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Field Grouting Summary Report on the WAG 4 Seeps 4 and 6 Removal Action Project, Oak Ridge National Laboratory, Oak Ridge, Tennessee

Volume 1. Text

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ECO Grouting Specialists Limited

contributed to the preparation of this document and should not be considered an eligible contractor for its review.

ORNL/ER-401/V1

Field Grouting Summary Report on the WAG 4 Seeps 4 and 6 Removal Action Project, Oak Ridge National Laboratory, Oak Ridge, Tennessee

Volume 1. Text

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Prepared by
ECO Grouting Specialists Limited
Cheltenham, Ontario, Canada
under subcontract 12X-ND857V

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PREFACE

This ***Field Grouting Summary Report on the WAG 4 Seeps 4 and 6 Removal Action Project at Oak Ridge National Laboratory, Oak Ridge, Tennessee*** **(ORNL/ER-401/V1–V3)** was prepared as part of the Oak Ridge National Laboratory Environmental Restoration Program removal action at Seeps 4 and 6 in Waste Area Grouping 4 at Oak Ridge National Laboratory. This report satisfies requirements of U.S. Department of Energy (DOE) Order 4700.1 as amended by DOE Order 430.1 dated August 1995. It was prepared under Work Breakdown Structure 1.4.12.6.1.04.21, Activity Data Sheet 3300, "ORNL WAG 4 Seeps 4 and 6."

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ACRONYMS AND ABBREVIATIONS

API	American Petroleum Institute
Bpf	blows per foot
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act of 1980
cm/sec	centimeters per second
cm	centimeter
^{60}Co	cobalt 60
^{137}Cs	cesium 137
DOE	U. S. Department of Energy
ECO	ECO Grouting Specialists Limited
F	Fahrenheit
ft	foot or feet
ft-lbs	foot-pounds
gal	gallon or gallons
GIS	geographical information system
gpm	gallons per minute
^{3}H	tritium
HAZWOPER	hazardous waste operations and emergency response
in	inch or inches
K_{pf}	pressure filtration coefficient
LMER	Lockheed Martin Energy Research
LMES	Lockheed Martin Energy Systems
lpm	liters per minute
mg/l	milligrams per liter
NAK	sodium and potassium
ORNL	Oak Ridge National Laboratory
ORR	Oak Ridge Reservation
PPE	personal protective equipment
psi	pounds per square inch
RAD	radiologically contaminated
rpm	revolutions per minute
RWP	radiation work permit
SLLW	solid low-level waste
^{90}Sr	strontium 90
SWSA	solid waste storage area
Th	thorium
TRU	transuranic
U	uranium
WAG	waste area grouping

ACKNOWLEDGMENTS

Project management and technical oversite were performed by Lockheed Martin Energy Systems, Inc. The removal action subcontract was managed by MK-Ferguson of Oak Ridge Company. International Technology (IT) Corporation was the removal action subcontractor. Two specialty grouting contractors formed a joint venture company as a sub to IT Corporation to perform grouting operations: Advanced Construction Techniques of Toronto, Canada and Rembco Engineering of Knoxville, Tennessee. Grouting technology, grouting design, and grouting engineering were provided by ECO Grouting Specialists Limited, Ontario, Canada.

EXECUTIVE SUMMARY

During the summer of 1996, a unique multi-phase, multi-stage, low-pressure permeation grouting pilot program was performed inside portions of four unlined waste disposal trenches at Waste Area Grouping (WAG) 4 at Oak Ridge National Laboratory (ORNL) in Oak Ridge, Tennessee. The project was deemed a non-time-critical removal action under the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA); however, due to a history of heavy precipitation in the fall, the schedule was fast-tracked to meet an October 31, 1996 grouting completion date.

The technical objective of the removal action was to reduce the off-site transport of Strontium 90 (^{90}Sr) by grouting portions of four waste disposal trenches believed to be responsible for over 70 percent of the ^{90}Sr leaving the site. A goal of the grouting operation was to reduce the average in situ hydraulic conductivity of the grouted waste materials to a value equal to or less than 1×10^{-6} cm/sec. This target hydraulic conductivity value was established to be at least two orders of magnitude lower than that of the surrounding natural ground.

