water content, and slow the setting time, thereby slowing early strength gain of grouts. These admixtures are beneficial when grout is transported long distances before placement. It should be noted that grout should never be mixed offsite for a foundation grouting project.

15.10.2.8 Expansion Aids/Fluidifiers:

Another common grout additive is an expansion aid/grout fluidifier, which is typically a combination of a dry HRWRA and an expansive agent such as aluminum powder. Some grout fluidifiers can contain thixotropic admixtures like gums as well. The actual effects of these admixtures should always be verified before use because the specific constituents of the cement can cause different expansion amounts. Overly high expansion can lead to significant loss of strength.

15.10.2.9 Silica Fume

Silica fume is another pozzolan that comes from the production of ferrosilicon materials. It is typically used for making high strength concrete and can be used in grouts with the same effect. In grout, silica fume can also add stability (increase resistance to pressure filtration) and reduce bleed and settling. They can also improve grout durability. Because of their very small particle size (about 1 percent of the diameter of the average cement grain size, about 0.1 micron), silica fume has a very high water demand. Thus, whenever silica fume is used, an HRWRA is almost always needed. Silica fume is typically dosed between 5 and 10 percent, by weight, of cement.

Silica fume can be delivered in a dry, densified form due to its very light weight. If this form of silica fume is used in grout, it is very important to ensure adequate mixing. If not, the densified particles will not break down, and the effectiveness of the silica fume can be greatly diminished. In essence, it becomes an expensive filler.

15.10.2.10 Sand

Sand is commonly used in a cement grout to act as bulk filler in large voids. The sanded grouts tend to exhibit reduced travel and are often used to build a barrier by pumping a limited volume on an intermittent basis. The time between pumping is often 4 hours after the sanded grout has set. Sand should only be used in cases where grout is free flowing into the hole under “suction” conditions (bleeder valve on the manifold remains open during grouting to ensure no pressure on the hole).

For foundation grouting, the typical ratio of water:cement:sand is 1:1:1, by volume. As an aid to keep sand suspended in the mix, bentonite can be included in the cement-sand mixes at the rate of 2 percent of the weight of cement (or more) with a 2-percent typical dosage rate.

The sand gradation used for foundation grouting varies. Sand can be successfully pumped if 100 percent of the sand particles pass the No. 30 sieve, and 15 percent or more of the sand particles pass the No. 100 sieve.

The use of sand in a cement grout does not necessarily result in a more economical mix. The yield of total solids in a neat cement grout is considerably greater than a sand-cement-bentonite grout when using equal volumes of dry material. Reclamation laboratory tests showed that two bags of cement in a neat cement grout with a water:cement ratio of 1:1, by volume, yielded 2.56 ft^3 of solids. On the other hand, one bag of cement and 1 cubic foot of sand plus 2 percent bentonite by weight of cement with a water:cement:sand ratio of 1:1:1 yielded only 1.92 ft^3 of solids. Laboratory results at Teton Basin Project showed that the use of neat cement grout was more economical based on yield and bid prices than grout containing cement, sand, and bentonite.

15.10.3 Grout Properties

A number of tests are used to measure the performance of cement grouts. The purpose of the tests ranges from helping establish field performance to verifying specification compliance. The most frequently used tests are described below.

- **Apparent viscosity** (ASTM D6910, Marsh Funnel, American Petroleum Institute (API) Recommended Practice 13B-1). Viscosity is a measure of flowability and is typically measured using a marsh funnel. The test is performed by filling a marsh cone to the bottom of the dump screen and then measuring the time for 1 quart of grout to flow through the funnel into a graduated cup.

- **Cohesion** (“Dam Foundation Grouting” [4]). Cohesion is yield stress and is related to how far and how easily a fluid will flow through a channel of a given size under a given pressure. Cohesion can be measured with a Lombardi plate (Lombardi, 2003).

- **Time to initial and final gel time** (“Dam Foundation Grouting” [4]). These are measured using vane shear apparatus. The tests measure the amount of time required for the grout to reach initial gel time (cohesion of 100 Pascals [Pa]) and final gel time (cohesion of 1,000 Pa).

- **Pressure filtration coefficient** (API Recommended Practice 13B-1). Measured using an API filter press. The test is performed by pouring a 400-milliliter grout sample into the top of the filter press. The sample is then pressurized to 100 lb/in^2. The test is run for 10 minutes. The value indicates how easily the cement grains will come out of the suspension to form a filter cake, which prevents further grout penetration. The value of the pressure filtration coefficient is then calculated with the following equation:

$K_{p\,f}$ = (volume of fluid extruded/initial volume of sample) x (time in minutes)$^{1/2}$

- **Bleed** (ASTM C940, Standard Practice for Use of Apparatus for the Determination of Length Change of Hardened Cement Paste, Mortar, and Concrete). Grout 'bleed' is a result of sedimentation/ separation of solid particles within the grout mixture. The bleed capacity of the grout can be measured in accordance with the method described in ASTM C940, with 1,000-milliliter graduate cylinder.

- **Specific gravity** (API Recommended Practice 13B-1, Recommended Practice for Field Testing Water-Based Drilling Fluids). Typically measured using a mud balance. The specific gravity of a grout can be measured in accordance with the method described in API Recommended Practice 13B-1, with a Baroid mud balance. The Baroid mud balance is a calibrated scale that is used to measure the specific gravity and density.

- **Initial and final set times** (ASTM C191, Standard Test Methods for Time of Setting of Hydraulic Cement by Vicat Needle). Measured using a Vicat needle. The initial and final set times can be determined with the Vicat needle testing apparatus.

- **Compressive strength** (ASTM C39, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens). Usually measured using 2-inch cubes or 2-inch-diameter by 4-inch-tall cylinders.

Thixotropy is a property that relates to the time in which a certain cohesion value is obtained and the impact of the shearing or mixing in obtaining an increase or decrease in cohesion. Several tests described above measure certain parameters related to thixotropy. There are various admixtures that can impact this, predominately HRWRAs and gums. For grout mixtures that have thixotropic admixtures, the cohesion and viscosity can rise rapidly as shear rate or mixing in a grout is reduced.

The critical elements of a grout mix are the water-to-cement ratio, particle size and distribution, pressure filtration, bleed, gel and set times, cohesion, and strength. Adjusting a grout mix to change one property will typically affect other properties. For instance, reducing viscosity by adding water typically reduces strength and may increase the pressure filtration coefficient (tendency of the grout to plug the grout pores or cracks and not penetrate well). However, if the viscosity is reduced by using a water-reducing admixture, the strength will likely not be reduced, the pressure filtration coefficient may go down (grout would better penetrate the grout medium), but bleed water may increase slightly.

15.10.4 Standard Reclamation Foundation Grouting Mixes

Unless large voids are known to exist, the use of the optimum mix theory dictates that the initial grout mix must be thinner than the expected optimum mix to avoid losing a stage if thinner grout is needed. For this reason, Reclamation grouting projects typically use neat cement grouts consisting of water, cement, and a super plasticizer with the initial grout mix having a water-to-cement ratio of 5:1, by volume, with an appropriate dosage of super plasticizer for each stage unless large voids are discovered during drilling or the primary focus of grouting is to only fill large voids. The initial mix is gradually thickened until the optimum mix is reached and the stage achieves the desired closure criteria without being prematurely slugged with thick grout. Reclamation typically progresses through the following series of grout mixes with an appropriate dose of super plasticizer until the optimum mix is reached: 5:1 → 4:1, 4:1 → 3:1, 3:1 → 2.5:1, 2.5:1 → 2:1, 2:1 → 1.5:1, 1.5:1 → 1:1, and 1:1 → 0.8:1, 0.8:1 → 0.6:1. Skipping grout mixes could result in premature closure of a grout stage.

Some may argue that super plasticizer is not necessary and has little benefit at high water-to-cement ratios. Reclamation began using super plasticizers in the early 1980s and has routinely observed increased penetrability of cement grouts at high water-to-cement ratios when a super plasticizer is added to the grout mix. Prior to the 1980s, Reclamation routinely used initial grout mixes with a water-to-cement ratio as high as 8:1 to 10:1, with the optimum mix routinely falling within the 6:1 to 3:1 range at many Reclamation projects. At the time, grouting personnel within Reclamation resisted the use of thicker initial grout mixes to avoid premature refusal of a grout hole stage, while Reclamation designers pushed for less ‘bleed’ water in the grout. The advent of super plasticizers struck a balance between design staff and field staff. Once the benefits of using super plasticizers were observed within grouts at high water-to-cement ratios, Reclamation grouting policy [10] was changed to begin with a 5:1 grout mix with an appropriate dosage of super plasticizer.

15.10.5 Balanced-Stable Grout Mixes

Balanced-stable grout mixes provide the benefit of minimal to no bleed water, a high resistance to pressure filtration, and only minor changes in the grout properties before and after injection. Grout ‘bleed’ is a result of sedimentation of solid particles within the grout mixture. Pressure filtration measures the amount of water pushed out of the grout mix during injection.

Balanced-stable grout mixes contain water, cement, and admixtures. Admixtures that are now routinely used in balanced-stable grout mixes are bentonite, welan or diutan gum, and super plasticizers. Water-to-cement ratios for balanced-stable grout mixes typically range from 3:1 to 1:1 (by volume). The bentonite dosage is often within 1 to 3 percent, by weight, of cement. The diutan or welan gum dosage is often about 0.1 percent, by weight, of cement. The super plasticizer dosage is often between 1 to 3 percent, by weight, of cement for grouts that contain bentonite.

If balanced-stable grout mixes are used for a foundation grouting project, a laboratory testing program must be performed prior to initiating grouting operations to determine the correct proportions of each material for each grout mix that will meet the desired properties, along with the batching instructions.

15.11 Drilling

Drill holes provide the means of conveying grout to the rock foundation. The grouting program and the drilling technique must work together to inject as much grout as possible at the optimum mix. It is important that the holes are drilled with relatively smooth and uniform borehole walls to facilitate seating of a packer. It is also important that the drill cuttings are sufficiently flushed out of the drill hole to prevent sealing of the fractures exposed on the borehole wall.

15.11.1 Drill Hole Sizes

Decades of Reclamation experience using unstable grout mixes have shown grout hole sizes of 2.5 inches or less are necessary to prevent premature refusal of a grout stage resulting from poor pressure filtration performance of unstable grouts. A 1.9-inch to 2.0-inch drill hole size is preferred unless the hole depth exceeds 400 feet. Since Reclamation uses unstable grouts in foundation grouting programs, Reclamation typically specifies an AX size (1.9 inches) drill hole diameter for grout holes. For grout holes that exceed a depth of 400 feet, Reclamation typically specifies a size BX (about 2.4 inches) drill hole diameter. At depths greater than 400 feet, the drill strings can experience excessive drill rod flexure, resulting in unacceptable changes in drill hole bearing and inconsistent hole diameter. BX size drill holes allow for the use of higher strength drill strings, resulting in straighter and more consistent diameter grout hole.

Given these hole size limits (less than 2 inches) and the use of the optimum mix, grout holes for Reclamation grout curtains are typically drilled with fluid rotary techniques. When using balanced-stable grout mixes, grout hole diameters up to 3 inches may be acceptable to accommodate less expensive drilling methods such as a water powered down-hole rotary hammer. However, a 3-inch-diameter grout hole is not recommended.

15.11.2 Acceptable Rock Drilling Methods for Grouting

Many methods exist for drilling in rock. Regardless of which method is selected for drilling the grout holes, Reclamation always requires the use of water as the flushing medium for drilling grout holes because it usually results in the most uniform and straight borehole, as well as the best method for removal of drill cuttings prior to grouting. The use of air as a flushing medium can result in excessive plugging of the fractures and prevent grout penetration. Additives to enhance the removal of drill cuttings or maintain a stable borehole, such as bentonite, are not allowed because they can partially or completely block off natural fractures in the rock. The use of bentonite clays as a drilling fluid additive is a separate process from the use of bentonite as a grout additive.

Reclamation prefers the use of water rotary methods for grouting. Rotary drilling techniques include the use of tri-cone bits, button bits, diamond bits, and diamond coring. Diamond coring is usually one of the slower methods of drilling, but it can result in the straightest drill hole with the most uniform borehole wall. It can be required where there is a need to verify rock conditions prior to grouting or when a very precise hole placement is needed. Reaming shells can be placed up the hole from the bit to keep the hole in alignment and to keep the borehole walls uniform in size and smoothness. Grout holes for Reclamation grout curtains are typically drilled with water rotary techniques due to the small hole diameters required (less than 2 inches).

Air or water powered rotary hammer drilling (rotary percussion), especially with top drive hammer drills, are some of the fastest drilling methods; however, it can result in the most irregular borehole and force the most cuttings into the formation. For top-hole percussion, the hammer is mounted on the drill, and the energy is applied to the bit through the drill rods. For downhole methods, the hammer is installed just above the bit and is activated by the drilling fluid, while the drill head rotates the string. In a Reclamation grouting program, the use of top drive hammer drilling may be acceptable for shallow grout holes, such as blanket holes, that are less than 20 to 30 feet deep, provided that excessive fines are not produced during drilling relative to standard rotary techniques. Given the relatively small hole sizes needed in a grouting program, at depth the drill strings can experience excessive drill rod flexure from the top drive hammer, resulting in unacceptable changes in drill hole bearing and inconsistent hole diameter. Inconsistent hole diameter can result in leakage around the packer or damage to the packer. For these reasons, Reclamation does not allow the use of top drive percussion methods for deeper grout holes.

Downhole hammer methods improve the efficiency of percussion drilling because the hammer is located at the end of the drill string. Downhole rotary hammer drilling typically uses some form of a button bit. Current bit technology and the use of steering tubes with guide ribs have resulted in borehole diameter consistency and wall smoothness approaching that of diamond rotary techniques.

In recent years, the grouting industry has used downhole hammers because modern equipment allows for drill hole sizes as small as 2.5 inches. Historically, downhole hammers could not accommodate the relatively small drill hole size necessary for a grout hole, due to the size of the hammer. Downhole hammers offer the benefit of reduced cost and drilling time compared to standard rotary techniques. In addition, downhole hammers significantly improve accuracy and are able to drill much deeper holes compared to top drive hammer methods. When balanced-stable grout mixes are used for a project, the use of larger drill holes (up to 3 inches) is acceptable and would allow for the use of downhole hammer methods.

Areas of restricted access, steep terrain, or grouting from a tunnel may require the use of smaller, portable drills for drilling grout holes. These rigs will usually have less torque and down pressure for drilling and will drill at a slower rate. Often, their small size requires the use of shorter drill rods. This also lengthens the time required to drill each grout hole. Figure 15.11.2-1 shows a small portable drill used on a recent Reclamation project.

Figure 15.11.2-1. Contractor using a Chicago-pneumatic 65 (CP-65) drill for drilling abutment grout holes on steep terrain.

15.11.3 Selection of Drilling Method

The selection of a drilling method should be based on a consideration of the properties of each rock unit that will be encountered. Many damsites have several different types of rock present in a foundation. Selection of a drilling method is often a compromise of what drilling method is most suited for the range of conditions and various rock types encountered within the dam foundation. Given the variety of drill bits and rigs currently available, adequate combinations are possible which will yield good borehole geometry with good drilling rates. Bits can be chosen specifically for the most common rock type or condition found at a damsite, but there will be an increased risk for changes in hole bearing, borehole straightness, wall integrity, blockage, and caving or complete loss of hole.

Plugging fractures with cuttings from drilling reduces the ability of the grout to penetrate those fractures effectively. This is the primary concern when discussing and choosing the grout hole drilling method. Some factors that could influence the plugging of fractures are:

- Fracture width
- Size of drill cuttings for drilling method
- Lithology
- Amount of weathering
- Hardness
- Clay/silt content
- Rotation speed and torque
- Pressure and volume of circulating fluid
- Type and sharpness of the drill bit
- Rate of penetration
- Water table

Usually, igneous and most metamorphic rocks do not produce significant fines relative to the rate of drilling that would clog the openings (fractures). In these types of formations, Reclamation would allow for downhole hammer methods if smaller hole sizes were available when using unstable grout mixes.

Argillaceous sandstones, claystones, and shales are more apt to produce fines relative to the rate of drilling that could clog fine fractures. For these reasons, Reclamation would recommend the use of water rotary drilling methods for these types of materials.

15.11.4 Hole Logging

A drill hole log should be kept for each grout hole drilled so verification can be made that grouting design assumptions for the rock mass are correct. The drill hole log can also provide advanced knowledge of potential zones of higher grout ‘takes’ and be used to adjust the grout program for special conditions. The drill hole log should include the rate of advance, zones of water loss or gain, down pressure used, water pressure used, nature of drill cuttings, location of any voids encountered, and character of drilling (how smooth or rough each drill run was).

Any cored holes should also include the core recovery for each drill run, location of joints encountered, character of joints encountered such as openness, healing, location of highly fractured or broken zones, and changes in rock type. Rock coring should be monitored and logged by a person trained in core logging and handling requirements as outlined in Reclamation’s *Engineering Geology Field Manual* [3].

15.11.5 Hole Straightness

Grout holes need to be drilled to the bearings and inclinations intended in the design to be most effective. Many factors can affect how straight a grout hole can be drilled. These include properties of the rock mass, initial hole orientation, size and depth of hole to be drilled, size and type of drill rig used, drill methods, bits, drill rods, and drill fluid used. Drilling a grout hole to the proper location starts with setting the standpipe to the correct bearing and inclination, proper initial set up of the drill rig, and orientation of the drill rods. A drill hole started on an incorrect bearing or inclination cannot be corrected in the ground.

In rotary drilling, the use of reaming shells helps with proper orientation. In downhole rotary hammer drilling, steering tubes with guide ribs behind the bit help keep the proper orientation. During drilling, the drill rig operator must maintain a balance between down pressure, advance rate, rotation speed, fluid volume, and fluid pressure to get the straightest hole possible.

Borehole straightness can be verified quickly with several types of instruments. Tri-axial magnetometers use the earth’s magnetic field direction and strength to determine the straightness of a borehole. However, they do not perform well in rock with higher iron content or in areas with localized magnetic anomalies. Tri-axial accelerometers detect changes in the instrument angle in relation to the earth’s gravity (referred to as the gravity roll angle) to determine changes in hole bearing and inclination. Many instruments operate on a combination of gravity and magnetic capabilities. Optical instruments use a laser beam to detect changes in hole orientation.

15.12 Grouting Equipment

The main components for a grout plant include a grout pump, grout mixer, agitator tank, pressure gauges, and a water meter. Additional necessary grouting equipment includes circulation lines, packer hose, valves, pressure gauges, pressure sensors, grout manifold, standpipes, and packers. Reclamation also requires the use of a flowmeter on any foundation grouting project. The use of a density meter should also be considered for large grouting projects. Figure 15.6.3-2 shows the layout of this equipment for a typical grouting job.

More sophisticated equipment monitoring during grouting, such as density meters and automated electronic instrumentation to record pressure, flow, volume, specific gravity, density, apparent Lugeon value, time, effective pressure, etc., are now regularly used throughout the grouting industry. Section 15.13 discusses the monitoring equipment.

Because a proper grouting design includes specifying the correct equipment, a brief discussion is provided below on the grouting equipment typically specified on a Reclamation foundation grouting project. Additional discussion on grouting equipment can be found in Houlsby [1] and Bruce and Weaver [4].

The most common type of grout plant is portable and is typically mounted on a trailer or skids. Given the remote location of many Reclamation dams at the time they were built, portable grout plants have often been used for Reclamation grouting projects (as shown in figures 15.12-1 and 15.12-2). Multiple grout plants can be used at various locations to concurrently grout multiple project features. For large grouting jobs involving large quantities of grout, a centralized, automated batch plant may be designed by the contractor. Centralized batch plants are capable of batching large quantities of grout and delivering it to multiple agitator tanks and pumps located closer to the point of injection. Figure 15.12-3 shows a centralized batch plant at New Waddell Dam.

Figure 15.12-1. Portable batch plant at Starvation Dam.

Figure 15.12-2. Portable batch plant at Ridges Basin Dam.

Figure 15.12-3. Centralized batch plant at New Waddell Dam.

15.12.1 Mixer

A grout mixer must have the ability to mix the grout, including admixtures, to a uniform consistency that is free of lumps. Reclamation requires the use of high-speed, colloidal-type mixer equipped with a high-speed centrifugal mixing pump operating at 1,500 to 2,000 revolutions per minute during mixing. The return flow from the centrifugal pump is directed tangentially into a cylindrical tub to create a vortex within the cylindrical tub. The centrifugal action tends to break up lumps and tends to limit segregation. The use of vertical paddle-type mixers is unacceptable given the superior mixing quality of a high-speed colloidal mixer.

Reclamation currently specifies a minimum volume of 17 ft^3 for the mixer and agitator tank. This volume is necessary in the event of a high ‘take’ grout stage.

15.12.2 Agitator

The agitator tank serves as a reservoir or holding tank, which keeps the grout in suspension after the grout is mixed. The agitator tank feeds the grout to the pump via a hose. Mixing tanks are typically vertical-tub, paddle-type tanks. In addition to the rotating paddle, baffles can be welded on the inside of the mixing tank to increase the mixing action. Reclamation Standard Drawing 40-D-6571 is included in every specification for a foundation grouting project to illustrate the requirements for the agitator tank.

After grout is mixed and dumped into the agitator tank, the next batch of grout can be mixed, while the grout in the agitator tank is being injected. Reclamation requires that all grout flowing into the agitator tank (includes inflow from the mixing tank and return flow from the manifold) pass through a U.S. Standard No. 16 sieve to remove any lumps from the grout..

15.12.3 Water Meter

Water meters or a water column are used to ensure that the water-to-cement ratio of the grout is accurate. The water meter or water column should be adequately sized to supply water to the mixer to allow for rapid mixing.

15.12.4 Pumps

Reclamation requires the use of a progressive cavity pump on a foundation grouting project. Reclamation also requires progressive cavity pumps because they produce uniform pressure without pulsations, have a large pumping capacity, and can pump a wide range of grout consistencies.

Reclamation does not allow the use of piston pumps because uniform pressure is difficult to maintain, and these pumps require frequent cleaning.

The grout pump should always be connected directly to the agitator tank. Pumps with holding hoppers and pumps with an open throat design are not acceptable for foundation grouting of an embankment dam.

The location of the grout pump should not be more than 20 feet above the elevation of the standpipe and no more than 200 feet from the grout hole to prevent excessive circulation line pressure relative to the grout injection pressure.

A standby grout pump is needed at each grout plant. This standby pump must be capable of being placed into operation within 15-minute's notice.

15.12.5 Valves

Valves used for grouting must remain operable after long durations of grouting, be easily cleaned, and be able to withstand the abrasive grout. For these reasons, Reclamation allows the use of ball valves or diaphragm type valves. All ball valves should have minimum 6-inch lever handles permanently fixed to the valve that clearly show the position of the valves. Only cast iron or steel bodied valves are acceptable. The use of PVC or other plastic valves are never allowed and are unacceptable.

15.12.6 Pressure Gauges and Sensors

Accurate pressure gauges at the manifold are highly important during grout injections. Reclamation typically requires two gauges on the grout line: one gauge at the pump and one gauge at the manifold. A faulty gauge can result in high pressures, which can lead to hydraulic jacking or hydraulic fracturing. A faulty gauge can also result in low injection pressures, leading to ineffective grouting. The rating of each gauge should not be more than three times the maximum grouting pressure of the stage being grouted.

Reclamation requires the use of a pressure sensor on the manifold during grouting. Figure 15.12.6-1 shows a pressure gauge and pressure sensor on a grout manifold. The pressure sensor is positioned on the manifold in the flow pattern behind the pressure control valve and ahead of the of the bleed-off valve (see figure 15.6.3-3 for Reclamation's standard grout manifold design drawing).
A pressure sensor allows for reliable pressure readings when pumping a cementitious grout. Reclamation specifies a Red Valve Series 42 pressure sensor, or equivalent, for all foundation grouting projects.

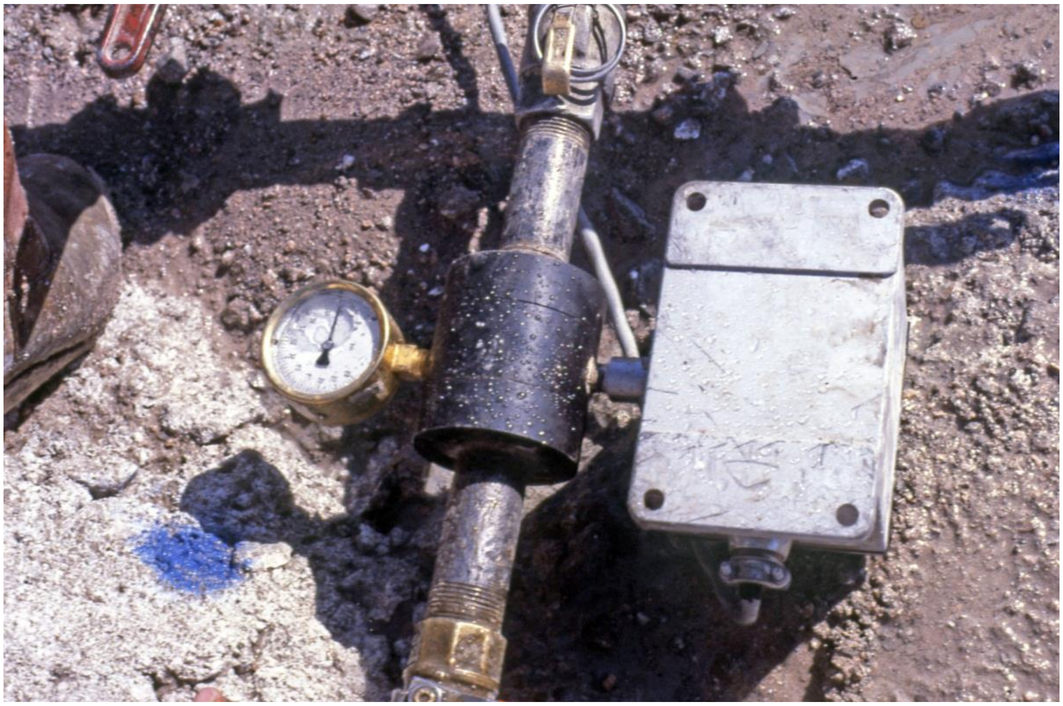

Figure 15.12.6-1. Pressure gauge and pressure sensor on a grout manifold (Series 42 Red Valve pressure sensor).

15.12.7 Circulation Lines

The circulating grout lines consist of a supply line to the manifold and a return line to the agitator tank. The supply and return lines permit continuous circulation of flow to the manifold. The minimum size of the supply line, along with all valves and fittings, is often specified at 1 inch; however, if the viscosity of the grout is high, and the expected ‘takes’ are high, 1.5-inch hose may need to be specified. High pressure flexible hose is often used by contractors for the circulation lines. Figure 15.12.7-1 shows circulation lines to a grout manifold.

Water should always be circulated through the grout lines prior to initiating grouting to ensure there are no blockages in the grout lines.

Figure 15.12.7-1. Supply and return grout circulation grout lines at the grout manifold.

15.12.8 Grout Manifold

Reclamation requires the use of a grout manifold on every grouting project. Figure 15.6.3-3 shows Reclamation's standard grout manifold design. The manifold is required to control pressure and flow into the grout hole.

Grout flows from the agitator tank, through the grout pump and the supply line, and then to the manifold. Prior to initiating grouting, the supply valve and return valve should be completely open, and the control valve should be closed. In this scenario, all of the grout flowing through the supply line is directed back to the agitator tank via the return line. With the pump circulating grout in the lines, pressure and flow into the grout hole are controlled at the manifold by control valve and return valve. As the control valve is slowly opened, the grout begins to flow into the grout hole. If added pressure and flow are needed after the control valve is completely open, the return valve can then be adjusted to provide increased pressure and flow. Typically, some return flow to the agitator tank should be maintained. As the stage is being grouted to refusal, the pressure will gradually build, which necessitates slowly opening the return valve and closing the supply valve on the manifold to maintain uniform pressure. It is important that the headerman and inspector maintain constant vigilance on the manifold to properly control pressure.

Multiple-port grout manifolds allow one grout pump to grout multiple grout holes at the same time. Because each port on the manifold is supplied by the same pump, the same grout mix must be used for each grout hole that is connected to the multiple-port manifold. For this reason, multiple-port grout manifolds are never acceptable on a foundation grouting project because each grout hole/stage could require a different grout mix as described in section 15.5.

Periodic back pressure and holding pressure measurements are necessary during grouting. Differentiating between back pressure and holding pressure takes place at the manifold by closing the top supply valve to the hole, while circulating grout in the line. If the pressure gauge returns to zero immediately, there is neither back pressure nor holding pressure. If the pressure gauge maintains pressure, the bleeder valve should be opened until the gauge reads zero, and then the bleeder valve should be closed. If the gauge remains at zero after bleeding, the gauge reading prior to bleeding is the holding pressure. If the gauge is then activated after closing the bleeder valve, this is the back pressure reading.

15.12.9 Flowmeter

Reclamation typically requires the use of a flowmeter on every foundation grouting project. A flowmeter provides real-time flow and volume measurements at the grout hole. Prior to flowmeters, an inspector would stand at the grout plant and insert a dipstick into the tub every 15 minutes. The elevation difference on the dipstick would be converted to a volume and a flow rate. A flowmeter reduces the number of inspectors required on a grouting project because the flow and volume can be read at the flowmeter instead of dip-sticking the agitator tank on regular intervals. It also allows for real-time flow and volume readings.

