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ANALYSIS OF THE BUREAU OF RECLAMATION'S USE OF GROUT AND GROUT CURTAINS— SUMMARY

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16. ABSTRACT

The foundation grouting programs of six large Bureau of Reclamation dams were reviewed and analyzed. The purpose of this program was to analyze the use of foundation grouting at Bureau of Reclamation structures to determine the effectiveness of the grout over the service life of the dams to date. Special attention was given to preconstruction geological conditions and changed or unexpected geological conditions discovered during the grouting activities. Recommendations are presented to improve future grouting operations.

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ANALYSIS OF THE BUREAU OF RECLAMATION'S USE OF GROUT AND GROUT CURTAINS – SUMMARY

by

Claude A. Fetzer

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Concrete and Structural Branch
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Engineering and Research Center
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UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. Administration.

PREFACE

This report is a summary of the following GR reports published by the Bureau of Reclamation:
GR-86-2, Analysis of Utilization of Grout and Grout Curtains – Hoover Dam
GR-86-3, Analysis of Utilization of Grout and Grout Curtains – Kortes Dam
GR-86-4, Analysis of Utilization of Grout and Grout Curtains – Hungry Horse Dam
GR-86-5, Analysis of Utilization of Grout and Grout Curtains – Flaming Gorge Dam
GR-86-6, Analysis of Utilization of Grout and Grout Curtains – Morrow Point Dam
GR-86-7, Analysis of Utilization of Grout and Grout Curtains – Heron Dam and Dike

The reports were prepared for the Bureau of Reclamation by Mr. Claude A. Fetzer, consulting geotechnical engineer. The contents of the reports reflect the views of Mr. Fetzer who is responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Bureau of Reclamation.

As directed by the statement of work in his contract, Mr. Fetzer reviewed the historical data and records for each of the dams discussed in this report series. He based his comments, conclusions, and recommendations on the results of this review. It should be understood that Mr. Fetzer's remarks relate to Bureau of Reclamation foundation grouting procedures that may have been accomplished as long ago as 50 years. During the interim, there have been tremendous advances in grouting. Design and construction practices also have been significantly changed and improved. Mr. Fetzer's reports thus comment on Bureau foundation grouting as performed in the past, not necessarily as is currently performed.

The GR reports and, particularly, this Summary Report contain a number of recommendations concerning Bureau foundation grouting practices. The majority of these recommendations have previously been incorporated into current grouting practice and require no further comment. The following discussion, however, is designed to clarify the Bureau's position with respect to several of the recommendations listed in the summary report:

> Recommendation 9.(f)(2) "Where communications between holes is occurring, consideration be given to pretesting and presetting packers in the advance holes as was used at Amistad."

The Bureau currently does this. However, if there is frequent occurrence of communication between holes, we will increase the spacing of the primary holes until communication ceases to occur. That is, if communication repeatedly occurs with primary holes spaced at 40 feet, we will increase the spacing to 80 feet or 160 feet as necessary. However, under certain geological conditions communication may be desirable.

> Recommendation 9.(g) "*Water-pressure tests.* The takes in the water pressure tests be used as a guide in selecting the initial grout mixes."

Mr. Fetzer also lists in the Summary Report several charts to this effect as used on other dam projects.

In the past, the Bureau did not normally do this because of the difficulty in differentiating between primary permeability and secondary permeability in some rock types. Currently, however, several dam foundations are being grouted where starting mixes are selected based on water pressure tests. We simply feel that the recommendation is too broad and cannot be applied to all foundations.

Kortes Dam – "Higher than normal pressures still exist near the downstream toe and installation of additional drains is recommended." A review of Structural Behavior foundation gauge pressures showed pressures higher than desirable, but not alarmingly high. Monitoring will continue and perhaps in several years another judgment will have to be made relative to stability. Also, due to the project purpose of power generation, the small reservoir capacity of 4,000 acre-feet and upstream proximity to Pathfinder Dam, additional drains do not seem necessary at this time.

Flaming Gorge Dam – "Additional information is needed on the uplift pressures in the sandstone and on the pore pressures in the shale in the right abutment; this information is needed to evaluate the stability of the narrow ridge forming the abutment." The diversion tunnel through the right abutment was plugged with concrete about in line with the dam's right abutment. Holes have been drilled in a radial pattern outward from the open portion of the tunnel to drain the right abutment. The volume of water has not been estimated

PREFACE – Continued

or measured. A sample measurement program could be initiated. In response to Mr. Fetzer, uplift and pore pressure would be helpful in evaluating the abutment stability, but the magnitudes may be small in light of surface seepage and drainage. The present program of monitoring should be sufficient.

Morrow Point Dam – "* * * fines are collecting behind one weir possibly indicating material is piping from a shear zone." This weir may be in the tunnel behind the powerplant. A review of weir measurement data sheets on file in Structural Behavior Section does not mention collection of fines. Thus, before evaluating piping, locate weir collecting fines and initiate a yearlong detailed measurement system.

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TABLES

AUTHORIZATION

This report was prepared by Claude A. Fetzer, Consulting Geotechnical Engineer, in accordance with the provisions of contract No. 2-07-DV-00148 dated February 10, 1982, issued by the Bureau of Reclamation, Engineering and Research Center, D-810, P.O. Box 25007, Denver, CO 80225. The title of the project was "Analysis of the utilization of grout and grout curtains, Portland Cement Grouting Research Program – Dam Safety."

PURPOSE

The purpose was to analyze the use of grout in concrete dams, the construction of grout curtains, the mixes and quantities used, and the apparent effectiveness of the curtains over the service life of the dam to date.

SERVICES TO BE FURNISHED BY ARCHITECT-ENGINEER

The services to be furnished were established by section C-2 of the specifications as follows:

> "a. Review and analysis of technical data, documents, and drawings related to preconstruction geology, foundation grout curtain design, estimated drilling and grout takes, design grout mixes, field water test and permeability, actual drilling, grout takes and mixes as constructed, closure criteria used, and any noted changed or unexpected geological conditions discovered during grouting for each of the six selected projects.
>
> *The Bureau will furnish materials for eight projects, two being alternates should materials from any of the selected six projects be insufficient.
>
> b. Conduct interviews with Bureau engineering personnel knowledgeable of and involved in design and/or construction of foundation grout features.
>
> c. Prepare a report inclusive of pertinent materials, data, and information analyzed for each of the six selected projects."

ARCHITECT-ENGINEER SUBCONTRACTORS

Subcontractors for the architect-engineer, Claude A. Fetzer, were Lloyd B. Underwood, Consulting Engineering Geologist, Omaha, Nebraska; and Richard M. Price, Consulting Engineering Geologist, Del Rio, Texas.

INTERVIEWS

The active and retired Bureau personnel interviewed regarding Bureau grouting practices included Fred Lippold, Lloyd Gebhart, Peter P. Aberle, Charlie Flagg, Richard Kramer, Errol McAlexander, Luther Davidson, and Lynn R. Burton.

DAMS SELECTED

The dams initially selected for analysis by the Bureau were Hoover, Hungry Horse, Flaming Gorge, Morrow Point, Heron, and Seminoe Dams. The two alternatives selected were Kortes and Swift Dams. Seminoe Dam was replaced by Kortes Dam because of the lack of pertinent data on Seminoe Dam. The only nonconcrete dam selected was Heron Dam, which is an earth and rockfill dam. Heron Dam was selected because it was constructed recently and because it had grout curtain extensions on each end, which could be drilled in the second phase of the program.

RESULTS OF ANALYSIS

A summary of the analysis along functional lines is presented in this report, and the detailed analysis for each dam is presented in GR reports as follows:

GR-86-2 – Hoover Dam
GR-86-3 – Kortes Dam
GR-86-4 – Hungry Horse Dam
GR-86-5 – Flaming Gorge Dam
GR-86-6 – Morrow Point Dam
GR-86-7 – Heron Dam and Dike

SUMMARY OF ANALYSES

Preconstruction Geologic Investigations

One measurement of the adequacy of the preconstruction geologic investigations is the changes encountered during construction related to the foundation work and the operational problems related to the grouting and drainage provisions. The problems encountered on the projects studied are shown in table 1.

On Hoover Dam, the preconstruction investigations were extremely limited for a dam this size, but there were few changes related to foundation conditions

Table 1. – Problems encountered on each dam project.

Dam	Construction changes	Operational problems
Hoover	2	7, 8, 9, 10
Kortes	3, 5	8, 10
Hungry Horse	2, 3, 6	–
Flaming Gorge	1, 2, 3, 4	7
Morrow Point	2, 3, 4	7, 9
Heron Dam	5	–
Heron Dike	5	7 (minor)

Key:
1. Significant overrun in foundation excavation
2. Significant overrun in grout hole drilling
3. Significant overrun in cement grout quantity
4. Significant increase in drainage holes
5. Slides or rockfalls
6. Unexpected faults
7. Excessive seepage
8. Excessive uplift
9. Additional grouting required
10. Additional drainage holes required

reported for the construction stage. Neither the preconstruction geology reports nor the grouting report on the original construction indicated the possibility of the seepage and uplift problems that developed, starting with initial filling. In volcanic formations the presence of intrusions, dikes, major fissures, and vent areas are difficult to predict. The dam breccia at Hoover Dam is a sedimentary rock, but it was intruded by volcanic lava flows. To have discovered all of the seepage paths and hydraulic connections found after impoundment in the 1938-1947 grouting program would have been almost impossible in a preconstruction investigation. However, the magnitude of the seepage problem would have been better understood if several deep exploratory holes had been drilled and the water-pressure had been tested in the valley bottom and abutments (horizontal holes at several levels).

At Kortes Dam, the preconstruction surface and subsurface geologic investigations revealed the conditions encountered during construction. However, the problem of rock falls was not identified in the geologic investigations until after the construction contract was awarded.

At Hungry Horse Dam, the investigations were fairly comprehensive; they included exploration of an adit in each abutment. However, the study sites were mostly located for a straight gravity dam and not for the arch dam eventually designed. A consulting board consisting of J. L. Savage, Dr. C. P. Berkey, and W. H. Irwin was employed during the design stage, and Savage and Berkey were engaged as a consulting board during construction. The preconstruction investigations failed to reveal five of the six faults, including three major faults and the lift seam that were found during construction.

At Flaming Gorge Dam, the investigations included explorations of many drill holes and of an adit on each abutment; however, many of these explorations did not penetrate deep enough into the valley bottom or far enough into the abutments to reveal the extent of weathering and open joints. Furthermore, many of the explorations were not appropriately located for the structure eventually designed. The keyways into the abutments had to be extended much deeper than shown on the plans, and the concrete cutoffs in the shale beds had to be extended much deeper than indicated in the specifications. In addition, the grouting and drainage provisions had to be increased considerably during construction. The original design of the grout curtain evidently did not take into account the information obtained from preconstruction surface geologic mapping. This information showed that the fault connections from Cart Creek to Green River below the dam were likely seepage paths after the reservoir was filled. A board of well-known consultants was engaged during construction, but not in the preconstruction stage.

At Morrow Point Dam, the preconstruction investigations were the most extensive of the projects reviewed. The explorations including abutment adits were appropriately located for the structure eventually designed. A well-known board of consultants reviewed the design shortly after the contract was awarded. Extensive rock bolting of the abutments was required during construction, and the grouting and drainage provisions had to be materially increased during construction. Additional grouting was required after construction to reduce seepage into the powerhouse drainage gallery. A problem not related to grouting also developed with the movement of a rock wall of the underground powerplant.

At Heron Dam and Dike, the preconstruction investigations were fairly extensive, but whether a number of holes along the final axis were ever drilled could not be determined. An abutment rock block slid along a bentonite seam during grouting even though relatively low pressures were being used. Extra blanket grouting was required at the spillway because of disturbances to the rock from excavation blasting. The grouting subcontractor made a claim based on the low grout take per pumping hour. More exploratory angle holes were needed in the abutments to determine the nature of the vertical joints; i.e., their filling, staining, openness, and continuity. Other exploratory drill holes to the full depth of the grout curtain were needed for the dam and the dike.

The term "investigations" includes not only the geologic mapping and explorations, but also the interpretation of the data as related to the proposed construction.

For the dams studied the preconstruction geologic investigations at Morrow Point were the most effective. Table 1 indicates that preconstruction investigations for Heron Dam and Dike were also effective.

Nevertheless, the perfect preconstruction geologic investigation will never exist, because even an infinite number of borings cannot reveal the conditions fully. Accordingly, Terzaghi [1]* wrote: "When nature created foundations for our structures she failed to comply with the specifications of the American Society of Testing and Materials, and we have to bear the consequences of the resulting uncertainties."

The inadequacies of preconstruction investigations at Bluestone Dam in the Ohio River Division, Corps of Engineers, were stated by Burwell [2] as follows:

> "In retrospect, it is apparent that geologic advice, unless fortified by adequate large diameter borings, is, not infrequently, a poor panacea for the prevention of foundation ills. That the small diameter borings at Bluestone, made during the design investigations, failed to reveal the foundation conditions in their proper light is evidenced by the fact that they were studied by two consulting geologists, two Department geologists and a board of consulting engineers without disclosure of the fault in question. The lesson many have yet to learn is, that all borings which do not recover approximately 100% of the core are negative holes. The cause of bigger and better holes in foundation exploration goes marching on."