Field grouting activities started in July 1996 and were completed October 21, 1996. Sleeve pipes (driven semi-remotely) were injected at least four times with multiple formulae of regular portland cement based grouts, ultrafine cement based grouts, and acrylamide grouts. Multiple-hole grout injection was monitored using real-time monitoring equipment allowing apparent Lugeon values to be continuously calculated by treating the entire grouting operation as an extended hydraulic conductivity test using grout as the test fluid. By comparing the in situ permeability of the formation to the apparent Lugeon values, the grout formulae were continually adjusted to ensure injection of the grout formulation containing the maximum possible solids content without limiting its ability to permeate the formation. Regular portland and ultrafine cement based grouts had delayed set-times and contained up to eight different ingredients.

Approximately 137,600 gallons of grout were injected. At the completion of production grouting, the effectiveness of the grout spread and the residual in situ hydraulic conductivity of the grouted mass were assessed. The results of hydraulic conductivity tests performed through check pipes after grouting indicate that the average in situ hydraulic conductivity of the waste materials was reduced to a value less than 1×10^{-6} cm/sec. The success of this project demonstrates the potential of low pressure permeation grouting as a predictable engineered containment technology for the treatment of low level nuclear waste burial sites.

This report describes brief background to the project, describes and analyzes the grouting operations, draws conclusions from the work performed, and presents some of the "lessons learned."

I. INTRODUCTION

The summer of 1996 marked the premier application of multi-phase, multi-stage, low-pressure, permeation grouting in unlined radioactive waste disposal trenches at ORNL.[1] The technical objective of the grouting program was to reduce the off-site transport of ^{90}Sr, the principal contaminant of concern. A site investigation[2] conducted in 1994 and 1995 concluded that portions of four waste disposal trenches were responsible for about 70 percent of the ^{90}Sr released from WAG 4 and about 25 percent of all ^{90}Sr discharged from the ORNL complex. The waste disposal trenches were seasonally inundated with percolating surface water and upwelling groundwater that transported contaminants off site. The goal of the grouting program was to sufficiently lower the in situ hydraulic conductivity of the waste materials such that groundwater would tend to flow around the grouted portions of the waste disposal trenches, rather than through them, reducing the off-site transport of ^{90}Sr. The entire lengths of the four trenches were not grouted; only the ^{90}Sr source areas of the trenches were grouted.

Although the project was classified as a non-time-critical removal action under CERCLA, the schedule was fast-tracked to meet a grouting completion date of October 31, 1996. This deadline was set to avoid having to perform work during the wet weather season. Design for the project was initiated in early January 1996 and field grouting was completed October 21, 1996.

A unique grouting methodology was designed to lower the average in situ hydraulic conductivity of the waste materials to a value equal to or less than 1×10^{-6} cm/sec. This target hydraulic conductivity value was established based on it being at least two orders of magnitude lower than the average in situ hydraulic conductivity of the surrounding natural ground. As part of the design scope of work, regular cement based and ultrafine cement based grout formulations, with up to eight ingredients were developed during a laboratory testing program at Mountainview Geotechnical Limited in Stoney Creek, Ontario, Canada. For the first time, natural pozzolans, retarders, and viscosity modifiers were combined with the standard additives used in regular and ultrafine cement based grouts. The properties of these formulations were further evaluated and adjusted during full-scale batching and testing at S&ME Singleton Laboratories in Louisville (near Knoxville), Tennessee. Development and performance of accelerated aging (compatibility) tests using contaminated groundwater from the trenches and grout cubes prepared at S&ME Singleton Laboratories were also attempted at ORNL. The laboratory testing programs will be described under separate cover. No laboratory testing was performed with the acrylamide solution grouts since engineering properties and durability are well-documented elsewhere.[3,4]

The following report summarizes the technical aspects of the field-grouting program. The report begins by describing relevant background information such as site description, training, medical monitoring for workers, and the procurement process. Background information is followed by detailed discussion and analysis of the field-grouting program, culminating with conclusions and a listing of some of the "lessons learned." During each grouting chapter, the intent and requirements are presented, followed by discussion and evaluation of the grouting subcontractor's actual execution.