Reclamation prefers that the flowmeter be placed between the bleeder valve and the standpipe. Either ultrasonic or magnetic flowmeters are typically specified. On most flowmeters, there is a digital readout screen, and the data on the screen can be stored in the readout device or be transmitted to a central data collection station. Figure 15.12.9-1 shows a Compu-flow ultrasonic flowmeter in use on a grouting project.

Figure 15.12.9-1. An ultrasonic flowmeter installed downstream of bleeder valve. The standpipe is not shown in the photo.

15.12.10 Packers

Packers are expandable devices that stop or prevent the injected fluid from returning past the packer and to the surface. Packers are used to water test and grout a section of hole by inserting the packer, which is attached to the packer hose, to the desired depth. Reclamation typically specifies the use of pneumatic packers. For single-stage grouting from the surface, a mechanical packer could be used. The packers are typically commercially available for many hole sizes, or they can be fabricated to suit any hole size. The packer should have a 1-inch-diameter hole, or larger, through the packer to avoid restricting grout flow through the packer. Figure 15.12.10-1 shows an inflatable packer being placed during a Reclamation grouting project.

15.12.11 Packer Pipe

On many recent projects, the grout supply pipe to the packer is a polyethylene gas pressure pipe with a minimum inside diameter of 1 inch and meets the criteria within ASTM D2513. If the contractor elects to use steel pipe, flush-coupled pipe is necessary. Standard pipe and couplings often cause withdrawal problems by raveling or caving holes. Figure 15.12.11-1 shows the installation of packer pipes into a grout hole on a recent Reclamation project.

Figure 15.12.10-1. Inflatable packer being placed into grout hole.

Figure 15.12.11-1. Packer pipe being installed into a grout hole.

Many contractors prefer the use of packer pipe grout reels. A packer pipe grout reel can accommodate any grout hole depth encountered on the project. If a packer pipe grout reel is not used, multiple lengths of packer pipe are needed to accommodate different grout hole depths. For this reason, the use of a packer pipe grout reel can save time between stages because frequent switching out of the packer pipe is not necessary.

When a packer pipe grout reel is used, the grout manifold is hooked to the grout reel. A packer is attached to the end of the packer pipe on the grout reel, and then it is lowered to the required stage depth. After grouting begins, the grout must flow through the entire length of packer pipe on the reel before it flows down the grout hole.

Despite the advantages provided by packer pipe grout reels, Reclamation does not allow packer pipe grout reels on projects that require low flow injection rates. At low flow rates, Reclamation has experienced issues with stage refusal occurring within the grout hose reel, rather than within the grout hole. At low flow rates, the grout may begin to gel in the bottom section of the reel, especially during hot weather, which causes faster gelling of the grout. Because most of Reclamation's foundation grouting projects require low flow injection rates, the use of packer pipe grout reels is not permitted by Reclamation.

15.12.12 Standpipes

Prior to drilling the grout holes, shallow standpipes are installed to mark the location of the hole, control leaks at the top of the hole, limit surface water inflow into the hole, and set the orientation and inclination of the hole. Figure 15.6.8-1 shows a typical detail for standpipe installation. Figure 15.12.12-1 shows 2-1/2-inch, schedule 40, black steel standpipes used on a recent Reclamation grouting project. In sound rock, embedment depths are usually 2 to 3 feet. In poor rock, the embedment depth should be increased until the standpipes are secure. The standpipes are usually capped when they are not being worked on. The inside diameter of the standpipes should be equal to, or within, one-half inch of the size of the hole being drilled through the standpipes. Reclamation requires the use of black steel pipe for standpipes. When grouting is complete, the standpipes are cut off flush with the final foundation surface before earthfill is placed.

Figure 15.12.12-1. Standpipes used for a recent Reclamation project.

15.12.13 Fluid Density Meter

If a real-time computer monitoring system is judged necessary for a grouting project, as discussed in section 15.13, Reclamation recommends the use of a fluid density meter. The density of the grout and, subsequently, the grout mix can be continuously recorded versus time. If a fluid density meter is needed, a straight tube type should be specified. The accuracy of these instruments is affected by the flow rate in the grout lines. To ensure accurate measurements, the fluid density meter should be located between the supply line and the grout manifold. The inside diameter of the density meter shall be specified to be equal to the grout line diameter.

15.13 Field Supervision, Inspection, and Monitoring

Foundation grouting is a somewhat unique construction activity within civil construction in that the design is adjusted and engineered in the field, based on the actual grout 'takes', drilling observations, and water test results. The design drawings provide the initial design for the grouting project; however, the design drawings are viewed as a dynamic document. For these reasons, Reclamation has always required that all drilling and grouting operations be directed by Reclamation staff. At the end of every Reclamation project, Reclamation is responsible for the safety of the embankment and its foundation. Reclamation firmly believes that the engineering field decisions for all foundation grouting features shall be made by Reclamation. There are many qualified grouting contractors within the United States, and their experience should be relied upon throughout drilling and grouting within the dam foundation or other project features; however, Reclamation must ultimately be responsible for the engineering design decisions that are made in the field.

Reclamation's specifications are written such that the contractor supplies the equipment, materials, and labor necessary to execute the drilling and grouting operations. This does not mean, however, that the contractor's expertise and ideas should be ignored. Experienced field engineering staff and inspectors must work with the contractors to foster an environment where all ideas are shared to make the project as successful as possible.

Reclamation has produced a training manual [11] that outlines the primary objectives of a grouting inspector. This training manual provides a good overview for what is required during inspection. The grouting inspectors and engineers should review this training manual before each grouting job. Some general inspection requirements that affect the overall design of a grouting program are the following:

- Grouting is not an activity where the inspector can roam the site and stop by every hour or so. A person(s) needs to be 100 percent dedicated to DIRECTING the drilling/grouting operations.

- Grouting of dam foundation will likely involve long hours, if not continuous work. Provisions need to be made to provide continuous (24-hour) supervision of the contractor's work. It is critical to ensure that there will be adequate inspection staff available for the duration of the grouting project.

- Each inspector is required to fill out the drilling and grouting forms for each drill hole and grout hole/stage. The data on the forms is very important for several reasons: (1) the contractor is paid based on the information contained on these forms; and (2) filling out the forms requires the grouting inspectors to stay engaged and "on top" of the situation. The grouting supervisor assesses the information collected by the inspectors to direct changes to the design (grout hole spacing, orientation, depth, grout mix proportions, pressures, etc.) when necessary.

- If the embankment designer and grouting engineer elect to use computer-aided monitoring of the drilling and grouting, field inspection is still necessary. A computer should never replace the need for an inspector to stand at the manifold or drill rig.

It is now common practice on many grouting jobs to use real-time computer monitoring. These real-time computer monitoring systems are able to measure, record, and graphically display the real-time injection pressure, rate of injection, fluid density, volume, Lugeon value, start/stop times, or any other information that is requested for the project. Equipment that is used to generate this data typically includes pressure transducers, flowmeters, and density meters. Data is transmitted via cables or wirelessly. After the data is received, computers will convert an analog signal to a digital signal.

The cost/benefit ratio of these real-time monitoring systems must be evaluated for each project. When remedial grouting is performed underneath an existing embankment dam, Reclamation requires a real-time monitoring system to reduce the likelihood of hydraulically fracturing the existing embankment during grouting. When the grouting program is expected to exceed $1 million, a real-time monitoring system should be considered.

Reclamation has used real-time, computer-aided monitoring for Ridgway Dam, Upper Stillwater Dam, Brantley Dam, Jordanelle Dam, and New Waddell Dam. The most recent large Reclamation grouting project, Ridges Basin Dam, did not use computer monitoring. With the exception of Ridgway Dam, the automated systems used at Reclamation dams were designed, operated, and maintained by the Government. The equipment and computer monitoring hardware and software Reclamation used in the 1980s and 1990s are now obsolete. Reclamation would likely use a real-time monitoring system provided by the grouting contractor for future large grouting projects if it was deemed economical. Real-time, computer-aided monitoring provides the ability to store large amounts of data, graphically display the grouting results in real time, and make more informed decisions if the data is properly evaluated.

If the Embankment Designer and Grouting Engineer elect to use computer aided monitoring of the drilling and grouting, field inspection is still necessary. Some contractors are now able to place all of the information collected from the monitoring system on a Web site. The Government should have someone reviewing this data in real time as it is collected. An electronic copy of this data should also be provided to the Government on a daily basis, and the data should be available to the Government in real time as well. Both the contractor and the Government are responsible for storing this information.

15.14 Construction Sequencing

Reclamation typically provides some construction sequencing criteria for grouting within the specifications. Guidance on typical construction sequencing issues is provided below:

- When possible, grouting should always be performed from the bottom up. In an ideal situation, grouting would begin at the lowest point in the valley and progress upward on each abutment. In many cases, grouting is broken up into stages to accommodate other construction activities such as excavation, foundation cleanup, and river diversions. If grouting commences on an abutment, grouting should begin at the lowest elevation possible and progress upward on the abutment.

- Prior to commencing foundation grouting beneath a new embankment dam, foundation overburden materials must be removed to expose the bedrock surface. Reclamation always performs geologic mapping prior to grouting. The cleanup requirements for geologic mapping are discussed in chapter 3 of *Design Standards No. 13 - Embankment Dams* [2].

- Reclamation typically requires the concrete lining for an outlet works tunnel, spillway tunnel, and gate chamber to be completed 100 feet upstream and downstream of the centerline of the grout curtain prior to drilling and grouting curtain holes within a minimum of 100 feet from the centerline of the tunnel/gate chamber.

- Blanket grouting is always performed ahead of curtain grouting. Reclamation specifications are written such that blanket grouting should be at least 80 feet ahead of curtain grouting.

- After grouting is initiated, an effort must be made to limit blasting in the vicinity of a completed closure pattern or areas being actively grouted. If blasting is necessary, the allowable peak particle velocity measured on the ground surface adjacent to an active grout hole or completed closure pattern should be determined for each site. When possible, work should be staged so that all blasting is completed prior to the start of grouting. The contracting officer must approve all blasting plans in the vicinity of an active grout hole or completed closure pattern if blasting is unavoidable.

- Reclamation does allow embankment placement to begin prior to completing grouting operations; however, the grouting must be complete at least 100 feet (measured along the slope from the collar of the grout hole) from the surface of the embankment material placement.

15.15 Reclamation Grouting Procedures

15.15.1 Initial Mix

When the purpose of grouting is to fill joints, fractures, fissures, bedding planes, or other openings similar in size, Reclamation's initial mix for each grout stage is typically 5:1 (water:cement ratio, by volume) with an appropriate dosage of super plasticizer.

When the purpose of grouting is to fill large voids and caverns, a balanced stable grout mix should be used to limit bleed water at the desired viscosity. Therefore, the section below, “Adjusting the Mix,” does not apply to filling large voids and caverns.

15.15.2 Adjusting the Mix

Several guidelines are provided below for adjusting the grout mix in the field to find the optimum mix. Because every grout stage is different, these guidelines are not set rules. It is not permissible to simply ignore what is happening in the field. Experience on a project and awareness of conditions in the field play a vital role in determining the best way to adjust the grout mix:

- Reclamation does not use the results of the water test to obtain the initial mix.

- Reclamation always takes a conservative approach when grouting a hole, which means that the starting grout mix is always thinner than the optimum mix. A grout mix can always be thickened when excessive ‘takes’ occur. The optimum mix technique, as described in section 15.5, should always be used. Never try to pump a grout mix that the rock will not readily accept.

- Reclamation typically thickens grout mixes (with an appropriate dosage of super plasticizer) progressively from 5:1 → 4:1, 4:1 → 3:1, 3:1 → 2.5:1, 2.5:1 → 2:1, 2:1 → 1.5:1, 1.5:1 → 1:1, 1:1 → 0.8:1, and 0.8:1 → 0.6:1. Reclamation recommends gradually thickening the grout. Skipping grout mixes could result in premature closure of a grout stage. On some projects, a 1.25:1 mix is necessary.

- Reclamation records the bag ‘take’ per hour at 15-minute intervals. The bag ‘take’ can be easily calculated by converting the flow rate readings at the flowmeter to bags/hour. A few examples are provided below:

  - The grout flow reading after 15 minutes has averaged about 4 gal/min at a 5:1 mix. The conversion to bags/hour from this flow reading is:

$$4\frac{gal}{\min} * \frac{60\min}{1hour} * \frac{1ft^3}{7.48gal} * \frac{1bag}{5.5ft^3 grout(5:1grout)} = 5.8bags/hour$$

  - If 8 ft^3 of 4:1 grout was pumped over a 15-minute interval, the conversion to bags/hour would be:

$$\frac{8\,ft^3}{15\min}*\frac{60\min}{1hour}*\frac{1bag}{4.5\,ft^3 grout(4:1grout)}=7.1bags/hour$$

To simplify this process, each inspector can be given tables to easily convert grout flows or volumes from a flowmeter to bags/hour.

- If the grout pump is able to reach the maximum allowable injection pressure, Reclamation typically pumps a 5:1 mix for at least 2 to 4 hours before adjusting the mix, provided that the grout ‘take’ stays constant. If the grout ‘take’ steadily decreases, then the optimum mix has been reached, and thickening of the grout mix should not be attempted.

- In stages where large grout ‘takes’ occur (>40 bags/hour), and the grout pump is unable to achieve the maximum allowable injection pressure after 30 minutes to 1 hour of pumping, consideration should be given to adjusting the mix.

- If the bag ‘take’ is steadily declining, regardless of the bag ‘take’ rate, wait an additional 2 hours and reevaluate whether or not the grout mix should be adjusted.

- In general, as the grout mix is progressively thickened, Reclamation experience has shown that the optimum grout mix is obtained when the grout ‘take’ decreases to about 10-15 bags/hour. After the bag ‘take’ falls below 15 bags/hour, the bag ‘take’ rate should be closely observed before thickening the mix.

- If the bag ‘take’ rate is generally steady and is more than 15 bags/hour after 2 to 4 hours of pumping, consideration should be given to thickening the mix.

- Each grout mix is generally given 2 to 4 hours of pumping time prior to thickening the grout mix. The amount of time for each mix may be adjusted after some experience is gained on a project.

- When the grout mix is thickened, the bag ‘take’ should remain the same or increase. If the grout mix is thickened and the bag ‘take’ drops immediately, the optimum mix may have been exceeded. If overthickening occurs, consideration should be given to using the previous, thinner grout mixes for subsequent grout batches mixed; however, Reclamation does not typically do this. If a thickening to a certain grout mix results in premature closure, caution should be used when deciding to thicken to this grout mix on future grout holes.

- If the grout 'take' is less than 10-15 bags/hour for a long duration (>8 hours) while using the same grout mix on a curtain hole or blanket hole stage, and closure has not been obtained, the grouting engineer must weigh the benefits of continuing to pump the same mix or thickening the mix. There are no set rules for this case. The decision made on this issue will vary from project to project and is a stage-by-stage decision. In general, Reclamation would likely hold a 3:1 mix or thicker for a longer duration relative to thinner mixes. Reclamation would likely switch to a thicker mix if there is high confidence that the use of a thicker mix will not result in premature closure of the hole due to slugging. If the field staff knows that the optimum mix is being pumped, Reclamation would likely keep grouting at the same mix for an extended duration. The geologic feature being grouted should factor into the decision. For example, closure must be achieved in a continuous stress relief joint in a dam abutment.

- The field staff will gain a lot of experience as to what works, and what does not work, at a damsite once grouting commences. This experience should be used to adjust any of these guidelines.

- If stable grout mixes are deemed necessary, a compromise between stable grout mixes and the optimum mix technique could be the following:

    - If closure is not obtained after progressively thickening to a 2:1 unstable grout mix, consideration could be given to switching to balanced-stable grout mixes. If this decision is made, the grout mix could be progressively thickened from 2:1 (unstable) → 2:1 (stable), 2:1 (stable) → 1.5:1 (stable), and 1.5:1 (stable) → 1:1(stable). If the bag 'take' immediately drops upon switching over to stable grout mixes, the use of unstable grout mixes should be resumed or thinner balanced-stable grout mixes should be used in the future.

    This compromise limits the likelihood of premature closure of a grout stage due to the use of an initial mix that exceeds the optimum grout mix.

Appendix A provides several examples for adjusting the grout mix.

15.15.3 Important Observations During Grouting

15.15.3.1 Uplift Monitoring

During grouting, the contractor is required to provide equipment for uplift monitoring. The system must be capable of providing real-time monitoring that is able to incorporate data from various types of instrumentation. Vertical displacements need to be accurate to at least 0.05 inch. If a predetermined amount of uplift is exceeded during grouting, the uplift monitoring system should be capable of providing warning.

Uplift monitoring can be accomplished using borehole extensometers, tiltmeters, surface measurement points, global positioning system (GPS) points, and lasers. Laser uplift monitoring is a popular choice among contractors for measuring uplift during foundation grouting. Recent Reclamation experiences with laser uplift monitoring have revealed problems with vibration, wind, and installation issues on uneven ground. These issues should be evaluated prior to commencing grouting operations.

15.15.3.2 Back Pressure and Holding Pressure

During grouting, it is important to determine whether or not a stage has back pressure or holding pressure. Differentiating between back pressure and holding pressure takes place at the manifold by closing the control valve. If the gauge pressure returns to zero, there is neither holding pressure nor back pressure. If the pressure gauge maintains pressure, open the bleeder valve in the manifold until the gauge reads zero, and then close the bleeder valve. If the gauge remains at zero, the gauge reading prior to bleeding was the holding pressure. If the pressure gauge is activated after closing the bleeder valve and shows pressure, that stage has back pressure, which could damage the formation. The cause for back pressure within a hole should be carefully evaluated. Back pressure often occurs when the formation being grouted is being lifted or squeezed, and the weight of the rock tends to force the grout back out of the hole. Back pressure could also occur if grout fills voids above the point of injection (as on a dam abutment) or methane gas or other gases in the foundation are compressed. Lowering the pressure and/or thickening the grout may reduce the potential for uplift. Under some conditions, it may be necessary to halt grouting to prevent further damage. For a completed stage with back pressure, a closed valve should be maintained at the standpipe until pressure has subsided. Holding pressure is a normal occurrence that gradually increases as the grout stage approaches refusal.

15.15.3.3 Leaks

During grouting on most projects, surface leaks are inevitable. When Reclamation’s optimum mix technique is used, most leaks start out as water or very thin grout and then progressively thicken. After grout at the leak is the same consistency as the grout being pumped, the leaks must be caulked. If caulking is needed, the super plasticizer must be temporarily removed from subsequent grout batches, and the injection pressure should be reduced until the leak is sealed. Materials that are typically used to caulk leaks in rock include lead wool, oakum, burlap, wooden wedges, and empty bags of cement (figure 15.15.3.3-1). If the rock is clean (i.e., a leak is observed during the water test and caulking is performed prior to grouting), sprayed foam and silicone based caulks have been successfully used if the product is able to bond to the rock. The contractor should always have the necessary tools onsite to caulk the leaks, such as chisels and hammers. If possible, sandbag rings around a leak may reduce leakage and aid in sealing the leak (figure 15.15.3.3-2). A manlift is sometimes used to access areas that are not readily accessible to seal leaks. If the leaks are inaccessible or cannot be sealed, sealing can be accomplished by some combination of intermittent pumping, thickening the mix, cutting the super plasticizer from the mix, and reducing the pressure. After the leaks are sealed, the use of super plasticizer should be continued.

Figure 15.15.3.3-1. Contractor sealing leak by pushing oakum into a crack in sandstone.

Figure 15.15.3.3-2. This leak has been caulked with oakum and wood wedges. A ring was formed with empty cement bags, earth materials, and grout to apply back pressure and further reduce leakage.

15.15.3.4 Communication to Adjacent Grout Holes

The likelihood of communication to adjacent grout holes is reduced by using the split spacing method and 80-foot closure patterns. If communication does occur, the following actions should be considered:

- If water or air is flowing into adjacent holes, but no grout is observed, no action is needed.

- If grout is observed flowing into adjacent grout holes, a packer must be placed in the adjacent grout hole to cap the connection. The packer should be placed just above the location where the grout is entering the adjacent hole. The pressure in the adjacent hole must be measured and be kept below the allowable grouting pressure for that hole. After refusal is obtained on the hole being grouted, the adjacent grout hole shall be immediately grouted to refusal.

15.15.4 Contractor Shifts

After grouting commences on a stage, Reclamation practice is to continuously inject grout until refusal is reached. This practice typically results in either three 8-hour shifts or two 12-hour shifts for the contractor on large grouting projects. After some experience is gained on a project, the water test results can be used to indicate when high grout ‘takes’ are expected and to assist the contractor in scheduling to avoid weekend work and overtime pay. In instances where the contractor has to shut down the operation for an unexpected reason, such as when the contractor runs out of cement, a minimum of 20 ft^3 of water should be injected into the grout hole immediately after grouting ceases. The reason for injecting water is to decrease the likelihood that refusal will be immediately reached when grouting resumes. Grouting of this stage would be resumed when the contractor resumes work.

15.15.5 Minimum Time Between Hookups

If no back pressure is observed upon completing a stage, Reclamation does not have a minimum time between hookups. The typical series of events during grouting is listed below:

- Drill the grout hole. If less than 50-percent fluid loss occurs during drilling, the grout hole is drilled to its full depth.

- The grout hole is washed out.

- The lowest stage is water tested.

- If the water 'take' exceeds 1 ft^3 over a 5-minute test period (or the particular value that is selected for grouting), the stage is grouted to refusal.

- If there is no back pressure, the packer is deflated, and water is injected through the packer pipe and packer until clean water exits the surface.

- The next stage is water tested.

- Repeat this process for each stage.

If back pressure is observed, the packer is typically left in the hole for several hours to allow the grout to partially set. Once the grout has partially set, the packer should be removed to avoid losing the packer and grouting of other stages can commence.

15.16 Postgrouting Assessment

As discussed above, the grouting specifications and drawings are viewed as a dynamic document. The design is adjusted in the field based on the actual grout 'takes'; therefore, it is necessary to continuously review the grouting results during construction.

Each grout hole should have a drill record and a grouting record. The grouting inspector is responsible for filling out these records. The grouting supervisor or shift leader is responsible for checking their accuracy before they become an official record. The grouting supervisor is responsible for totaling the pay items listed on these forms. The grouting data should be summarized in tables that summarize the results for each feature (curtain grouting, blanket grouting, pressure grouting for appurtenant features such as an outlet works tunnel or drainage adit). For blanket grouting and curtain grouting, the summary tables should also be further separated based on location (e.g., left abutment, right abutment, and valley section). These summary tables should indicate the grout 'takes' for each series of holes. Appendix C provides some examples.

The best way to track progress to evaluate closure, when using the split spacing method, is to plot the grouting results. Every foundation grouting project should have an up-to-date plan and profile of the grouting results in the field office. The process for generating these drawings varies, depending on whether or not real-time computer monitoring is used.

On every foundation grouting job, a specified number of verification core holes are included in the contract. The verification core holes are drilled within a completed 80-foot grout curtain pattern. The purpose of these verification core holes is to observe the discontinuities after closure has been reached and perform additional permeability tests to verify that the permeability of the formation has been sufficiently reduced. Within each verification hole, the rock is cored, the condition and amount of grout observed within the discontinuity are documented, water tests are performed, and some limited geophysical testing is performed. If the water tests exceed 1 ft^3 in 5 minutes, these stages would be grouted to refusal.

On a recent large Reclamation foundation grouting projects that did not use real-time computer monitoring, the grouting supervisor and field geologist were responsible for compiling all of the data collected during grouting to assess closure and keep track of the pay items. Upon receiving the inspector's daily reports, the grouting data is then transferred to a geotechnical borehole log by a geologist, and the field engineer illustrates this data on a working copy of the plan and profile grouting drawings in the field.

The drawings and borehole logs should include the following:

- Identification of hole (station number and series)
- Hole inclination angle
- Ground surface elevation
- Contact surface for geologic units
- Stage lengths
- Water 'take' and injection pressure for each stage
- Bag 'take' for each stage
- Total bag 'take' for grout hole
- Date grouted

A separate plan view drawing should be created to track any information on surface leaks and grout hole communication.

The inspector's grouting records are also provided to a computer-aided design (CAD) technician. The grouting engineer in the field is periodically provided with an updated plan and profile grout drawing to replace the working copy that is in use in the field. Appendix C shows examples of plan and profile drawings. The updated CAD drawings and working copies in the field contain the same information.

The field staff typically produce daily and/or weekly reports that briefly summarize the completed work, a summary of pay items, and any noteworthy issues that occurred. At the end of each month, Reclamation also requires that a Monthly Grouting Progress Report (the L-10 report) be completed by the grouting engineer in the field. The L-10 report briefly summarizes the areas being grouted during that month, specifies the contractor equipment, staging areas, contractor shift schedules, updated quantities for pay items, updated grout summary tables, updated drawings, selected construction photos, and a brief summary of any issues that came up during that month.

If real-time computer monitoring is used, several contractors are now able to interface the real-time grouting data to produce CAD drawings and grout summary reports. The updated grouting drawings and reports would then be available on an Internet site that would allow all authorized personal to access this information and track real-time grouting progress. It should be noted that the use of this technology is not free. The need for this technology should be made on a project-to-project basis. If computer monitoring is used, the specifications should be written such that the electronic data must be available to the Government in real time, and electronic copies of all reports and drawings must be supplied to the Government in the desired format.

As discussed above in section 15.6.7.4, the information provided on these drawings is then used to adjust the grouting design in the field based on the grout 'takes'. The grouting engineer should review these drawings on a daily basis to verify that closure is achieved.

Upon completing all grouting for the project, a final grouting report is generated to summarize the results of the project. The report should provide a complete summary of the project, along with final pay item quantities, final grout summary tables, as-built drawings, and selected construction photos.

15.17 Case Histories

The foundation grouting programs at Ridgway Dam, McGee Creek Dam, Jordanelle Dam, New Waddell Dam, and Heron Dam were designed and generally constructed using methods similar to those described in this chapter. These projects are judged to be representative of the performance observed at Reclamation facilities that were grouted using the best practices described herein. The data shown in table 15.17-1 is included to provide additional insight into the magnitude of these projects.

Since their original construction, these dams have experienced relatively little seepage. There have been few incidents of wet spots, and seepage measurements through toe drains have typically been on the order of 0 to 5 gal/min. The lack of seepage at these dams indicates that seepage through the bedrock has not been a concern.

Table 15.17-1. Grout program statistics

Dam	Blanket grouting		Grout curtain		Structural dam height (ft)	Length of grout curtain (ft)	Crest length (ft)
	Linear feet drilled	Bags of cement used	Linear feet drilled	Bags of cement used			
Ridgway Dam	38,009*	22,168 [a]	26,344 [a]	9,996 [a]	330	2,500	2,460
McGee Creek Dam	5,182	1,223	22,465	11,872	161	2,096	1,968
Jordanelle Dam	29,820	6,581	50,977	12,520	345	3,673	3,820
Heron Dam	394	5,548	81,737	134,676	269	2,900	1,220
New Waddell Dam	221,306 [c]	68,114 [c]	833,651[c]	919,036 [c]	440	10,900	4,900 [b]

[a] Data not available for grouting in left abutment due to technical difficulties with tracking during grouting. Actual grout totals are greater due to omission of these quantities.

[b] Crest length is independent of curtain grouting that occurred on the right abutment ridge.

[c] Approximate values; reported Stage I values for blanket and curtain grouting were not separated.

15.18 Quantity Estimates

The pay items that are typically included in Reclamation foundation grouting specifications include the following:

- **Lump sum payment for mobilization and demobilization**. Includes cost of moving equipment onto the site, assembling the equipment, disassembling the equipment, site cleanup, and removing equipment from the site.

- **Standpipes.** Includes cost for materials and installation of standpipes required for drilling and grouting.

- **Drill setups.** Includes cost for each time the drilling equipment must mobilize to the grout hole. Upstage grouting will require one drill setup for each hole unless 50-percent or more drill fluid loss occurs during drilling.

- **Drilling grout holes.** Includes cost for measured length of drilling for grout holes. Different types of drilling should be paid for separately. For example, sonic drilling through an embankment would have a different unit price than rock drilling.

- **Casing for grout holes.** Includes cost for casing within overburden or an existing embankment.

- **Hookups to grout holes.** Includes costs for setting packer and equipment needed for grouting. Cost is typically limited to one hookup per stage.

- **Grouting time.** Includes cost for operation of grout pump measured from the time grout is first injected into a hole until closure is reached. Reclamation provides the contractor with ½ hour of setup and cleanup time on days when grouting is performed.

- **Cement.** Includes cost for number of bags of cement actually injected into grout holes. No payment is made for cement that is wasted.

- **Admixtures.** Includes costs for any admixtures that are used in the grout mix.

- **Water tests.** Includes cost for each water test performed.

The pay items for any grouting project may vary to suit the needs of the project.

Foundation grouting differs from all other construction activities associated with civil engineering construction. The full extent of the required work is unknown until the grouting project is complete. Any quantity estimate should be considered as an approximate cost. Some guidance for estimating quantities is provided below:

- The information collected during the geologic explorations must be evaluated to determine the formation's response to grouting.

- The judgment used in providing grouting estimates should be discussed with the project designers, the grouting engineer, and geologists.

- A thorough geologic investigation is necessary for providing reasonable quantity estimates for foundation grouting. Overruns and underruns of schedule quantities related to foundation grouting can result from inadequate geologic data.