The inadequacy of the preconstruction geologic investigations stems partly from the approach to the problem used by the engineering geologist. An approach that emphasizes proving that unfavorable geologic features do not exist has been espoused by Nieto [3] as follows:

> "More specifically, the most important and most difficult task of an engineering geologist at a damsite is not merely to show that the amount of exploration already performed has not disclosed unfavorable geologic features. By the proper choice of exploration techniques, he is to prove, to everyone's satisfaction, that certain adverse geologic features do not exist at a given site. His proper course of action at the different stages of the project depends on his ability to anticipate the geologic features that are likely to be present at the site, and his ability to understand how these features, if present, could affect the design, construction, and performance of the dam."

This approach has also been suggested by Terzaghi [1] as follows:

> "Hence, if the geologic conditions do not strictly exclude the presence of weak seams or spots in the natural formations supporting our structures, every construction operation involves at least a remote possibility of unanticipated developments."

The key is for the engineering geologist to know what defects are expected in the site formulations and to direct his efforts to prove they do not exist. In the case of Hungry Horse Dam, clay seams in the dam foundation formation were found by two geologists 2 years before construction. These clay seams were found in road cuts just downstream of the dam, in tunnel A in the right abutment and in tunnel B in the left abutment. The clay seam in tunnel B could almost be projected to the lift seam found in that abutment during construction. Therefore, at Hungry Horse Dam the existence of clay seams could not be ruled out because there was direct evidence of their existence in the Siyeh limestone. Instead of assuming the clay seams were only near-surface defects, investigations should have been directed toward proving that the clay seams did not exist below the stripping level, particularly on the left abutment where the bedding planes were parallel to the surface. Because clay seams are easily eroded and carried out with the drilling water, any lost core should have been considered a clay seam until proven otherwise. Either large diameter borings (5½-in) or calyx holes for visual examination would have been needed to disprove the existence of clay seams because TV cameras were not available then.

On most projects in this study, the preconstruction drill holes did not give full coverage in depth and lateral extent to the designed grout curtain. The coverage of explorations and geologic surface mapping should be sufficient to identify the extent of defects such as relief joints, crushed zones, cavities, etc., which could affect the seepage flow at the dam after the reservoir fills. For example, if relief joints exist at the site, angle or horizontal drill holes should be carried into the abutments until the joints are tight enough that grouting is not required to protect the structure or to prevent excessive leakage. In all cases, the explorations with water-pressure tests should provide full coverage to the anticipated grout curtain.

None of the available preconstruction geologic reports related the geologic conditions to the design of the grout curtain. Although some reports referred to the need for grouting, none presented an analysis connecting the geologic conditions to a specific grout curtain design or to the design of the drainage system. The geologic report should have included recommendations on the depth, lateral extent, angle, and spacing of holes and an estimate of the grout

* Numbers in brackets refer to entries in bibliography.

takes based on the geologic conditions. Although the design engineer may have wanted to retain ultimate authority over the final design, all available knowledge should have been applied to this sensitive and critical phase of design and construction. Wherever possible the geologist performing the preconstruction investigations should also have been assigned to the construction staff because the design for grouting and drainage continued through construction.

Consulting boards were used on Flaming Gorge and Morrow Point Dams during the construction stage. A consulting board was used at Hungry Horse Dam during the preconstruction and the construction stages. The use of a consulting board at Hoover Dam could not be clearly established from available records, although individual consultants were used. In no case did preconstruction consultants address the grouting provisions.

To be effective, a board of consultants should be engaged early enough to independently review the adequacy of the geologic information and the analysis regarding the grout curtain and the interrelated foundation excavation and treatment. The board should be authorized to recommend additional investigations and changes in the design, as it sees fit, and the board should be continued through the construction stage. The engineer (the Bureau) should retain ultimate authority on all matters related to the design and construction; nevertheless, in today's contractual climate any advice from a board that would avert changes in the contract would be extremely worthwhile.

Design

Introduction – Limits of Cement Grouting. – Too much is often expected of cement grouting. At best, cement grouting should be expected to fill the voids in the bedrock opening. Many of the open joints, bedding planes, shear planes, fault zones, solution caverns, and other rock defects are partly filled with soil and weathered rock particles. Cement grouting should not be expected to fill the voids in the soil or other joint fillings.

Removal of material in joints is extremely difficult, as was found in the lift seam at Hungry Horse Dam. Removal of material in joints in the ordinary grouting job by washing from one hole to another must be considered wasted effort because only a rat hole is usually developed between the two holes. If cement grouting is relied upon to improve the bearing capacity, the results may be disappointing because the shear strength and compressibility of the rock may still depend on the soil remaining in the joints. Nor can cement grouting be expected to fill the minute voids in sandstone or highly fissile shale or other thinly bedded rocks. This was recognized at Flaming Gorge Dam, and cutoff shafts were excavated back into the shale zones until competent shale with a very low permeability was encountered. This was also recognized recently at the Bureau's Red Fleet Dam, where a partially grouted zone of shale and gypsum was removed until relatively tight shale was encountered.

The difficulty of grouting weathered rock with cement grout is described in a paper by Cole [4] on Dartmouth Dam. Attempts were made to grout a weathered rock seam on the right abutment using a 5:1 water-cement ratio grout. Even when the pressure was raised to 50 percent above the design pressure (410 kPa = 59.5 lb/in^2) and later to 100 percent above the design pressure, the grouting was unsuccessful (Brian R. Cole, personal communication, July 21, 1982). Chemical grout was then used to seal the seam.

The main points of this discussion are (1) the capabilities of cement grouting are limited; (2) rock removal is a more positive solution for bearing capacity than consolidation grouting where the bedrock openings are partly filled with soil and debris; and (3) highly weathered or fissile-type materials cannot be reliably grouted with cement grouts and should be removed or cutoff with concrete walls when near the surface; if these materials occur at depth an impermeable chemical grout may be used.

Design of Grout Curtains and Drainage Systems. – The main prerequisite for designing a grout curtain is a thorough knowledge of the site geology. (This subject was previously discussed under the heading "Preconstruction Geologic Investigations.") The grout curtain should be designed to seal bedrock openings that would permit excessive seepage and excessive uplift after the reservoir is impounded. For earth dams, the fractures in the rock must be sealed at the embankment-rock contact by mortar or concrete because grouting cannot be expected to prevent piping of erodible soil into the rock fractures.

No magic rule of thumb (such as 0.4 net head) exists for the depth of the grout curtain; the depth of the curtain can be decreased as you ascend the abutments. Open relief cracks, as found at Morrow Point Dam, and an open cavity, as found in the Arizona abutment at Hoover Dam, may require an even greater depth near the top of the dam than at the bottom of the dam; or the grout curtain may have to be extended beyond the end of the dam, as at Flaming Gorge and Heron Dams.

Depth of hole is defined as the penetration into rock, regardless of the angle of the hole. The key to the depth of hole or even the requirement for the grout hole is the need to seal an opening in the bedrock. At Patoka Dam [5] the double-line grout curtain was eliminated across the valley bottom when check holes indicated that the founding rock, Elwren shale, would not take any grout. Patoka has been tested at a high pool for an extended period of time with no evidence that the grout curtain across the valley bottom was needed. It is understood that grouting was not provided by the Bureau at Choke Canyon Dam because the foundation was composed of relatively impervious claystone and siltstone.

Where regional joints or crushed zones extend too deep to reasonably construct a grout curtain, installing additional drainage lines may be more economical than deepening the grout curtain, provided the bedrock is not limestone or some other soluble rock. The design analysis in the cases for non-soluble rock should include flow nets to develop an estimate of the seepage quantities.

The depth of the B-holes should be based on penetrating the most prevalent near-surface bedrock openings. The extent of coverage over the foundation depends on whether the B-holes are intended as a seepage barrier or as a foundation reinforcement.

For the A-holes, there is no logic in the present procedure of carrying the primary holes to a greater depth than the secondary, tertiary, etc. During construction, the intermediate holes could be shortened if conditions indicate the intermediate holes need not penetrate to the same depth as the primary holes.

The angle of the grout holes should be selected to intersect the majority or most prominent bedrock openings as close to 90 degrees as possible. In this manner the holes have the best chance of intersecting the greatest number of openings. The specifications should permit flexibility in the angle of the holes because joints exposed during construction may indicate different angles than determined during design.

No logical method for selecting the ultimate spacing of the holes has been developed. Experience in similar formations on the spacing required to achieve closure is probably the best guide. However, in rocks of volcanic origin experience may not be a reliable guide, and a very close spacing should be specified.

The drainage holes should be designed to intercept the same bedrock openings as the grout curtain. This means that the drainage holes would be almost as deep as the grout holes. The drainage holes should also be angled to intercept the predominate bedrock openings. Many dams are designed with downstream drainage galleries or with other positive means for controlling the seepage.

Drainage holes were installed in one downstream abutment at Morrow Point Dam. Inclined drainage holes have also been installed in downstream valley walls to control slides that developed along the bedrock-overburden contact at East Fork Dam and Brookville Dam in the Louisville District, Corps of Engineers. Because the cost of drainage holes is only a fraction of the cost of slide correction, the use of inclined drains in the downstream abutments should be considered in the design of all dams.

Grouting Methods and Procedures

Specifications. – The general requirements of the specifications were satisfactory; however, more specific requirements should have been included to clarify the requirements for equipment and methods. Minimum requirements should have been stated for the size and capacity of the mixing and holding tanks, the capacity of the grout pump at maximum pressure, the size of the grout injection tube, the distance between drilling and grouting operations, the amount of cement on hand to prevent shortages for high-take holes, and the time period for water-pressure tests. The specifications should also have included the expected range of grout mixes. The maximum pressure for water tests and for grouting should have been required at the packer rather than at the gauge. The use of pigtails (a single line to the grout hole) should have been prohibited. Restrictions in the grout lines and grout tubes should also have been prohibited.

Cement Pay Item and Estimated Quantities. – It could not be determined in this study whether it was advantageous to use a variable pay item for cement based on take per lineal foot of hole, as was used at Heron Dam and Dike. A claim was submitted on the Heron Dam contract based on the slow rate of take. If the pay item for cement were based on the rate of take per pumping hour, the record keeping effort would have been very high. The traditional practice of paying a single bid item price per sack of cement is therefore advisable.

Extensive studies should be made on each project to correctly estimate the quantity of cement required. Overruns and underruns have been the bases for claims on many dams. These studies should include: data on the water-pressure tests in the exploratory drill holes (as noted on several projects, the pump capacity in the exploratory work should be increased because the takes often exceeded the capacity of the pump); a detailed study of the geologic conditions that would take grout; and analyses of grout takes on previous dams having similar geologic conditions.

A study comparing water acceptance in cubic feet per minute and grout acceptance in cubic feet has been made for the upstream and downstream barrier holes at Peace Canyon Site One. This study, which was prepared jointly by B. C. Hydro Construction Division and Knight & Piesold, Ltd., has been reproduced in appendix A with permission of B. C. Hydro. This study was based on using a standardized gauge pressure of 40 lb/in^2 in the water tests, which reduces the variables involved. As shown on the plot on figure 1 of appendix A, the amount of grout acceptance increased with the water acceptance, although there was a large spread in the results.

A study of the water takes in the exploratory drill holes versus the grout takes in the B-hole was made for Flaming Gorge Dam (See figures 12, 24 and 25 of GR-86-5). The data available were very limited. A comparison of grout takes per foot versus water takes per foot could not be made because the exploratory holes did not extend to the depth of the grout holes and because the length of hole taking grout was unknown. However, the study did indicate that high grout takes could be expected in the areas where the exploratory holes had high water takes.

These studies indicate that analysis of the water-pressure tests in drill holes should help estimate cement quantities.

Drilling Methods. – With the exception of the percussion drilling used for the B-holes at Hoover Dam, rotary drilling with water as the circulatory medium was used for drilling the grout and drainage holes on the dams in this study. Other agencies permit percussion drilling. For instance, percussion drilling was used at W. A. C. Bennet Dam [6], and both percussion and rotary drilling were used at Peace Canyon Site One, with alternating air and water used for cleaning holes.

Percussion drilling with air may be satisfactory for shallow blanket holes where the cuttings can be easily blown out. Percussion drilling or rotary drilling with air may be preferable where expansive clays in cavity fillings tend to plug the holes when water is used. Such a situation was described at Amistad Dam [7], where rotary drilling was used, with air or water permitted as the circulatory medium:

> "*Pressure Washing:* The holes were pressure washed on completion of drilling and were then pressure tested and grouted in zones by the split-spacing, stop-grouting method as follows: Specifications required that on completion of drilling, the holes were to be washed by circulating clear water under pressure from the bottom of the hole until all sludge and cuttings were removed, or until the return water had cleared. In the early stages of the operations, it was noted that a rather large number of holes were 'bridged' with clay and required cleaning out by drilling to facilitate setting packers at the lower zones. It was determined that the bridge was formed by swelling or caving of clay from filled or partially-filled cavities as a result of excessive wetting during the pressure washing operations. It was also noted that a relatively large number of the air-drilled holes were 'tight' on grouting, and it was suspected that air cuttings were filling openings which ordinarily would have taken grout.
>
> "To check the adequacy of the curtain that had already been grouted on 20-foot centers and to determine the best method of cleaning the holes, a number of experimental split-spaced holes were drilled and tested by the methods described below:
>
> a. Twenty-six holes were drilled with water, bottom hole washed, and pressure tested prior to grouting.
>
> b. Twenty-four holes were drilled with air, bottom hole washed with water, and immediately pressure tested and grouted.
>
> c. Twenty-four holes were drilled with air, air cleaned, and pressure tested just prior to grouting, as the work load permitted.
>
> A comparative study of the unit take, in sacks per foot of drill hole, in the various tests indicated that air drilling and air cleaning was the most satisfactory method of drilling and cleaning the holes. As a result of these tests, water washing of the holes was discontinued, and pressure testing of the individual holes was deferred until the packer was set for grouting. After the bottom hole washing was discontinued, there was a noticeable decrease in the clay cleanout operations."