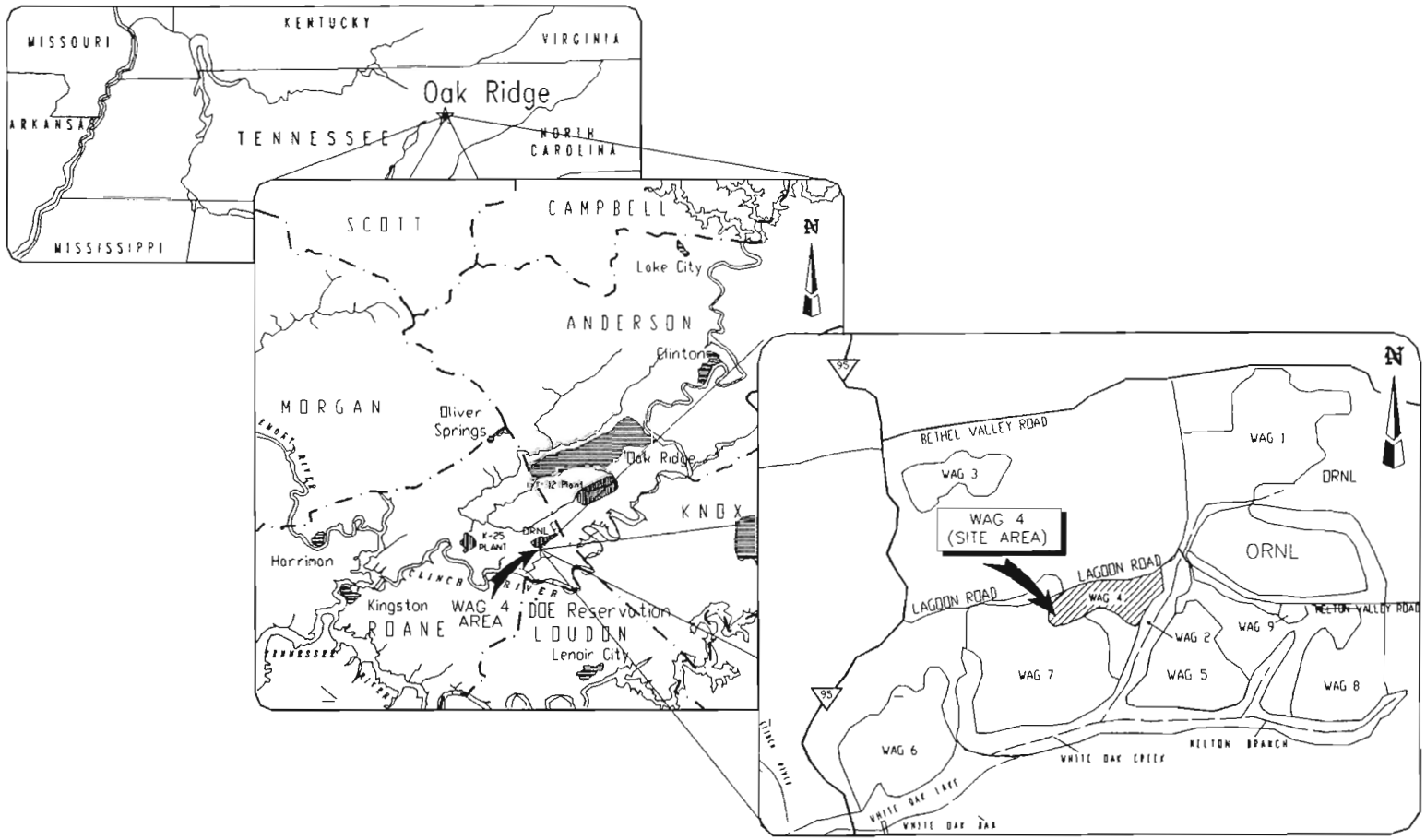
This report was prepared by ECO Grouting Specialists Limited (ECO) for Lockheed Martin Energy Systems, Inc. (LMES) under subcontract no. 12X-ND857V. Final editing of the manuscript was performed by LMES.