- High quality water test data is necessary for providing reasonable quantity estimates for foundation grouting. The procedures, equipment, and calculations used in the water tests should be closely reviewed.

- Previous grouting experience with similar geologic formations should be used. Appendix D contains spreadsheets that show pertinent drilling and grouting information from many of Reclamation's previous projects. The reference data provided in appendix D, when combined with actual site data for a new damsite, can further contribute to the effectiveness of grout quantity estimates. These spreadsheets could be used to view the grouting results of dams that were constructed in similar geologic settings. A final grouting report for existing Reclamation projects should be available for any project that was constructed after 1970.

- Judgment from a grouting expert is extremely valuable and should be factored into the quantity estimate.

- Estimates for standpipes, drilling, casing, setups, water tests and hookups can be estimated based on the number of grout holes provided on the design drawings. On every grouting job, some number of additional closure holes not shown on the design drawings will be required to obtain closure. An estimate needs to be made for the number of additional holes, the average depth of the additional holes, and the number of water tests, setups and hookups. For non-scheduled grout holes, the number of setups and hookups per grout hole is typically 1.

- The number of water tests, setups and hookups must account for the potential for drill fluid losses and borehole wall collapse that results in some downstage grouting.

- An estimate must be made for the number of hookups per grout hole. The number will vary based on the number of stages within each grout hole series. The number of hookups per hole should gradually decrease from the primary grout hole series to the quaternary holes series since the quaternary grout holes are typically shallower than the primary grout holes and the formation should be substantially less permeable when grouting the quaternary grout holes relative to the primary grout holes.

- Grouting time, cement 'takes' and the required amount of admixture are often very difficult to estimate.

  For a new embankment dam, the cement 'takes' are often estimated between 0.25 and 3 bags/ft of linear drill hole length, with 0.5 to 1.0 bag/ft being the most common range. The number is adjusted based on the results of the geologic investigations, experience in similar geologic units, and expert advice. For an existing embankment dam, the amount of remedial grouting could be estimated by reviewing the original construction grouting quantities and adjusting them to suit the needs and purpose of the remedial grouting project.

  For curtain grouting, the grouting time estimate is typically based on a range of 8 to 15 bags/hr. For jobs with a high bag 'take' estimate, such as 3 bags/ft, the grouting time estimate would be on the high end of the 8 to 15 bags/hr range, and vice versa. For blanket grouting, the grouting time estimate is typically based on a range of 5 to 12 bags/hr.

The amount of admixture is estimated based on the amount that is necessary for each bag of cement (such as 8 ounces of super plasticizer for every bag of cement).

15.19 References

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Appendix A

Case Histories of Pressure Grouting Programs within Reclamation

Contents

Purpose

The purpose of this appendix is to briefly summarize the performance of multiple foundation grouting projects that were generally designed and constructed using Reclamation's best practices discussed in this design standard. Design components documented in this appendix include:

- Brief description of embankment
- Brief geologic description of the damsite
- Foundation grouting design
- Closure pattern
- Brief summary of grout 'takes'
- Grout hole orientation
- Depth of holes
- Grout mixes utilized (by volume)

Factors discussed to evaluate the effectiveness of the grout programs include:

- Pre-construction foundation permeability values
- Post-grouting seepage observed (from historical seepage data)
- Post-grouting foundation permeability values

Case Histories

Ridgway Dam

Ridgway Dam was constructed from 1978 to 1987 on the Uncompahgre River in southwestern Colorado. The dam is a rolled earthfill dam with a clay central core zone and various zones. The structural height is 330 feet and the hydraulic height is 200 feet. The crest length is 2,460 feet and the reservoir has a maximum capacity of 93,945 acre-feet.

Site Geology

The geologic description of the damsite can be simplified as including various deposits of Quaternary Alluvium overlying the Jurassic Morrison Formation. Within the impervious core footprint, the alluvium was excavated to bedrock for construction of a cutoff trench composed of core material (Figure A-1). However, the alluvium material remains below the upstream and downstream portions of the dam below the shell material.

Figure A-1 – Photograph of core trench construction at Ridgway Dam.

The Morrison Formation is primarily composed of shales and mudstones, with random thin beds of sandstone and siltstone. Clay seams several inches thick occur throughout the formation. The Morrison Formation has a regional dip of about 4 to 5 degrees toward the northeast (the right abutment).

Site investigations were extensive to allow for design of an effective grout program. Site investigations included geologic mapping, air photographic interpretation, drill holes, test pits and trenches, surface and downhole geophysical testing, and laboratory testing.

Grout Design

The grout program was divided into two stages (Figure A-2) due to a slide which developed on the left abutment during excavation of the overburden materials prior to initiating foundation grouting. Stage one included a single row grout curtain between Stations 7+05 and 24+75, as well as blanket grouting at specified locations. Grouting within the river channel (Stations 13+05 to 16+25) was completed last to allow the river to be diverted.

Stage two grouting consisted of curtain grouting and blanket grouting on both the left and right abutments (Figure A-2). Grouting of the right abutment included two rows; a row located 5 feet upstream and another located 5 feet downstream. Grouting was intended to fill faults, joints, shear zones, springs, and any foundation defects that required treating.

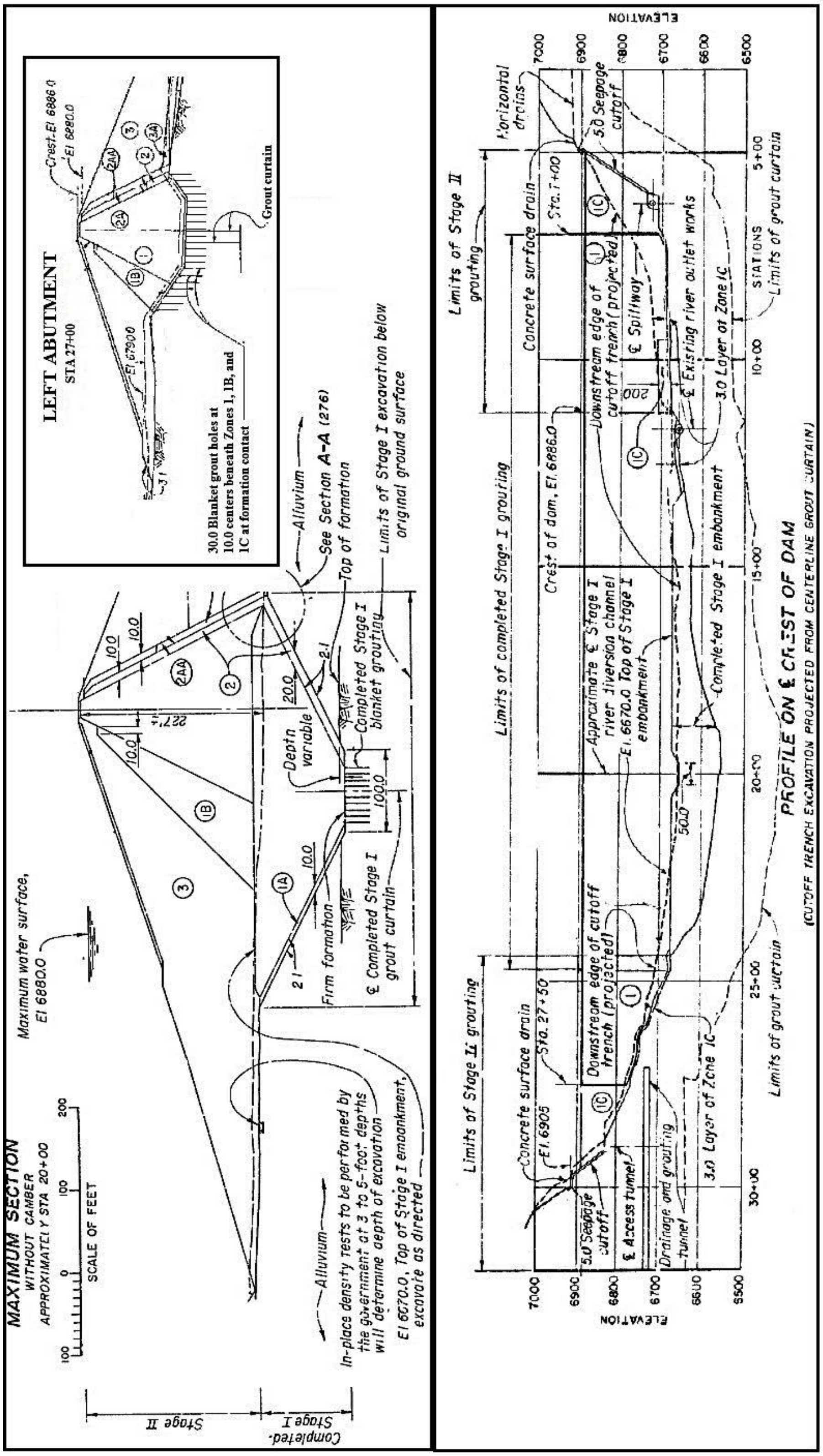


Figure A-2 – Section view of original grout curtain design at Ridgway Dam.

Grout Hole Patterns, Depths and Orientations

The main dam grout curtain was originally designed using the split-spacing closure method, with final closure holes spaced at 10-foot centers and primary holes spaced at 80-foot centers. To obtain closure, additional holes spaced at 5-foot centers were periodically included between stations 18+90 and 24+53.

The grout curtain holes between stations 7+00 and 29+82 were angled 15 degrees from vertical toward the left abutment. The upstream grout holes on the right abutment (between 4+80 and 7+50) were angled 30 degrees from vertical toward the right abutment. The downstream grout holes on the right abutment (between 4+80 and 7+50) were angled 15 degrees from vertical toward the right abutment.

The specifications also required blanket grouting the foundation to a depth of 30 feet at all locations specified by the Contracting Officer. Blanket holes were provided to permit special treatment of open cracks, jointed areas, high grout take areas, and other defects in the foundation. Blanket holes were drilled to a depth of 30 feet and were spaced on 10 foot centers. Blanket grouting was performed after the completion of curtain grouting.

Grouting Method

After setting the nipples, grout holes were drilled to depth to allow for stage-up grouting. Each hole was grouted in 30-foot stages. Before grouting a stage, a water test was performed. Each stage was pressurized to 0.75 psi per foot of depth to the bottom of the packer. Grouting was required within the stage when water-take was greater than 2 ft^3 over 5 minutes. Grout injection pressures measured at the manifold were 0.75 psi per foot of depth to the packer.

The water-cement ratio (by volume) within grout mixes typically varied from 8:1 to 2.5:1, although grout mixes with a 1:1 ratio were sometimes used to treat surface leaks. Large artesian water flows were encountered in the holes between stations 18+25 to 23+05. The flows were up to 35 gallons per minute (gpm) with pressures up to 70psi and were successfully grouted off using grout with a water-cement ratio ranging from 8:1 to 3:1.

The blanket grouting resulted in a total of 38,009 linear feet of drilling, with 22,168 bags of cement being injected into the foundation. The curtain grouting resulted in a total of 26,344 linear feet of drilling, with 9,996 bags of cement being injected into the foundation. These quantities do not include the grout takes in the left abutment due to technical difficulties with the automated data collection system. A detailed summary of the average cement take for various areas is provided in Table A-1.

Table A-1 – Summary of Grouting at Ridgway

Hole Series	Spacing (ft)	Hole Length (ft)	Angle (degrees) from vertical	Avg. Cement-Take (sacks/ft)
Stage 1 - Grout Curtain, Main Dam (Station 7+05 to 13+05 and Station 16+25 to 24+75)				
Primary	80	150	-15	0.89
Secondary	40	90		0.50
Tertiary	20	60		0.42
Quaternary	10	30		0.45
Quinary	5	30		0.11
Check Holes	Random	130-150	Not available	0.20
Stage 1 - Grout Curtain, Main Dam (Station 13+15 to 16+15)				
Primary	80	150	-15	0.15
Secondary	40	90		0.08
Tertiary	20	60		0.10
Quaternary	10	30		0.21
Quinary	5	30		0.01
Check Holes	Random	Not available	Not available	0.00
Stage 2 - Grout Curtain, Right Abutment (Station 4+81.5 to 7+50)				
4+81.5 to 7+50[b] 4+92 to 7+50[c]	Typical (80, 40, 20, 10)	150, 90, 60, 30	+15[b]/30[c]	0.27
Stage 2 - Grout Curtain, Left Abutment (Station 25+51 to 29+82)				
24+51 to 29+82	Typical (80, 40, 20, 10)	150, 90, 60, 30	-15	N/A
Stage 1 - Blanket Grouting, Main Dam (7+07 to 13+05 and 16+25 to 24+75)				
	10 x 10	30	-15	0.31
Stage 2 - Blanket Grouting, Right Abutment (4+93 to 11+30)				
115 feet u/s to 75 feet d/s	10 x 10	30	+30	0.74
Stage 2 - Blanket Grouting, Left Abutment (24+50 to 29+60)				
245 feet u/s to 345 feet d/s	10 x 10	30 (50)a	-30	N/A

[a] Holes above elevation 6750 and downstream of the dam centerline were grouted to a depth of 50 feet
[b] Right abutment upstream curtain holes (5 feet upstream of centerline)
[c] Right abutment downstream curtain holes (5 feet downstream of centerline)
[d] Positive indicates angled towards R abutment, negative indicates angled towards L abutment

Foundation Permeability Values Prior to Grouting

Foundation permeabilities were calculated from water tests performed during preliminary geologic investigations [1]. These tests were performed primarily using pressure packers over 10- to 12-foot spans within each hole. Permeability was calculated for each span in accordance with Reclamation's Designation E-18 (now designated as USBR-7310). Permeabilities were most frequently calculated to be in the 0 to 200 ft/yr (low permeability), but were sometimes calculated to be in the 200-1000 ft/yr range (moderate permeability), and were even calculated to be greater than 1000 ft/yr (high permeability) in a few instances. Testing indicated that large water losses were related to jointing, while smaller losses could be through rock pores. Also, tests indicated that the mudstone of the Morrison Formation tends to become less permeable with depth [1].

Grouting Issues

Several surface cracks were observed in the left abutment during foundation excavation and grouting operations. In this (and similar) cases, new holes were drilled nearby and grouted to greater depths to provide additional protection. Also, several water tests were performed in holes drilled after completion of grouting within an area. Tests typically showed the grouted areas to be water-tight. In the instances when they were not, additional grouting was performed.

Remediation of the large foundation block in the left abutment consisted of installation of 51 rock tendons to pin the block to the abutment, completion of the grout curtain, 23 horizontal drains from the abutment surface, extension of the left abutment drainage tunnel drains into the slide mass to reduce pore-water pressures, and instrumentation to monitor the water levels and movement of the block [2].

Grouting also took place within a drainage tunnel at the left abutment. The drainage tunnel allowed for additional grouting in the left abutment. Additional grouting was desired to provide additional stability, as several slides had already occurred on the left abutment.

Post-Grouting Seepage Observed

The 2010 CFR describes there to be "very little seepage anywhere at Ridgway Dam" and that "the downstream area of the dam is dry [2]." There are presently four active seepage monitoring locations. V-notch weirs located in inspection wells at the outfalls of the left and right toe drains have "been essentially dry since first filling [2]." The one seepage monitoring location that consistently registers measurable flows is a V-notch weir in the left abutment drainage tunnel. Flows have been found to correspond with the reservoir level, and have historically averaged 14 to 22 gpm. The low flow quantities through the drainage tunnel indicate that the grout curtain is successfully limiting flow through the foundation.

McGee Creek Dam

McGee Creek Dam was constructed from 1982 to 1987 on McGee Creek in southern Oklahoma. The dam is a zoned earthfill embankment. The dam has a crest length of 1,968 feet, and the reservoir has a maximum capacity of approximately 199,700 acre-feet. The structural height is 161 feet and the hydraulic height is 154 feet. The dam was founded on interbedded sandstone, shale, and siltstone formations. The shale deposits were found to contain dispersive clays.

Site Geology

The dam and appurtenant structures are founded on interbedded shale, sandstone, and siltstone of the Pennsylvanian Atoka Formation. The strike of the beds is at approximately 60 degrees from the dam axis, and the dip varies from about 35 degrees in the right abutment to about 10 to 15 degrees in the left abutment. Two prominent joint sets were readily recognized during construction, with one having joints spaced from 0.33 to 2.30 ft, and the other having joints spaced from 3.28 to 13.12 ft. Major faulting was not observed in the foundation area of the dam and structures [3].

Grout Design

A single row grout curtain (Figure A-3) was constructed between Stations 0+11 and 6+50[1], with a grout cap being constructed between Stations 1+49 and 4+52 to facilitate grouting and setting of nipples. Blanket grouting was performed between Stations 0+29 and 1+54 to consolidate the foundation materials and prevent piping of the dispersive clay embankment material into jointing along the abutment in this area. Curtain grouting was performed to create a positive seepage path cutoff [4].

Grout Hole Patterns, Depths and Orientations

The grout curtain (Figure A-4) was constructed using the split-spacing closure method, with final closure holes spaced at 10-foot centers and primary holes spaced at 40-foot centers. To obtain closure, additional holes spaced at 5-foot centers were periodically included. A second line of curtain holes was also added approximately 6 feet upstream of the main curtain at select locations to further establish closure.

[1] Dam Stationing is in meters. Stationing for other case histories is in feet.

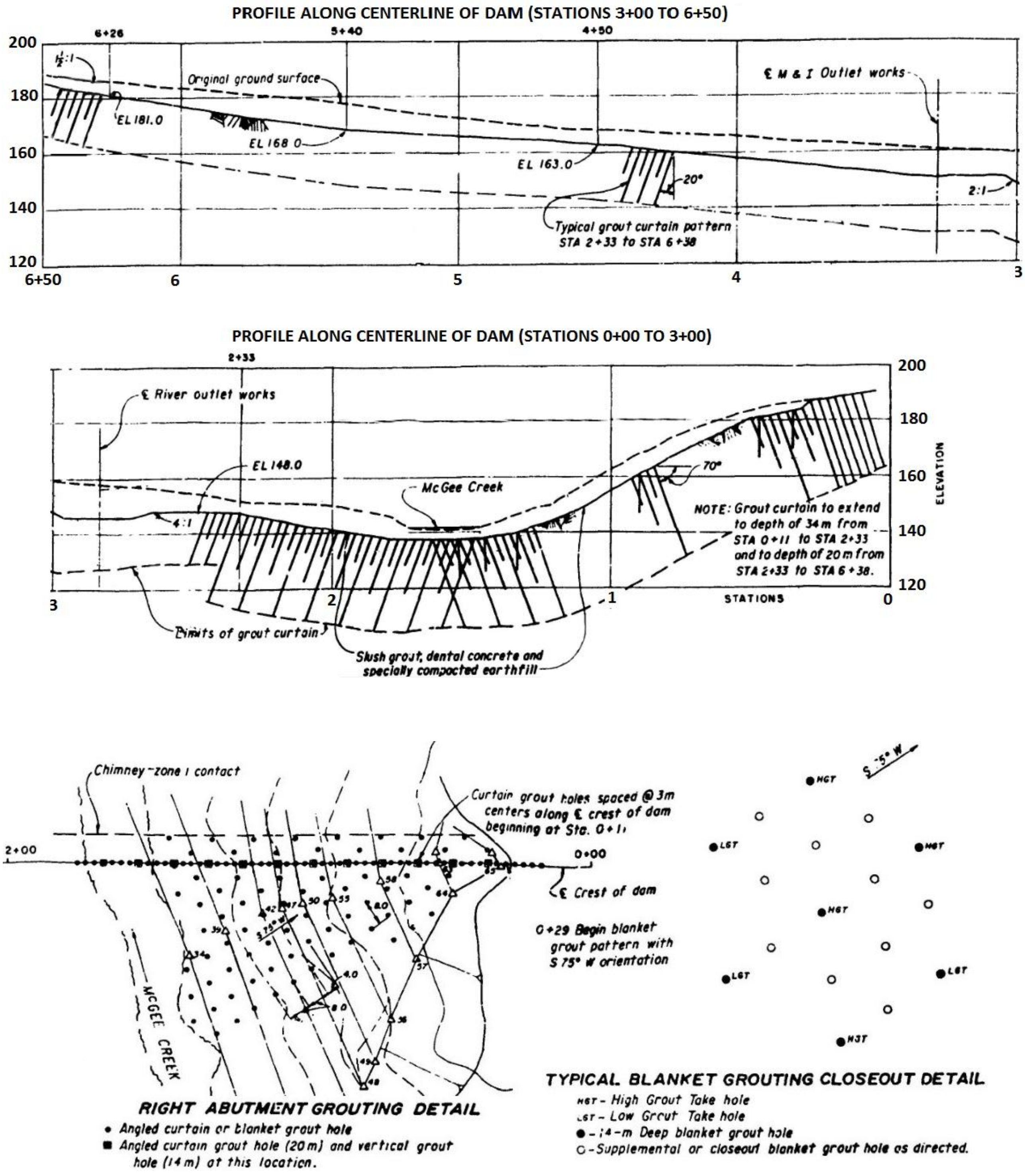


Figure A-3 – Section view of original grout curtain design at McGee Creek Dam

Figure A-4 – Photograph of curtain grouting at McGee Creek Dam

Grout curtain holes between stations 0+11 and 1+60 were angled 20 degrees from vertical towards the right abutment. Curtain holes between stations 1+50 and 6+50 were angled 20 degrees from vertical towards the left abutment.

The grout blanket consisted of 107 grout holes, constructed in a 26-foot diamond grid (75-degree angle between adjacent holes, as opposed to 90-degree angle for typical rectangular grid). Holes were angled 20 degrees from vertical towards the right abutment, with holes varying in depth from 33 to 53 feet.

Grouting Method

After setting the nipples, curtain grout holes were drilled to their scheduled depth. After drilling curtain holes, water testing and grouting proceeded in 20-ft stages. Grouting of a stage was required if the water-take under a given pressure was greater than 1 cubic foot per 5 minutes. The pressure used was based on the depth of the packer to the top of the stage. When the packer was at the ground surface, a pressure of 10 psi was used. When the packer was placed down the hole, the injection pressure measured at the manifold was 0.88psi/ft of depth plus 5 psi.

After drilling blanket holes, packers were used to test and grout the holes in two stages. Blanket grout holes were typically separated into two stages, 0 to 10 feet and 10 to 30 feet. The injection pressure measured at the manifold used was based on the depth of the packer at the top of the stage. When the packer was at the ground surface, an injection pressure of 5 psi was used. When the packer was placed down the hole, the injection pressure measured at the manifold was increased by 0.66 psi per foot of depth to the bottom of the packer.

The starting water-cement ratio (by volume) for each stage was determined based on the results of the water tests and is summarized in Table A-2.

Table A-2 –Starting grout mix (water:cement ratio) based on water loss observed

Water loss (ft3/ 5 minutes)	Starting mix (water:cement)
1.1 to 3.5	8:1
3.6 to 7.0	6:1
> 7.0	5:1

The blanket grouting resulted in a total of 5,182 linear feet of drilling and 1,223 bags of cement were injected into the foundation. The curtain grouting resulted in a total of 22,465 linear feet of drilling and 11,872 bags of cement were injected into the foundation. A detailed summary of the average cement take for various areas is provided in Table A-3.

Table A-3 – Summary of Grouting at McGee Creek Dam

Hole Series	Spacing (ft)	Hole Length (ft)	Angle (degrees)	Avg. Cement-Take (sacks/ft)
Right Abutment + Main Dam Grout Curtain (0+11 to 1+50)				
Primary	40	40 to 119 ft	-20	0.18
Secondary	20		-20	0.04
Tertiary	10		-20	0.09
Quaternary	5		-20	0.03
Check Holes	N/A		-20	0.01
Left Abutment + Main Dam Grout Curtain (1+50 to 6+50)				
Primary	40	40 to 138 ft	+20	1.29
Secondary	20		+20	0.85
Tertiary	10		+20	0.34
Quaternary	5		+20	0.25
Check Holes	N/A		+20	0.10
Right Abutment Blanket Grouting (0+29 to 1+54)				
Primary	8 x 8 m grid	33 to 53 ft	-20	0.27
Secondary			-20	0.24
Tertiary			-20	0.42
Quaternary			-20	0.07
Closure			-20	0.13

Foundation Permeability Prior to Grouting

Joints and fractures within the top 5 to 10 feet of ground surface are mostly tightly closed or filled, as evidenced by field permeability test results [5]. Maximum water take during field permeability tests was about 30.4 gpm. Most water takes were less than 2.5 gpm. Depth to tight rock (rock with water take equal to or less than 1.3 gpm) varied from 10 to 120 ft [4].

The construction design indicated that "with the cutoff beneath the dam excavated to moderately to slightly weathered bedrock and with effective grouting there will be no potential for significant reservoir losses due to under seepage and deep percolation [5]."

Grouting Issues

No significant issues were noted during grouting in the available records.

Post-Grouting Seepage Observed

McGee Creek is heavily monitored, so seepage issues could be expected to be more readily observed. Five seepage measurement points are monitored at the dam. Flows at these measurement points are typically less than 1 or 2 gpm, and a maximum flow of 5 gpm was measured on one occasion [6].

Flow is also monitored at three additional locations downstream on the right abutment. These areas started as wet spots before they started flowing in 1987. The maximum flow measured for any of the wet spots was 1.35 gpm. However, these areas have been drying up in recent years [6]. The grouting program and foundation treatment performed at the site appears to have been satisfactory as seepage has been minimal [6].

Jordanelle Dam

Jordanelle Dam was constructed from 1987 to 1992 on the Provo River approximately 25 miles southwest of Salt Lake City. The dam is a zoned earthfill structure with relatively steep upstream and downstream slopes. The dam has a crest length of 3,820 feet, and the reservoir has a maximum capacity of 361,000 acre-feet. The structural height is 345 feet, while the hydraulic height is 283 feet.

Site Geology

The dam and appurtenant structures are primarily founded on an intrusion of andesite porphyry of the Jordanelle Stock unit, although a portion of the downstream toe is founded on volcanic breccia of the Coyote Canyon unit. Surficial deposits above these units consist of alluvium, colluvium, slope wash, talus, landslide deposits, and alluvial fan deposits. The intrusion of the andesite porphyry created stresses that resulted in areas of shearing, fracturing, and extensive weathering in the foundation. These areas have been referred to as faults, structure zones, shears, and hydrothermally altered zones during the preconstruction investigations. They lack continuity, are steeply-dipping, vary widely in width, and do not extend beyond the andesite boundary. The primary rock crystals within these areas have been pseudomorphically replaced by clay minerals.

Grouting was expected to be "difficult in the foundation … [as the] high variability of the jointing patterns … [would] make it difficult to predict spacing and orientations for grout holes [7]."

Grout Design

The grout program consisted of both blanket grouting and curtain grouting along the entire length of the dam (Station 1+97 to 38+70, Figures A-5 and A-6). Blanket grouting always took place before curtain grouting. Alluvium was excavated down to bedrock within the footprint of where zone 1 material was to be placed, allowing blanket grouting to be performed within the area.

Construction was divided into two stages to allow the Provo River to be diverted. This allowed grouting to take place across the entire channel without being disrupted by the river. The first stage took place from Station 14+65 to 20+20, while the second stage took place from Station 1+95 to 15+25, and from Station 19+90 to 38+60.

The width of the grout blanket spanned 100 feet (from 168 ft to 268 ft upstream of the dam centerline) using six rows of grout holes. Five of these rows (located 168, 188, 228, 248, and 268 feet upstream, designated as Lines 1, 2, 3, 5, and 6, respectively) made up the grout blanket. A sixth row (located 208 ft upstream of the dam centerline, designated as Line 4) made up the grout curtain. This pattern is depicted in Figure A-7.

Additional holes needed for closure of the grout blanket were located in intermediate rows (designated as Lines 0.5, 1.5, 2.5, 4.5, and 5.5, based on their location) between the existing rows. Most of the additional closure holes needed for the grout curtain were located 10 feet upstream (Line 3.5) of the grout curtain centerline (Line 4), although in some instances additional holes were also located 5 feet upstream (Line 3.75) of the grout curtain centerline.

Grout Hole Patterns, Depths and Orientations

The hole spacing pattern for the blanket hole rows was constructed using the split-spacing closure method, with final closure holes spaced at 5-foot centers, and primary holes spaced at 40-foot centers. To obtain closure, additional holes spaced at 2.5-foot centers and at 1.25-foot centers were periodically included.