Rotary drilling with water has proved satisfactory on the six Bureau projects studied, however, there may be special situations where other drilling methods and circulatory and cleaning mediums may be needed.

Grouting Methods. – Ascending-stage grouting with packers set at the top of each stage was used as the basic method of grouting on the projects reviewed. Descending-stage grouting without packers was used intermittently in the 1938-1947 grouting at Hoover Dam. The specifications on the other projects included provisions for using descending-stage grouting at the discretion of the contracting officer. The descending-stage grouting was to be used when drill water losses occurred, but this was not stated in the specifications. The specifications did not provide for payment for the extra drill setups and extra grouting hookups required by descending-stage grouting; however, current Bureau specifications do provide for payment for the extra hookups. The Bureau's requirement for descending-stage grouting of setting a packer at the bottom of each preceding stage is correct; otherwise, the increased pressure applied with increased depth would be applied to the top of the hole. From this study of six dams, it was found that ascending-stage grouting had been used successfully as the basic grouting method.

One of the problems encountered in ascending-stage grouting is that holes drilled ahead of the grouted hole may be grouted by transmission. This problem was encountered at Amistad Dam [7] and was corrected as follows:

> "After several holes had been grouted, it was found that approximately 50% of the holes already drilled were being grouted or partially grouted by grout transmission from hole to hole, with no indication as to where the grout entered the adjacent hole or holes. A number of holes grouted by transmission were cleaned out and regrouted, with the result that approximately 63% of the holes accepted grout, indicating that grouting by transmission was incomplete. In addition, when the primary holes were grouted by transmission, it was found that the first series of split-spaced holes frequently took more grout than these particular primary holes, a further indication that the initial grouting was ineffective. That redrilling and cleanout of holes grouted by transmission was effective is indicated by the fact that of a total of 95 holes cleaned out and regrouted, 68 holes, or 71.6 percent of the holes, accepted additional grout.
>
> "The procedure adopted to improve the grouting was to pressure test two holes in advance of the hole to be grouted to predetermine the zones of possible grout take ahead of the grouting schedule. The packers were then left in each hole just above the lowest take zone, and the holes were grouted under the maximum allowable pressure for the zone. This allowed the header to be shifted from hole to hole and thus increased the effectiveness of the grout job. As soon as this procedure was adopted, the successive split-spaced holes showed a progressive decrease in grout take, indicating the procedure was more effective. In addition, having advance information as to where the adjacent holes accepted was valuable in determining the depth required for drilling split-spaced holes. The three-packer operation also reduced the requirement for redrilling grouted or partially grouted holes."

This procedure is similar to that required by the Bureau specifications, which require that packers be set in the advance holes after grout flows from the advance holes. However, at Amistad Dam the packers were preset in the advance holes before grouting was started. One disadvantage of the Amistad Dam procedure is that the packer could get grouted-in if grout flowed into the hole above the packer, which had been set just above the lowest water-take zone. However, this did not occur at Amistad Dam. The reported success of the procedure developed at Amistad Dam merits consideration of its use.

Continued use of ascending-stage grouting with packers is recommended; however, geologic conditions may require that descending-stage or circuit grouting be the basic grouting method.

Water-Pressure Tests. – The available records indicate that water-pressure tests were performed for 5 minutes at the planned maximum grout pressure. On some projects where interconnections with other holes were found, the pressure tests were continued for another 5 minutes or more.

The results of the water-pressure tests can be used as a guide for selecting the initial water-cement ratio for the grout. The data in table 2 was used as a guide by Richard M. Price [8] for the moderately solutioned limestone at Clarence Cannon Dam.

At the Amistad Dam and Reservoir Project [6] the guides in table 3 were used in the highly solutioned limestone.

A discussion on the use of Lugeon units can be found in the subsection entitled "Spacing of holes and closure."

Grout Mixes. – Data from tests at the Waterways Experiment Station [9] indicate that for cracks 0.02 to 0.03-inch wide, type III cement with a water-cement ratio of 0.64 could be injected at a pressure of 25 lb/in^2. For cracks 0.01-inch wide, a minimum water-cement ratio of 4.0 was required for an injection pressure of 25 lb/in^2, and a minimum water-cement ratio of 2.0 was required for a pressure of 50 lb/in^2. These tests were made for a separated horizontal void between two concrete slabs that had much smoother surfaces than most.

Table 2. – Guide for selecting water-cement ratio at Clarence Cannon Dam.

Water take, ft^3/min	Initial water-cement ratio, by volume
≤ 4	6:1
4–6	5:1
6–7	4:1
>7	3:1

Table 3. – Guides for selecting initial grout mix and water-cement ratio at Amistad Dam.

Embankment dam foundation	
Water loss, gal/min	Initial grout mix, by volume
1–30	5:1
30–60	3:1
>60	2:1
Concrete dam foundation	
Water take, ft^3/min	Water-cement ratio to start grouting, by volume
0.2–4.0	3:1
4.0–8.0	2:1
>8.0	1:1

The Waterways Experiment Station tests showed the following disadvantages in using high water-cement ratios: (1) bleeding becomes more severe the higher the unit water content, and squeezing out the excess water in the grouting tests at 100 lb/in² was found to be impossible; (2) the setting time increased with water content, and neat cement grout with a water content as high as 3.8:1 (5.7:1 by volume) required 10 days to set, in the setting-time tests, and had not set in 13 days when spread into a thin film in the grouting tests; and (3) low strengths occurred for high water-cement ratios.

These laboratory data indicate a tremendous advantage in using low water-cement ratios. At Hungry Horse Dam, the washed void in the lift seam was filled with grout having water-cement ratios ranging from 2:1 to 0.75:1, with the greatest amount at 0.8:1; the gauge pressures used ranged from 5 to 40 lb/in². This field experience indicates the feasibility of using grouts having relatively low water-cement ratios. When grouting dry rock above the water table, some of the water in the grout is undoubtedly absorbed by the rock even though water has been previously introduced during drilling, during the prewashing, and during the water-pressure tests. Even in dry rock, it appears advantageous to use a minimum amount of thin grout and to progressively thicken the grout to determine the thickest grout the hole will accept.

Where large quantities of thin grout are injected into one hole under suction conditions, the mix should be thickened to increase pressure. Where large takes of thin grout occur, exploratory holes should be used to determine whether the grout is spreading beyond the limits needed for grouting.

Although extremely thin grouts with water-cement ratios of 20:1 or greater were used successfully in the additional grouting at Hoover Dam, the need for such thin grouts has not been demonstrated in laboratory tests or on more recent projects. Extremely thin grouts should probably be used only where there is a demonstrated need.

Grout Injection Pressures. – The minimum and maximum grout pressures as determined from the available data are shown in table 4.

The bases for establishing and using grout and water-test pressures at Kortes Dam are explained in detail by J. R. Anderson in memoranda to "Construction Engineer" dated May 28, 1948, and December 14, 1949. Extracts from these memoranda are presented in appendix B. They indicate that considerable care was given to the use of the pressures and that many factors were considered during the grouting operations. The observations and adjustments made during the grouting operations are extremely important for success. They fully indicate the need to have competent personnel, such as J. R. Anderson, in the field at all times.

In reviewing the grout pressures (all references to grout pressures refer also to water-test pressures), three items are questioned: (1) the basis for establishing the maximum pressures (see discussion on this question in GR-86-3, section 13(c); (2) the need for the high pressures used; and (3) the use of collar pressure rather than the pressure at the packer as a criterion.

It is recognized that while grouting B-holes and C-holes from top of rock, field observations of leaks have been used to indicate when to adjust the grout pressures. However, this practice can cause considerable damage to the rock structure before that damage is visible. For instance, considerable movement of a rock block was recorded by instrumentation in the powerplant roof at Morrow Point Dam, even though that movement was not visible. Visual observations during grouting of the A-holes from the

Table 4. – Minimum and maximum grout pressures at dams studied.

	Collar pressures in lb/in²					
	B-Holes		C-Holes		A-Holes	
Dam	Min.	Max.	Min.	Max.	Min.	Max.
Hoover (1933-35)	–	400	–	750	–	1000
Hoover (1938-47)	–	–	–	–	–	700
Kortes	1	200	–	–	100	425
Hungry Horse	0	150	75	300	100	500
Morrow Point	5	80	–	–	75	475
Heron	0	10	–	–	0	150
Heron Abut. Ext.	–	–	–	–	15	240
Heron Dike	0	15	–	–	10	130

foundation galleries are extremely limited. The first indication of excess pressure is when movement of the dam is recorded or recognized, at which time the structure and the rock have already been damaged. Therefore, the movement of the structure should not be used as a safety guide for the grouting pressures.

Pressure is used to move the grout into the rock voids to form a blockage of sufficient width to resist the reservoir pressure. The pressure is required to make the grout flow through the lines and to overcome the friction along the rock walls within the void. Grout pressure is also needed to overcome the back pressure from high ground water and from reservoir water pressure.

The objective is to apply enough pressure to make the grout flow into the rock voids without displacing the rock or creating new voids in the rock. Lifting or splitting the rock at any stage of construction may cause damage because all the voids created may not be filled with grout later. Therefore, the foundation may have less shear strength and may be more compressible after damage from grouting than it was before grouting. Even a small displacement of the rock can cause breaks in previously grouted joints in the rock, breaks in healed rock joints and bedding planes, and breaks in previously grouted horizontal and vertical construction joints in the structure. It can be concluded that a prime objective in the grouting program is to apply only the pressure needed to move the grout and fill the voids.

The key to finding safe grouting pressures is to determine the effective load and/or the effective strength needed to resist the effective grouting pressures. The effective grouting pressure includes the gauge pressure plus the pressure from the fluid in the line to the packer – that is, to the point of application. When the pressure is applied to a hole 30 feet deep with a nipple or packer set 2 feet below the surface, it must be remembered that the full uplift pressure is applied to the rock 2 feet below the surface, and that shallow rock can be lifted easily or split in a vertical direction. Flow of water or grout is not necessary for rock movement because the static pressure does the damage.

Calculations to determine the effective uplift pressure and the effective vertical load at Hungry Horse Dam are presented on figure 33 of GR-86-4. These calculations did not include any resistance to uplift caused by the strength of the rock because the area was cut with several nearly vertical faults. The permissible collar pressure computed for a safety factor of 1.0 was almost equal to 1 lb/in^2 per foot of depth to the packer; however, about 40 percent of the effective unit load was caused by the upstream rock being 70 ft higher than the base of the structure. If the structure had been based at the top of rock, the spread of the structural load at a depth of 135 ft would have reduced the computed effective unit load from that shown. It must be concluded that rules of thumb, such as 1 lb/in^2 per ft of depth to the packer, are not a reliable guide to safe pressures.

Numerous factors should be considered when computing safe pressures for grouting and water testing. Some of these factors are attitude, strike and dip of bedding and joints, strength of shear seams, fault zones, bentonite seams, temporary excavations, existing internal rock stresses, relief joints, groundwater elevation, artesian pressure, reservoir water pressure, structural load, etc. Determining the safe pressure requires a full understanding of the geologic conditions and the loads and stresses that will be applied. Collaboration of the geologist and engineer is needed to develop the maximum allowable grouting and water-test pressures throughout the entire dam.

Abutments with relief joints present a difficult problem for determining safe grouting pressures. Rock has no strength across an open joint, and the resistance to sliding for the riverside block may be the only strength across a weak shale seam or a presheared seam altered to clay. At Heron Dam, a sizeable block of rock in the abutment was moved with a relatively low grout pressure. Just filling a relief joint with fluid grout to a depth of 25 to 30 ft may cause a block of rock to slide, and adding a high pressure at the collar places the rock block in double jeopardy. Where relief joints are present, rock bolts should be installed deep into the abutment after the excavation is completed and before the grouting operations are started.

While drilling the grout holes, water under a high artesian head is often encountered in the valley bottom. The pressure on this water is sometimes so high that the grout pressure can only be raised a slight amount over the artesian pressure without lifting the rock. This condition was encountered in the foundation for Pipestem Dam, Omaha District, Corps of Engineers. These are just two examples of the problems that may be encountered.

The pressures used must be based on an analysis of the geotechnical conditions for each grouting operation and for each segment of the dam. An engineer and geologist who know the site conditions should establish the maximum grouting pressures for all holes and for all depths. Furthermore, observations should be made continuously during grouting operations to determine whether any adverse effects are developing, because unknown conditions may result in failure even at computed safe pressures.