II. SITE DESCRIPTION

ORNL is a federal facility managed by Lockheed Martin Energy Research, Inc. (LMER) for the U.S. Department of Energy (DOE). ORNL is on the Oak Ridge Reservation (ORR) in East Tennessee at the Anderson and Roane County lines, approximately 24 km (15 miles) west of Knoxville, Tennessee, and 16 km (10 miles) southwest of the city of Oak Ridge. WAG 4 is situated in Melton Valley, southwest of the ORNL main complex. A site vicinity map is presented as Figure 1.

An aerial view of WAG 4 is shown in Figure 2. The area of interest within WAG 4 is Solid Waste Storage Area (SWSA) 4, a 23-acre area used for burial of radioactive and industrial wastes originating from ORNL and other DOE and U.S. Department of Defense facilities and their contractors over a 23-year period, beginning in 1951. The majority of the early waste disposal records were destroyed by fire in 1957; consequently, disposal records and information regarding waste disposal trench locations are very limited. Primarily, solid wastes were disposed at the site. The waste form is highly variable consisting of materials such as absorbent paper, clothing, glassware, scrap metal, dirt, filters, lumber, irradiated piston rings, oils, powders, wire, depleted uranium, carcasses of animals used in biological experiments, transuranic (TRU) wastes, waste oils, solvents, a few large pieces of equipment, and a small metal building. Some waste materials were buried in metal, wooden, plastic, fiber, or concrete containers; others were dumped into the trenches without containers. Potentially explosive elemental sodium and potassium (NaK), packaged in metal drums or glass containers, may be present. The contaminants of concern are ^{90}Sr, ^{137}Cs, ^{60}Co, ^{3}H, U, Th, and TRU isotopes; however, ^{90}Sr is the principal contaminant of concern based on its concentration, mobility, and radiotoxicity.

The sections of waste disposal trenches targeted for grouting were about 100 to 175 ft long, 6 to 12 ft wide, and 12 to 20 ft deep. The trenches were excavated down through residual soils to weathered shale and limestone in an uphill-to-downhill direction. Waste was typically end-dumped out of trucks into the trenches, then covered with about 3 to 4 ft of loose soil. In the case of boxes, more orderly stacking may have been performed. Parts of the site were later covered with construction debris. A portion of one of the trenches was capped with lean concrete averaging about 2 ft thick, prior to addition of the soil/construction debris cover. No geotechnical information was available for the waste disposal trenches. Groundwater information was available from the site investigation previously cited [2] and ongoing groundwater monitoring efforts.



Note figure was taken from Action Memorandum (Jacobs ER Team 1996)

NOT TO SCALE

Fig. 1	Location of the WAG 4 seeps DOE – ORNL , Waste Area Grouping 4 – Oak Ridge, Tennessee

Figure 2. An aerial view of WAG 4 taken in 1980. The ORNL main plant can be seen in the background (upper left corner). The bare soil patches show where subsidence has occurred and therefore mark some of the trench locations.

The waste disposal trenches were seasonally inundated by percolating surface water and upwelling groundwater. This water flowed down through the waste materials, collecting contaminants and discharging down hill in the form of contaminated seeps and contaminated groundwater. Figure 3 shows the layout of the waste disposal trenches, the trench sections targeted for grouting, and the locations of the corresponding groundwater seeps. A conceptual model of the contaminant transport mechanism [2] is shown in Figure 4. The lower portions of the trenches remain submerged year round. During the wetter seasons, the downhill ends of some trenches become completely full of water and spill over at the ground surface. Hence, the trenches have been referred to as bathtubbing."

Unusually high rainfall was received in July 1996 in East Tennessee. Consequently, water levels in the target trench sections at the time of grouting were higher than normal, varying from 0 to 10 ft below the ground surface (ground surface refers to the surface prior to placement of crushed stone described subsequently).