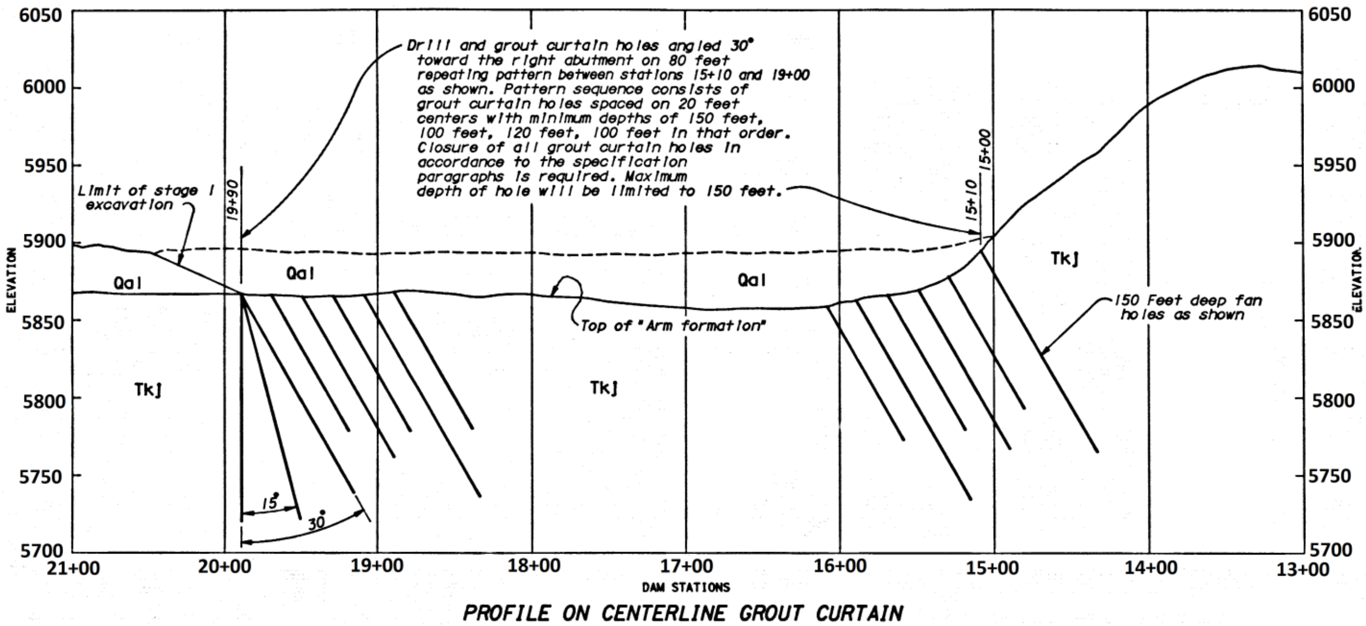


Figure A-5 – Section view of original grout curtain design at Jordanelle Dam – Stage 1

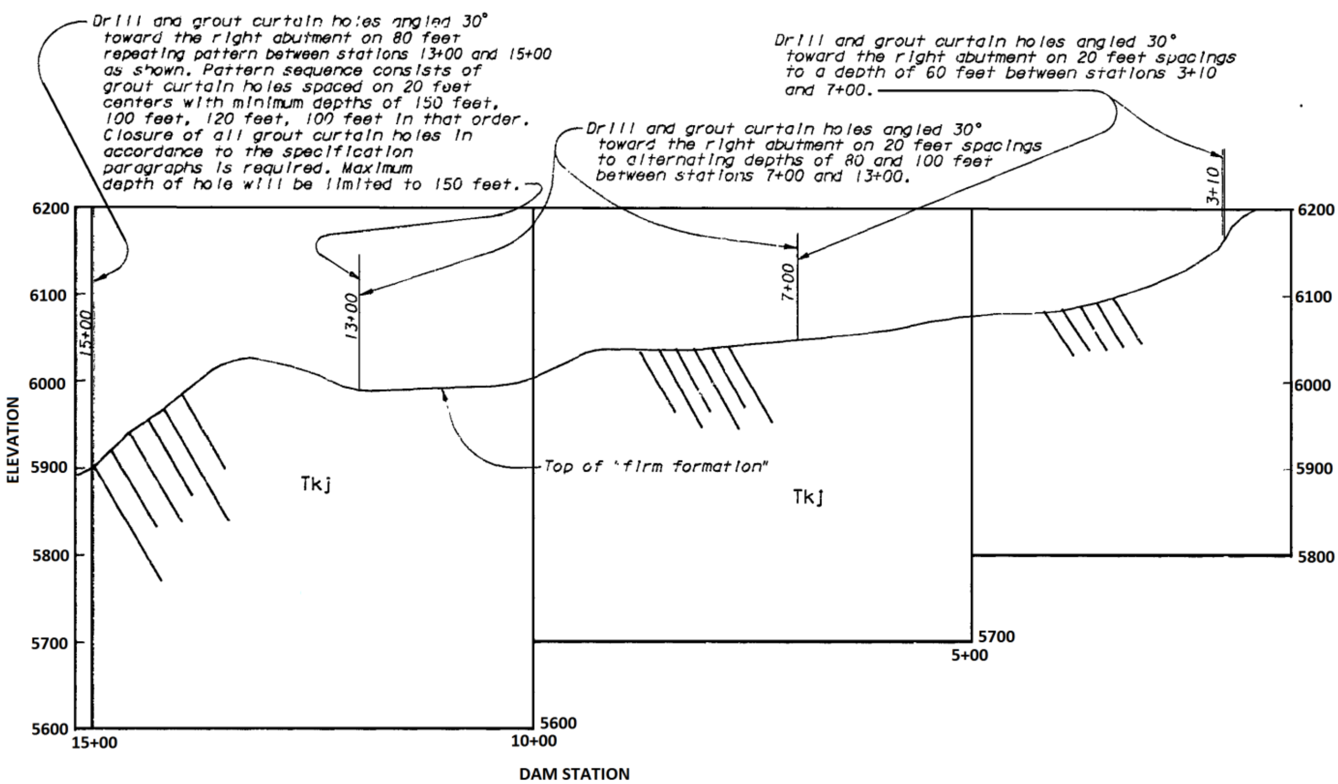


Figure A-6 – Section view of original grout curtain design at Jordanelle Dam – Stage 2 (1 of 2)

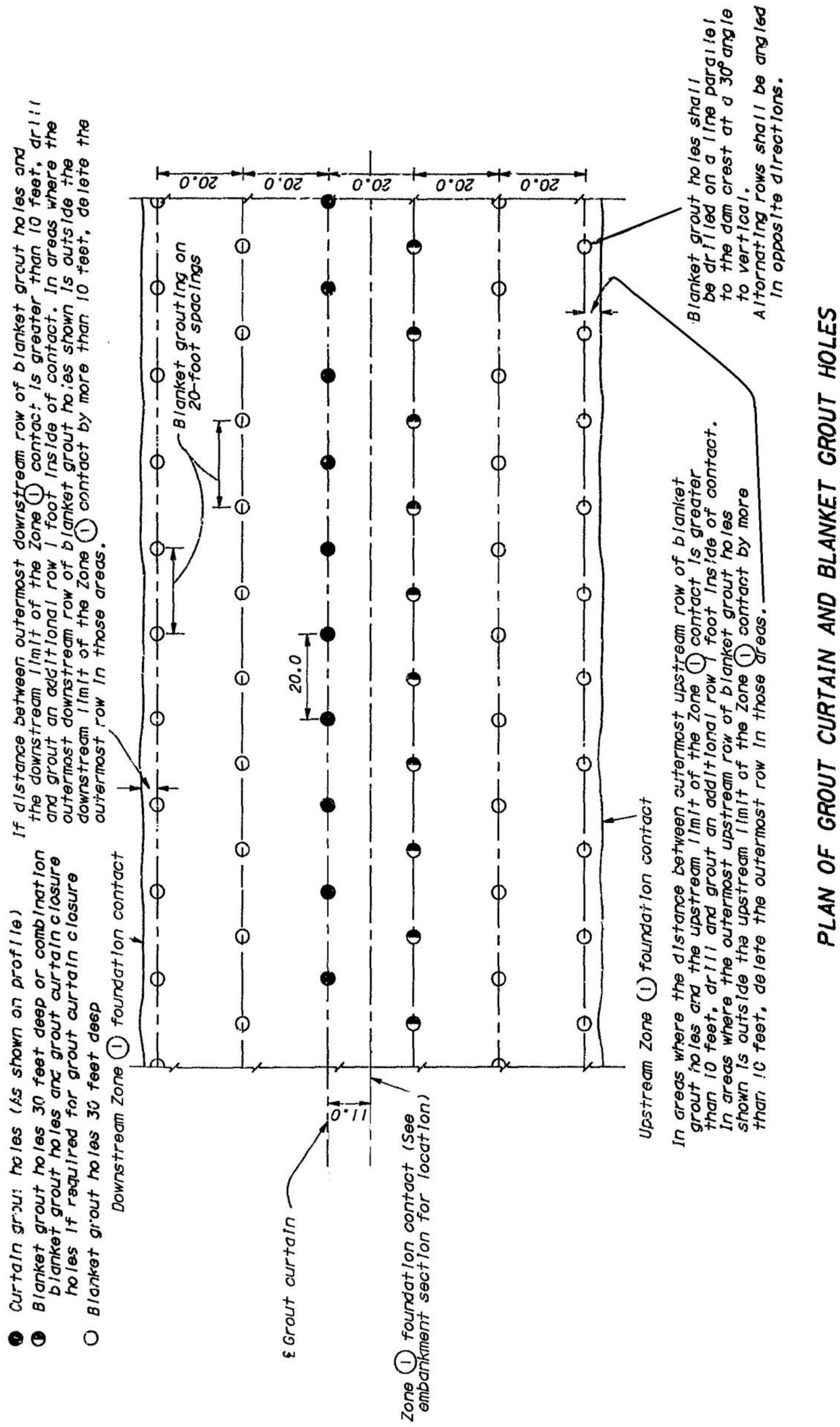


Figure A-7 – Grouting pattern at Jordanelle Dam.

The hole spacing pattern for the grout curtain holes was also constructed using the split-spacing closure method. Final closure holes were spaced at 10-foot centers, and primary holes spaced at 80-foot centers. To obtain closure, additional holes spaced at 5-foot centers and at 2.5-foot centers were periodically included.

Blanket holes were drilled 30 feet deep and were oriented either vertically, or dipped 30 degrees from vertical (towards the nearest abutment). Curtain holes were drilled to depths varying from 100 to 150 feet for primary, secondary, and tertiary holes, and to depths varying from 20 to 150 for closure holes. Hole orientation was often modified from the orientation presented in the specifications (Figure A- 8) in order to better intercept faults and seams.

Grouting Method

Blanket grouting was performed first. After setting the nipples, grout holes were drilled to their scheduled depth. Upstage grouting methods were utilized. Packers were then used to water test and grout the holes in two stages, 0 to 10 feet and 10 to 30 feet, with the injection pressure based on the depth of the packer. For the upper stage in the blanket holes, an injection pressure of 10 psi at the manifold was used. An injection pressure of 15 psi measured at the manifold was used when the packer was placed at a depth of 10 feet.

After completing the grout blanket, construction proceeded on the grout curtain. Upstage grouting methods were utilized. After drilling curtain holes, packers were used to perform water tests in 20- or 30-foot stage lengths. The stage length increased from 20 to 30 feet at depths greater than 120 feet. The injection pressure measured at the manifold used for both water testing and grouting was equal to 1 psi per foot of depth to the bottom of the packer. Grouting of a stage was required if the water-take under a given pressure was greater than 1 cubic foot per 5 minutes. When the grout-take dropped below 2 bags per hour, or below 1 ft^3 of grout mix per 10 minutes, grouting of the stage was considered completed. Grout mixes normally consisted of a water-cement ratio (by volume) between 5:1 and 1:1.

The blanket grouting resulted in a total of 29,820 linear feet of drilling and 6,581 bags of cement were injected into the foundation. The curtain grouting resulted in a total of 50,977 linear feet of drilling and 12,520 bags of cement were injected into the foundation. Due to errors associated with the grout monitoring software, the variance grout sequences (primary, secondary, etc.) were often mislabeled. This led to the final grouting report being misleading in that closure holes (and later-sequence holes) did not always reflect smaller grout-takes. For this reason, Table A-4 does not indicate grout takes for these sequences. However, it was noted that “each line was completely closed in accordance with standard grouting procedures [8].”

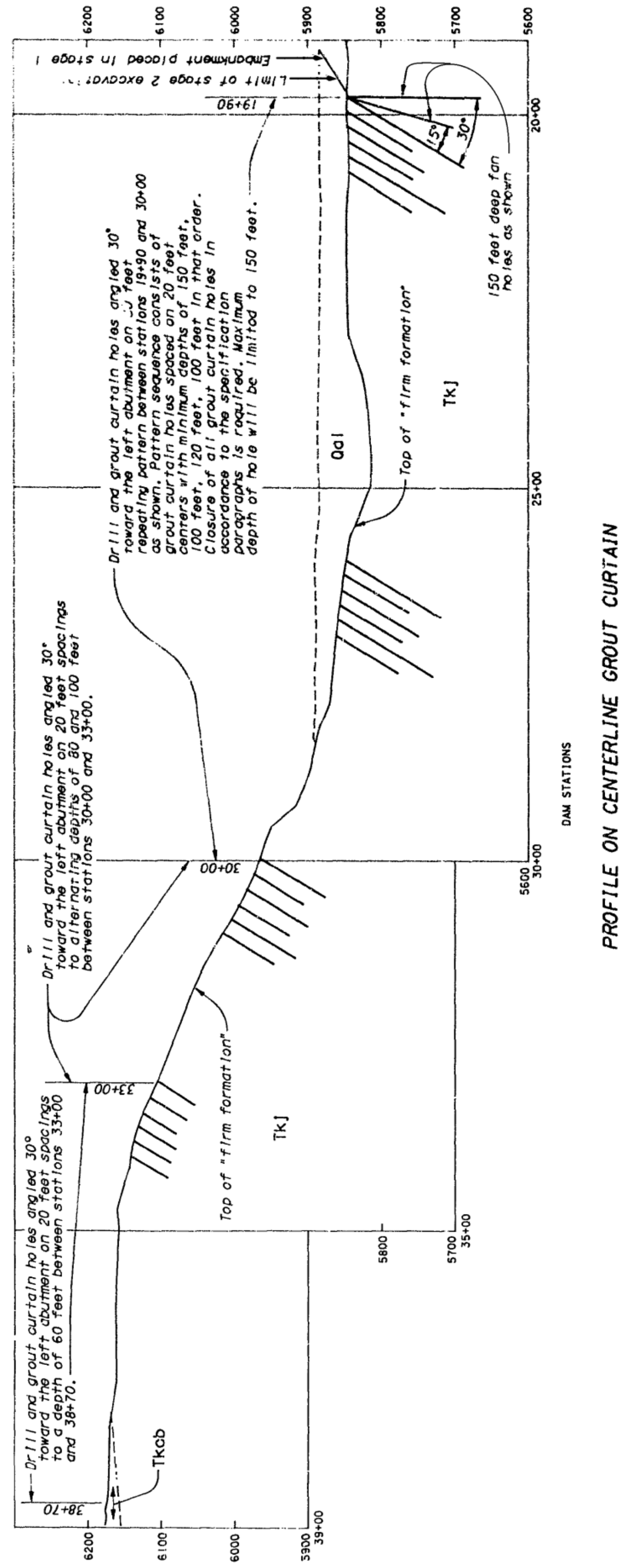


Figure A- 8 – Section view of original grout curtain design at Jordanelle Dam – Stage 2 (2 of 2)

Table A-4 – Summary of Stage 1 Grouting at Jordanelle Dam

STAGE 1 (14+65 to 20+20)					
Blanket*			Curtain (Line 4)		
Sequence	Spacing (ft)	Depth (ft)	Sequence	Spacing (ft)	Depth (ft)
Primary	40	30	Primary	80	150
Secondary	20	30	Secondary	40	120
Tertiary	10	30	Tertiary	20	100

STAGE 2 (1+95 to 15+25, and 19+90 to 38+60)					
Blanket*			Curtain**		
Sequence	Spacing (ft)	Depth (ft)	Sequence	Spacing (ft)	Depth (ft)
Primary	40	30	Primary	80	150
Secondary	20	30	Secondary	40	120
Tertiary	10	30	Tertiary	20	100
Quaternary	5	30	Quaternary	10	20-150
Quintary	2.5	30	Quintary	5	20-150
Sextary	1.3	30	Sextary	2.5	20-150

* Blanket included lines 1, 2, 3, 5, and 6, plus additional closure lines 0.5, 1.5, 2.5, 4.5, and 5.5
** Curtain included line 4 and additional closure lines 3.5 and 3.75

Foundation Permeability Prior to Grouting

Permeability values for the foundation were documented in Appendix 1 of the Foundation Construction Geology Report [7]. Permeabilities were calculated to be less than 200 ft/yr (low permeability) in about 75 percent of tests, were found to be in the 200-1000 ft/yr range (moderate permeability) in about 20 percent of tests, and were found to be greater than 1000 ft/yr (high permeability) in about 5 percent of tests. Packer tests indicated that “permeability in the foundation is variable and unpredictable, but generally small [7].”

Grouting Issues

Casing was required for some holes to keep the hole from caving in on itself. Uplift was monitored using installed elevation measurement points every 40 to 50 feet along the grout curtain line. No change in elevation was detected at these points. Surface leaks were “caulked” with oakum and cement, but were relatively ineffective in sealing the surface. Pressure transducers and flowmeters were used to monitor grouting operations but occasionally were inactive due to technical difficulties. Reclamation’s grouting software was also not entirely compatible with the grout hole layout due to the complexity of the site and the significant number of grout lines. This resulted in errant values in the grout-take report [8].

Post-Grouting Seepage Observed

Jordanelle Dam is well instrumented and seepage has been minimal [9]. The CFR states that “there is a lack of any appreciable seepage emanating from the downstream toe areas.”

Total seepage through the toe drain system is typically between 15 and 30 gallons per minute, depending on the reservoir level." This is a "very low volume of seepage given the size and hydraulic height of Jordanelle Dam [9]." Since 2001, seepage has been measured greater than 32 gpm on four occasions, with maximum seepage being measuring at 60 gpm. These increased seepage values were observed when "snowmelt at the damsite is sufficient to appreciably raise tailwater levels [above the invert of the toe drain collection pipe]" [9].

Heron Dam

Heron Dam was constructed from 1967 to 1971 in northern New Mexico on Willow Creek (Figure A-9). The reservoir has a maximum capacity of 401,320 acre-feet. The dam has a crest length of 1,220 feet, a structural height of 269 feet, and a hydraulic height of 249 feet [10].

Figure A-9 – Photograph of grout hole construction at Heron Dam

Site Geology

The dam was founded on the sedimentary bedrock units of the Dakota and Morrison Formations. These formations each contain beds of both shale and sandstone. At the dam site, the sedimentary foundation beds dip gently at 5 to 10 degrees. Two primary joint sets were noted; numerous north-south joints were spaced a few inches to about a foot apart; east-west joints were less common and were spaced 50 to 100 feet apart. Extracted rock cores found joints within the sandstone to be open 1/8 -inch to 1/4-inch, but to be tight within the shale. Open stress relief joints at the surface of the sandstone were found to be open from a fraction of an inch to two or more feet for lengths up to 150 feet.

Site investigations were extensive to allow for design of an effective grout program. Site investigations included subsurface mapping and drilling, and surface mapping.

Grout Design

The grout program (Figure A-10) included the following components: a grout curtain beneath the dike; the main dam grout curtain along the valley bottom and extending up the abutments; a grout cap above the main dam grout curtain (with a width of 3-feet and a depth ranging from 3 to 6-feet); a second grout curtain (constructed without a grout cap) located twenty feet upstream of the main curtain along the channel bottom and extending partway up the abutments; *extended grout curtains* extending 1,000 feet beyond each abutment; grout blanket over portions of the impervious core footprint; and grouting along features such as the outlet works intake structure, tunnel, gate chamber, adit to gate chamber, and access shaft.

The main dam grout curtain was designed to prevent seepage through relief joints in the abutments, and to prevent seepage through the valley bottom. The grout curtain extending beyond the abutments was to prevent seepage in relief joints parallel to the main river channel.

Grout Hole Patterns, Depths and Orientations

The main dam grout curtain was originally designed using the split-spacing closure method, with final closure holes spaced at 10-foot centers and primary holes spaced at 80-foot centers. However, need for additional grouting resulted in some final closure holes being spaced as closely as at 5-foot centers. Hole spacing for the second grout curtain and the extended grout curtains are also presented in Table A-5.

Grout curtain holes were angled at 24 degrees from the vertical, but in opposite directions on each abutment. These hole-orientations were extended from the abutments towards the center of the channel-bottom, with the two orientations overlapping by about 68-feet.

Grout mixtures used within the curtain had water-cement ratios (by volume) ranging from 12:1 to 1:1, with most of the grout having an 8:1 ratio. Grout injection pressures measured at the manifold were 1 psi per foot of depth to the packer setting (and a maximum pressure of 150 psi) plus 10 to 15 psi.

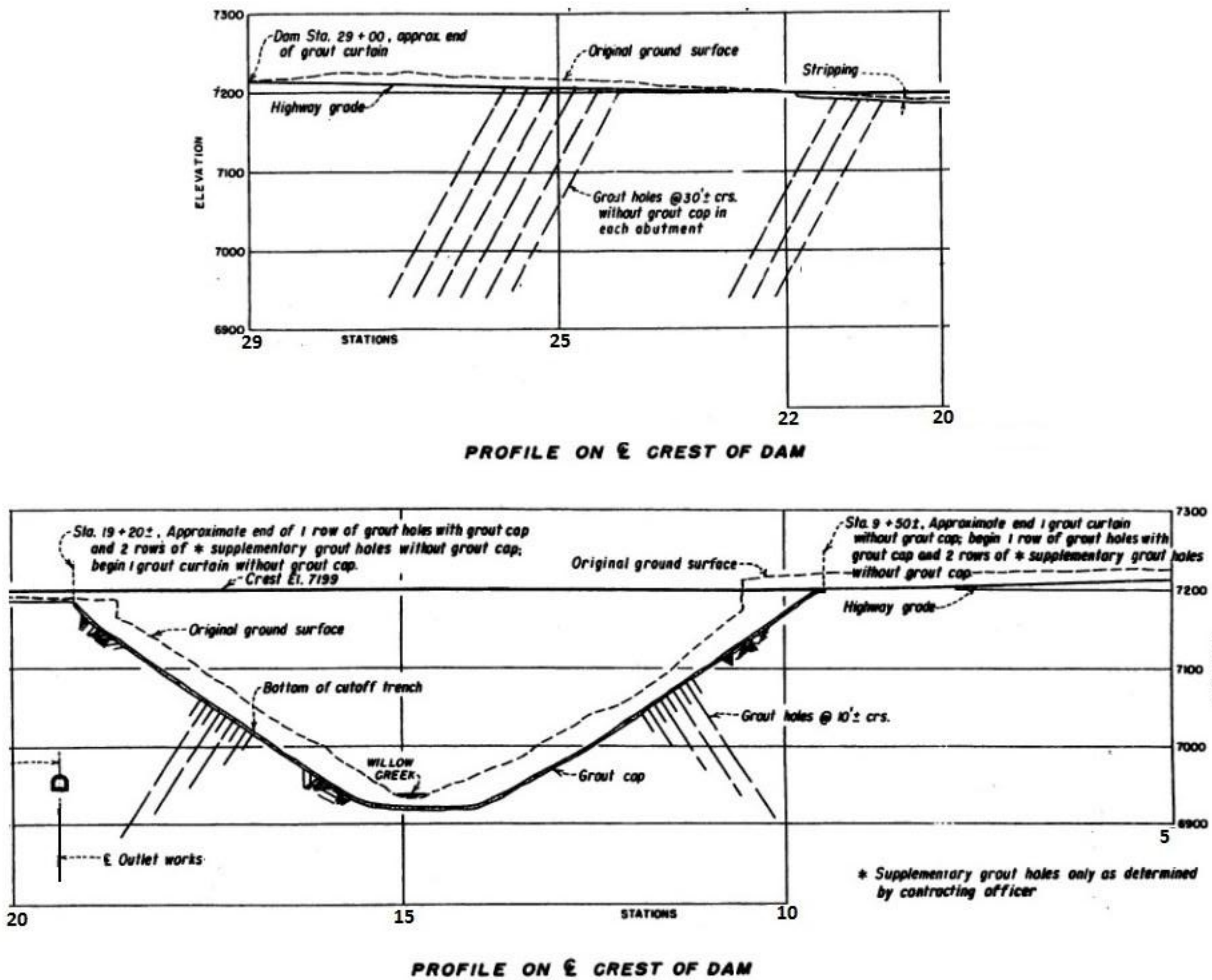


Figure A-10 – Section view of original grout curtain design at Heron Dam.

Table A-5 – Summary of Grout-Takes at Heron Dam

Hole Series	Spacing (ft)	Hole Length (ft)	Angle (degrees) from vertical**	Avg. Cement-Take (sacks/ft)
Main Dam Grout Curtain				
Primary	80	30 to 260 (30 to 150)*	±24	9.11
Secondary	40		±24	5.62
Tertiary	20		±24	2.48
Quaternary	10		±24	0.77
Quinary	5		±24	0.57
Upstream Curtain (20-feet upstream)				
Primary	40	30 to 250 (30 to 140)*	±24	0.49
Secondary	20		±24	0.30
Tertiary	10		±24	0.33
Quaternary	5		±24	0.00
Extended Curtain (Left Abutment)				
Primary	120	60 to 260	+24	8.08
Secondary	60		+24	3.64
Tertiary	30		+24	2.26
Quaternary	15		+24	0.69
Quinary	7.5		+24	0.33
Senary	3.75		+24	0.03
Extended Curtain (Right Abutment)				
Primary	120	90 to 260	-24	2.81
Secondary	60		-24	1.65
Tertiary	30		-24	0.70
Quaternary	15		-24	0.34
Quinary	7.5		-24	0.10

* The long grout holes (primary, secondary, etc.) along the channel bottom were 150 ft long, while the holes were lengthened to 260 ft along the abutments
**Positive angle indicates angled towards R abutment, negative angle indicates angled towards L abutment

Thirty-six blanket holes were drilled and grouted. These holes ranged from 5 to 40 feet deep, and were oriented vertically. Grout mixtures used for blanket holes had water-cement ratios ranging from 5:1 to 1:1. Pressures were limited to 10psi at the manifold for each hole.

The blanket grouting resulted in a total of 394 linear feet of drilling and 5,548 bags of cement were injected into the foundation. The curtain grouting resulted in a total of 81,737 linear feet of drilling and 134,676 bags of cement were injected into the foundation. A detailed summary of the average cement take for various areas is provided in Table A-5.

Grouting Method

Upstage grouting methods were utilized. After drilling curtain holes, packers were used to perform water tests in stages. Grouting was required when water-take was greater than 2 ft^3 per 5 minutes. Stages were set in 30-foot increments for spans of the holes deeper than 100 feet, and in 20-foot increments for spans of the holes shallower than 100 feet. Within each hole, grouting was stopped when grout-take was less than 1 ft^3 for the following conditions:

- 10 minutes (at less than 50 psi)
- 7.5 minutes (at 50 to 100 psi)
- 5 minutes (at greater than 100 psi)

To ensure that closure was taking place, grout-take quantities were compared from sequential closure hole groups (primary, secondary, etc.). This data is also presented in Table A-5.

Foundation Permeability Values Prior to Grouting

Foundation permeability values were not included within the documents reviewed for this appendix. Likewise, packer test data performed during grouting operations was not located.

Grouting Issues

No significant issues were noted during this review.

Post-Grouting Seepage Observed

There are thirteen seepage monitoring points; eight below the dike and 5 below the dam. Seepage has been monitored since 1975. An inspection dated September 11, 1980 found minimal seepage downstream of the dam, indicating that the grout curtain “undoubtedly blocked seepage that would have developed through the relief joints paralleling the canyon walls [11].” The CFR states that “measurable seepage has been very low or nonexistent since construction [10].” The dam has operated with reservoir elevation at least ten feet below the top of active conservation pool (Elevation 7186.1) since 2003. This timeframe corresponds to a drought within the area. No seepage has been reported from the toe drains during this time. Prior to the drought, the maximum seepage measured from any of the toe drains was 4 gpm in 1983.

New Waddell Dam

Construction on New Waddell Dam was completed in 1992. The dam is approximately 35 miles northwest of Phoenix, Arizona on the Agua Fria River. The dam impounds Lake Pleasant, which has a maximum capacity of 902,000 acre-feet. The dam has a crest length of 4,900 feet, a structural height of 440 feet, and a hydraulic height of 300 feet. The dam was founded on bedrock except near the maximum section, where the foundation excavation extends only to the top of a dense, permeable, older alluvium. The foundation preparation included grouting, shaping of bedrock surfaces, filters, and a concrete cutoff wall through the alluvium where the embankment bears on it [12].

Site Geology

Geology at the dam site is complex, characterized by thin alluvial soils and a conglomerate unit overlying interlayered volcanic units. The alluvial soils were identified as a younger alluvium (Qal) and an older alluvium (Qoal). The younger alluvium was all removed from the dam foundation. The older alluvium, which ranges from about 95- to 120-feet thick, was excavated or compacted and treated where loose. The conglomerate unit is encountered below the alluvial units. Below the conglomerate unit are the various volcanic units, which were grouped into three rock types: tertiary andesite, brecciated andesite, and tertiary tuff.

The conglomerate is mostly horizontally bedded, with some cross-beds which dip up to 30 degrees, predominantly in the southeast to southwest direction. The dip of the bedding tends to become steeper towards the right. Volcanic units were emplaced over an irregular erosional surface, typically as flows.

Faulting at the site is believed to be inactive. Joints and fractures vary widely among the bedrock units and across the dam foundation. The orientation of bedrock jointing is essentially random, especially in the volcanic units. Within the footprint of the dam, prominent joints in the conglomerate are nearly vertical and tend to strike nearly perpendicular to the dam axis. These joints are widely to extremely widely spaced, and range from wide open to tight.

Site investigations were extensive to allow for design of an effective grout program. Site investigations included over 300 borings. Many of these borings were then pressure tested (with water) to determine permeability.