Spacing of holes and closure. – The B-holes in the consolidation grouting were usually drilled on a predetermined spacing with occasional holes added. However, the takes achieved did not appear to be the basis for providing additional closure holes, nor did the grouting records indicate a criterion for providing additional closure holes. At Heron Dam and Dike, considerable flexibility was used in locating blanket holes, and additional holes were used as considered necessary to tighten up an area.

On the deep curtain A-holes, the primary holes were usually drilled at a spacing of 80 ft, and subsequently split-spaced to 5-ft to 10-ft spacings. No criterion for how the ultimate spacing was determined was presented in the grouting report. A review of the data on the grouting profiles indicated many places where additional split spacing may have been beneficial. In some cases, holes on either side of a hole having a high take in the lowest stage were not carried to the depth of that hole. In other cases, additional split spacing was not done below spacings of 5 or 10 ft, even though the adjacent holes had high takes. See analysis under this heading in GR-86-2 through 86-7.

Criteria should be established for field guidance on split spacing to obtain a tight foundation for the B-holes and a tight curtain with the A-holes. The Bureau's practice is not to drill holes for water-pressure tests and permeability determinations after the grouting is completed; therefore, the only guidance for further split spacing is the grout takes.

Houlsby [10] advocates the use of the Lugeon unit as a measure of permeability and as a criterion for the tightness of a grout curtain. The method of performing the pump-in tests and the formula for computing the values are presented on page 2 of the Australian *Grouting Manual* [11]. In English units: 1 Lugeon unit = 0.0107 $ft^3/ft/min$ at 142 lb/in^2.

For a test of a 20-ft-depth increment at 142 lb/in^2, one Lugeon unit would represent a take of 1.07 ft^3 in 5 min. The takes per Lugeon would vary with the pressure used and the length of the zone tested. Unless all water tests were standardized at 1 bar (approximately 15 lb/in^2) conversions would be required for each test.

Under current procedures, grouting is not attempted in a zone if the water take is less than 1 ft^3 in 5 min. This criterion would approximate 1 Lugeon unit for a 20-ft-test zone at a test pressure of 150 lb/in^2, 0.5 Lugeon unit at 300 lb/in^2, and 3 Lugeon units at 50 lb/in^2.

The use of Lugeon units as a measure of permeability implies that the rock has uniform permeability over the length tested; whereas, in reality there may be one opening of 1-inch or less that takes all the water in the 20-ft zone tested. Under current procedures no permeability calculations are made, but it is assumed that one or more open fractures will take grout if the water take is greater than 1.0 ft^3 in 5 min at the anticipated grouting pressure. The current procedure is simple and direct, and its continued use appears to be justified.

The use of Lugeon units to determine whether further split spacing is necessary would essentially require an entire extra order of split spacing. The current procedure of split spacing until a low order of grout takes is achieved is simple and direct, and the performance of the grout curtains installed by this method in this study has been satisfactory. Therefore, justification for extra drilling and testing to determine Lugeon units at the end of grouting does not appear to be necessary, except for research purposes.

Refusal Criteria. – The specifications defined refusal on a sliding scale; i.e., 1 ft^3 of mixture in 20 min at 50 lb/in^2, 1 ft^3 of mixture in 15 min at pressures from 50 to 100 lb/in^2, and 1 ft^3 of mixture in 5 min at pressures greater than 100 lb/in^2. At Heron Dam the time to refusal was reduced to 50 percent of the specified time. The times actually used at Heron are more realistic than the specified times. Because hookups are not even made to stages having a water take less than 1 ft^3 in 5 min, it does not appear reasonable to require pumping of perhaps a very thin grout mixture for 20 min to obtain a take of 1 ft^3.

Final Backfill of Grout Holes. – The flow of water from previously grouted criss-cross holes at Flaming Gorge Dam probably occurred because the holes were not filled to the surface with solid grout. The Omaha and Kansas City Districts of the Corps of Engineers had problems with water flowing from previously grouted holes at Long Beach, Smithfield, and Pipestem Dams. Soundings of many holes with a heavy steel rod indicated that there was often only a thin cap of grout at the top and that much of the remainder of the hole was filled with water.

Research was conducted on simulated grout holes in the Missouri River Division Laboratory [12]. The tests were conducted in water filled 2½-inch-diameter 30-foot-long tubes. Some tubes were held in a vertical direction and others at a 30° angle from the vertical. The results of the tests indicated that a solid tube of grout could be obtained by injecting a mixture of 1 part water to 1 part cement through a grout tube placed within 1 foot of the bottom. The 1:1 grout introduced through a tube 10 feet from the bottom or poured in at the top resulted in numerous gaps in the grout column. When grout with a water-cement ratio of 3 to 1 was placed through a tremie tube, only from slightly less than ½ to about ¾ of the tube was filled with grout. In the tests with a 3:1 mixture, the solid grout was in the bottom of the tube. The 3:1 mixture took longer to set up and had less strength than the 1:1 mixture.

The final grouting reports on the projects reviewed did not discuss the procedures used in the final backfill of the completed grout holes. The Bureau's present procedures are to sound completed grout holes with a grouting tube to determine the top of solid grout, then to inject thick grout (1:1 to 0.8:1) through the grout tube initially held close to the bottom and gradually withdrawn as the grout rises. These procedures are very good. However, these procedures and payment therefor should be stated in the specifications.

Grouting Records and Reports. – The field-record forms used by B. C. Hydro and Power Authority are included in appendix C. These forms require more information than the drilling inspector's daily report and the grouting inspector's daily report found in TM (Technical Memorandum) 646 Revised, "Pressure Grouting," dated June 1957 (figs. 35 and 36, respectively). One important chart is the mix change and grout take report, which provides considerable guidance to the inspector. Forms that require information equivalent to the information on the B. C. Hydro forms should be developed. In addition, these forms should require recording the data from the water-pressure tests for each stage.

The monthly drilling and grouting (L-10) reports contain much valuable information, and continuance of these reports is highly recommended. The final grouting report and final geologic reports relating the grouting takes to the geologic conditions are invaluable records. These reports can be used by geologists and engineers working on future dams. They would be extremely helpful if seepage problems develop later. Key personnel at the jobsite should not be transferred until these reports are completed.

Unexpected Geologic Conditions Encountered During Grouting

Unexpected geologic conditions were encountered during grouting on all six projects. On Hoover Dam, large unexpected voids were evidently encountered on the Nevada abutment because the holes had large takes and were abandoned before refusal was achieved. Also, the presence of warm springs caused flash setting of the grout in several holes. In the additional grouting from 1938-1947, mud seams, sand seams, soft and broken rock zones were encountered unexpectedly. Open vertical fissures or faults were encountered unexpectedly, and vertical grout travel in one hole exceeded 550 ft. Hydraulic connections over an elevation range of 740 ft were also encountered.

At Kortes Dam, unexpected leakage developed in the left abutment during an intermediate reservoir rise while grouting was in progress.

At Hungry Horse Dam, five additional faults were found during the foundation stripping, and a lift seam filled with clay along a bedding plane was found on the left abutment before the grouting started. The difficulty of washing out the clay of the lift seam even with closely-spaced holes was discovered during the remedial treatment. Clay was also found in some of the C-holes along the upstream toe of the dam.

At Flaming Gorge Dam, a three-fold increase from the bid quantity of 52,000 sacks to 154,318 sacks of cement used in grouting indicates that the rock had more voids than expected.

At Morrow Point Dam, additional grouting beyond the limits of the design curtain was required to control excessive seepage into the powerhouse drainage tunnel.

At Heron Dam, a large block of rock in the right abutment moved during grouting, despite relatively low grouting pressures. It was reported that the block moved on a bentonite seam. Rock bolts were used to hold the block in place as the grouting proceeded up the abutment.

Grout Takes as Related to Geology

The grout takes on all six dams could be related to the geologic conditions. At Hoover Dam, the large grout takes were usually associated with broken zones, open seams, faults, and fissures. However, there were some low grout takes, even though the drill holes made large quantities of water.

At Kortes Dam, large grout takes were associated with the intrusive diabase dike and with the sheeted zones. The grout takes were much larger in the left abutment, where high takes occurred in the water-pressure tests in the exploratory drill holes, than in the right abutment, where the exploratory drill holes had lower water takes.

At Hungry Horse Dam, high grout takes occurred in the holes alongside the faults and in the fault cutoff shafts. The disturbance of the adjacent rock by movement of the fault would be expected to result in voids that the grout filled. Some grout takes at Hungry Horse may have been caused by excessive pressure lifting the rock.

At Flaming Gorge Dam, the grout takes were usually associated with relief joints in the abutments and with a crushed zone in the valley bottom. High grout takes also occurred in the grout holes along the switchyard barrier curtain. The limited number of exploratory holes in the vicinity of the barrier curtain had drill water losses and high takes in the water-pressure tests within the depth zone grouted.

At Morrow Point Dam, the grout takes were usually associated with relief joints in the abutments and with a halo of fractured rock in the valley bottom, which probably resulted from unloading as the overlying rock was removed by erosion.

At Heron Dam, the grout takes were usually associated with relief joints in the abutments and with jointing in the valley bottom.

Evaluation of Grouting and Drainage

At Hoover Dam, the grouting and drainage work accomplished during construction was only partly successful. Even during the initial reservoir filling, the uplift pipes indicated excessive pressure in percent of net head, and excessive seepage developed in the galleries and on the downstream Nevada abutment. The additional grouting and drainage accomplished from 1938 to 1947, were successful in reducing the uplift pressures to safe levels and in reducing the seepage to acceptable quantities.

At Kortes Dam, excessive uplift pressures developed during the early stages of operation and additional drain holes had to be installed to reduce the pressures to acceptable levels. Higher than normal pressures still exist near the downstream toe; therefore, installation of additional drains is recommended.

At Hungry Horse Dam, the grouting and drainage provisions have adequately controlled the uplift pressures and seepage quantities. A disproportionate increase in water inflow into the galleries occurs at the highest reservoir stages; however, this water inflow is related to cracks in the concrete dam joints and not to foundation seepage.

At Flaming Gorge Dam, the uplift pressures on the base of the dam are within safe levels, and the amount of seepage inflow into the galleries is very low. There are several springs on both abutments downstream of the dam, but their flows are evidently not increasing. More information is needed on the uplift pressures in the sandstone and on the pore pressures in the shale in the right abutment to evaluate the stability of the narrow ridge forming the abutment. The effectiveness of the switchyard barrier curtain on the right abutment is questionable because seepage into the switchyard is apparently passing through or bypassing the curtain. However, this seepage does not endanger the dam.

At Morrow Point Dam, additional grouting was required after the original grouting was completed to reduce the seepage into the powerplant drainage tunnel to acceptable levels. The total volume of seepage within the dam and drainage tunnels now ranges from 30 to 55 gal/min, which is characterized as "tolerable." However, fines are collecting behind one weir, possibly indicating that material is piping from a shear zone. There are no uplift measuring pipes, and installation of such pipes is recommended.

At Heron Dam and Dike, the seepage is very minor. This indicates that the grouting is effective. Piezometers installed in the foundation rock beneath the dam indicate a somewhat gradual reduction in head from the upstream side of the grout lines to the downstream toe, rather than a large reduction in head at the grout lines.

The absence of downstream seepage is no indication of the water pressure in the rock that could affect the stability of the dam. Before the failure of the Uniontown Locks and Dam Stage 1 dam cofferdam [13], no seepage was observed from the foundation rock, and observers reported that it was the driest cofferdam on the Ohio River. An exploratory hole drilled there was found to be making water before the failure, indicating there was water pressure in the rock or in coal seams. Pressure relief holes were provided for the reconstructed cofferdam because it was thought that uplift pressure contributed to the failure of the original cofferdam.

Pierre Londe [14] reported that Malpasset Dam was inspected one-half hour before its failure on December 2, 1959, and there was no indication of seepage through the ground. Rock mechanics studies conducted later indicated that this failure was caused by the increase of water pressure on a wedge of foundation rock, and that drain holes angled sharply upstream would have been required to relieve this pressure. The paper [14] closes with these statements: "In fact, positive monitoring of safety should rely on piezometers, since as Malpasset shows, the important thing to know above all else, is the distribution of the underground water pressure. Today, piezometers are installed in rock abutments of dams."

RECOMMENDATIONS

Preconstruction Geologic Investigations

(1) Perform thorough geologic investigations at the final design site.

(2) List the adverse conditions that could be present in the formations at the site and establish an investigative program to prove that the adverse conditions do not exist.

(3) Consider any lost core to be a rock defect that requires further investigation by positive means, such as examination of hole by TV camera, or by visual examination from large-diameter, calyx holes.

(4) Wherever possible use trenches, adits, and preliminary stripping to determine the depth of weathered rock and the zone of highly fractured rock.

(5) Extend explorations into foundation and abutments to determine the point where rock joints or other defects are sufficiently tight that grouting would not be needed.

(6) Interpret the geologic conditions revealed by all of the available information in the most unfavorable light; i.e., in the way the defects in the rock can most adversely affect the structure.

(7) Analyze the geologic conditions in relation to the need for and the design of the grouting and drainage provisions.

(8) Have direct communication between the geologist and the design engineer to ensure that the design engineer fully understands the site conditions.

(9) Employ consulting boards at an early preconstruction stage to permit input on the adequacy of investigations and on the design of grouting and drainage provisions and the foundation treatment.