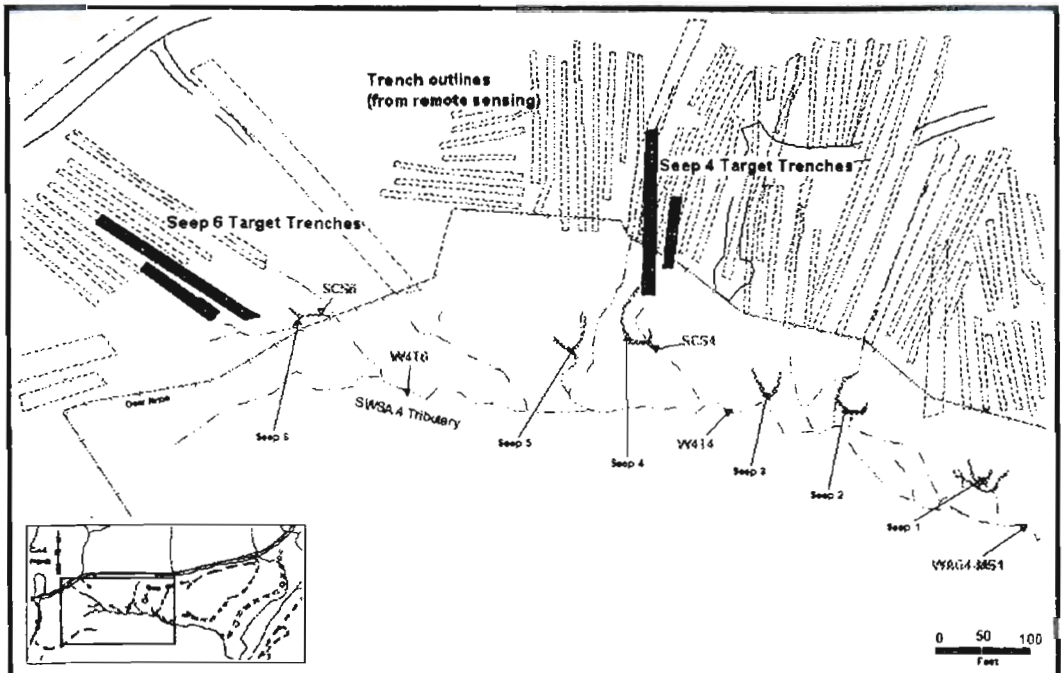


Figure 3. This map shows the WAG 4 target trenches and associated seeps. This figure was generated by Dr. Dale Huff of LMER with a geographical information system (GIS) that was used to display information as the project progressed.

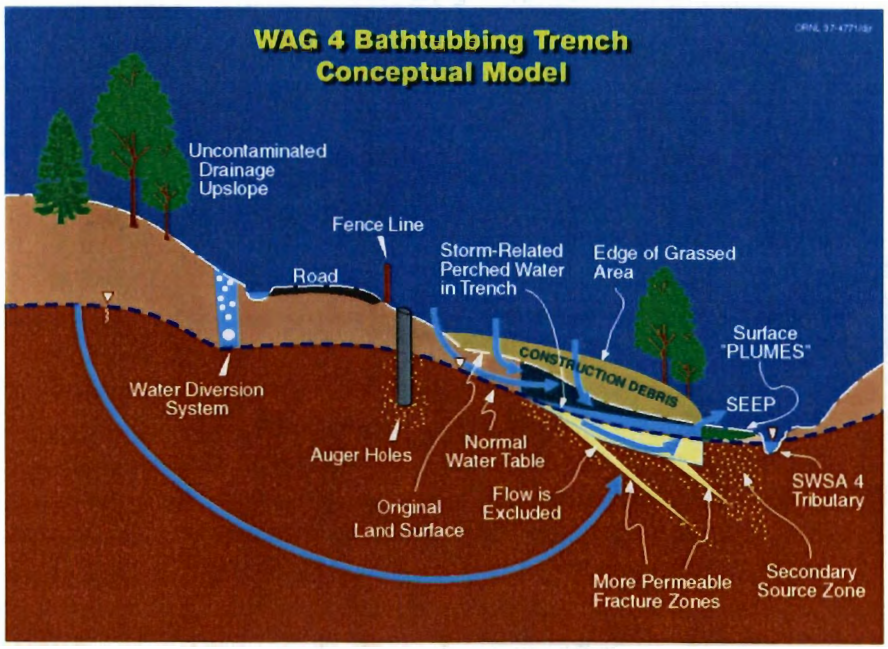


Figure 4. Schematic cross section of WAG 4 showing conditions that lead to release of contaminants from waste trenches. (Model by Dr. Dale Huff of LMER).