Grout Design

The grout program consists of blanket and curtain grouting along the foundation of the dam, and curtain grouting along the ridge west of the right abutment. Curtain grouting was performed in two parts: 1) exploratory grouting and 2) closure grouting. Individual holes were grouted using the upstage grouting method, with stages set in 30-foot increments for spans of the holes deeper than

100 feet, and in 20-foot increments for spans of the holes shallower than 100 feet. A grout cap was not used because the quality of the foundation rock was sufficient so that nipples could be grouted in place.

Construction of the dam was performed in two stages, with Stage II being divided into two phases. Stage I was originally designed to include all blanket and curtain grouting for the left and right abutments, and exploratory grouting of the right abutment ridge. However, due to time constraints, certain sections of abutment grouting were delayed; these sections were instead included in Stage II of construction. The grouting operations performed during Stage I and II are shown in Table A-6.

Table A-6 – Summary of Stage I and II Grouting Operations

<table>
<tr><th rowspan="2"></th><th colspan="2">Main Dam</th><th>Right Abutment Ridge</th></tr>
<tr><th>Curtain</th><th>Blanket</th><th>Curtain</th></tr>
<tr><td>Stage I</td><td>Single line exploratory grouting:
9+80 to 57+30*

Closure grouting:
28+00 to 36+55</td><td>9+80 to 33+30**
42+50 to 57+30**</td><td>Exploratory curtain grouting holes</td></tr>
<tr><td rowspan="2">Stage II</td><td rowspan="2">Closure grouting:

7+00 to 28+00
36+40 to 37+60
41+00 to 44+00
47+00 to 53+00</td><td rowspan="2">Sections of the abutments that were not grouted during Stage I:

15+00 to 20+00
22+00 to 28+00
36+55 to 37+90
41+00 to 45+30</td><td>Phase I:
Row 1 holes, spacing ranges from 10- to 80-foot (centers)</td></tr>
<tr><td>Phase II:
Closure grouting:
Additional holes for row 1; additional rows 2 and 3</td></tr>
</table>

* Exploratory grouting was not performed in vicinity of outlet works tunnel construction, or in channel section

** Blanket grouting not performed during Stage I due to time constraints and design modifications

Grouting was also performed along an existing concrete cutoff wall, along several secant pile walls, around the river outlet works intake structure and gate chamber, and around the CAP outlet works tunnels. Additionally, a grout gallery was constructed to facilitate future grouting should it be needed. Grouting performed for these additional features is not described in this report.

Grout Hole Patterns, Depths and Orientations

Curtain holes constructed along the dam axis during Stage I (Figures A-11 and A-12) were drilled to depths ranging from 100 to 250 feet. The main line of exploratory holes (located 5 feet downstream of the dam centerline) was drilled and grouted first, with adjacent holes spaced at intervals of either 10 or 20 feet. Based on water and grout takes in these holes, additional closure holes were added as needed. The additional holes were added using the split-spacing method, reducing the spacing between adjacent grout holes to as little as 5 feet in some areas. Additional rows were also added upstream and downstream of the main line where needed. Curtain holes were orientated at angles ranging from vertical to 30 degrees from vertical. An AX-size (about 2 inches) hole diameter was used for drilling. Curtain grouting along the right abutment ridge (Figures A-13 and A-14) took place using holes drilled to depths as deep as 550 feet.

Blanket grouting was performed in the foundation within the footprint of the Zone 1 core material. Blanket holes were drilled to a depth of 30 feet. They were located on a staggered, 10-foot by 10-foot grid pattern across the foundation, for the full width of the Zone 1 material. Holes were grouted in two 15-foot stages. Blanket holes were also orientated at angles ranging from vertical to 30 degrees from vertical, and were also drilled using an AX hole-size (diameter of about 2 inches).

Grout mixtures used for both blanket and curtain holes started at water-cement ratios of 5:1 (water:cement), and were varied as necessary down to a ratio of 1:1. Grout injection pressures measured at the manifold were limited to 10psi within the top stage of each hole. For blanket holes, the injection pressure measured at the manifold for the bottom stage was 20psi. For curtain holes, the injection pressure measured at the manifold within the lower stages was increased by a maximum of 1.5psi per linear foot of depth of the hole, to a maximum of 400 psi.

A high-range, water reducing admixture (HRWRA) plasticizer was used in the grout mix to improve the pumpability of the grout and to achieve greater penetration of grout into the foundation. The amount of plasticizer in the grout varied between 4 and 16 fluid ounces per bag of cement.

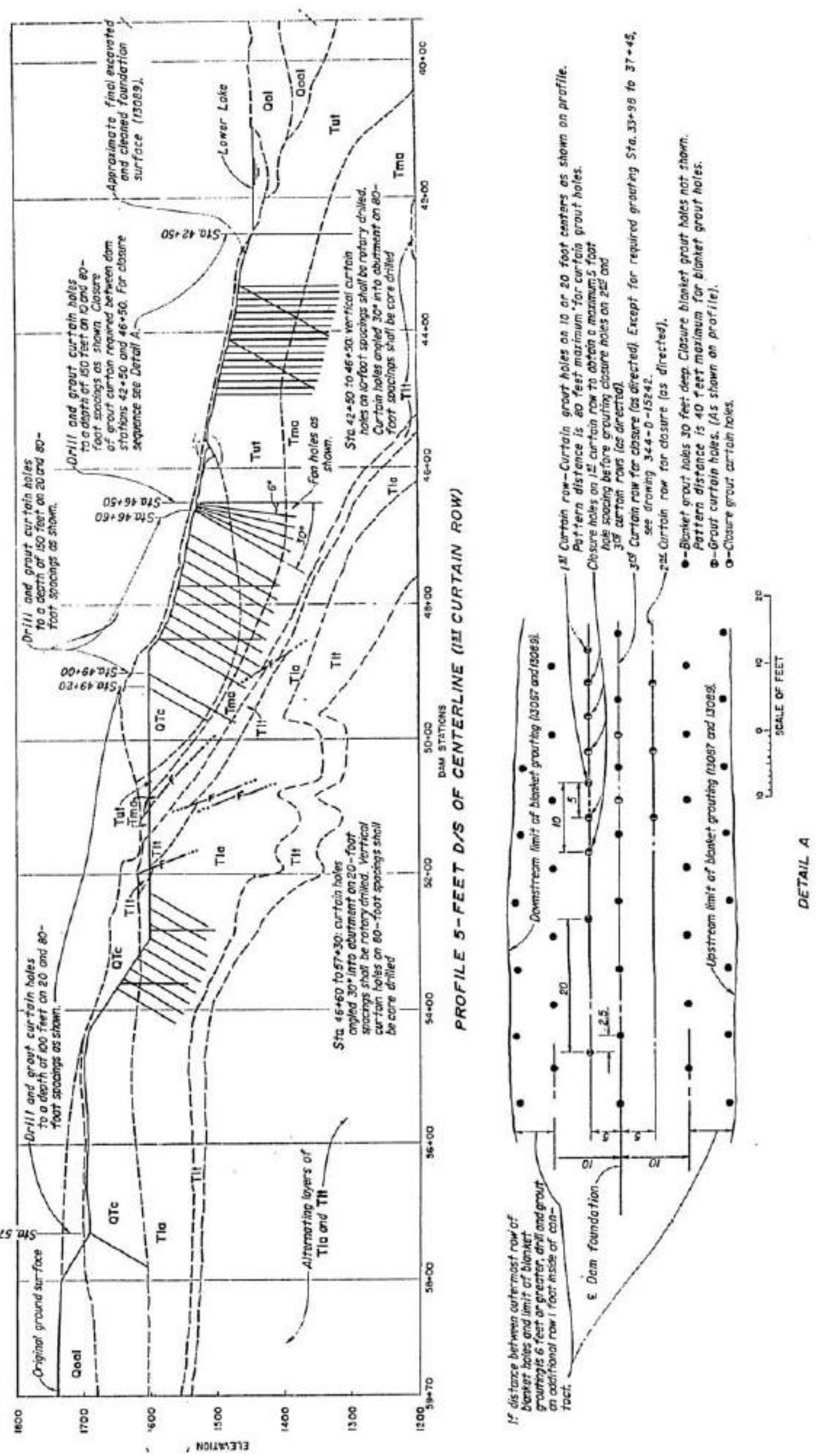


Figure A-11 – Section view of original grout curtain design at New Waddell Dam

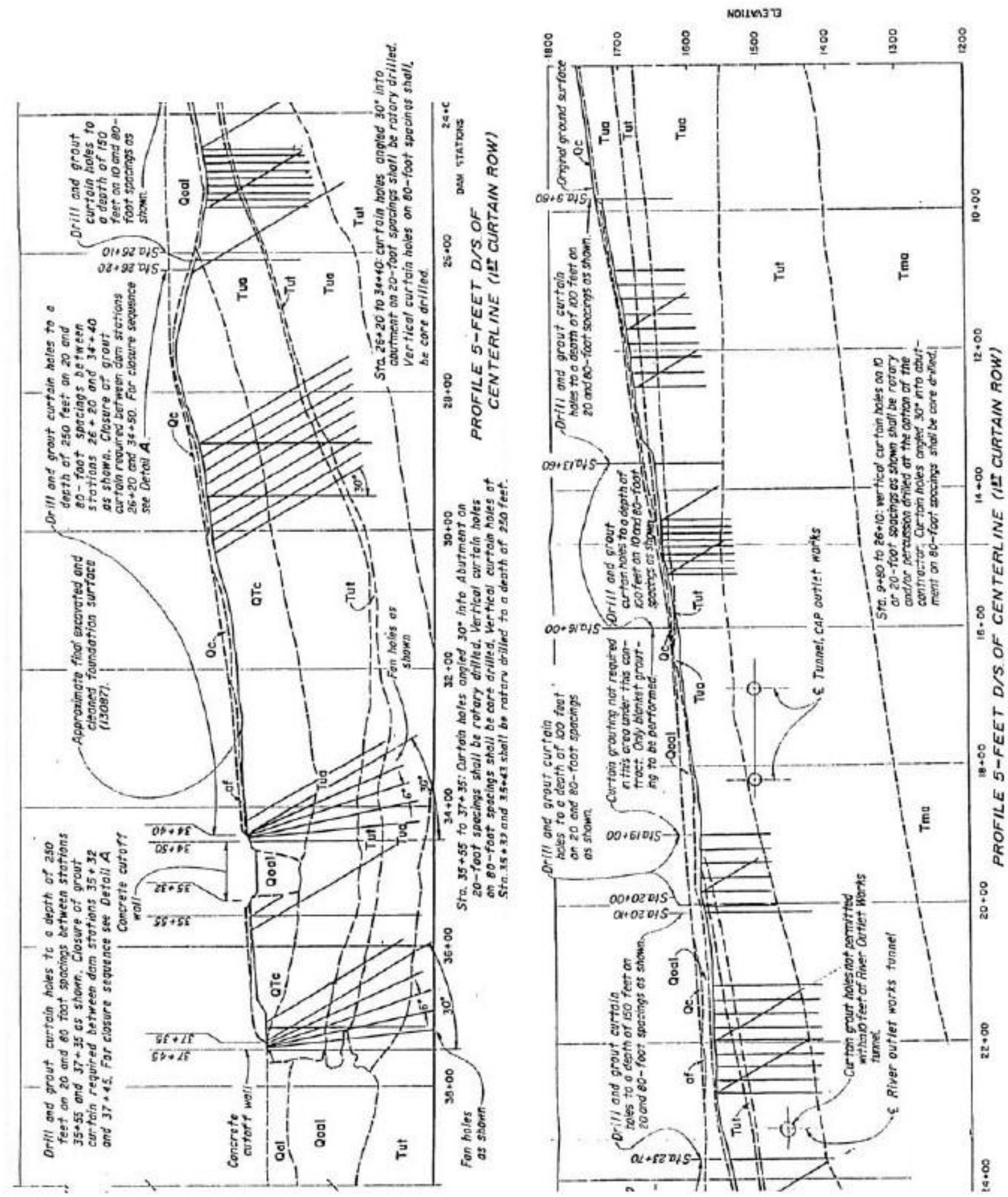


Figure A-12 – Section view of original grout curtain design at New Waddell Dam

Grouting Method

Grout holes were first drilled to their full prescribed depth. After drilling a hole, a packer was inserted and the hole was water-tested in stages, with each stage being grouted if needed. Grouting was required when water-take was greater than 2 ft^3 per 5 minutes in curtain holes, and greater than 3 ft^3 per 5 minutes in blanket holes. Grouting was stopped within each stage of a hole once it met refusal criteria, which was defined as when grout-take was less than 1 ft^3 for the following conditions:

- 20 minutes (at less than 50 psi)
- 15 minutes (at 50 to 100 psi)
- 10 minutes (at greater than 100 psi)

Table A-7 provides a detailed summary of the average cement take within curtain and blanket holes.

Table A-7 – Summary of Dam Foundation Grouting

	Stage	Type	Total Linear Feet Drilled	Total Grout Injected (bags)	Avg. Cement-Take (bags/ft)
Main Dam	Stage I	Blanket and Curtain	273,331	128,519	0.47
		Blanket*	82,500*	41,500*	0.50*
		Curtain*	190,831*	87,019*	0.46*
	Stage II	Blanket	138,806	26,614	0.17
		Curtain	137,067	46,524	0.34
Right Abutment Ridge	Stage I	Exploratory Curtain	12,639	25,644	2.03
	Stage II – Phase I	Curtain Closure	90,266	458,062	5.07
	Stage II – Phase II	Curtain Closure	402,848	301,787	0.75

*Records show linear footage drilled and total grout injected quantities combined for blanket and curtain grouting. Values divided based on estimates of blanket grouting quantities from design estimates [13].

Grouting Issues

No significant issues were noted during this review.

Post-grouting Seepage Observed

There are three areas where seepage has been noted to daylight from the abutments or foundation: 1) on the cut-slope behind the pumping/generating plant, 2) on the left abutment, and 3) at the embankment toe near station 32+00. Seepage quantities are very small at the latter two locations; seepage at the toe near station 32+00 has been reduced substantially since a French drain was installed. A V-notch weir at mid-height on the cut-slope currently measures peak flows of about 15 gpm when the reservoir is up, although greater measurements have been made in the past (including a maximum seepage quantity of 35 gpm in 1994).

Seepage is also measured using various instruments located at the dam; there are 9 seepage weirs in the toe drain inspection wells, 2 outlet pipes from the toe drain system, and 7 seepage flow locations which are measured using bucket and stopwatch. Seepage at these locations is typically monitored monthly, and flows at these locations have typically been found to vary with the elevation of the reservoir. Flows from the left toe drain outfall have been measured at 17 gpm during peak reservoir surface elevations. A weir on the right side of the embankment has measured flows (from surface flows as well as subsurface seepage) as high as 60 gpm during peak reservoir surface elevations; there is no flow when the reservoir is below el. 1660. The CFR states that seepage at these locations "…has not changed significantly over time… [12]."

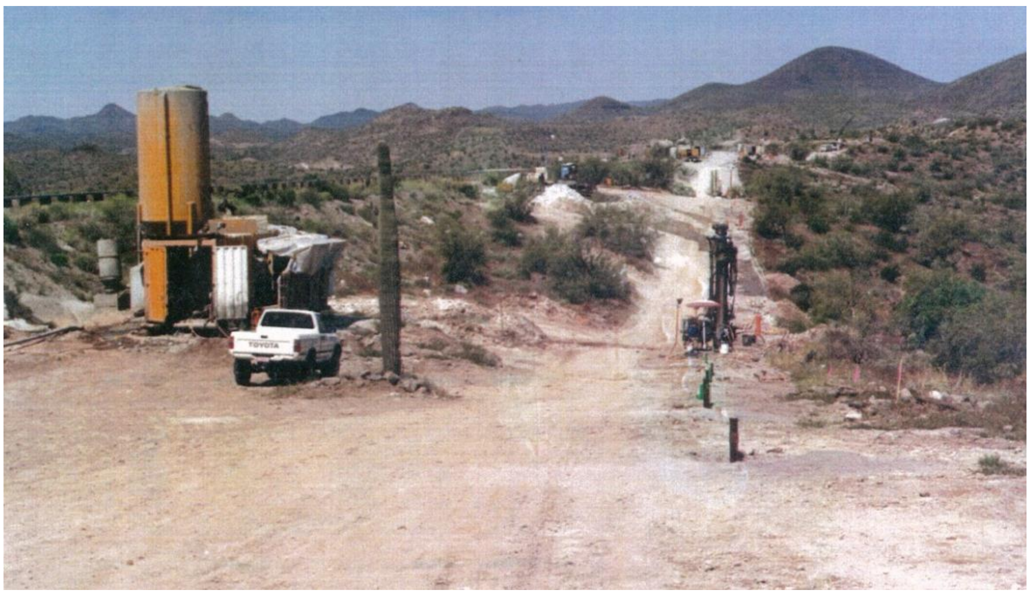


Figure A-13 – Photograph of dam alignment along right abutment ridge

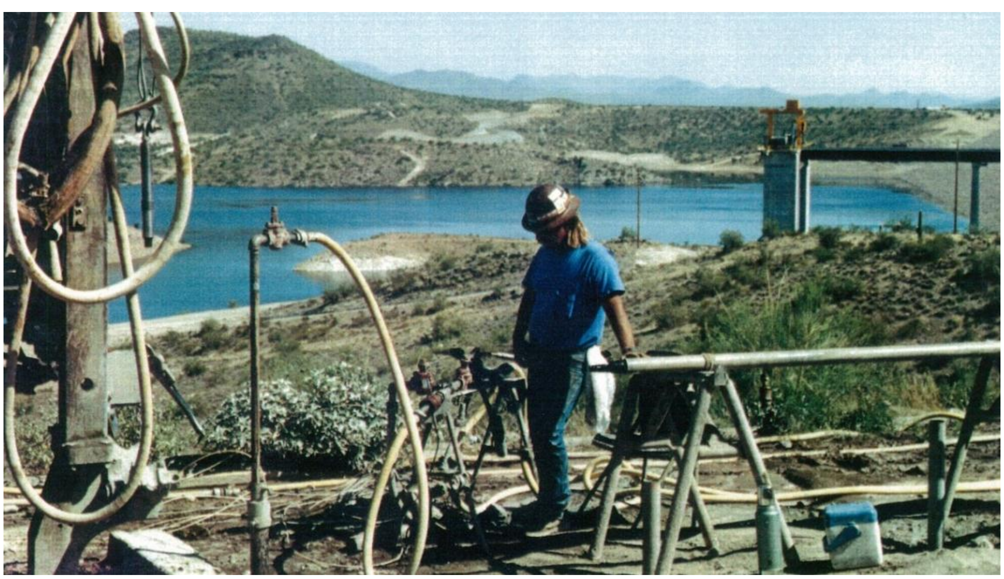


Figure A-14 – Photograph of grouting performed along right abutment ridge

Summary

The foundation grouting performed at Ridgway Dam, McGee Creek Dam, Jordanelle Dam, Heron Dam and New Waddell was generally designed and constructed using methods similar to those described in this design standard. These projects are judged to be representative of the performance observed at Reclamation facilities that were grouted using the best practices described herein. The data shown in Table A-8 has been included to provide additional insight into the magnitude of these projects.

Table A-8 – Grout Program Statistics

Dam	Grout Blanket		Grout Curtain		Length of Grout Curtain (ft)	Crest Length (ft)
	LF Drilled	Bags of Cement Used	LF Drilled	Bags of Cement Used		
Ridgway	38,009 [a]	22,168 [a]	26,344 [a]	9,996 [a]	2,500	2,460
McGee Creek	5,182	1,223	22,465	11,872	2,096	1,968
Jordanelle	29,820	6,581	50,977	12,520	3,673	3,820
Heron Dam	394	5,548	81,737	134,676	2,900	1,220
New Waddell Dam	221,306 [c]	68,114 [c]	833,651 [c]	919,036 [c]	10,900	4,900 [b]

[a] Data not available for grouting in left abutment due to technical difficulties with tracking during grouting. Actual grout totals are greater due to omission of these quantities.
[b] Crest length is independent of curtain grouting that occurred on the Right Abutment Ridge.
[c] Approximate values; reported Stage I values for blanket and curtain grouting were not separated.

Since their original construction, these dams have experienced relatively little seepage. The small amounts of seepage collected at these dams indicate that the foundation grouting performed at these structures was very effective in limiting seepage through the foundation.

References

[1] "Geologic Specification Design Data," Ridgway Dam and Reservoir, Dallas Creek Project, Montrose Projects Office, Bureau of Reclamation, Montrose, CO, April 1979.

[2] "Ridgway Dam - Comprehensive Facility Review," Bureau of Reclamation, Technical Service Center, Denver, CO, December 2010.

[3] "McGee Creek Dam - Final Construction Geology Report," Bureau of Reclamation, Farris, OK, July 1998.

[4] "McGee Creek Dam - Construction Considerations," Bureau of Reclamation, Denver, CO 1983.

[5] "McGee Creek Dam - Final Construction Report," Bureau of Reclamation, Oklahoma-Texas Area Office, Oklahoma City, OK, October 2000.

[6] "McGee Creek Dam - Comprehensive Facility Review," Bureau of Reclamation, Technical Service Center, Denver, CO, June, 2010.

[7] "Jordanelle Dam - Foundation Construction Geology Report, Stage 1," Bureau of Reclamation, Boneville Construction Office, Provo, Utah, March, 1988.

[8] "Jordanelle Dam - Final Grouting Report (L-10); Stages 1 and 2 - Foundation and Outlet Works," Volume 1 of 2, Bureau of Reclamation, Provo Area Office, Provo, UT, June 1997.

[9] "Jordanelle Dam - Comprehensive Review," Bureau of Reclamation, Technical Service Center, Denver, CO, January 2013.

[10] "Heron Dam - Comprehensive Facility Review," Bureau of Reclamation, Technical Service Center, Denver, CO, September 2007.

[11] "Analysis of Utilization of Grout and Grout Curtains - Heron Dam," Prepared by Claude A. Fetzer for Bureau of Reclamation under Contract No. 2-07-DV-00148, Denver, CO, February 1986.

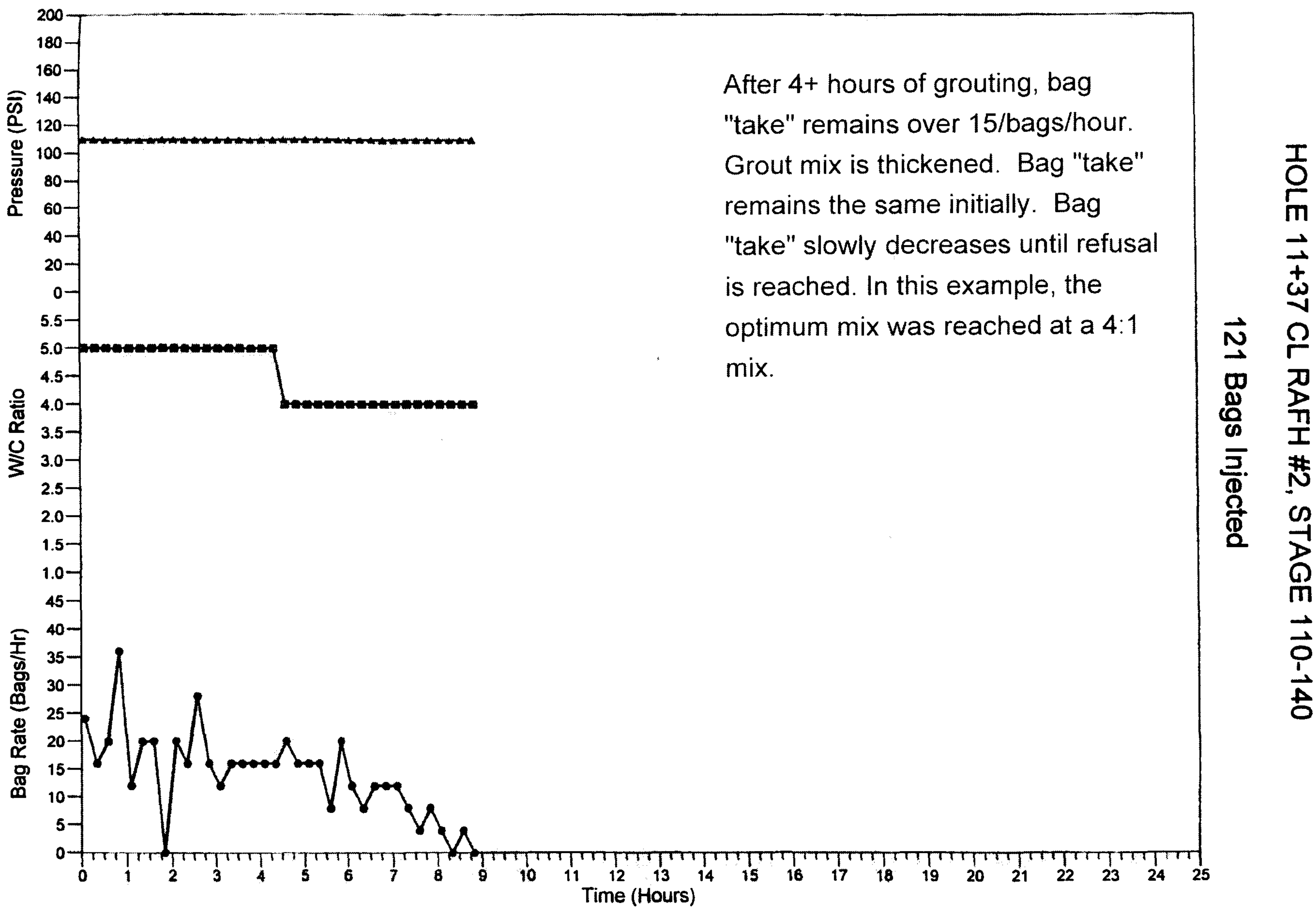
[12] "New Waddell - Comprehensive Facility Review," Bureau of Reclamation, Technical Service Center, Denver, CO, July 2011.

[13] “New Waddell – Comprehensive Facility Review,” Bureau of Reclamation, Technical Service Center, Denver, CO, July 2011.