Design of Grout Curtains and Drainage Systems

(1) The design of the grouting and drainage provisions should be based on a thorough knowledge of the geologic conditions and on an analysis of the seepage patterns that will develop after impoundment of the reservoir by the structure.

(2) The design should be based on the factors requiring grouting at the site and not on an off-the-shelf design from a previous project.

(3) Considerable engineering effort should be made in designing the grouting and drainage provisions because these items are critical to the safety of the dam and because they are on the contractor's critical path (all changes have a ripple effect throughout the remainder of the contract).

(4) The design for cement grouting should be based on knowledge of the limitations of cement grouting and should be coordinated with the foundation treatment for removal of nongroutable rock.

(5) The cement grouting should be supplemented with chemical grouting where applicable.

(6) Grouting should not be relied upon to protect erodible embankment material from being piped into bedrock openings. Sealing of all bedrock fractures under the embankment should be required under foundation treatment.

(7) Consideration should be given to providing drains in the downstream valley walls for both concrete and earth dams to prevent instability after impoundment.

(8) Provide a second line of drain holes near the downstream toe for concrete dams with wide bases.

(9) Require the consulting board to review the grouting and drainage provisions.

Specifications

Provide sufficient details in the specifications so that the contractor knows which equipment, supplies, grout mixes, etc., are expected.

Cement Pay Item and Estimated Quantities

(1) The bid item should be based on a single price for all cement.

(2) To develop the estimated cement quantity, extensive studies should be made of all geologic data and water-pressure tests in the exploratory drill holes and of takes on other projects having similar formations.

Drilling Methods

Rotary drilling with water should be used as the standard method, but other drilling methods and other circulatory and cleaning mediums should be considered for special situations.

Grouting Method

(1) Ascending-stage grouting with packers should be used as the standard method, but other methods should be considered for special situations.

(2) Where communications between holes occurs, pretesting and presetting packers in the advance holes, as at Amistad Dam, should be considered.

Water-Pressure Tests

The takes in the water-pressure tests should be used as a guide in selecting the initial grout mixes.

Grout Mixes

(1) Grout mixes should be progressively thickened, depending on the pressure buildup and take of the hole, so that the thickest possible grout is injected.

(2) The spread of thin grouts should be checked before large quantities are injected.

(3) Extremely thin grouts, as used at Hoover Dam, should be used only where there is a demonstrated need.

Grout Injection Pressures

(1) Engineers and geologists should determine maximum permissible grouting pressures (and water-test pressures) on the basis of effective unit loads, and the effective shear strength in the direction of force versus the effective grouting pressure at the point of application.

(2) The permissible maximum grouting pressures should be determined for selected segments of the dam for the various geotechnical and loading conditons and for each depth of packer setting for each selected segment.

(3) Observations and measurements should be made to determine whether adverse effects are developing that would indicate the pressures should be lower than the computed values.

(4) Abutments with relief joints or weak shear seams at an unfavorable angle should be secured with rock bolts before water-pressure testing and grouting.

Spacing of Holes and Closure

(1) Criteria should be developed for determining the final spacing of holes and the closure, but all final decisions should be made by the engineers and geologists at the project site.

(2) Lugeon units should not be used as a basis for final closure.

Refusal Criteria

Refusal criteria should be defined in the specifications, as actually used at Heron Dam.

Final Backfill of Holes

The method of sounding and backfilling completed grout holes and payment therefor should be established in the specifications.

Grouting Records and Reports

(1) The preparation of monthly drilling and grouting (L-10) reports should continue.

(2) Final grouting and final geologic reports should continue to be written by job-site personnel.

(3) Data required by the B. C. Hydro forms in appendix C should be considered in any revision of field forms.

Grout Takes as Related to Geology

This subject should be covered in detail in the final geologic report and should include an analysis of grout takes versus water takes in the pressure tests of the grout holes.

Evaluation of Grouting and Drainage

(1) An evaluation (to date of report) of the grouting and drainage should be provided in either the final grouting report or the final geologic report.

(2) In addition to uplift pipes at the base of concrete dam, piezometers should be installed at the depth in zones critical to the stability.

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APPENDIXES

APPENDIX A

SECTION 19, CONSTRUCTION REPORT ON FOUNDATION GROUTING FOR DAM INTAKES, SPILLWAY AND DAM – PEACE CANYON SITE ONE, JUNE 1978

Prepared jointly by B. C. Hydro Construction Division
and Knight & Piesold, LTD. Vancouver, Canada

SECTION 9 – MISCELLANEOUS

9.1 GROUT MIXES

Table 10 shows the grout mixes used for the grouting program under the Phase A Structures. The grout batching was measured by volume (i.e. cu. ft. of water and bags of cement) as opposed to batching by weight. Control of the grout mix proportions was achieved with a water meter on the supply line that read in cubic feet and the cement was supplied in 80 lb bags. Batching by volume simplified the calculations required for thinning and thickening of the residual mixes in the agitating tank which was also graduated in cubic feet. All water:cement ratios referred to in this report are by volume.

9.2 WATER TESTING

Extensive water testing was done to many of the foundation grout holes prior to grouting. The results of the water testing for the Dam-Intakes upstream and downstream barrier are shown in Table 11. This testing was done at 40 psi in multiple packer settings to isolate the areas below BP–4, at BP–4, and above BP–4. In many cases the water acceptance rate at BP–4 was so high that a pressure of 40 psi could not be attained. The testing was done with single packers set in ascending stages although some of the holes were done with a double packer arrangement. The water testing showed BP–4 to have a high acceptance rate in every hole of Blocks D2, D3, D4, D5 and D6. Block D1 was tight at BP–4, with the exception of the upstream hole closest to Block D2. The water tests show BP–X to be opening up at the downstream right corner of D3. BP–X is open at the downstream end of Block D4 and D5 is open both upstream and downstream. Block D6 had no downstream barrier holes, however, the upstream primary holes showed BP–X to be open. The water test results of BP–X fluctuate between 0 and 1.56 cu. ft./min. indicating the plane to be discontinuous or tight in places. The largest water acceptances at the contact were in Blocks D1 and D5. Bedding Planes 1 and 2 are remaining under Block D1 in the area of high water acceptance.

Table 12 shows the tabulation of water testing done in Spillway Blocks S2 and S3. The upstream barrier testing was done by the same methods as the Intake barrier holes. The testing of the grid holes (PS-series) was done in accordance with the instructions of Drawing A–10 of Appendix A. These tests showed BP–X to be open in the same manner as Blocks D4, D5 and D6 of the intakes (i.e. fluctuating readings). The upstream barrier holes showed BP–4 to be tight in S2 and S3, however, one must consider the heavy grouting previously in the adjacent Block S1. The water tests in the grid were done with a single stage combining the contact area and BP–4. As such the results for BP–4 cannot be separated from the contact. The water test results for the 33 psi tests were adjusted to 40 psi for plotting on Figure 1.

Table 13 shows a summary of the water tests and solids acceptances for various areas of the foundation. This data was used to plot Figure 1 which shows a plot of the average water acceptances vs: the average cement solids injected, for various areas. Average acceptances were used because of the large grout spreads. That is a hole could have a large water acceptance prior to the grouting of adjacent holes, and then be grouted by interconnection with other holes.

Average solids injected were used as opposed to average grout acceptances because of the varying

SITE 1 DEVELOPMENT
STANDARD GROUT MIXES
USED FOR 1977-1978 FOUNDATION GROUTING PROGRAM

NOMINAL MIX NO.	WATER cu. ft.	CEMENT Bags	YIELD cu. ft.	W/C RATIO	
				By Volume	By Weight
6	6	1	6.44	6:1	4.68:1
5	5	1	5.44	5:1	3.9 :1
4	4	1	4.44	4:1	3.12:1
3	3	1	3.44	3:1	2.34:1
2	2	1	2.44	2:1	1.56:1
1	1	1	1.44	1:1	0.78:1
½	1	2	1.88	0.5:1	0.39:1

TABLE 10

SITE ONE DAM INTAKES
SUMMARY OF WATER TEST DATA OF u/s and d/s BARRIERS HOLES
DRILLED PRIOR TO GROUTING

HOLE NO.	WATER ACCEPTANCE, cu. ft./min (GROUT ACCEPTANCE, cu. ft.) BELOW BP-4	AT BP-4	ABOVE BP-4	REMARKS
Block D1				
P4	0.0 (0.0)	0.09 (0.0)	0.37 (48)	u/s Barrier
P8	0.0 (0.0)	0.0 (0.0)	2.06 (64)	"
P12	0.0 (0.0)	0.17 (0.0)	0.95 (27)	"
P16	0.04 (0.0)	0.78 (19)	0.0 (0.0)	"
P2008	0.0 (0.0)	0.0 (0.0)	0.02 (0.0)	d/s Barrier
P2016	0.0 (8)	0.0 (0.0)	0.03 (0.0)	"
D1 Average	0.0 (1.3)	0.17 (3.0)	0.58 (23.0)	Averaged over 6 holes
Block D2				
P20	0.0 (0.0)	1.04 (39)	0.0 (0.0)	u/s Barrier
P24	0.02 (0.0)	2.5 (120)	0.0 (0.0)	"
P28	0.0 (0.0)	2.84 (48)	0.0 (0.0)	"
P32	0.0 (0.0)	2.88 (26)	0.0 (0.0)	"
P36	0.04 (0.0)	0.94 (16)	0.0 (0.0)	"
P2024	0.02 (0.0)	2.8 (112)	0.86 (0.0)	d/s Barrier
P2032	0.06 (0.0)	2.64 (111)	0.12 (0.0)	"
D2 Average	0.02 (0.0)	2.23 (67.4)	0.14 (0.0)	Averaged over 7 holes
Block D3				
P40	0.0 (0.0)	3.14 (111)	0.0 (37)	u/s Barrier
P44	0.1 (0.0)	2.67 (216)	0.0 (0.0)	"
P48	0.0 (0.0)	2.84 (221)	0.0 (0.0)	"
P52	0.0 (0.0)	2.8 (0.0)	0.0 (25)	"
P56	0.0 (0.0)	3.1 (113)	0.0 (0.0)	"
P2044	0.0 (0.0)	2.0 (0.0)	0.0 (0.0)	d/s Barrier
P2052	0.64 (0.0)	3.4 (0.0)	0.02 (0.0)	"
D3 Average	0.11 (0.0)	2.85 (94.4)	0.0 (8.9)	Averaged over 7 holes
Block D4				
P60	0.12 (0.0)	2.4 (0.0)	0.04 (0.0)	u/s Barrier
P64	0.0 (0.0)	2.2 (0.0)	0.0 (0.0)	"
P68	0.0 (0.0)	2.3 (51)	0.0 (0.0)	"
P72	0.04 (0.0)	3.16 (92)	0.0 (0.0)	"
P76	0.04 (0.0)	2.9 (0.0)	0.0 (0.0)	"
P2064	0.12 (0.0)	3.6 (116)	0.72 (0.0)	d/s Barrier
P2072	1.48 (17)	0.28 (10)	0.44 (0.0)	"
D4 Average	0.26 (2.4)	2.4 (38)	0.17 (0.0)	Averaged over 7 holes
Block D5				
P80	1.02 (0.0)	0.78 (0.0)	0.0 (0.0)	u/s Barrier
P84	0.0 (0.0)	2.46 (100)	2.06 (0.0)	"
P88	0.0 (0.0)	2.12 (185)	0.0 (0.0)	"
P92	0.4 (0.0)	2.82 (60)	1.04 (0.0)	"
P96	1.56 (16)	1.10 (35)	0.0 (8.0)	"
P2084	1.36 (18)	3.65 (92)	0.18 (0.0)	d/s Barrier
P2092	1.32 (0.0)	3.36 (0.0)	2.98 (8)	"
D5 Average	0.81 (5.0)	2.32 (67.4)	0.89 (2.3)	Averaged over 7 holes
Block D6				
P100	1.48 (0.0)	1.16 (13)	0.24 (0.0)	u/s Barrier
P104	0.4 (8)	0.0 (0.0)	0.3 (12)	"
D6 Average	0.76 (4)	0.58 (7)	0.27 (6)	Averaged over 2 holes

Notes:

All testing done at 40 psi if possible

TABLE 11

WATER TEST RESULTS
S2 and S3 V/s Barrier

HOLE NO.	ABOVE BP-4		BP-4		BELOW BP-4		REMARKS	
	W.A.	G.A.	W.A.	G.A.	W.A.	G.A.		
P120	0.0	0.0	0.0	0.0	0.0	0.0	S2	u/s barrier may have
P124	0.0	0.0	0.26	0.0	0.0	0.0	"	been contaminated by
P128	0.0	0.0	0.0	0.0	1.2	277	"	S1 grouting
P132	0.4	0.0	0.0	0.0	1.0	0	S3	
P136	0.0	0.0	0.48	0.0	1.6	13	"	
P140	0.0	0.0	0.0	0.0	0.0	0.0	"	
S2,S3 Ave	0.6	0.0	0.12	0.0	0.63	48	Averaged over 6 holes	

S2 and S3 Grid
Stage 1 (Includes contact and BP-4)