The site was prepared for grouting access by the construction manager's direct hire forces. Figure 5 shows the prepared site during grouting. The purpose of this stage of the project was to remove any physical obstructions and to reduce the potential exposure of the grouting subcontractor's personnel and equipment to contamination and other hazards. Site preparation included the creation of a "clean" working surface, so the threat of worker contamination and associated needs for protective clothing could be minimized. The working areas over the trenches and other areas accessed regularly by site personnel were covered with multiple layers of geotextile and crushed stone to avoid worker contact with the contaminated ground surface. The crushed stone pads over the trenches were also designed to drain surfacing groundwater and grout away from the work area. The site entrances were enlarged to allow access by semi-trucks for delivery and removal of equipment and supplies. Simultaneously, overhead power transmission lines and communication lines which formerly traversed the trench areas to be grouted were re-routed to the south side of Lagoon Road to avoid interference with subsequent operations. Straw bales, silt fencing, and sandbags were installed around the trenches to be grouted to control movement of grout that would surface during grouting operations. Figure 6 shows the grout barriers constructed in the western side of the site near the Seep 6 area. Also, the concrete cap which was discovered over Trench II was pre-drilled semi-remotely (from a distance) to allow easy penetration of the grout pipes into the trench. A view of the pre-drilling operation is provided in Figure 7. Site preparation activities were performed in March through May 1996.

III. TRAINING AND MEDICAL MONITORING

Extensive training and medical monitoring were required for all persons working on this hazardous and radiologically contaminated work site. The grouting subcontract listed 17 different types of required training. The extent of required training and medical monitoring for an individual depended on the duties that person would perform and the areas of the site within which that person would be working. The most rigorous training and medical monitoring were required for workers performing work in an exclusion zone of the site. An exclusion zone is defined as the area where contamination does or could exist. The exclusion zone was limited to the area in the immediate vicinity of the trench being grouted. Personnel working in the exclusion zone were required to take General Employee, Radiological Worker II, 40-hr Hazardous Waste Operations and Emergency Response (HAZWOPER), and respirator protection training. These personnel were required to have a baseline medical examination by a physician specializing in occupational medicine. They were also required to have entrance and exit bioassays, lung counts, and whole body counts, and were required to take a respirator fit test. Personnel handling acrylamide materials received specific training in that regard. As it turned out, respirators were only used in the cement batching areas.