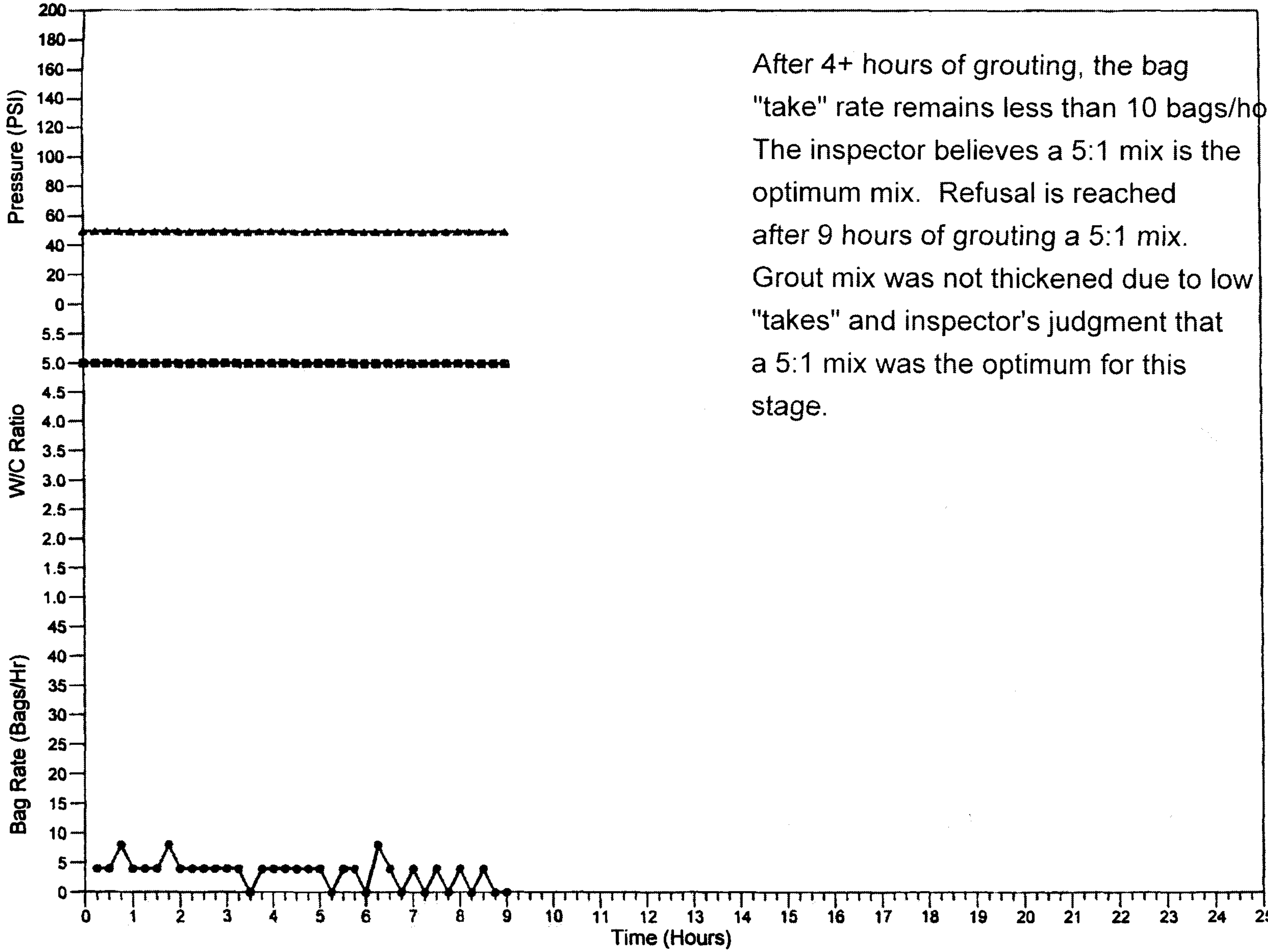
Appendix B

Grouting Mix Adjustment Examples

GROUTING PERFORMANCE GRAPHS



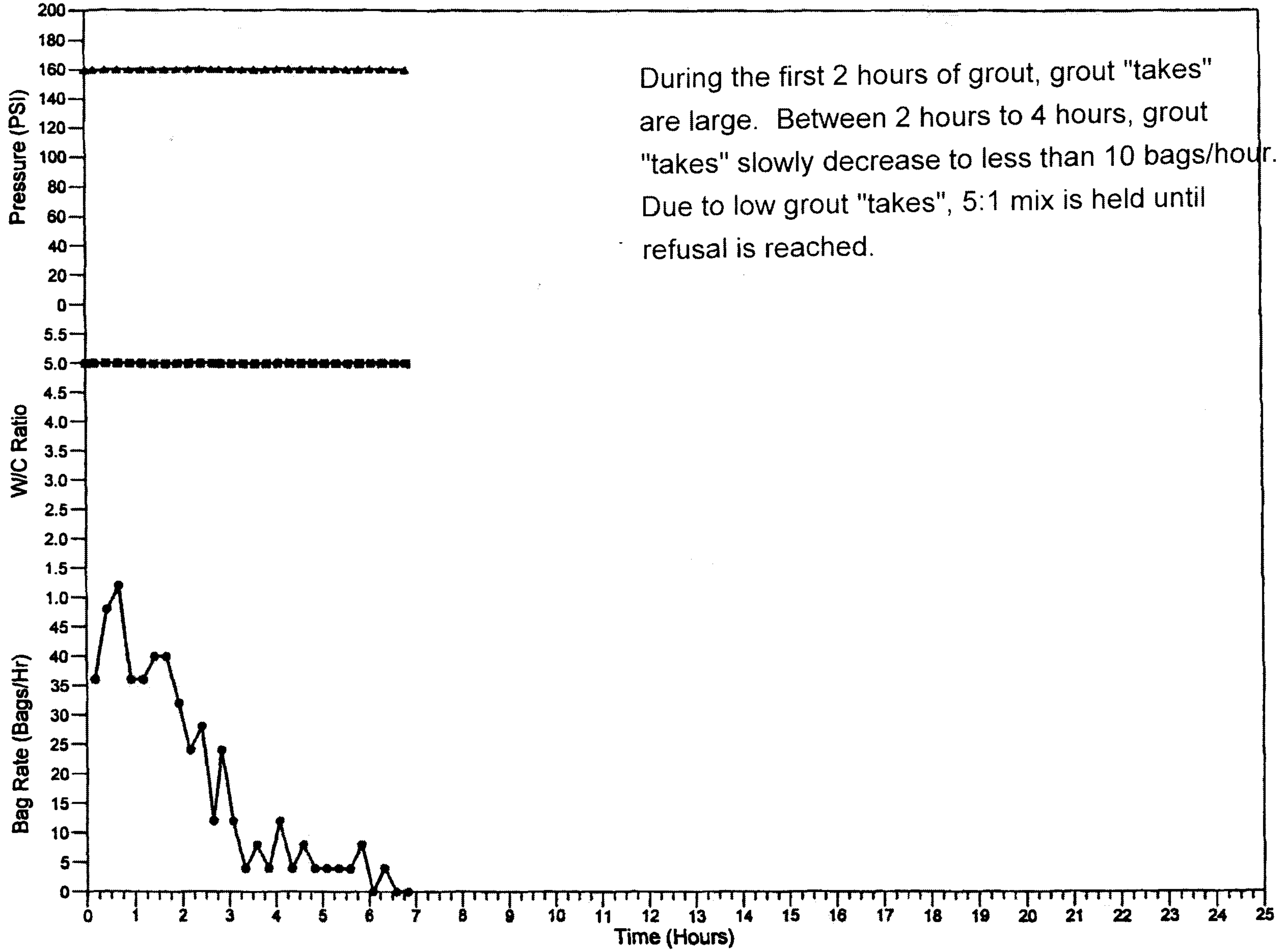
GROUTING PERFORMANCE GRAPHS



HOLE 25+86 U37, STAGE 50-70

30 Bags Injected

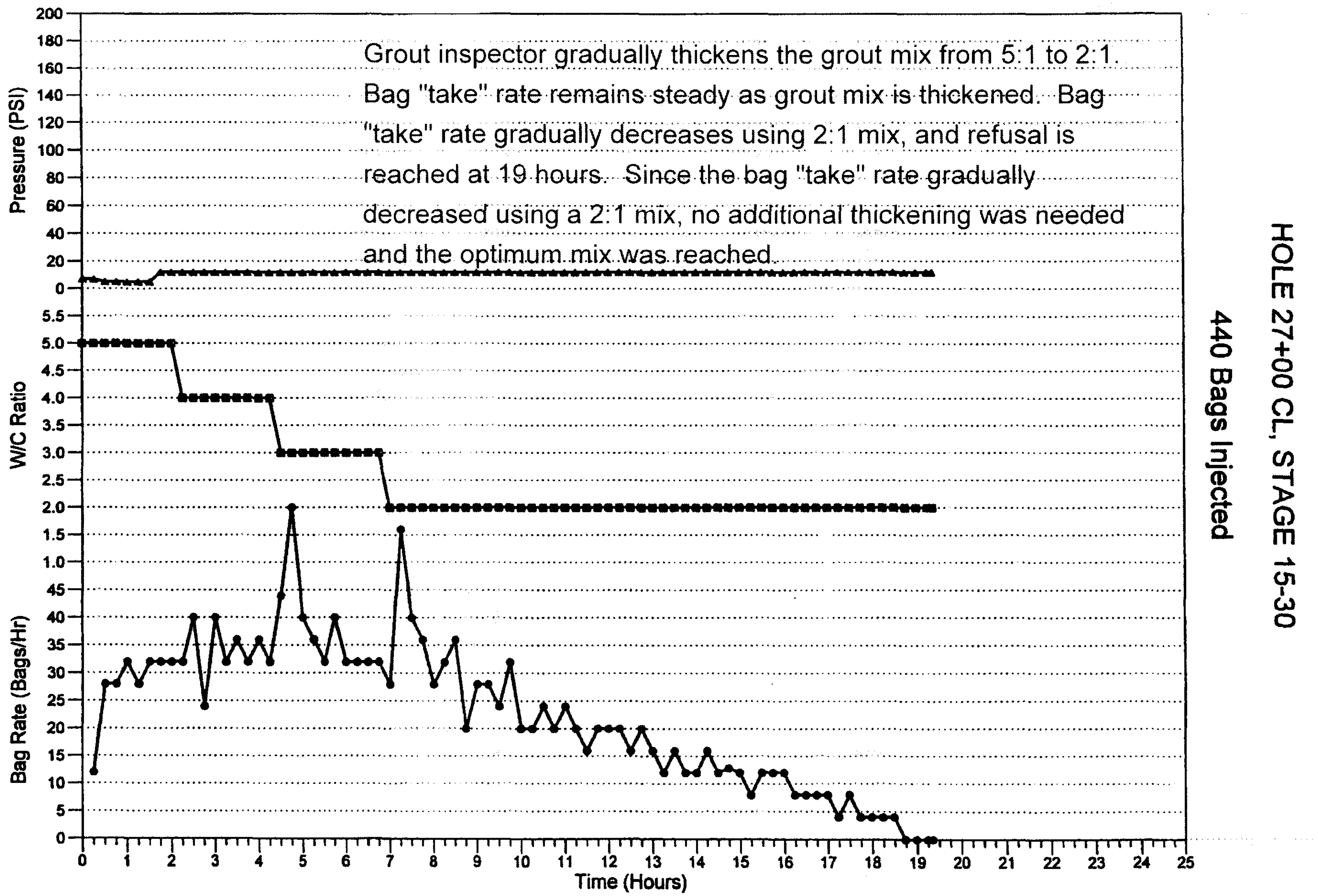
GROUTING PERFORMANCE GRAPHS



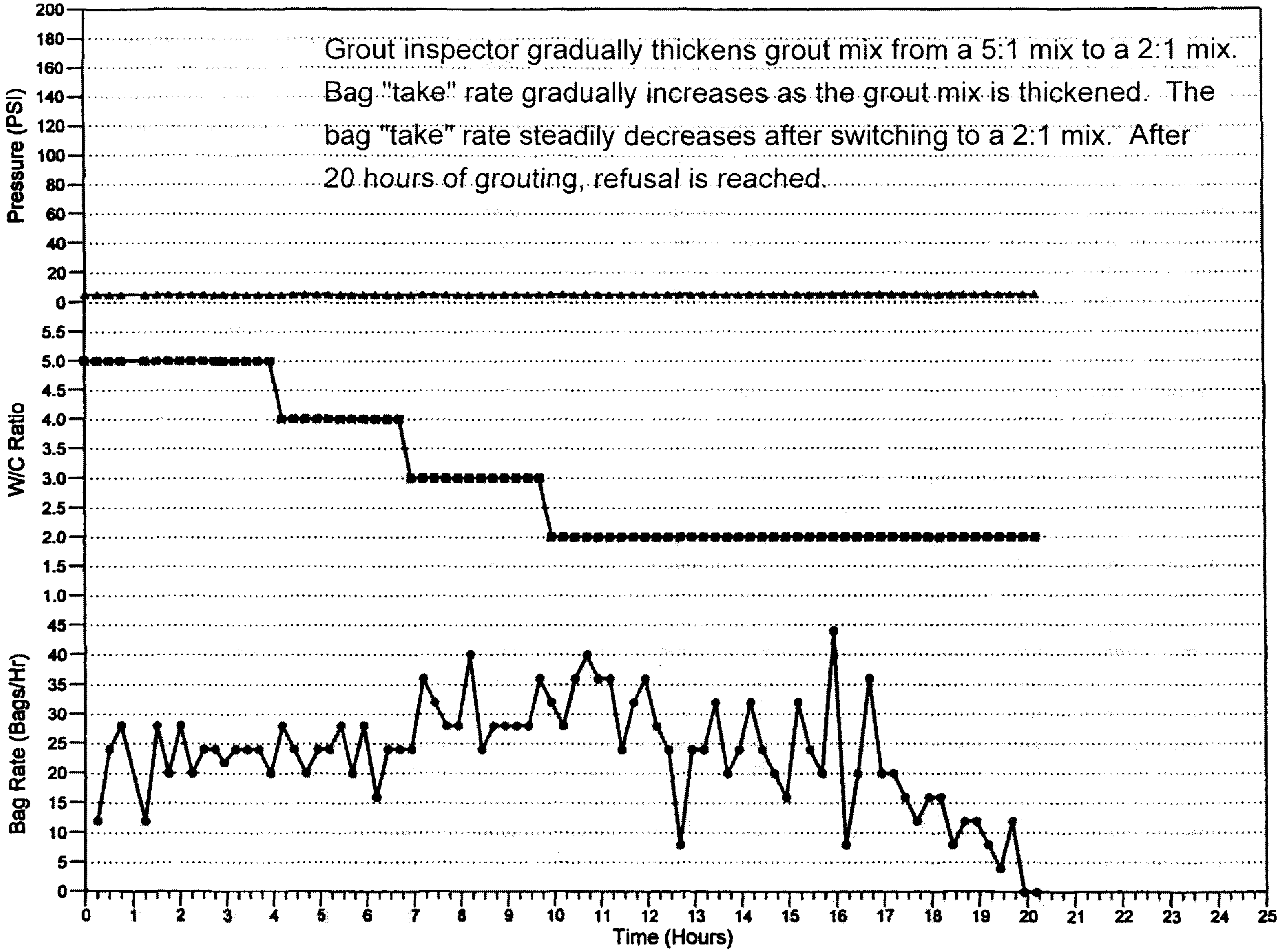
HOLE 25+60 U42, STAGE 160-210

117 Bags Injected

GROUTING PERFORMANCE GRAPHS



GROUTING PERFORMANCE GRAPHS

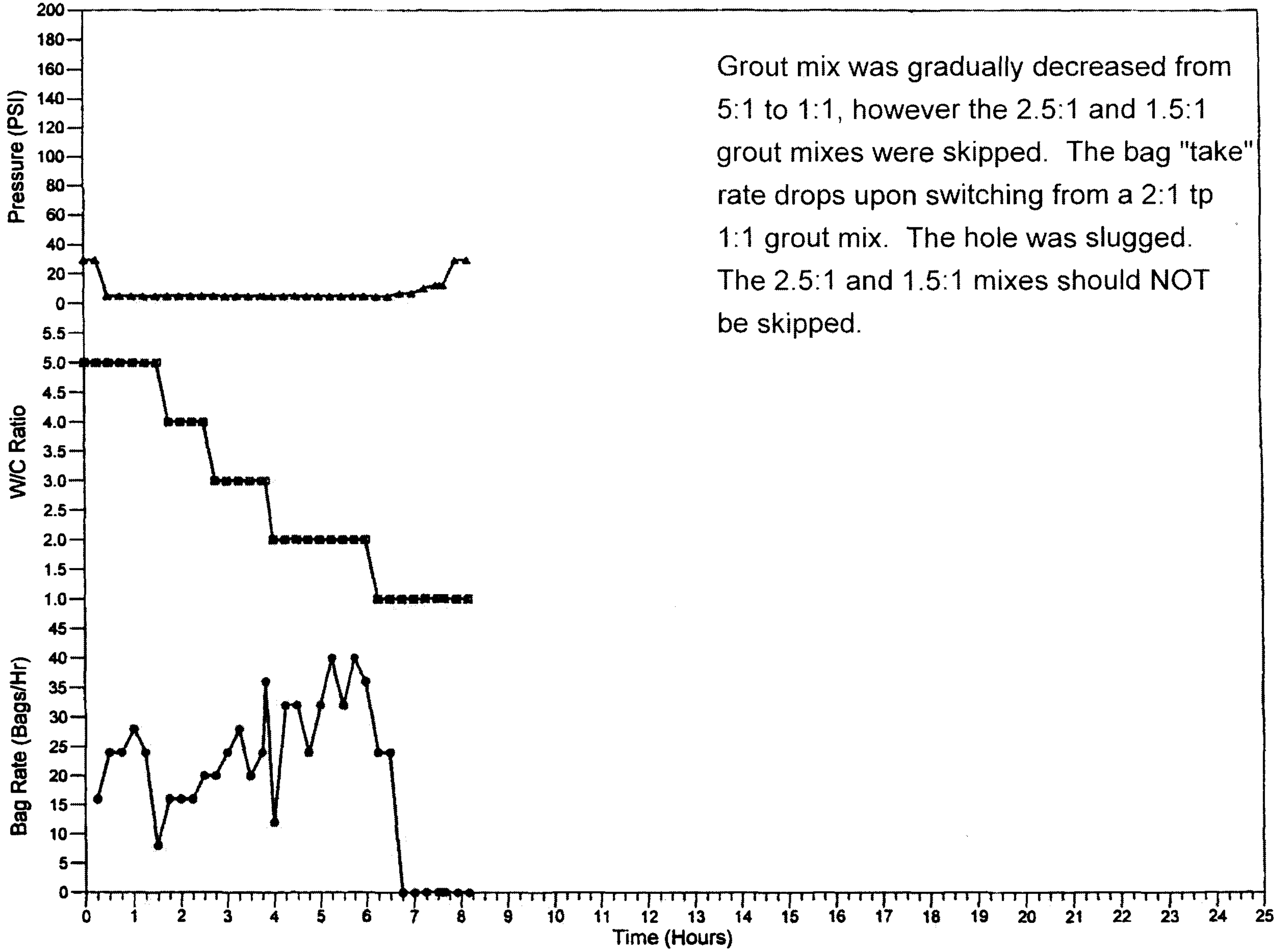


HOLE 27+20 CL, STAGE 5-11

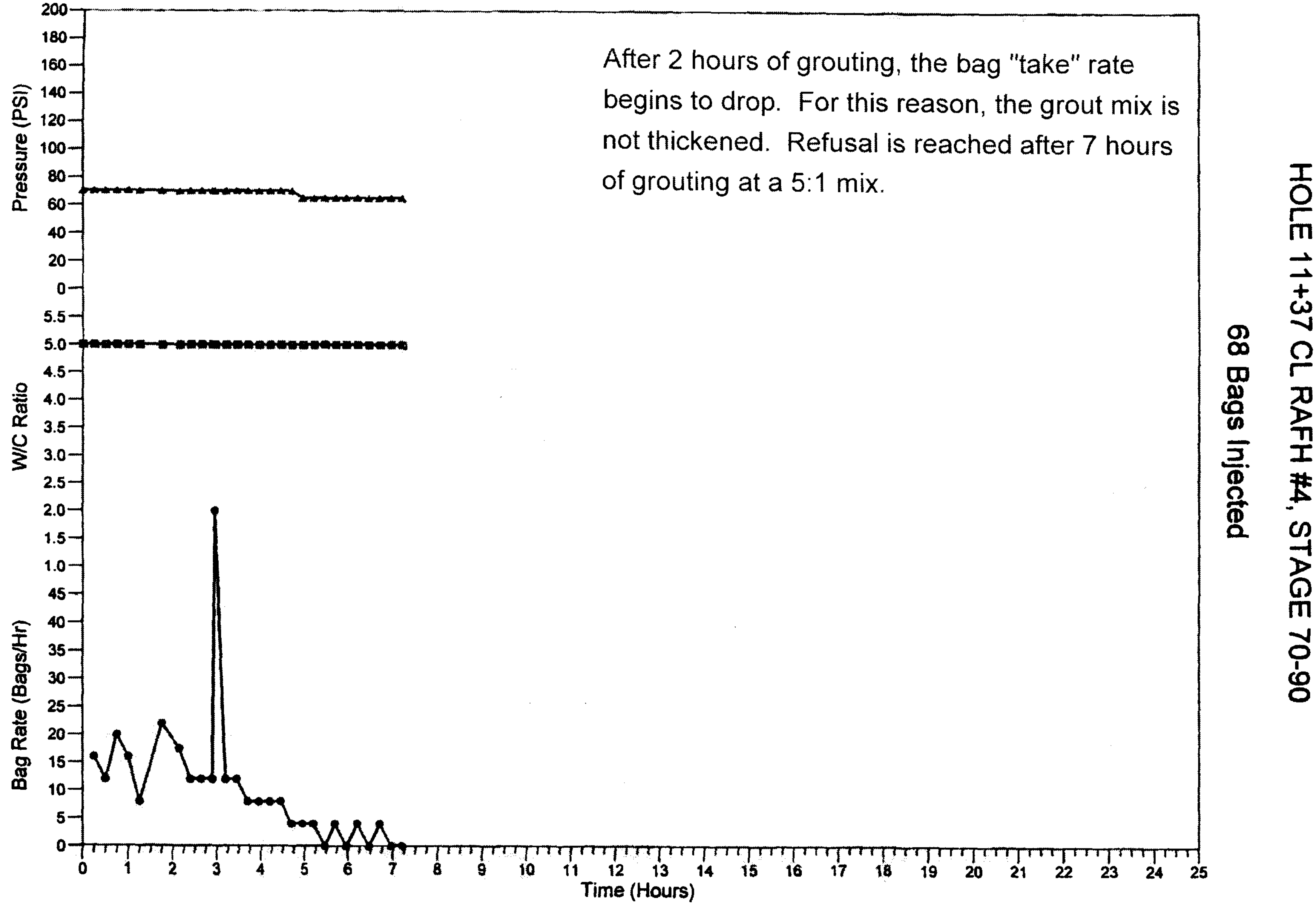
469 Bags Injected

HOLE 27+80 U01, STAGE 30-45

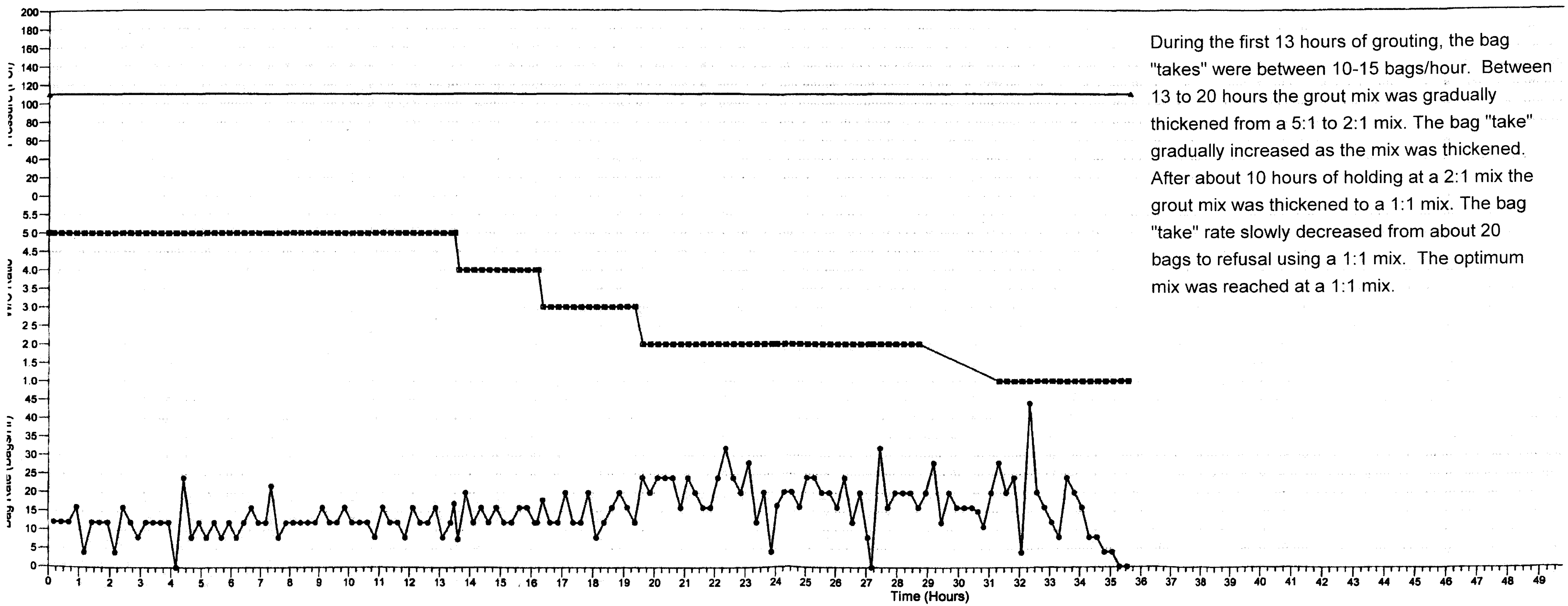
161 Bags Injected



GROUTING PERFORMANCE GRAPHS



GROUTING PERFORMANCE GRAPHS HOLE 27+80 U01, STAGE 110-140 533 Bags Injected



During the first 13 hours of grouting, the bag "takes" were between 10-15 bags/hour. Between 13 to 20 hours the grout mix was gradually thickened from a 5:1 to 2:1 mix. The bag "take" gradually increased as the mix was thickened. After about 10 hours of holding at a 2:1 mix the grout mix was thickened to a 1:1 mix. The bag "take" rate slowly decreased from about 20 bags to refusal using a 1:1 mix. The optimum mix was reached at a 1:1 mix.

Appendix C

Grout-Takes and Plan and Profile Drawings

GROUTING PROGRAM REPORT

PROJECT: Animas La Plata Project
FEATURE: Ridges Basin Dam
SPECIFICATION: 04-NA-40-8064

DATE: 12/17/2007

Blanket Total

GROUTED FROM 4/12/2005 TO 6/7/2006

	DRILLING				WATER TESTING			GROUTING								
	DRILLING (Linear FT)	TOTAL HOURS DRILLING	RATE (Linear FT/HR)	TOTAL HOURS REDRILL	AVE TAKE (FT^3 in 5 min)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	TAKE BAGS	TAKE BAGS/FT	TOTAL HRS GROUTING	AVE HRS GROUTING (Per Stage)	RATE (Bags/HR)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	AVE MIX RATIO W:C
CATAGORY:															
Primary Holes	5698	335.42	16.99	5.90	2.24	7	3	6494	1.14	609.45	4.23	10.66	7	2	3.7
Secondary Holes	4677	257.48	18.16	1.92	1.83	7	3	3967	0.85	433.47	3.47	9.15	7	3	3.4
Tertiary Holes	5847	318.18	18.38	2.45	1.38	7	3	2902	0.50	386.02	3.06	7.52	6	3	3.3
Quaternary Holes	3360	181.47	18.52	0.00	0.84	7	4	1199	0.36	124.08	2.76	9.66	6	3	3.0
Quinary Holes	750	37.97	19.75	0.00	0.69	8	4	341	0.45	25.10	3.59	13.59	4	1	2.4
Senary Holes	60	3.07	19.57	0.00	1.95	9	3	17	0.28	2.47	2.47	6.89	3	0	2.8
Septenary Holes	60	3.72	16.14	0.00	1.58	9	4	7	0.12	1.50	1.50	4.67	2	0	3.4
TOTALS	**20452**	**1137.30**	**17.98**	**10.27**	**1.61**	**7**	**3**	**15030**	**0.73**	**1582.08**	**3.52**	**9.50**	**7**	**3**	**3.5**

GROUTING PROGRAM REPORT

PROJECT: Animas La Plata Project
FEATURE: Ridges Basin Dam
SPECIFICATION: 04-NA-40-8064

DATE: 12/17/2007

Curtain Grouting Total

GROUTED FROM 4/15/2005 TO 2/6/2007

	DRILLING				WATER TESTING			GROUTING								
	DRILLING (Linear FT)	TOTAL HOURS DRILLING	RATE (Linear FT/HR)	TOTAL HOURS REDRILL	AVE TAKE (FT^3 in 5 min)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	TAKE BAGS	TAKE BAGS/FT	TOTAL HRS GROUTING	AVE HRS GROUTING (Per Stage)	RATE (Bags/HR)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	AVE MIX RATIO W:C
CATAGORY:															
Primary Holes	6424	240.60	26.70	30.48	3.17	75	36	27075	4.21	1194.78	10.96	22.66	108	59	3.6
Secondary Holes	5765	217.85	26.46	2.12	2.17	76	44	17446	3.03	696.95	9.82	25.03	133	62	3.8
Tertiary Holes	10938	410.35	26.66	5.37	1.50	73	44	13791	1.26	806.05	5.84	17.11	112	58	4.1
Quaternary Holes	16395	598.02	27.42	5.98	0.84	69	43	9179	0.56	730.35	4.77	12.57	103	55	4.1
Quinary Holes	15883	578.27	27.47	3.73	0.63	70	45	6495	0.41	429.82	4.43	15.11	137	60	4.2
Senary Holes	8490	238.07	35.66	6.88	0.44	72	52	578	0.07	109.65	3.23	5.27	124	60	4.4
Septenary Holes	1380	16.63	82.97	0.00	0.38	52	32	27	0.02	10.58	1.32	2.55	88	44	5.0
TOTALS	**66193**	**2371.43**	**27.91**	**54.57**	**1.20**	**71**	**44**	**77179**	**1.17**	**4113.32**	**6.51**	**18.76**	**115**	**57**	**3.9**

GROUTING PROGRAM REPORT

PROJECT: Animas La Plata Project
FEATURE: Ridges Basin Dam
SPECIFICATION: 04-NA-40-8064

DATE: 12/17/2007

Grand Total

GROUTED FROM 4/6/2005 TO 2/6/2007

Page: 1

	DRILLING				WATER TESTING			GROUTING								
	DRILLING (Linear FT)	TOTAL HOURS DRILLING	RATE (Linear FT/HR)	TOTAL HOURS REDRILL	AVE TAKE (FT^3 in 5 min)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	TAKE BAGS	TAKE BAGS/FT	TOTAL HRS GROUTING	AVE HRS GROUTING (Per Stage)	RATE (Bags/HR)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	AVE MIX RATIO W:C
CATAGORY:															
Primary Holes	12582	584.13	21.54	36.60	2.71	33	15	33868	2.69	1854.75	6.92	18.26	70	23	3.6
Secondary Holes	10672	473.17	22.55	4.03	1.96	34	19	21518	2.02	1150.92	5.70	18.70	81	27	3.6
Tertiary Holes	16875	729.82	23.12	7.82	1.44	41	24	16709	0.99	1194.73	4.51	13.99	76	33	3.8
Quaternary Holes	19815	781.00	25.37	5.98	0.84	53	33	10382	0.52	856.38	4.28	12.12	88	45	4.0
Quinary Holes	16693	617.07	27.05	3.73	0.63	65	42	6836	0.41	454.92	4.37	15.03	130	57	4.1
Senary Holes	8550	241.13	35.46	6.88	0.46	72	51	595	0.07	112.12	3.20	5.31	121	59	4.4
Septenary Holes	1440	20.35	70.76	0.00	0.43	50	30	34	0.02	12.08	1.34	2.81	77	39	4.8
TOTALS	**87545**	**3518.32**	**24.88**	**65.05**	**1.37**	**48**	**29**	**92633**	**1.06**	**5771.03**	**5.22**	**16.05**	**83**	**34**	**3.8**

GROUTING PROGRAM REPORT

PROJECT: Animas La Plata Project
FEATURE: Ridges Basin Dam
SPECIFICATION: 04-NA-40-8064