Pressure PSI	HOLE NO. PS – WATER ACCEPTANCE IN cu. ft./min.									Ave W.A.	90 In-crease	Grout accept Ave GA/Hole Primaries
	404	412	420	424	824	1012	1204	1220	1224			
33	1.77	2.6	0.22	0.11	0.75	3.15*	1.0+	0.87	0.95	1.27	–	25
67	2.52	3.6	0.42	0.2	1.0	4.1**	–	0.29	1.24	1.80	42%	
100	3.23	7.4*	0.78	0.3	1.4	4.1**	–	1.52	1.64	2.55	100%	
STAGE 2 (below BP-4)												
33	0.28	0.04	0.005	0.1	0.41	0.63	0.19	0.82	1.04	0.39	–	23
67	0.23	0.06	0.007	0.2	0.55	0.85	0.26	1.14	1.31	0.51	31%	
100	0.51	0.18	0.19	0.3	0.77	1.13	0.56	2.74	1.54	0.88	126%	

NOTES

1. * Maximum pressure obtainable was 70 psi
2. ** Maximum pressure obtainable was 50 psi
3. + Packer leaking and unable to seal
4. Calculation for adjustment from 33 psi to 40 psi
   Average % increase in W.A. from 33 psi to 67 psi is approximately 1%
   to adjust from 33 psi to 40 psi 1.08 (W.A. at 33 psi) = W.A. at 40 psi

   Average W.A. at Stage 1 = 1.27 (1.08) = 1.37 for 40 psi
   Average W.A. at Stage 2 = 0.39 (1.08) = 0.42 for 40 psi

TABLE 12

SUMMARY OF AVERAGE WATER TESTS
FOR U/S AND D/S BARRIERS AND SPILLWAY S2 and S3 GRID

Location of Water Tests	No. of Holes Water Tested	No. of Holes Grouted	Total Solids	Total W.A.	Ave Solids Injected/Hole	Ave W.A./Hole cu. ft./min.
U/S BARRIER						
Primaries						
Below BP-4	37	37	168	9.32	4.54	0.25
At BP-4	37	37	456	49.89	12.05	1.35
Contact	37	37	49	7.86	1.32	0.21
Secondaries						
Below BP-4	35	35	28	4.41	0.8	0.12
At BP-4	35	35	17	2.28	0.49	0.06
Contact	35	35	0	1.48	0	0.04
D/S BARRIER						
Primaries						
Below BP-4	10	24	8	5.0	0.33	0.5
At BP-4	10	24	280	21.8	20.0	2.18
Contact	10	24	6.5	5.37	0.24	0.54
SPILLWAY GRID						
Primaries S2 and S3						
Below BP-4 (Stage 2)	9	28	228	3.78	8.14	0.42
BP-4 and Contact (Stage 1)	9	47	325.5	12.33	6.92	1.37

NOTES

1. The water tests and cement acceptances are averaged over the number of holes in an area.
2. Data extrapolated from Drawings B-3 and B-18 of Appendix B and Table 3 of miscellaneous section.

TABLE 13

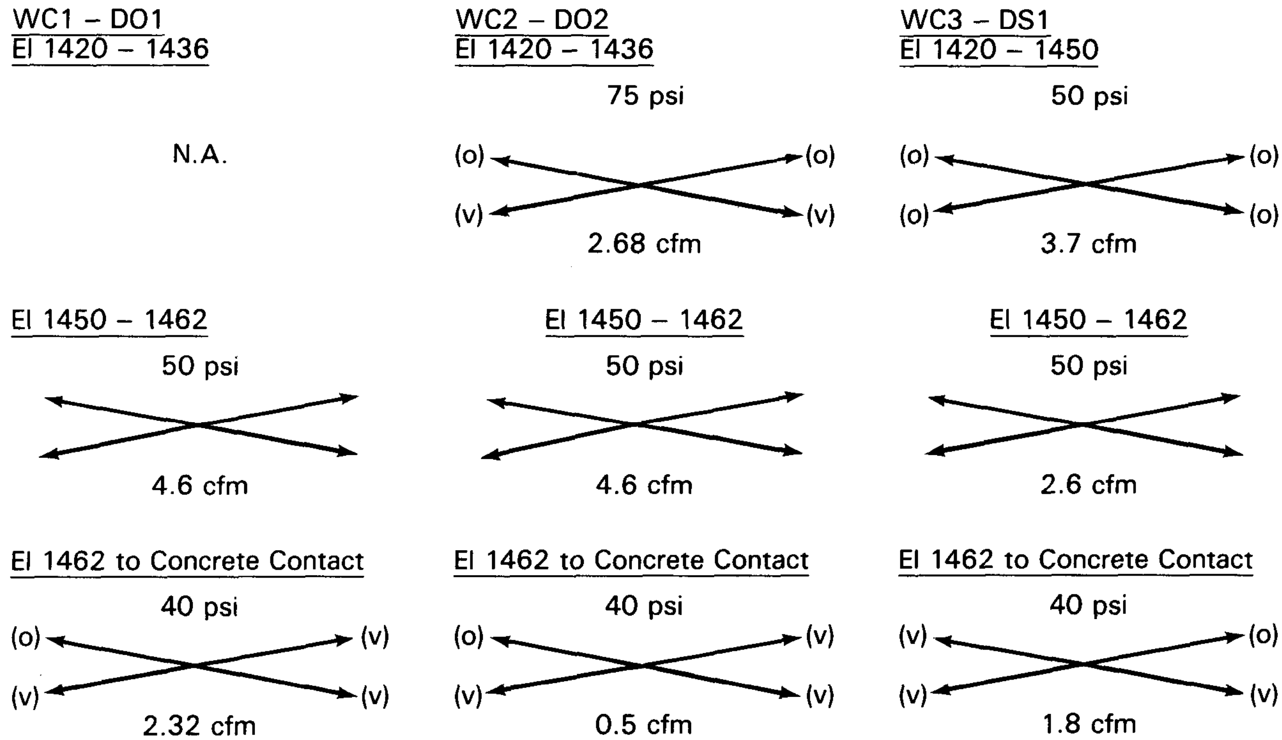
mix consistencies. The plot shows a good relationship between the water acceptances and solids acceptances.

The water tests from individual Block S1 holes were not plotted because of the larger variation in pressures used in conducting the tests (0 to 60 lb/in^2).

The large variation in pressures were due to the large acceptance rates in the area. The pressure would not build to 40 lb/in^2 in many areas.

The wash cell test data in Block S1 depicted below:

(v) indicates water return observed, and;
(o) indicates no water return



The 4.6 cfm readings at BP–4 are among the highest recorded on site.

9.3 GROUT SETTING TIMES VS: $CaCl_2$ AND TEMPERATURES

Tests were performed at the concrete laboratory on site to determine the effects of calcium chloride on the setting time type 30 cement at varying temperatures. Table 14 summarizes the results of the tests. The cooler temperatures delayed the setting times on the neat grout. On the grout that was mixed without $CaCl_2$ the temperature had a significant effect on the setting times, this effect was further increased when $CaCl_2$ was added to the grout. At 4 percent $CaCl_2$ the initial setting time was reduced to 10 minutes. This would not allow enough time for a grouting sequence to be completed. The initial grout setting time of one hour at 2 percent $CaCl_2$ was considered marginal. The only place $CaCl_2$ was used in the foundation grouting program was in Spillway Block S1. The $CaCl_2$ was used in the grout to seal off the contact prior to the concrete repairs following the foundation grouting program. The large number of open holes and considerable back venting to previously grouted holes was the reason for going to a faster setting time. The amount of $CaCl_2$ added was 2 percent. Two upstream barrier holes in S1 were also grouted with the $CaCl_2$ additive for the same reason.

9.4 GROUT STRENGTHS AS AFFECTED BY FLOURESCENE DYE

Table 15 summarizes a series of tests conducted using varying percentages of flourescene as a dye. The testing showed that very small amounts of this organic compound significantly reduced the strength of the grout. An additional feature was that the brightly coloured green dye lost its colouration once the grout had set. Similar tests were performed with Phenolthaleon dye and the results were similar. Neither of these dyes were used in the grouting on site.

9.5 GROUT STRENGTHS AT LOW TEMPERATURE FOR TYPE 10 AND TYPE 30 CEMENT

Table 16 summarizes the results of tests conducted on site to investigate the strength of type 10 and type 30 cement grout mixed and cured at lower temperatures. The tests showed much higher early strengths for the type 30 cement used in the grouting program. Type 30 cement was the only cement used in the program.

GROUT SETTING TIMES
vs:
TEMPERATURE AND CALCIUM CHLORIDE

GROUT MIX			% CaCl	MIXING & CURING TEMPERATURES °F	INITIAL SET hrs-min	FINAL SET hrs-min
Cement gm	Water gm	CaCl gm				
500	134	0	0	70	1:50	2:50
500	134	0	0	68	2:20	3:10
500	134	0	0	45	3:10	4:10
500	134	0	0	39	4:10	6:10
500	134	2.5	0.5	40	1:55	3:20
500	142.5	5.0	1.0	40	1:45	2:45
500	142.5	10.0	2.0	40	0:50	1:15
500	142.5	10.0	2.0	70	1:00	1:20
500	147.5	20.0	4.0	40	0:10	0:25
500	147.5	20.0	4.0	70	0:15	0:25

NOTES

1. Mixes are approximately 28.5% water by weight.
2. Cement – Type 30 Portland.
3. Samples moist cured at 100% humidity.

TABLE 14

GROUT STRENGTHS
vs:
FLOURESCENE

SAMPLE NO.	MIX = 2:1 BY WEIGHT				GROUT STRENGTHS (psi)		
	Cement gms	Water gms	Flourescene gms	Flourescene oz/bag of C	3 day	7 day	28 day
1	363.2	566.6	Nil	Nil	62.5	1000	1231
2	726.4	1133.2	0.3	½ oz (1 tsp)	53.6	679	933
3	363.2	566.6	0.3	1 oz	45.5	600	867
4	726.4	1133.2	1.1	2 oz	41.7	567	833
5	363.2	566.6	1.1	4 oz	32.1	406	656

TABLE 15

GROUT COMPRESSIVE STRENGTH RESULTS
for
TYPE 30 CEMENT AND TYPE 10 CEMENT

Age Days	TYPE 30 CEMENT		Strength psi	Age Days	TYPE 10 CEMENT		Strength psi
	Area sq. in.	Load lbs			Area sq. in	Load lbs	
7	2.625	1200	460	7	3.0	300	100
7	2.625	1300	500	7	3.0	400	130
14	2.625	3100	1180	14	3.0	900	300
14	2.625	2900	1100	14	2.75	1100	400
28	2.75	3500	1270	28	2.75	1800	660
28	2.625	3400	1300	28	2.625	1700	650

NOTES

1. At 3 days the grout was not as yet strong enough for testing.
2. Settlement of grout left 37.5% water at the top of cube.
3. Mixes have 2:1 w:c ratio by weight.
4. Grout Mixed and Cured at 6°C.

TABLE 16

9.6 SETTLEMENT IN GROUT-DYE MIXTURES

Figure 2 is a plot of the percent settlement of solids in grout as affected by iron oxide dyes. This mix used was 500 gms of type 30 cement, 75 mls of water, and 18.75 grains of dye. The mixtures using neat grout, grout with red dye, and grout with yellow dye were mixed for 15 minutes, poured into 400 millilitre 2″ diameter calibrated glass cylinders and allowed to settle. The readings were taken every 5 minutes. The results show that the dyes kept the solids in suspension longer and had a lower overall settlement at the end of the test. Grout strength tests performed on the grout using the dyes were inconclusive but did indicate a reduction in strength of the grout with the use of these red and yellow oxide dyes. These dyes were extensively used throughout the foundation grouting program.

AVERAGE WATER ACCEPTANCE
VS:
AVERAGE GROUT ACCEPTANCE

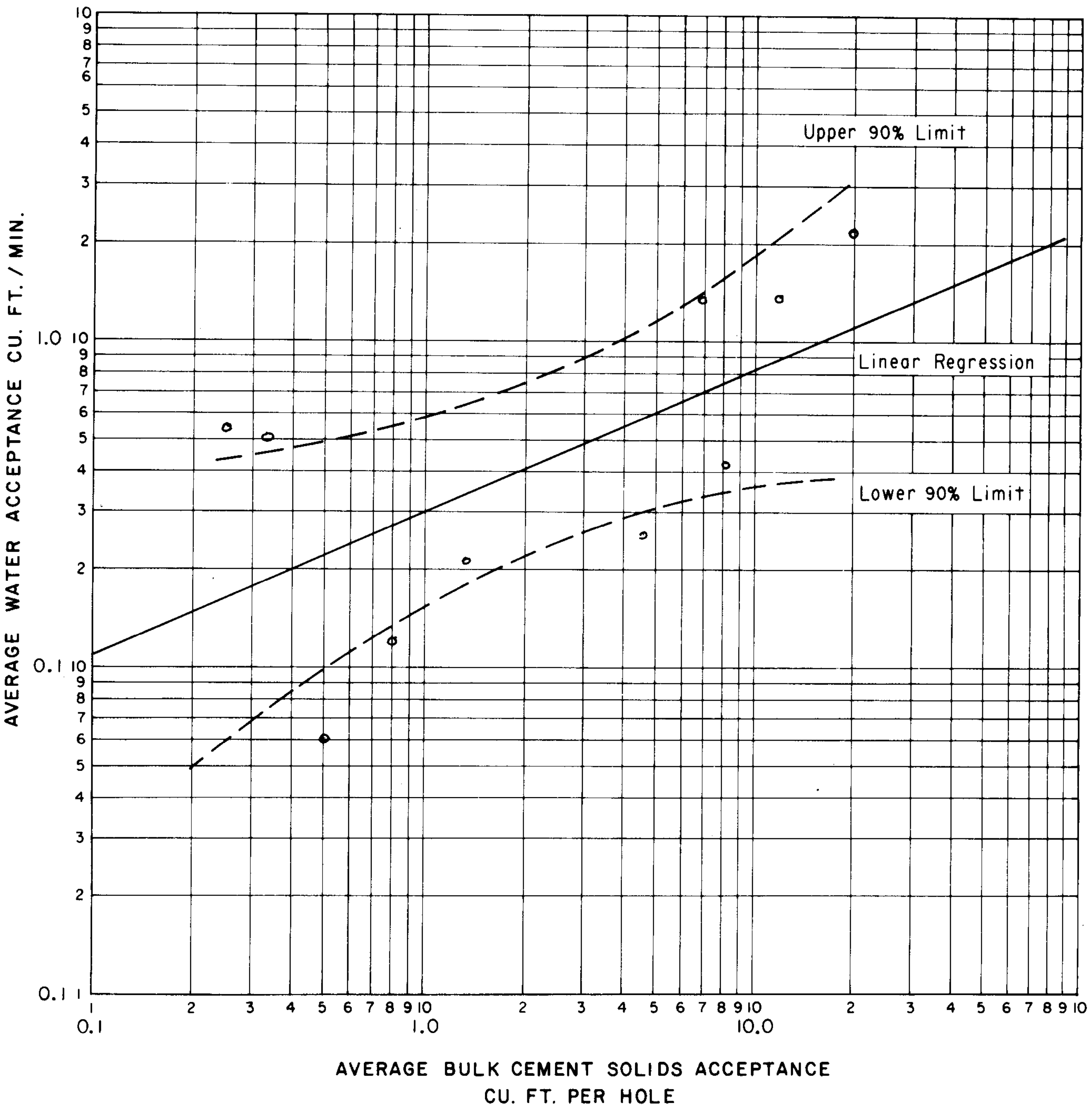


FIGURE 1

SETTLEMENT IN DYE - GROUT MIXTURES

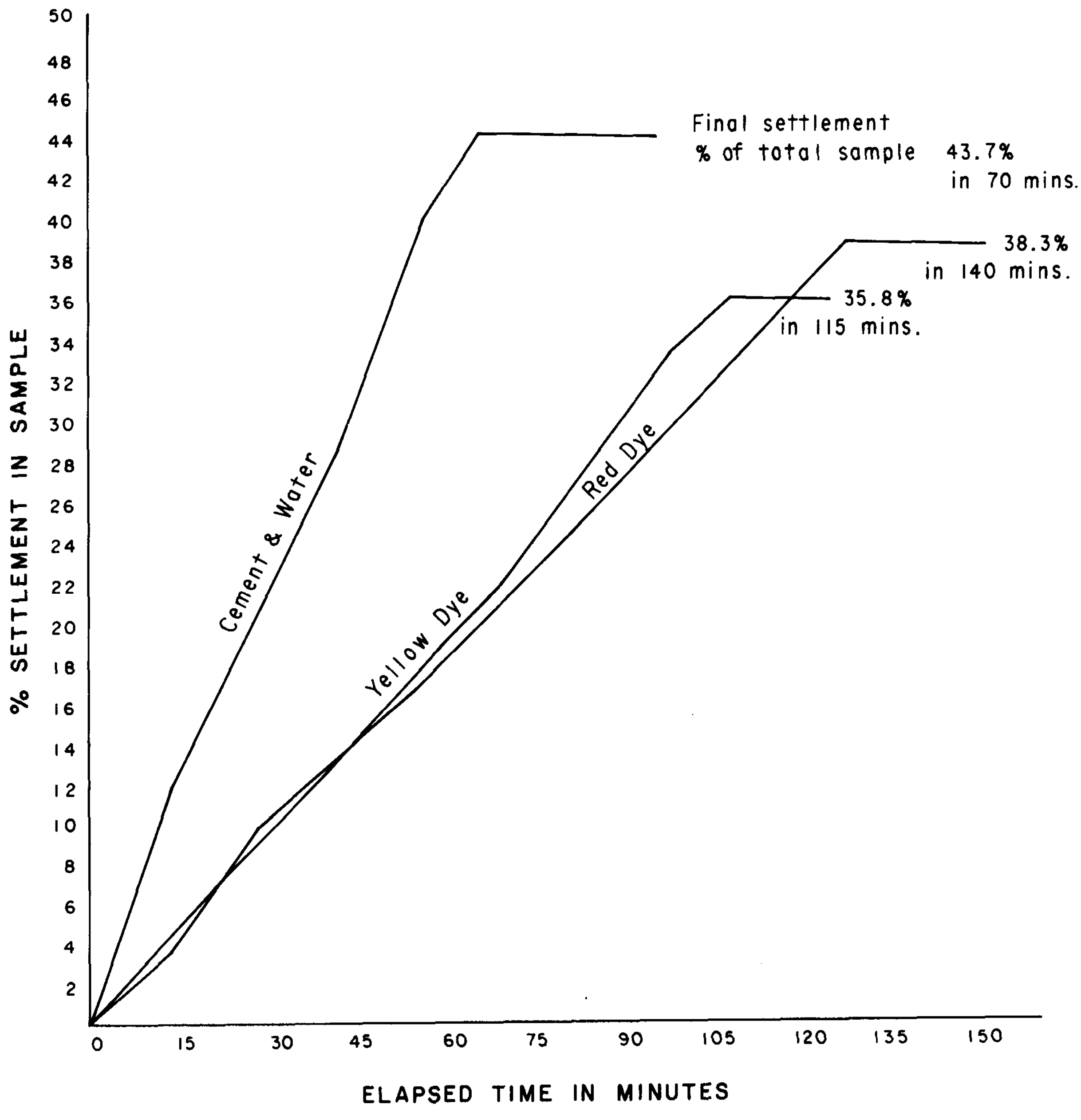


Figure 2

APPENDIX B

EXTRACTS FROM J. R. ANDERSON MEMORANDA DATED MAY 28, 1948, AND DECEMBER 14, 1949

APPENDIX B

Extract from Memorandum dated May 28, 1948 from J. R. Anderson to *Construction Engineer*, subject: Foundation Treatment Report – May 1948 – Kortes Dam and Power Plant – Missouri River Basin Project – Wyoming

METHODS OF TREATMENT – CONSOLIDATION GROUTING

Due to the jointed and fractured condition of the foundation bedrock and the variation in the spacing of the joints, a definite or detailed pattern plan of treatment holes cannot be set up for the entire foundation. The following stops are being used in executing the drilling phase of the program.

Drilling Operations:

1. First, a thorough study is made of the physical condition of the bedrock in any local area to be treated. Second, using the information gained by these studies, a pattern of holes is laid out on a 15 to 25 foot grid system that, when drilled, intersects the largest number of seams and joints. Third, when drilling operations are begun, a "BX" size ($2\frac{5}{16}$" O.D.) core bit is used to drill the top section of the hole. The purpose of this drilling is to make provisions for installing 1½ inch nipples to be used during grouting operations. The length of the nipple and depth of the "BX" drilling varies with the physical condition of the bedrock. It is expected that the maximum depth of the "BX" drilling for nipple setting will be about four feet. Fourth, when "BS" drilling operations are completed, a 1½ inch nipple is placed in the hole and the "BX" section of the hole is drilled. The placing of the 1½ inch nipple in the hole before the "EX" drilling is started makes it possible to properly center the "BX" hole at the bottom of the "BX" section. When all drilling operations have been completed, the nipple is securely anchored in place with a mixture of alummite and Portland cement. However, before the mixture of alummite and cement is placed around the nipple, a small packer is set at the top of the "BX" section of the hole. This is necessary to prevent possible leakage of the mixture at the bottom of the nipple from entering the hole.

2. All consolidation grout holes in the right abutment area are being drilled normal to the slope and to an average depth of 30 feet. All consolidation grout holes in the river channel will be drilled vertical and to minimum depth of 30 feet. It is expected that consolidation grout hole drilling operations on the left abutment will be executed in a manner similar to that being used on the right abutment.

Water Tests

1. When all drilling operations have been completed on a hole, a water test is made. These tests are conducted in the following manner. For the first test a packer is set in the hole approximately three feet from the surface. Water is pumped into the hole at this packer setting at a pressure of 50 to 100 pounds per square inch. These tests are repeated progressively downward at five foot intervals with corresponding higher pressures until a point is reached that is considered to be the top of the tight zone. The time required for each of these tests is governed by the amount of water the hole takes and the extent of surface leakage. The location of all surface leaks and the amount of water used are recorded on each of these tests. The hole and adjacent seams and joints are thoroughly washed during the testing operations.

Grouting Procedures

1. Before grouting operations are begun on a hole, a careful study is made of the results of the water tests. From these studies a procedure for grouting the hole is formulated. A zoning system is used in performing the grouting phase of the operations. From the results of the water tests the hole is divided into three zones according to the tightness of the bedrock. In most cases the tight zones appear at the bottom half or more of the hole, the moderately tight zones from approximately 6 to 15 feet from the surface, and the leaky zones from the surface to a depth of 5 or 6 feet. The tight zone is grouted first, moderately tight zone second and the leaky zone last. This method permits the grouting of small seams in the tight zones, by using thin grout mixes and higher pressures, that might otherwise be left ungrouted if the entire hole is grouted in one operation. Also, this method makes it possible to use thick grout mixes and to increase or decrease the pressure as may be necessary in grouting the moderately tight and leaky zones.

2. If, after an area has been treated using the above drilling and grouting procedures, it is considered that the spreading of the grout through all of the seams and joints was not accomplished, intermediate holes are drilled and grouted in the manner described above for the first pattern.

Extract from Memorandum dated December 14, 1949 from J. R. Anderson to *Construction Engineer*, subject: Foundation Treatment Report – September, October and November 1949 – Kortes Dam and Power Plant – Missouri River Basin Project, Wyoming.

Blanket Grouting

1. During the period a total of 24 holes with an average depth of 30 feet, was drilled in the area of the foundation to be covered by Block 2. A total of 1,511.7 cu ft of cement was used during the treating of these holes. The average cement consumption per hole was 62.9 cu ft or an average of 2.1 cu ft per lin ft of hole.

2. Certain modifications of the original treatment procedures were necessary as a result of excessive surface leakage. This direct surface leakage was particularly excessive during the grouting of the near surface zone. The opened jointed bedrock condition was first encountered during the treating of rows of holes at elevation 6045 and 6050. Preventative measures employed to alleviate the direct surface leakage above elevation 6050 was the installation of 2″ pipe on bedrock at the desired hole locations and extending on the proper bearings and inclination through, at least, 5 feet of concrete preparatory to drilling and grouting.

Cut-Off Curtain Treatment

a. Method of Treatment

1. Treatment of the cut-off curtain is being accomplished by the Split-grouting sequence series – Stop-grouting method. The split-grouting sequence series layout of this method consists of treating the first or primary series of holes at 80 foot spacing, horizontally, and progressively treating the successive first, second, third and fourth intermediate series – each successive intermediate series of holes located so as to equally split the horizontal distance between the holes of all preceding completed series. Zone or stop grouting of each hole consists of grouting at vertical intervals of 25 feet to 35 feet beginning approximately 25 feet from the bottom of the hole and progressively setting the packer at the desired injection points up the hole until the entire length of hole is grouted.

b. Grout Injection Pressures

1. Computation of allowable ranges of minimum and maximum grout injection pressures were based on the following factors:

Superimposed Structure	– 1 pound p.s.i. for each vertical foot of concrete directly above hole being grouted.
Rock Factor	– 2 pounds p.s.i. for each lin. foot of rock as measured between packer setting and nearest point of rock-concrete contact.
Previous grouting	– 50 pounds p.s.i. – added factor as a result of blanket grouting.
Series hole factor	– Pressure increased for each successive series by 12½% over the preceding series.

2. Devising a formula for the computation of definite grout injection pressures cannot be accomplished before the commencement of actual treatment operations, inasmuch, as there are many unknown governing factors not outlined in the foregoing paragraph. The nature and results of these unknown factors are revealed as the treatment operations progresses. Included below are the most important of these factors: (a) Extent of grout spreads as revealed by interconnections with other grout holes and by surface leaks outside the limits of the foundation, (b) Nature of grout spreads with reference to direction of exerted force, i.e., whether grout is traveling along one plane or several parallel planes in bedrock which produces force in one direction or, whether grout is traveling through an irregular pattern of joints where the pressures or forces are counteracting, (c) presence of groundwater, (d) rate of grout injection and (e) consistency or fluidity of grout mixture.

3. Establishing an ideal injection pressure for each injection point which will produce effective grout spreads, yet will not cause deformation or uplift required due consideration of all the factors included in the preceding paragraphs. Preparatory to the treating of the primary series of holes pressure ranges were established for all the prearranged locations of injection points based on the applicable controlling factors outlined in paragraph 1. The extremes, highs and lows, of these pressure ranges varied from 50 to 100 pounds p.s.i. The selection of the actual pressure used, appearing within the limits of the pressure ranges, was governed by the controling factors in paragraph 2.