DATE: 12/17/2007

Right Abutment Curtain Total

GROUTED FROM 9/8/2005 TO 9/13/2006

	DRILLING				WATER TESTING			GROUTING							
	DRILLING (Linear FT)	TOTAL HOURS DRILLING	RATE (Linear FT/HR)	TOTAL HOURS REDRILL	AVE TAKE (FT^3 in 5 min)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	TAKE BAGS	TAKE BAGS/FT	TOTAL HRS GROUTING	AVE HRS GROUTING (Per Stage)	RATE (Bags/HR)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	AVE MIX RATIO W:C
CATAGORY:															
Primary Holes	2280	112.82	20.21	26.42	4.14	86	43	16841	7.39	525.07	14.59	32.07	113	71	3.0
Secondary Holes	2020	113.70	17.77	1.17	2.22	84	55	11694	5.79	336.12	16.81	34.79	140	77	3.4
Tertiary Holes	4153	231.93	17.91	1.17	1.56	78	53	9441	2.27	370.37	8.05	25.49	126	67	3.7
Quaternary Holes	5540	258.67	21.42	1.92	0.70	79	52	3578	0.65	215.08	5.38	16.64	123	67	4.1
Quinary Holes	4526	239.93	18.86	1.70	0.53	74	51	3340	0.74	105.08	6.57	31.78	169	90	3.8
Senary Holes	2400	35.58	67.45	6.88	0.32	85	66	44	0.02	8.45	1.69	5.21	149	92	5.0
Septenary Holes	380	6.17	61.62	0.00	0.68	62	27	13	0.03	5.17	1.29	2.52	96	58	5.0
TOTALS	**21299**	**998.80**	**21.32**	**39.25**	**1.31**	**79**	**52**	**45040**	**2.11**	**1567.50**	**9.33**	**28.73**	**128**	**72**	**3.5**

GROUTING PROGRAM REPORT

PROJECT: Animas La Plata Project
FEATURE: Ridges Basin Dam
SPECIFICATION: 04-NA-40-8064

DATE: 12/18/2007

Tunnel Total

GROUTED FROM 7/26/2006 TO 1/25/2007

	DRILLING				WATER TESTING			GROUTING								
	DRILLING (Linear FT)	TOTAL HOURS DRILLING	RATE (Linear FT/HR)	TOTAL HOURS REDRILL	AVE TAKE (FT^3 in 5 min)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	TAKE BAGS	TAKE BAGS/FT	TOTAL HRS GROUTING	AVE HRS GROUTING (Per Stage)	RATE (Bags/HR)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	AVE MIX RATIO W:C
CATAGORY:															
Primary Holes	1802	163.10	11.05	0.00	4.38	86	35	2010	1.12	150.28	5.01	13.37	89	48	4.5
Secondary Holes	1100	96.53	11.40	0.00	3.29	91	46	435	0.40	63.07	3.00	6.90	94	47	4.9
Tertiary Holes	2587	224.58	11.52	0.00	2.13	83	41	744	0.29	112.28	2.55	6.63	72	45	4.4
Quaternary Holes	125	12.75	9.80	0.00	0.93	35	20	35	0.28	4.80	2.40	7.29	38	30	3.8
Quinary Holes	0	0.00	0.00	0.00	0.00	0	0	0	0.00	0.00	0.00	0.00	0	0	0.0
Senary Holes	0	0.00	0.00	0.00	0.00	0	0	0	0.00	0.00	0.00	0.00	0	0	0.0
Septenary Holes	0	0.00	0.00	0.00	0.00	0	0	0	0.00	0.00	0.00	0.00	0	0	0.0
TOTALS	**6174**	**561.80**	**10.99**	**0.00**	**2.67**	**81**	**40**	**3246**	**0.53**	**339.27**	**3.29**	**9.57**	**82**	**46**	**4.5**

GROUTING PROGRAM REPORT

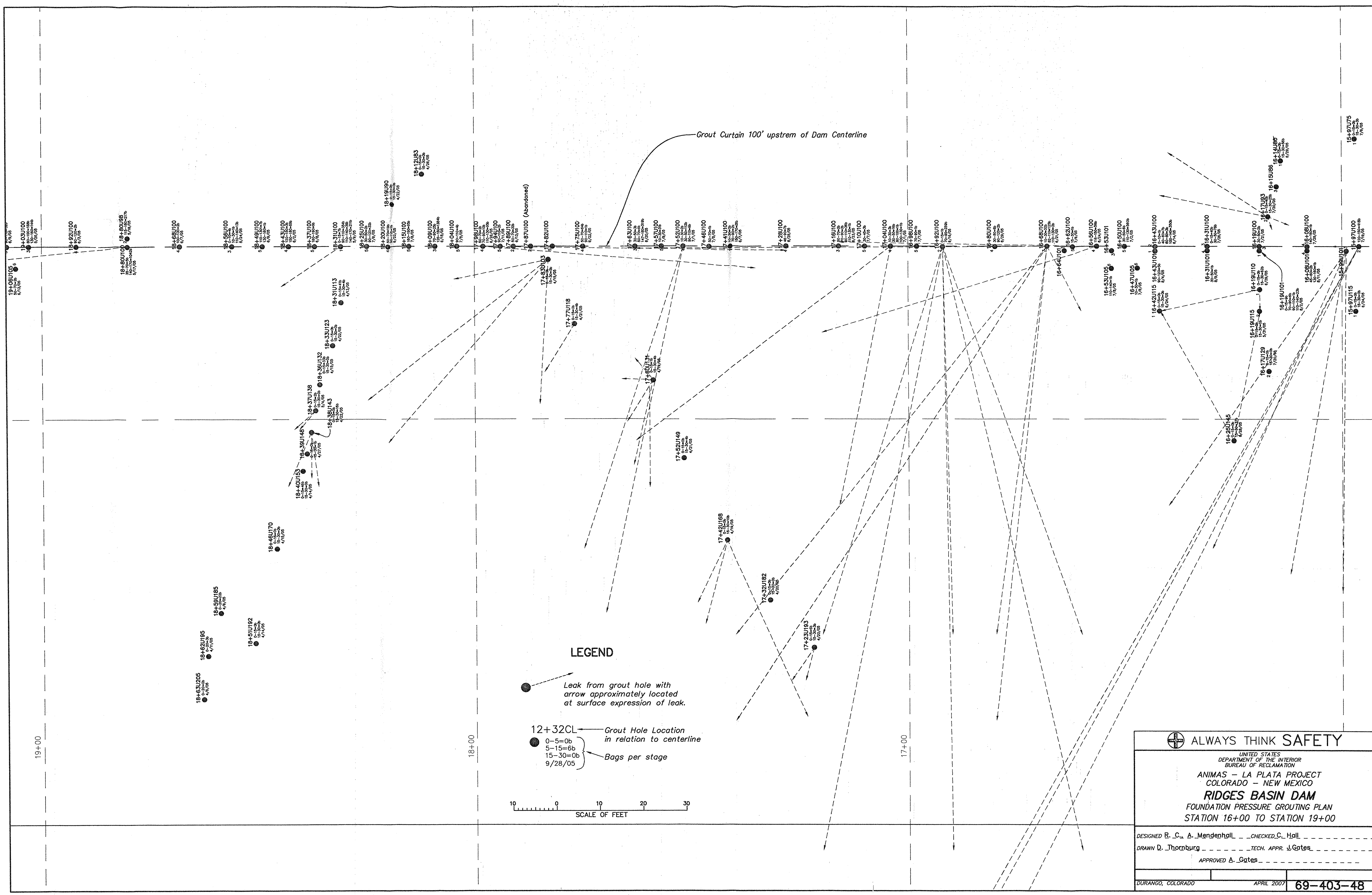
PROJECT: Animas La Plata Project
FEATURE: Ridges Basin Dam
SPECIFICATION: 04-NA-40-8064

DATE: 12/17/2007

Right Abutment Blanket Total

GROUTED FROM 6/27/2005 TO 6/7/2006

	DRILLING				WATER TESTING			GROUTING								
	DRILLING (Linear FT)	TOTAL HOURS DRILLING	RATE (Linear FT/HR)	TOTAL HOURS REDRILL	AVE TAKE (FT^3 in 5 min)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	TAKE BAGS	TAKE BAGS/FT	TOTAL HRS GROUTING	AVE HRS GROUTING (Per Stage)	RATE (Bags/HR)	AVE PUMP PRESSURE (PSI)	AVE BACK PRESSURE (PSI)	AVE MIX RATIO W:C
CATAGORY:															
Primary Holes	2490	212.78	11.70	0.65	2.69	7	3	2437	0.98	263.45	3.99	9.25	7	3	3.4
Secondary Holes	2067.5	154.17	13.41	1.67	1.98	7	3	1761	0.85	187.78	3.61	9.38	6	3	3.2
Tertiary Holes	2997.5	187.88	15.95	2.45	1.23	7	3	1188	0.40	181.67	2.98	6.54	6	3	3.2
Quaternary Holes	1560	98.92	15.77	0.00	1.04	8	4	403	0.26	61.52	2.80	6.55	5	3	3.1
Quinary Holes	300	9.75	30.77	0.00	0.87	8	3	171	0.57	16.58	3.32	10.31	4	1	2.3
Senary Holes	0	0.00	0.00	0.00	0.00	0	0	0	0.00	0.00	0.00	0.00	0	0	0.0
Septenary Holes	0	0.00	0.00	0.00	0.00	0	0	0	0.00	0.00	0.00	0.00	0	0	0.0
TOTALS	**9415**	**663.50**	**14.19**	**4.77**	**1.76**	**7**	**3**	**5960**	**0.63**	**711.00**	**3.45**	**8.38**	**6**	**3**	**3.2**



ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

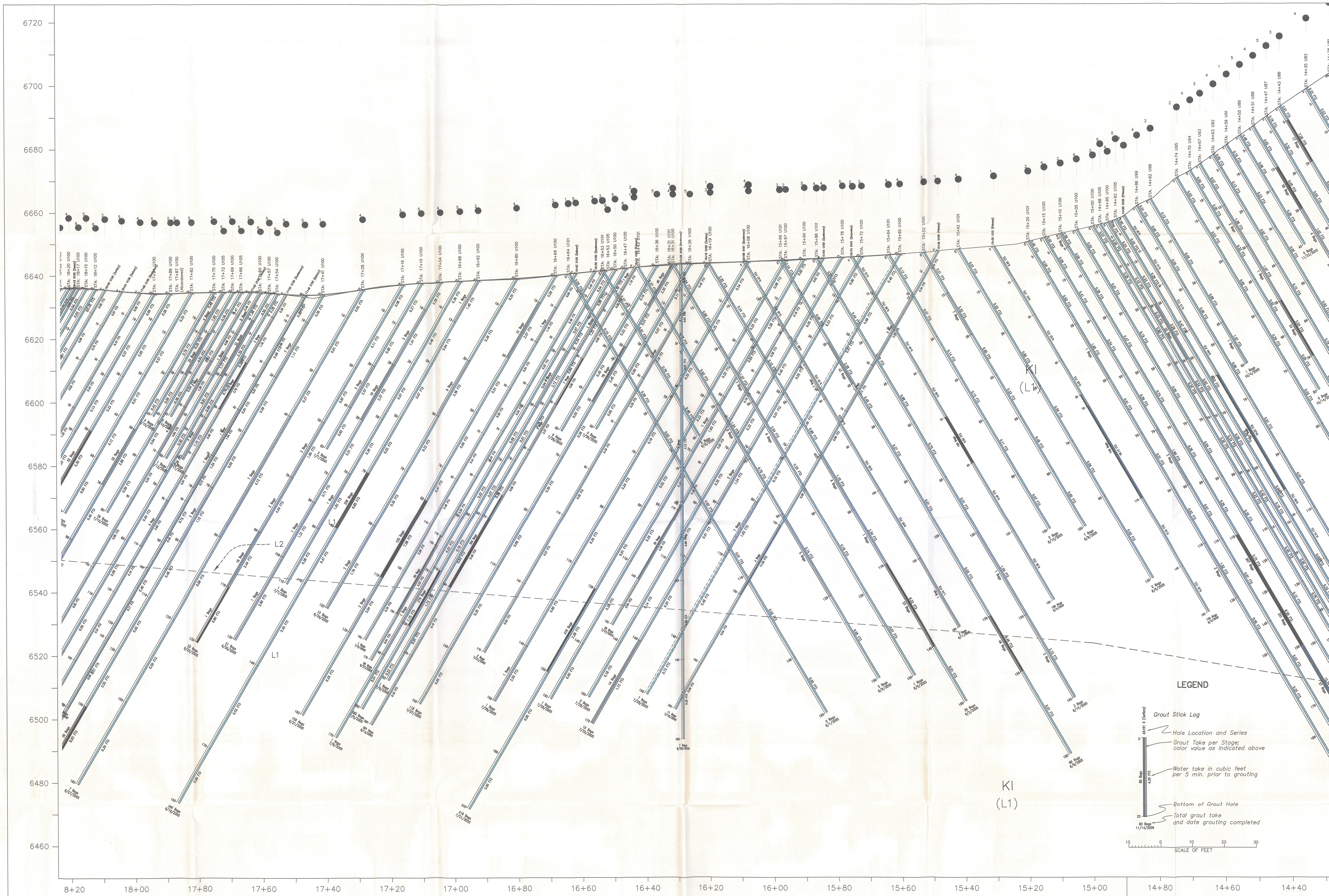
ANIMAS – LA PLATA PROJECT
COLORADO – NEW MEXICO

RIDGES BASIN DAM

FOUNDATION PRESSURE GROUTING PLAN
STATION 16+00 TO STATION 19+00

DESIGNED R. C., A. Mendenhall CHECKED C. Hall
DRAWN D. Thornburg TECH. APPR. J. Gates
APPROVED A. Gates

DURANGO, COLORADO APRIL 2007 69-403-48



ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

ANIMAS – LA PLATA PROJECT
COLORADO – NEW MEXICO

RIDGES BASIN DAM

GROUT CURTAIN PROFILE
STATION 15+20 TO STATION 17+28

GEOLOGY

TECH. APPR. J.G.
SUBMITTED
APPROVED J. Gates
MANAGER TECHNICAL SERVICES DIVISION

DESIGNED R. C., A. Mendenhall CHECKED C. Hall
DRAWN D. Thornburg TECH. APPR. J.Gates
APPROVED A. Gates

CADD SYSTEM AutoCAD	CADD FILENAME Unknown
DURANGO, COLORADO	APRIL 2007

69–403–45

Appendix D

Drilling and Grouting Data

DRILLING AND GROUTING DATA - EMBANKMENT DAMS

DAM				GENERAL DESCRIPTION		FOUNDATION	DRILLING OF HOLES							GROUTING OF HOLES							
Name	Location		Year Constructed	Type of Dam	Structural Height in Feet	Geology	Type of Grouting	number of rows of holes	number of holes drilled	spacing of holes in feet	maximum depth		total linear feet of holes	total cubic feet of grout			mix ratio w/c by volume	cf / lf of hole		% cost of total cost	remarks
	State	River									curtain	blanket		totals	cement	sand		grout	cement		
Lahontan	Nevada	Carson	1911-1915	Zoned Earthfill	162	sediments over clay, mudstone and cong.	curtain	2	83	3	50	-	2593	1174	1174	0	7:1 to 2:1	0.5	0.5	N/A	numerous faults
Tieton	Washington	Tieton	1923-1925	Zoned Earthfill	235	andesite over shale, blocky, tight joints	curtain	1	533	variable	15	-	7991	7738	3607	4131	4:1 to 0.8:1	1	0.5	N/A	
McKay	Oregon	McKay Creek	1923-1927	Diaphragm Earthfill	165	basalt, hackly, tight fanglomerate, permeable	curtain structures	3	340	2 to 10	32	-	11200	4601	4001	600	2.5:1 to 0.9:1	0.4	0.4	N/A	
Guernsey	Wyoming	North Platte	1925-1927	Diaphragm Earthfill	135	limestone, and sandstone dip 3 deg upstream	curtain structures	1	111	N/A	30	-	1100	1620	1620	0	3:1 to 1:1	1.5	1.5	N/A	
Echo	Utah	Weber	1927-1931	Zoned Earthfill	158	shale, sanstone, cong. Few joints	curtain structures	1	53	variable	30	-	446	1483	705	778	2:1 to 1:1	3.3	1.6	N/A	
Cle Elum	Washington	Cle Elum	1931-1933	Zoned Earthfill	160	mostly glacial detritus, some ss. And sh	tunnel lining	-	40	-	-	-	800	34372	20489	13883	1:1	42.9	25.6	N/A	mostly backfill around tunnels
Agency Valley	Oregon	North Fork Malheur	1934-1935	Zoned Earthfill	110	shale basalt, volcanic tuff	curtain structures	1	64	15	50	-	1578	8072	5493	2579	4:1 to 0.75:1	5.1	3.5	N/A	
Hyrum	Utah	Little Bear	1934-1935	Homogenous Earthfill	116	lacustrine clay, sand and gravel lenses	curtain structures	1	126	5	30	-	1796	7787	7787	0	1:1	4.3	4.3	N/A	
Pine View	Utah	Ogden	1934-1937	Homogenous Earthfill	132	limestone - thin bedded, fractured	tunnel	-	37	1 to 6	50	-	370	10108	10108	0	3:1 to 0.5:1	27.3	27.3	N/A	mostly placed in drains and well points
Rye Patch	Nevada	Humbolt	1935-1936	Homogenous Earthfill	75	unconsolidated clayey sands	tunnel lining	-	64	-	10	-	146	1001	1001	0	2:1 to 0.75:1	6.9	6.9	N/A	
Taylor Park	Colorado	Taylor	1935-1937	Zoned Earthfill	206	jointed schist, quartzite, dolomite	curtain structures	2	N/A	5	50	-	12400	26300	26300	0	6:1 to 0.6:1	2.1	2.1	N/A	
Moon Lake	Utah	West Fork, Lake Fork	1935-1938	Zoned Earthfill	101	shale and clayey gravel with boulders	curtain structures	1	116	variable	30	-	1600	8000	8000	0	5:1 to 0.75:1	5	5	N/A	
Alcova	Wyoming	North Platte	1935-1938	Zoned Earthfill	265	limestone= few cavities, jointed ss. Dip 13 deg d.s.	blanket	N/A	727	20	-	75	45306	210403	96441	113962	4:1 to 0.8:1	4.6	2.5	31	hot water flows in drains and well points
Alcova	Wyoming	North Platte	1935-1938	Zoned Earthfill	265	limestone= few cavities, jointed ss. Dip 13 deg d.s.	curtain	N/A	304	20	219	-	21567	26394	17337	8957	4:1 to 0.8:1	1.2	0.8	N/A	
Alcova	Wyoming	North Platte	1935-1938	Zoned Earthfill	265	limestone= few cavities, jointed ss. Dip 13 deg d.s.	tunnels and galleries	N/A	229	variable	-	-	2914	2337	2337	0	4:1 to 0.8:1	0.8	0.8	N/A	
Alcova	Wyoming	North Platte	1935-1938	Zoned Earthfill	265	limestone= few cavities, jointed ss. Dip 13 deg d.s.	spillway	N/A	69	variable	131	-	7250	4488	2100	2388	4:1 to 0.8:1	0.6	0.3	N/A	
Island Park	Idaho	Henry's Fork	1935-1938	Zoned Earthfill	91	rhyolite, tuff, jointed, weathered	blanket	-	659	10 to 20		50	11908	24384	23442	962	2:1 to 0.6:1	2.1	2	N/A	many springs in foundation
Island Park	Idaho	Henry's Fork	1935-1938	Zoned Earthfill	91	rhyolite, tuff, jointed, weathered	curtain	1	300	10 to 20	50	-	10452	37043	36165	878	2:1 to 0.6:1	3.5	3.5	N/A	
Island Park	Idaho	Henry's Fork	1935-1938	Zoned Earthfill	91	rhyolite, tuff, jointed, weathered	spillway	-	104	10 to 20	50	-	1728	9946	4306	5640	2:1 to 0.6:1	5.8	2.4	N/A	
Alamogordo	New Mexico	Pecos	1936-1937	Zoned Earthfill	164	jointed ss. And cong., shale	curtain structures	2	241	10	100	-	26446	70561	70561	0	4:1 to 0.75:1	2.7	2.7	14	
Unity	Oregon	Burnt	1936-1938	Zoned Earthfill	83	tuff-breccia and basalt - jointed	curtain structures	1	N/A	5	50	-	2819	6084	6084	0	N/A	2.2	2.2	5.5	
Caballo	New Mexico	Rio Grande	1936-1938	Zoned Earthfill	96	siltstone, claystone, ss, cong., and alluvium	curtain structures	N/A	155	10	50	-	3949	24342	25342	0	0.75:1	6.4	6.4	N/A	
Fresno	Montana	Milk	1937-1939	Homogenous Earthfill	111	interbedded ss and sh	tunnel and elsewhere	-	N/A	-	-	-	253	9700	5456	4244	N/A	38.3	21.6	N/A	
Grassy Lake	Wyoming	Grassy Creek	1937-1939	Zoned Earthfill	118	rhyolite, lake sediments and glacial deposits	curtain and blanket	N/A	238	N/A	50	N/A	5424	5567	5567	0	5:1 to 0.6:1	1	1	N/A	
Boca	Nevada	Little Truckee	1937-1939	Zoned Earthfill	116	tuff and tuff breccia, fractured basalt flows	curtain	2	N/A	10	100	-	643	1191	1191	0	N/A	1.9	1.9	4	
Deer Creek	Utah	Provo	1938-1941	Zoned Earthfill	235	limestone, and quartzite fracture and jointed	blanket	-	94	-	-	75	3015	4113	4113	0	5:1 to 0.75:1	1.4	1.4	N/A	
Deer Creek	Utah	Provo	1938-1941	Zoned Earthfill	235	limestone, and quartzite fracture and jointed	curtain	1	310	N/A	75	-	15812	48340	48340	0	5:1 to 0.75:1	3.1	3.4	N/A	
Vallecito	Colorado	Pine	1938-1941	Zoned Earthfill	162	glacial till over horizontal ss,sh and mudstone	curtain structures	2	260	10	50	-	6692	4471	4471	0	2.3:1 to 0.67:1	0.7	0.7	N/A	
Green Mountain	Colorado	Blue	1938-1943	Zoned Earthfill	309	faulted sh, ls, ss, up injected dikes and sills	curtain structures	4	413	N/A	150	-	15706	33754	33754	0	5:1 to 1:1	2.2	2.2	N/A	
Wickiup	Oregon	Deschutes	1939-1948	Zoned Earthfill	100	lava and glacial deposits	outlet structure	-	N/A	N/A	N/A	-	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	only info available
Anderson Ranch	Montana	South Fork Boise	1941-1950	Zoned Earthfill	456	granite-jointed many shear zones	curtain structures	2	980	10	150	30	46028	55080	55080	0	5:1 to 0.5:1	1.2	1.2	N/A	
Big Sandy	Wyoming	Big Sandy Creek	1941-1952	Zoned Earthfill	85	shale, sandstone, platy siltstone	curtain	1	N/A	10	60	-	9100	27784	27784	0	N/A	3.1	3.1	8	
Deerfield	South Dakota	Castle Creek	1942-1946	Zoned Earthfill	133	schist	curtain	2	N/A	5	60	-	6904	N/A	6660	N/A	N/A	N/A	1	N/A	no info available
Jackson Gulch	Colorado	Offstream	1942-1949	Zoned Earthfill	180	shale and sandstone surface joints	curtain structures	1	196	10	100	-	5090	6850	6850	0	3:1 to 0.6:1	1.2	1.2	N/A	right abutment grouting
Davis	Arizona	Colorado	1942-1950	Zoned Earthfill	200	granite gneiss-jointed, facutred and weathered	curtain structures	1	N/A	10	350	-	367823	244015	244015	0	N/A	0.6	0.6	5	major portion under spillway
Scofield	Utah	Price	1943-1946	Zoned Earthfill	125	interbedded shale and sandstone	curtain	-	N/A	10	50	-	N/A	105138	N/A	N/A	N/A	N/A	N/A	N/A	only info available
Dixon Canyon	Colorado	Offstream	1946-1946	Zoned Earthfill	240	sandstone, limestone, shale, dip 30 deg east	curtain structures	1	138	5 to 10	160	-	10106	21703	21703	0	5:01	2.2	2.2	N/A	

DRILLING AND GROUTING DATA - EMBANKMENT DAMS

DAM				GENERAL DESCRIPTION		FOUNDATION	DRILLING OF HOLES							GROUTING OF HOLES							
Name	Location		Year Constructed	Type of Dam	Structural Height in Feet	Geology	Type of Grouting	number of rows of holes	number of holes drilled	spacing of holes in feet	maximum depth		total linear feet of holes	total cubic feet of grout			mix ratio w/c by volume	cf / lf of hole		% cost of total cost	remarks
	State	River									curtain	blanket		totals	cement	sand		grout	cement		
Cascade	Idaho	North Fork Payette	1946-1948	Zoned Earthfill	107	granite-jointed and sheeted	curtain, blanket	N/A	N/A	10	100	20	N/A	N/A	N/A	N/A	4:1 to 2:1	N/A	N/A	N/A	only info available
Long Lake	Washington	Offstream	1946-1948	Zoned Earthfill	130	basalt with interflow zones	curtain structures	1	N/A	10	110	-	19689	37400	37400	0	N/A	1.9	1.9	14	
Horsetooth	Colorado	Offstream	1946-1949	Zoned Earthfill	155	solution channels in ls. Sh. Ss, 38 degree dip to east	curtain structures	1	197	5 to 10	200	-	12869	54270	54270	0	N/A	4.2	4.2	N/A	
Soldier Canyon	Colorado	Offstream	1946-1949	Zoned Earthfill	226	sandstone limestone, shale-dip 30 deg east	curtain structures	1	176	5 to 10	150	-	10779	5784	5784	0	5:01	0.5	0.5	N/A	
Spring Canyon	Colorado	Offstream	1946-1949	Zoned Earthfill	220	sandstone shale, limestone - dip 30 deg east	curtain structures	1	153	5 to 10	160	-	10377	25726	25726	0	N/A	2.5	2.5	N/A	
Dry Falls	Washington	Offstream	1946-1949	Zoned Earthfill	123	basalt with interflow zones, jointed	curtain structures	1	N/A	5 to 10	110	-	41961	28992	28992	0	5:1 to 0.6:1	0.7	0.7	13.4	
O'Sullivan	Washington	Lower Crab Creek	1946-1949	Zoned Earthfill	200	basaltic lava flows and interflow zones	curtain structures	1	N/A	10	150	-	77780	261789	261789	0	5:1 to 0.5:1	3.4	3.4	10.2	
Granby	Colorado	Colorado	1946-1950	Zoned Earthfill	298	fractured granite schist and gneiss	curtain	2	N/A	10	160	-	32635	61106	61106	0	N/A	1.9	1.9	N/A	does not include dikes 1 and 2
Olympus	Colorado	Big Thompson	1947-1949	Zoned Earthfill	70	granite, gneiss and schist all sheared and broken	curtain structures	1	N/A	10	100	-	12035	9133	9133	0	N/A	0.8	0.8	N/A	
Boysen	Wyoming	Wind	1947-1952	Zoned Earthfill	220	jointed granite gneiss, ss. And sh. Faulted	curtain-structure	1	N/A	5 to 10	110	-	22430	41373	41373	0	N/A	1.8	1.8	1.5	
North Dam	Washington	offstream	1949-1951	Zoned Earthfill	145	granite, basalt, sh. Interflow tuffs	curtain-structure	1	N/A	10	210	-	9046	10828	10828	0	N/A	1.2	1.2	N/A	
Platoro	Colorado	Conejos	1949-1951	Zoned Earthfill	165	fractured quartz latite, few faults, rockslides	curtain	1	N/A	10	160	-	12532	2358	2358	0	N/A	0.2	0.2	6.6	sediments are disturbed
Keyhole	Wyoming	Belle Fourche	1950-1952	Zoned Earthfill	168	sandstone and shaly ss., widely spaced jointes	curtain	1	N/A	10	160	-	9953	4833	4833	0	10:1 to 1:1	0.5	0.5	2	
Carter Lake	Colorado	offstream	1950-1952	Zoned Earthfill	190	sandstone, shale, limestone, 15 deg dip downstream	curtain-structure	1	N/A	10	110	-	16489	25960	25960	0	5:1 to 1:1	1.6	1.6	8.5	
Soda Lake Dike	Washington	offstream	1950-1952	Zoned Earthfill	57	basalt-columnar jointing	curtain	1	N/A	10	60	-	8089	1549	1549	0	N/A	0.2	0.2	8.4	
Cachuma	California	Santa Ynez	1950-1953	Zoned Earthfill	279	siltstone and shale tuffaceous to siliceous	curtain-structure	1	N/A	10	160	-	36530	20787	20787	0	N/A	0.6	0.6	3	intensely sheared and fractured
Rattlesnake	Colorado	rattlesnake creek	1951-1952	Zoned Earthfill	130	shale, sandstone, soft biotite schist	curtain	1	N/A	10	110	-	4791	265	265	0	7:1 to 1:1	0.1	0.1	8.3	
Glen Anne	California	offstream	1951-1953	Zoned Earthfill	152	massive ss. With sh. And cong.	outlet tunnel	-	N/A	-	20	-	1885	120	120	0	N/A	0.06	0.06	0.8	joints and bedding tight
Willow Creek	Colorado	Willow Creek	1951-1953	Zoned Earthfill	127	severely fractured basalt with siltstone, ash, ss.	curtain-structure	1	N/A	10	110	-	15832	19071	19071	0	N/A	1.2	1.2	15.7	
Jamestown	North Dakota	James	1952-1953	Zoned Earthfill	110	shale (jointed claystone)	curtain	1	70	10	60	-	3396	8197	8197	0	5:1 to 0.5:1	2.4	2.4	3	
Kirwin	Kansas	North Fork Solomon	1952-1955	Zoned Earthfill	169	fractured chalk, sand, silt, clay, shale	curtain-structure	1	N/A	10	110	-	4970	N/A	19883	N/A	N/A	N/A	4	N/A	only information available
Tiber	Montana	Marias	1952-1956	Zoned Earthfill	205	sandstone, shale with bentonite and gypsum seams	curtain-structure	1	231	10	160	-	18390	24669	24669	0	N/A	1.3	1.3	1.1	gypsum partially dissolved
Pactola	South Dakota	Rapid Creek	1952-1956	Zoned Earthfill	230	amphibolite, schist, slate, all jointed	blanket, curtain-structure	1	725	5 to 10	250	20	64531	50053	50053	0	8:1	0.8	0.8	8.4	
Webster	Kansas	South Fork Solomon	1952-1956	Zoned Earthfill	154	massive chalky limestone, shale	curtain-structure	1	219	10	125	-	11258	5574	5574	0	5:1 to 1:1	0.5	0.5	N/A	
Sly Park	California	Sly Park Creek	1953-1955	Zoned Earthfill	190	quartzite and phyllite, some fracturing	curtain	2	N/A	10	150	-	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	only information available
Wanship	Utah	Weber	1954-1957	Zoned Earthfill	175	sandstone jointed shale	curtain-structure	1	271	10	110	-	14133	36235	36235	0	5:1 to 0.75:1	2.6	2.6	4.6	
Pineview Enlargement	Utah	Ogden	1955-1957	Zoned Earthfill	132	limestone, quartzite	curtain- left abutment	1	15	10	110	-	1201	643	643	0	N/A	0.5	0.5	N/A	
Palisades	Id	South Fork Snake	1955-1957	Zoned Earthfill	270	andesite - jointed clayey, sand and gravel	curtain-structure	1	1586	10	210	-	42573	106173	106173	0	5:1 to 1:1	2.5	2.5	2.2	only andesite grouted
Glendo	Wyoming	North Platte	1955-1957	Zoned Earthfill	190	shale, limestone, jointed sandstone	curtain-structure	1	790	5 to 10	160	-	37992	33780	33780	0	5:1 to 0.75:1	0.9	0.9	2.8	
Haystack	Oregon	offstream	1956-1957	Zoned Earthfill	105	interbedded rhyolite flows, and tuff - faulted	curtain-structure	1	106	10	110	-	6650	35209	27417	7792	5:1 to 0.75:1	5.3	4.1	21.5	
Twitchell	California	Cuyama	1956-1958	Zoned Earthfill	241	rhyolite tuff, moderately jointed	curtain-structure	1	225	20	110	-	7525	6800	6800	0	N/A	0.9	0.9	0.7	clean alluvial deposits in channel
Casitas	California	Coyote Creek	1956-1959	Zoned Earthfill	334	ss., claystone, and siltstone, faulted	curtain-structure	1	464	10	160	-	16170	8588	8588	0	7:1 to 1:1	0.5	0.5	1.5	many joints and shears
Helena Valley	Montana	offstream	1957-1957	Zoned Earthfill	91	broken indurated tuff, conglomerate and gravels	curtain	1	88	N/A	N/A	-	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	
Howard Prairie	Oregon	Beaver Creek	1957-1958	Zoned Earthfill	100	basalt over andesite, claystone, ss. And siltstone	curtain-structure	1	53	10	110	-	3135	600	600	0	N/A	0.2	0.2	2.5	
Keene Creek	Oregon	Keene Creek	1957-1959	Zoned Earthfill	78	basalt and tuff breccia jointed	curtain	1	31	10	110	-	1078	1112	1112	0	N/A	1	1	0.6	
Trinity	California	Trinity	1957-1962	Zoned Earthfill	538	meta-andesite and tuff joint, shears, faults	curtain-structure	2	2112	10	260	-	95113	94287	94287	0	N/A	1	1	1.5	many fractures somewhat open
Littlewood R.	Ida	Littlewood	1958-1960	Zoned Earthfill	122	basalt with interflow zones	curtain	1	281	20	60	-	7609	77212	68488	8724	N/A	10.1	9	34.3	