4. To provide a means of detecting uplift or block tilting as a result of grouting operations, dial gages were installed at construction joints 2-3, 3-4, and 4-5 in the foundation gallery.

c. Grout Mixture

1. The design of suitable grout mixtures was based on the results of previous treatment operations, the results of water tests and by the results of trial investigation measures employing the use of several mix ratios. Grout mixes with water over cement ratios of 4.0 to 8.0 by volume have been used. Water tests and grouting results to date indicate that the foundation bedrock along the cut-off curtain line in Blocks 3, 4 and 5 is broken by many small and tight joints that will accept only a very thin mix.

APPENDIX C

FIELD-RECORD FORMS, B.C. HYDRO AND POWER AUTHORITY

DRILLING REPORT

These reports are of importance when abnormal conditions are encountered. Record all erratic or unusual drilling events or conditions in the remarks column. (Use the whole sheet if you need the space.) On conclusion of drilling, the hole should be washed with water under pressure.

Under remarks include notes on drilling difficulties; caving, rod drop, changes in quantity of water and color return and any other changes in the routine of steady drilling. Careful note should be made as to the color and whether any fine material is removed with any return wash water.

						GROUTING RECORD	
CONTRACTOR						CONTRACT NO.	
			DRILLING REPORT				
DATE OF START			HOLE DIA.		IN.	HOLE TYPE R ▭ P ▭	
DRILL TYPE			BIT TYPE			DRILL MOUNT	
SHIFT– N ▭ D ▭ A ▭			DESIGN DEPTH		FT.	STAGE ____ FT. TO ____ FT.	
DATE	TIME		DEPTH IN FT.		DRILL WATER RECOVERY %	REMARKS	
	FROM	TO	FROM	TO			
TOTAL FOOTAGE FOR THIS STAGE						FEET	
CONTRACTORS INSPECTOR							
						HOLE NO.	

GROUT HOLE WASHING REPORT

This form is to be used to record washing operations. The form is self explanatory. Record only the time during which equipment is actively washing. Describe colour of return water and any recovered solids and give estimate of quantity under Remarks. Delays and difficulties should also be recorded and described under Remarks.

GROUTING RECORD

CONTRACTOR: | CONTRACT NO.

GROUT HOLE WASHING REPORT

DATE – | HOLE TYPE R ☐ P ☐

SHIFT – N ☐ D ☐ A ☐ | STAGE ______ FT. TO ______ FT.

TIME FROM	TIME TO	ELAPSED TIME MIN.	REMARKS
TOTAL		MINUTES	

CONTRACTORS INSPECTOR

HOLE NO.

GROUTING REPORT

The data collected on the grouting report will be used for assessing the grouting program.

Data are to be recorded continuously from beginning to completion of a grouting stage. Fill in all the data required or explain when it is not possible to do so.

Under Remarks record such items as mixing time, grout pump pressure, equipment defects and breakdowns. For every stage grouted either fill out a surface Leakage Report or state "No surface leaks" in Remarks column of Grouting Report.

			GROUTING RECORD
CONTRACTOR:			CONTRACT NO.
	GROUTING REPORT		
DATE	HOLE ELEV. FT.	HOLE DIA. IN.	HOLE TYPE- R ☐ P ☐
SHIFT- N ☐ D ☐ A ☐	SEQ. NO.		
WEATHER	AIR TEMP. °F	GROUT TEMP. °F	STAGE ____ FT. TO ____ FT.

(DATE)TIME	W/C	MIX NO.	CEMENT LB.	FLUID LB.	SAND C.F.T.	CU.FT. INJECT	HEADER PRESSURE P.S.I.	FLOW CONE SEC.	REMARKS (ALWAYS SHOW PACKER DEPTH)
TOTALS									

CONTRACTORS INSPECTOR	CEMENT TYPE:
	FLUIDIFIER;
	HOLE NO. ________________

SHEET ______ OF ______

MIX CHANGE AND GROUT TAKE REPORT

The chart has been attached for illustration purposes only. For charts to be used, refer to Drawing 1007-C21-D6409.

Grouting of all holes shall commence initially with 5:1 grout mix ("50"-see Note 5). After 5 minutes active grouting the take (Not including grout used for filling lines or hole) is measured. An "X" marked in the first "time" column i.e. under the 5, and opposite the take measured. Grouting is continued using the mix shown in the square to the right of the "X."

After a further 5 minutes the take, for the 5 minutes, is measured. An "X" is now placed in the second time column, i.e. under the 10, and opposite the take measured. Grouting is continued using the mix shown in the square to the right of the last "X."

The above procedure is repeated across all the time columns shown or until the take falls below ½ cu. ft. in 5 minutes.

When the take falls below ½ cu. ft. in 5 minutes the "X" is marked over the figure for the active grouting time in minutes, the hole shall be considered to have refused and grouting shall be terminated.

When the take has not fallen below ½ cu. ft. in 5 minutes, when the "Xs" reach the right side of the chart grouting shall be suspended and the hole shall be washed out after the initial set has taken place.

This form shall be completed for every stage or part of stage grouted.

Do not use partial batches. Change mix between regular size batches.

MIX CHANGE PROCEDURE AND GROUT TAKE PATTERN

GROUT TAKE CU. FT. FLUID IN 5 MIN	ACTIVE GROUTING TIME IN MINUTES SINCE INJECTION COMMENCED																									
	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130
1/2-1	50	50	50	50	50	50	50	50	50	50	50	50	50	50	50	50	50	50	30	30	30	20	20	10	8	6
1-2	50	50	50	50	50	50	50	50	50	50	50	50	50	50	50	50	30	30	30	30	30	20	10	8	6	
2-3	50	50	50	50	50	50	50	50	50	50	50	50	50	50	30	30	30	30	30	20	20	10	8	6		
3-4	50	50	50	50	50	50	50	50	50	50	50	50	50	30	30	30	30	30	30	20	10	8	6			
4-5	50	50	50	50	50	50	50	50	50	50	50	50	30	30	30	30	30	20	20	10	8	6				
5-6	50	50	50	50	50	50	50	50	50	50	30	30	30	30	30	30	30	20	10	8	6					
6-7	50	50	50	50	50	50	50	50	50	30	30	30	30	30	30	20	20	10	8	6						
7-8	50	50	50	50	50	50	50	50	30	30	30	30	30	20	20	20	10	8	6							
8-9	50	50	50	50	50	50	50	30	30	30	30	20	20	20	20	10	8	6								
9-10	50	50	50	50	50	50	30	30	30	20	20	20	20	20	10	8	6									
10-11	50	50	50	50	50	50	30	30	20	20	20	20	20	10	8	6										
11-12	50	50	50	50	50	30	30	20	20	20	20	10	10	8	6											
12-13	50	50	50	50	30	30	20	20	20	10	10	10	8	6												
13-14	50	50	50	50	30	30	20	20	10	10	10	8	6													
14-15	50	50	50	30	30	20	20	10	10	10	10	8	6													
15-16	50	50	50	30	20	20	10	10	10	8	6															
16-17	50	50	30	30	20	10	10	10	8	8	6															
17-18	50	50	30	20	20	10	10	8	8	6																
18-19	50	50	30	20	10	10	8	8	8	6																
19-20	50	30	20	20	10	8	8	6	6																	
20-21	50	30	20	10	8	8	6	6																		
21-22	50	30	20	10	8	6	6	6																		
22-23	50	30	20	10	8	6	6	6																		
23-24	50	30	20	10	8	6	6	6	6																	
24-25	50	30	20	10	8	6	6	6	6																	
25+	50	30	20	10	8	6	6	6																		

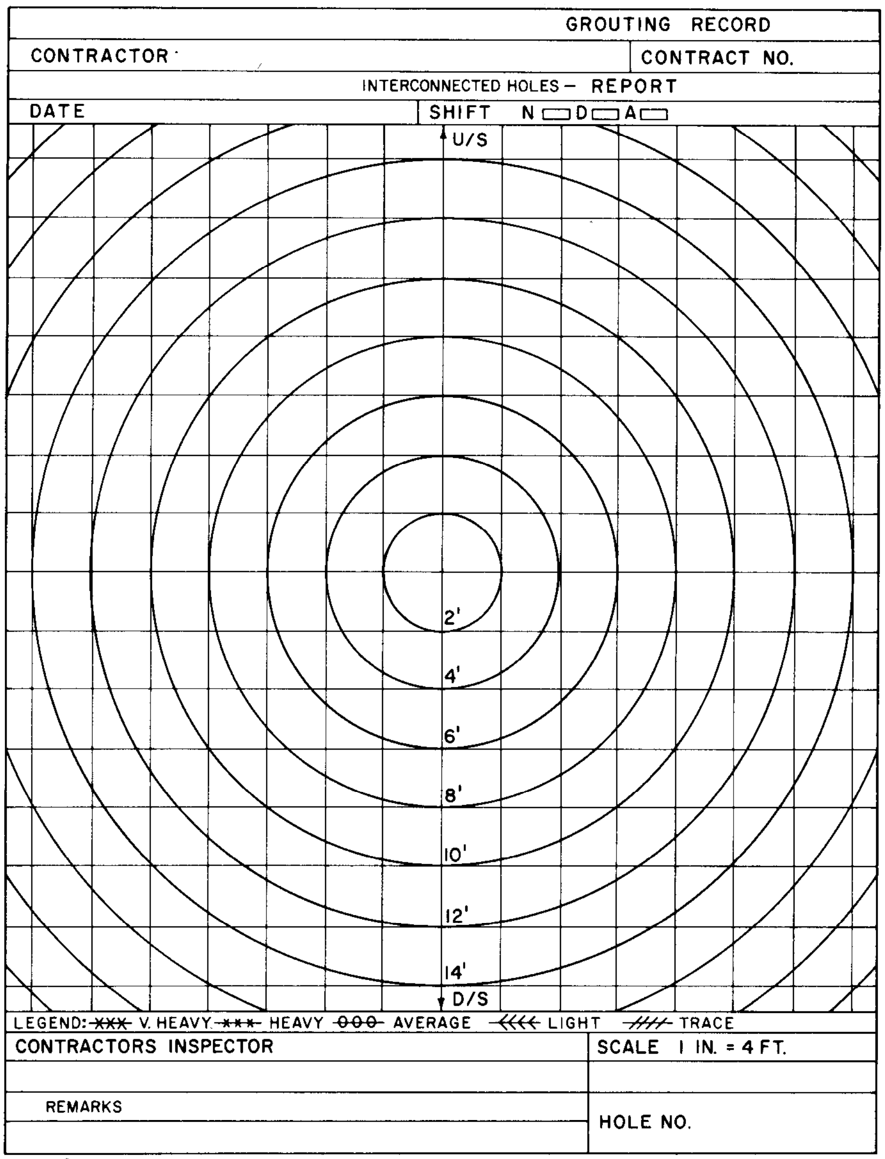
NOTES:
1. DO NOT REVERT TO THINNER MIXES AS FLUID TAKE DECREASES.
2. MAXIMUM ALLOWABLE PRESSURE IS TO BE DEVELOPED AS SOON AS POSSIBLE AND MAINTAINED UNTIL INJECTION IS STOPPED.
3. STOP GROUTING WHEN RATE OF TAKE FALLS BELOW ONE HALF CU. FT. IN 5 MINUTES. THIS IS REFUSAL.
4. WHERE REFUSAL IS NOT OBTAINED WITHIN THE MIXES AND TIMES SHOWN ON THE CHART STOP GROUTING. AFTER INITIAL SET HAS TAKEN PLACE, WASH OUT THE HOLE.
5. MIX NUMBERS ARE RATIOS OF WATER TO 1 PART OF CEMENT BY WEIGHT. MULTIPLIED BY 10.

DATE	
STAGE: FROM	TO
HOLE NO.	

INTERCONNECTED HOLES REPORT

This form is to be used for recording any grout flow between holes.

The hole being grouted is at the centre of the "target" on the report form. The lines on the form are radial and grid scales to indicate the location of interconnections. When an interconnection occurs assess its intensity and plot its location on the grid using the symbols shown in the legend.


GROUTING RECORD
CONTRACTOR
CONTRACT NO.
INTERCONNECTED HOLES – REPORT
DATE
SHIFT N D A
U/S
2'
4'
6'
8'
10'
12'
14'
D/S
LEGEND: -XXX- V. HEAVY -***- HEAVY -OOO- AVERAGE -<<<<- LIGHT -////- TRACE
CONTRACTORS INSPECTOR
SCALE 1 IN. = 4 FT.
REMARKS
HOLE NO.

Form R8

BACKFILLING REPORT

This form is to be used for the completion of each grout hole.

If the volume of grout backfill exceeds the net volume of the hole, check for possible causes and record.

						GROUTING RECORD
CONTRACTOR :						CONTRACT NO.
BACKFILLING REPORT						
DATE		SHIFT- N☐ D☐ A☐			AIR TEMP	°F
TIME	MIX NO.	CEMENT LB.	FLUID LB.	SAND C.F.T.		R E M A R K S
TOTALS						
CONTRACTORS INSPECTOR						
						HOLE NO.

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The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.

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