DRILLING AND GROUTING DATA - EMBANKMENT DAMS

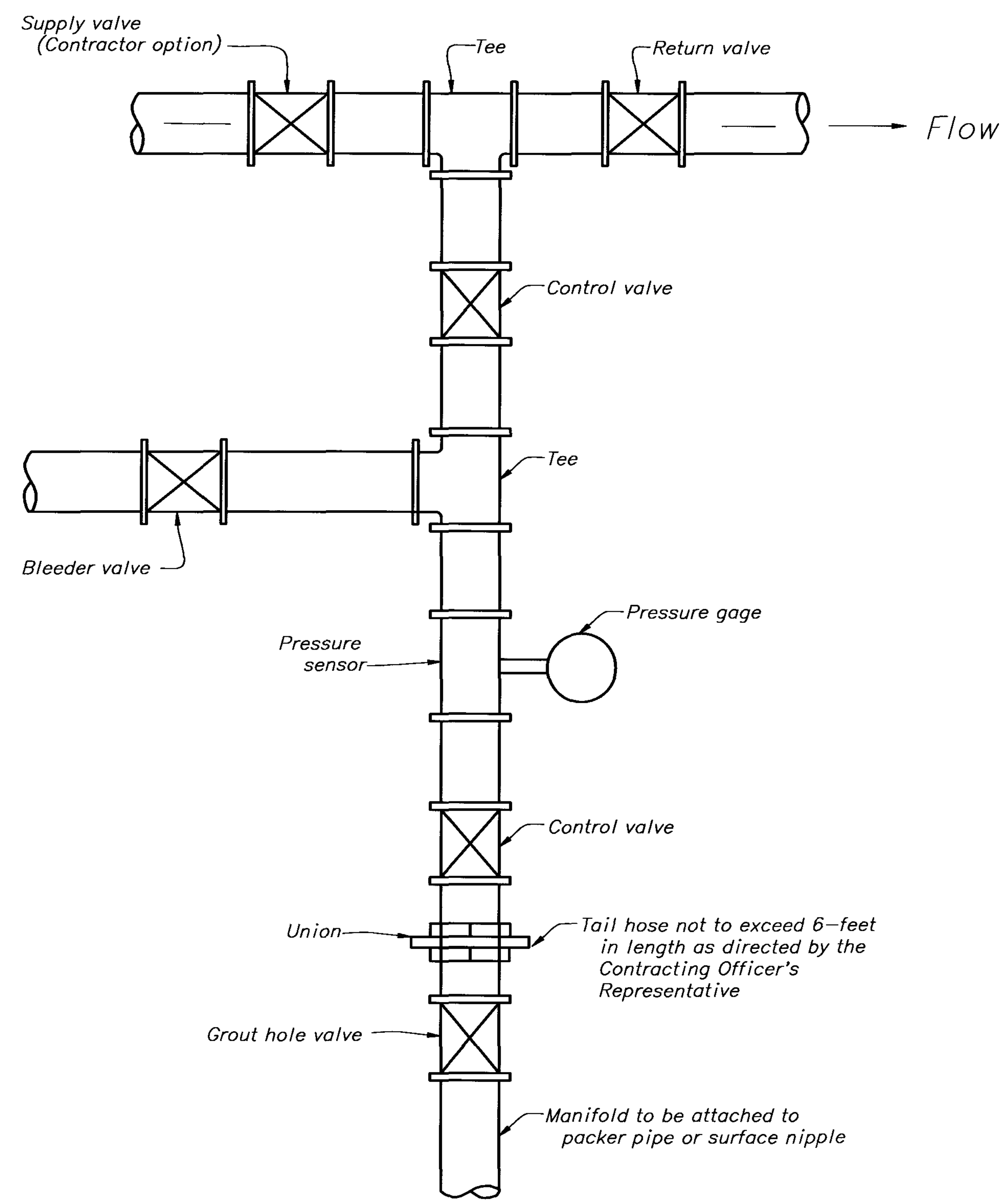
DAM				GENERAL DESCRIPTION		FOUNDATION	DRILLING OF HOLES							GROUTING OF HOLES							
Name	Location		Year Constructed	Type of Dam	Structural Height in Feet	Geology	Type of Grouting	number of rows of holes	number of holes drilled	spacing of holes in feet	maximum depth		total linear feet of holes	total cubic feet of grout			mix ratio w/c by volume	cf / lf of hole		% cost of total cost	remarks
	State	River									curtain	blanket		totals	cement	sand		grout	cement		
Emigrant	Oregon	Emigrant Creek	1958-1961	Zoned Earthfill	196	sandstone - some shale and cong., dikes and sills	curtain-structure	1	267	20	160	-	8466	11109	11109	0	N/A	1.3	1.3	3.5	
Prineview	Oregon	Crooked	1958-1961	Zoned Earthfill	245	basalt lava flows, tight seams	curtain-structure	1	186	10	60	-	4452	2729	2729	0	5:1	0.6	0.6	2	
Navajo	New Mexico	San Juan	1958-1963	Zoned Earthfill	408	sandstone-stress relief joints	curtain-structure	1	1276	10	260	-	66034	119501	119501	0	N/A	1.8	1.8	2.8	
Steinaker	Utah	offstream	1959-1961	Zoned Earthfill	162	sandstone and shale	left abutment repair	1	51	10	110	-	4327	5409	5409	0	N/A	1.3	1.3	N/A	grouted after reservoir filled
Paonia	Colorado	Muddy	1959-1962	Zoned Earthfill	199	interbedded shale and sandstone	curtain-structure	1	272	10	110	-	10375	36261	36261	0	5:1	3.5	3.5	6	
Prosser Creek	California	Prosser Creek	1959-1962	Zoned Earthfill	157	tuff breccia, basalt, andesite, sand and gravel	curtain-structure	1	243	5 to 10	125	-	21844	65482	65482	0	5:1 to 1:1	3	3	11.8	rock is fractured and cavernous
Crawford	Colorado	Iron Creek	1960-1962	Zoned Earthfill	162	sandstone-jointed some shale	curtain-structure	1	283	10	110	-	8503	16999	16999	0	8:1 to 0.5:1	2	2	5.7	
Twin Buttes	Texas	South and Mid Concho	1960-1963	Zoned Earthfill	134	sh., ls., dolomite, ss., siltstone, tight joints	curtain-structure	1	86	10	60	-	4052	6198	6198	0	5:1 to 0.8:1	1.5	1.5	0.3	
Whiskeytown	California	Clear Creek	1960-1963	Zoned Earthfill	278	metarhyolite and meta-diorite dikes, faulted	curtain-structure	2	1084	10	160	-	58080	52869	52869	0	5:1 to 1:1	0.9	0.9	7.4	grouting pressure = 50 + 1.5 depth to packer
Lewiston	California	Trinity	1961-1962	Zoned Earthfill	70	greenstone-cut by diorite dike	curtain	1	103	10	60	-	3683	1616	1616	0	N/A	0.4	0.4	1.7	joints and shears fairly tight
Lemon	Colorado	Florida	1961-1963	Zoned Earthfill	284	shale , siultstone, ss., minor fractures	curtain-structure	1	437	10	150	-	18200	8754	8754	0	7:1 to 2:1	0.5	0.5	2.2	
Spring Creek Debris	California	Spring Creek	1961-1963	Zoned Earthfill	210	quartz diortie cut by dikes, tight joints	curtain-structure	1	165	10	110	-	7720	4801	4801	0	N/A	0.6	0.6	2.6	shears and faults fairly tight
Clark Canyon	Montana	Beaverhead	1961-1964	Zoned Earthfill	148	fractured thin bed ls, tuffs and cong.	curtain struct.	1	240	5 to 10	160	-	25824	46413	46413	0	N/A	1.8	1.8	8	
Fontenelle	Wyoming	Green	1961-1964	Zoned Earthfill	139	shale and sandstone, shrinkage cracks and open bedding planes	curtain struct.	2	971	5 to 10	110	-	45497	143376	143376	0	5:1 to 0.75:1	3.2	3.2	8.5	2nd curtain on right abutment
Bully Creek	Oregon	Bully Creek	1962-1963	Zoned Earthfill	125	Basalt, Volcanic breccia, ss. Clayston, siltstone	curtain	1	74	10	110	-	3180	4399	4399	0	N/A	1.4	1.4	2.5	
Sanford	Texas	Canadian	1962-1965	Zoned Earthfill	228	sh. Siltstone, dolomite, gypsum, breccia, filled chimneys	curtain struct.	2	932	10	160	-	54883	42185	42185	0	5:1 to 1:1	0.8	0.8	1.7	2 rows on abutments
Causey	Utah	South Fork Ogden	1962-1965	Zoned Earthfill	206	limestone and dolomite, fractured, solution cavities	curtain struct.	2	536	10	160	-	28254	67559	67559	0	5:1 to .75:1	2.4	2.4	5.5	
Blue Mesa	Colorado	Gunnison	1962-1966	Zoned Earthfill	340	granite gneiss, schist, few joints	curtain blanket	1	190	10	310	N/A	16000	14124	14124	0	7:1 to 1:1	0.9	0.9	1	
Blue Mesa	Colorado	Gunnison	1962-1966	Zoned Earthfill	340	granite gneiss, schist, few joints	spillway and outlet	-	N/A	-	-	-	18708	4929	4929	0	N/A	0.3	0.3	N/A	
Lost Creek	Utah	Lost Creek	1963-1965	Zoned Earthfill	190	shaly limestone thin bedded	curtain struct.	1	364	5 to 10	160	-	14276	36501	36386	115	N/A	2.6	2.6	9	
Joes Valley	Utah	Seely Creek	1963-1966	Zoned Earthfill	220	sandstone, shale, coal seams, few joints	curtain	1	N/A	10	160	-	10199	13144	13144	0	N/A	1.3	1.3	N/A	P=25+d; d=depth to packer
San Luis	California	San Luis Creek	1963-1967	Zoned Earthfill	384	cong. Sandstone, silty sands, and gravels	curtain struct.	1	N/A	10	300	-	167691	65011	65011	0	N/A	0.4	0.4	2	many faults - fairly tight
Fontenelle Repair	Wyoming	Green	1964-1965	Zoned Earthfill	139	shale, sandstone, siltstone	curtain	10	608	5 to 10	160	-	56376	203552	203552	0	5:1 to 0.75:1	3.6	3.6	91	10 rows on right abutment only
Los Banos	California	Los Banos Creek	1964-1965	Zoned Earthfill	164	sandstone with some shale fractures fairly tight	curtain	1	153	10	110	-	8788	1674	1374	0	5:1 to 1:1	0.2	0.2	2.5	
Los Banos	California	Los Banos Creek	1964-1965	Zoned Earthfill	164	sandstone with some shale fractures fairly tight	outlet works	-	164	-	30	-	3568	2860	2860	0	1:01	0.8	0.8	N/A	
Senator Wash	California	Senator Wash	1964-1966	Zoned Earthfill	95	andesite - vesicular jointed, fractured	curtain	1	268	10	60	-	11222	1633	1633	0	N/A	0.2	0.2	3	
Arbuckle	Oklahoma	Rock Creek	1964-1966	Zoned Earthfill	140	shale, ls, ss, cong. Marlstone, fault gouge	curtain	1	137	10	110	-	8981	1710	1710	0	5:01	0.2	0.2	4	dip mostly vertical to steep, some low angle dips
Mann Creek	Idaho	Mann Creek	1964-1967	Zoned Earthfill	150	basalt lava with tuff breccia	curtain	1	149	10	110	-	9997	6969	6969	0	N/A	0.7	0.7	3	tuff lake beds near top of each abutment
Rifle Gap	Colorado	Rifle Creek	1964-1967	Zoned Earthfill	150	ss and sh dip is vertical ss is fractured	curtain	1	225	10	110	-	8466	11896	11896	0	5:1 to .75:1	1.4	1.4	4.1	
Ruedi	Colorado	Frying Pan	1964-1968	Zoned Earthfill	325	sandstone - open bedding planes jointed, dip 40 deg downstream	curtain	1	912	10	260	-	25757	222920	222920	0	6:1 to 1:1	8.6	8.6	11.3	
Glen Elder	Kansas	Solomon	1964-1969	Zoned Earthfill	185	fractured shale, bentonite, chalk, ls, minor faults	curtain struct.	1	391	10	110	-	21950	15290	15290	0	8:1 to 1:1	0.7	0.7	N/A	
Agate	Oregon	Dry Creek	1965-1966	Zoned Earthfill	90	basalt (fractured) over tuff breccia	curtain struct.	1	63	10	120	-	3087	4691	4691	0	10:1 to .5:1	1.5	1.5	4.1	
Mason	Oregon	Powder	1965-1968	Zoned Earthfill	190	meta-andesite - jointed and faulted	curtain struct.	1	N/A	10	260	-	10211	7363	7363	0	5:1 to 1:1	0.7	0.7	N/A	alluvium on right abutment
Contra Loma	California	Offstream	1966-1967	Zoned Earthfill	105	soft claystone, sandstone, and siltstone	curtain	1	N/A	10	110	-	19829	10904	10904	0	5:1 to 1:1	0.6	0.6	N/A	open joints in weathered material
New Melones	California	Stanislaus	1966-1979	Earth and Rockfill	625	meta-volcanics, meta-basalt agglomerate, meta-sediments	curtain	3	N/A	5 to 10	200	-	89965	12148	N/A	N/A	6:1 to .6:1	0.2	0.2	N/A	
Starvation	Utah	Strawberry	1967-1970	Zoned Earthfill	205	sandstone-vertical joints shale shrinkage cracks	curtain struct.	1	612	10	160	-	33154	35968	35968	0	8:1 to 1:1	1.1	1.1	3.9	
Heron	New Mexico	Willow Creek	1967-1971	Zoned Earthfill	280	ss., sh. And ls. Badly jointed	curtain blanket	2	N/A	5	260	-	99122	146438	146438	0	15:1 to 1:1	1.5	1.5	N/A	

DRILLING AND GROUTING DATA - EMBANKMENT DAMS

DAM				GENERAL DESCRIPTION		FOUNDATION	DRILLING OF HOLES							GROUTING OF HOLES							
Name	Location		Year Constructed	Type of Dam	Structural Height in Feet	Geology	Type of Grouting	number of rows of holes	number of holes drilled	spacing of holes in feet	maximum depth		total linear feet of holes	total cubic feet of grout			mix ratio w/c by volume	cf / lf of hole		% cost of total cost	remarks
	State	River									curtain	blanket		totals	cement	sand		grout	cement		
Conconully Rehab	Washington	Salmon	1968-1968	Hydraulif Earthfill		granite and meta-sediments altered and fractured	curtain struct.	2	N/A	10	110	-	6588	26224	18224	0	8:1 to 1:1	4	4	N/A	
Stampede	California	Little Truckee	1968-1970	Zoned Earthfill	250	tuff breccia - fairly tight fractures	curtain struct.	1	222	5 to 10	160	-	25550	7057	7057	0	10:1 to 1:1	0.3	0.3	N/A	
Bottle Hollow	Utah	Offstream	1969-1971	Zoned Earthfill	72	sandstone, minor jointing, shale	curtain struct.	1	265	10	110	-	8061	5383	5383	0	5:1 to 2:1	0.7	0.7	N/A	
Toa Vaca	Puerto Rico	Toa Vaca	1969-1972	Zoned Earthfill	240	andesite breccia, fractured, open joints in weathered rock	curtain	1	274	10	110	-	13662	13123	13123	0	7:1 to 1:1	1	1	1.9	
Soldier Creek	Utah	Strawberry	1970-1974	Zoned Earthfill	272	interbedded, gradational, interlensed sandstone, siltstone	curtain, blanket	1	N/A	10	160	N/A	26541	N/A	25633	N/A	N/A	N/A	1	N/A	
Ririe	Idaho	Willow Creek	1970-1977	Earth and Rockfill	253	valley alluvium underlain by basalt/ basal sediment	curtain	1	N/A	5	160	N/A	54628	55418	N/A	N/A	3:1 to .5:1	1	1	N/A	
Currant Creek	Utah	Currant Creek	1974-1975	Zoned Earthfill	164	sandstone, siltstone, shale	curtain	1	N/A	N/A	160	-	21315	6075	N/A	N/A	N/A	0.3	0.3	N/A	
Red Fleet	Utah	Big Bush Creek	1977-1980	Zoned Earthfill	175	shale, siltstone, sandstone	curtain, blanket	1	N/A	10	65	N/A	N/A	16653	N/A	N/A	8:1 to 3:1	N/A	N/A	N/A	
Ridgway	Colorado	Dallas Creek	1978-1987	Zoned Earthfill	333	sandstone, shale, coal beds	curtain, blanket	1	N/A	10	150	N/A	64353	32164	32164	0	8:1 to 1:1	0.5	0.5	N/A	
McGee Creek	Oklahoma	McGee Creek	1981-1987	Zoned Earthfill	161	Sandstone, Siltstone, shale	curtain, blanket	1	N/A	5 to 10	138	N/A	27647	13095	13095	0	8:1 to 1:1	0.5	0.5	N/A	
McPhee	Colorado	Dolores	1983-1984	Zoned Earthfill	295	sandstone, siltstone	curtain, blanket	1	N/A	10	180	30	N/A	23412	23412	0	10:1 to 1:1	N/A	N/A	N/A	
French Canyon	Washington	Cowiche Creek	1983-1985	Zoned Earthfill	74	basalt, cong.	curtain	3	N/A	20	50	-	N/A	N/A	N/A	N/A	N/A	0 to 1	0 to 1	N/A	
Brantley	New Mexico	Pecos	1984-1989	Zoned Earthfill	159	dolomite, siltstone, sandstone, anhydrite, gypsum	curtain, blanket	1 / 3	N/A	20	55	N/A	61165	28217	N/A	N/A	N/A	0.4	0.4	N/A	ft drilled and cement sacks used include abutments
Jordanelle	Utah	Provo	1987-1992	Zoned Earthfill / Rockfill	345	andesite porphyry, volcanic breccia	curtain, blanket	3	N/A	2.5 to 10	150	N/A	80797	19101	N/A	0	5:1 to 1:1	0.3	0.3	N/A	
New Waddell	Arizona	Agua Fria	1987-1994	Zoned Earthfill	440	conglomerate	curtain, blanket	1 / 3 / 5	N/A	10 to 20	250	N/A	1054957	987150	N/A	N/A	N/A	N/A	0.9	N/A	
Ridges Basin	Colorado	Basin Creek	2003-2011	Zoned Earthfill	273	sandstone, siltstone	curtain, blanket	1	N/A	10	260	N/A	87545	92633	N/A	0	N/A	1.1	1.1	N/A	

Appendix E

Standard Drawings, Inspector Drilling and Grouting Forms



NOTE

1. All valves shall have 6-inch long lever handles.
2. Pipe size to be determined by the specifications.

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

STANDARD DESIGNS

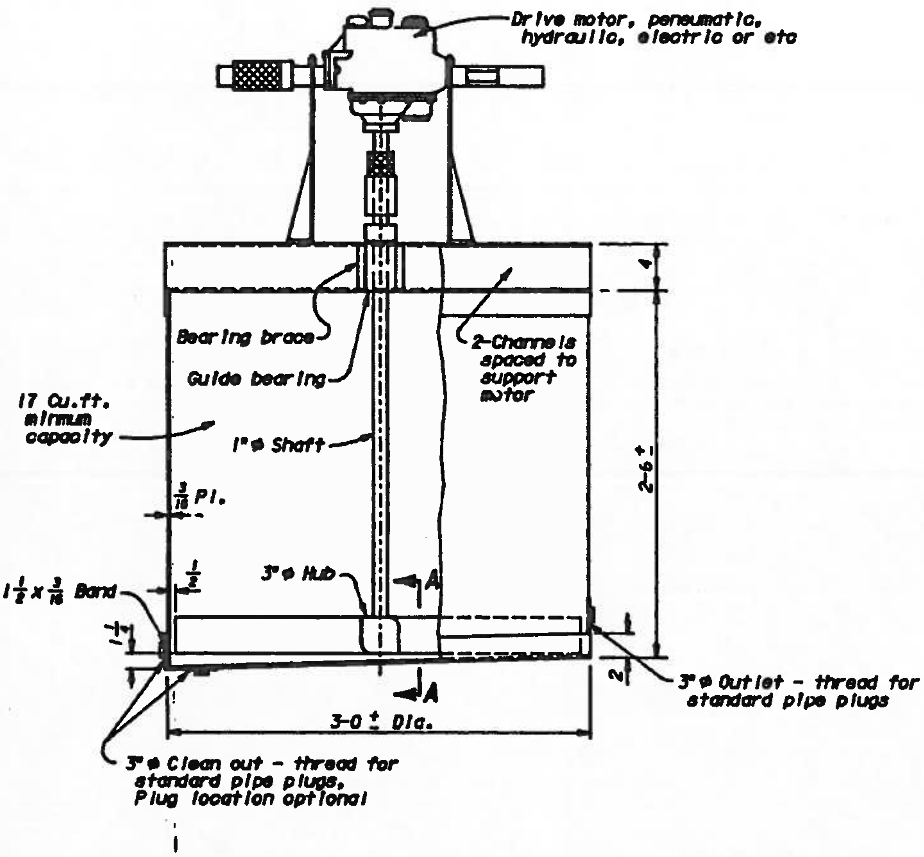
GROUT MANIFOLD FOR CIRCULATING LINE SYSTEM

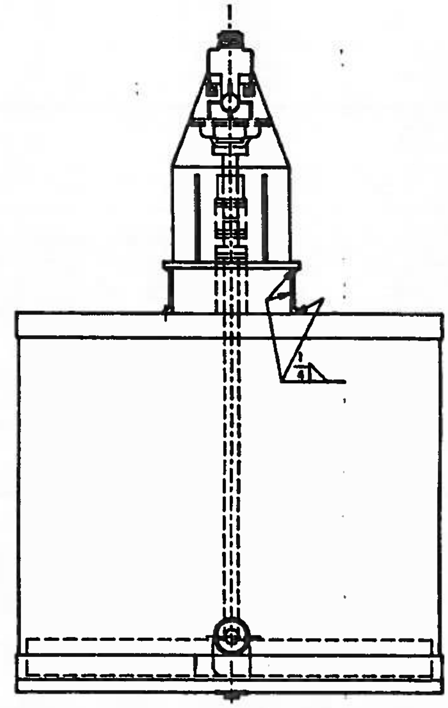
DESIGNED P. P. ABERLE
CHECKED
DRAWN
TECH APPR P.E.
APPROVED P.E.

CADD SYSTEM AutoCAD Rel. 15.06
CADD FILENAME 40-D-7100.DWG
DENVER, COLORADO JANUARY 9, 2006

40-D-7100

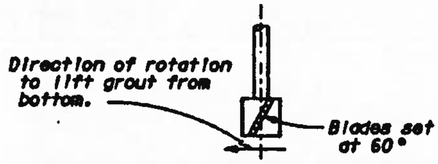
FILE PATH: Q:\ACOMITA DAM
PLOTTED BY: LWALKER
OCTOBER 4, 2005





NOTE

Normal operating speed 30 R.P.M., for cleaning 100 R.P.M.
Construction of agitator may be variable.
Three inch diameter outlet must be 2 inches above agitator floor.
Minimum capacity 17 cubic feet.



SECTION A-A

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

STANDARD DESIGN

GROUT AGITATOR USED IN FOUNDATION GROUTING

DESIGNED P.P. Aberle — TECHNICAL APPROVAL [signature]
DRAWN R.O. Albrecht — SUBMITTED [signature]
CHECKED P.P. Aberle — APPROVED [signature]

DENVER, COLORADO
COMPUTER DRAFTING — AUG. 7, 1990 — 40-D-6571

DIVISION OF CONSTRUCTION

INSPECTOR' DRILL RECORD

PROJECT: **CONTRACT NO. :**

FEATURE: **TYPE HOLE : BLANKET - CURTAIN - TUNNEL - CORE**

STATION: ____________________

Planned Depth ________ **ft.** **DETAIL:** **BIT SIZE:** ______________

CATEGORY: 1 - 2 - 3 - 4 - 5 - 6 - 7 **NIPPLE SIZE:** **Drill Type:**

DRILL TIME		DEPTH FT.		PERTINENT INFORMATION	DRILL # / DRILLER	SHIFT & DATE	INSP. INITIALS
START	STOP	FROM	TO				

GROUTING INSPECTOR'S REPORT

PROJECT :
FEATURE :
CONTRACT NO. :
START DATE: ____/____/____ **END DATE:** ____/____/____

STATION : ____________________

TYPE OF HOLE: BLANKET CURTAIN TUNNEL

CATEGORY : 1 2 3 4 5 6 7

DEPTH DRILLED : __________ **FT.**

CEMENT SUMMARY					
Gum	Bentonite	Plast. Oz.	Bags Injected	Waste (Bags)	
lbs	lbs			GOV	CONT

STAGE	DATE	TIME	CEMENT (BAGS)			Flow	Total	W/C	PRESSURE			PUMP NO.	INSP. INITALS	REMARKS
			TOTAL	CHANGE	BAGS/HR	Rate	Volume	RATIO	PUMP	HLD	BCK			