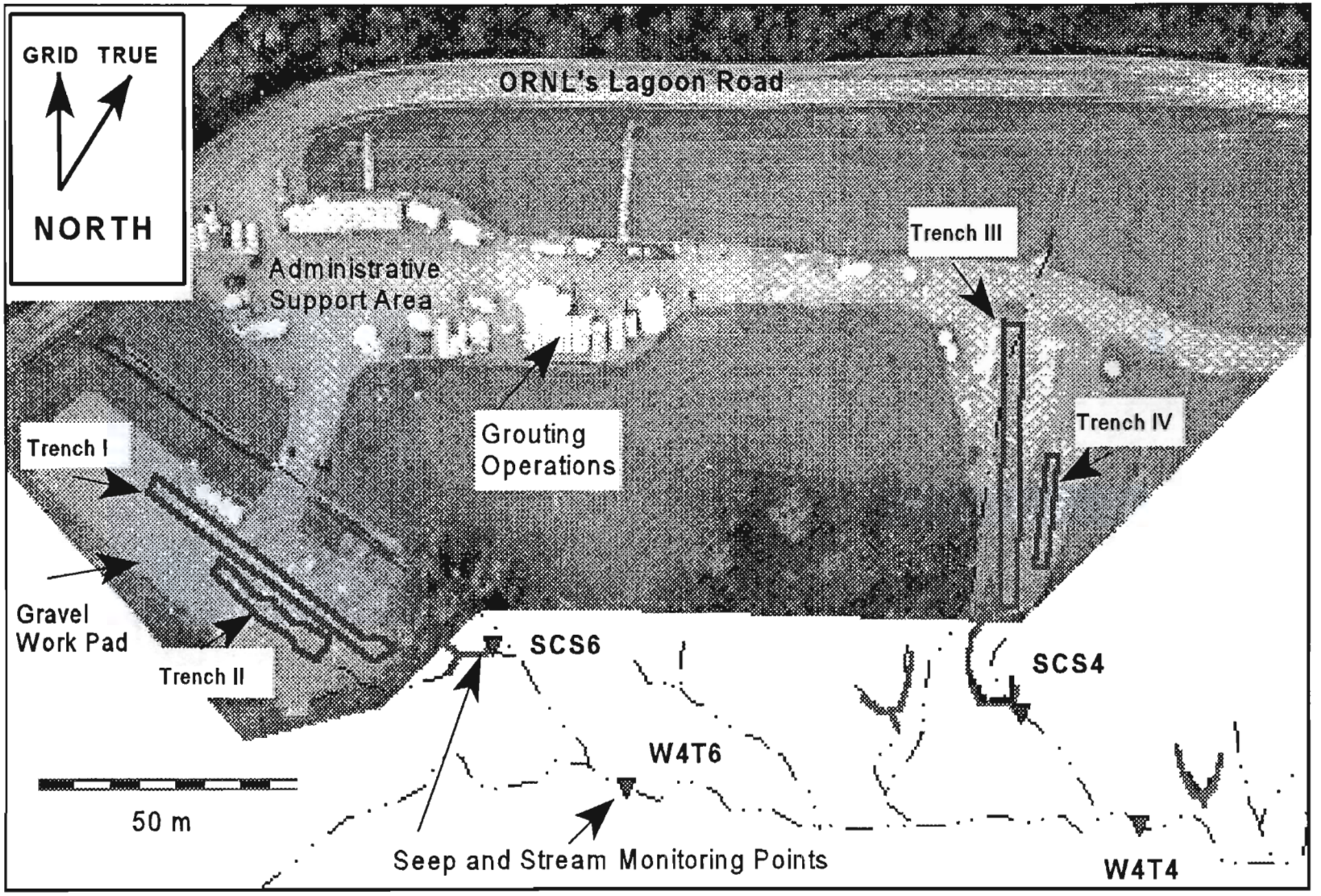


Figure 5. Aerial view of WAG 4 taken during grouting.

Figure 6. Hay bale and silt fence barriers were used to prevent grout from flowing into the tributary. Shown is the barrier system in the Seep 6 area.

Figure 7. Modified track-hoe used for augering holes through concrete cap covering part of Trench II. Augering was performed before the grouting subcontractor arrived on site. The track-hoe arm enabled the operator to maintain a minimum safe distance of 25 feet away during augering.

IV. PROCUREMENT OF GROUTING SUBCONTRACT

Early on, the project team realized the highly specialized nature of the work and the need to prequalify grouting subcontractors. The preference was to perform an evaluated procurement. However, because of the fast-track schedule, there was not enough time to perform an evaluated procurement. Typically, with an evaluated procurement, offer evaluations are performed after the design is prepared. A point system is applied to technical qualifications and price. Also, the offeror scoring the highest total points is awarded the subcontract. In the case of this project, the prequalification step had to take place while the design package was being prepared. As a compromise, the project team elected to perform two-step procurement. During the first step, potential offerors were evaluated based on a set of technical criteria. The technical proposals were reviewed and scored by an impartial committee of judges. In essence, the prequalification criteria were developed to establish a minimum acceptable level of qualifications. Potential offerors who were deemed to be technically qualified were then given the opportunity to submit a price proposal for the work during the second step of the procurement process. The successful offeror was then the offeror with the lowest price proposal. The offers were reviewed to verify that they were reasonable and not unbalanced.

The prequalification scoring covered a wide range of criteria such as relevant experience of the firm, experience of key personnel assigned to the project, equipment to be used, knowledge of relevant grouting techniques, similar project experience, and the firm's "claiming history." Minimum acceptable requirements were stated for the firm's safety record, as well. Key personnel were required to be assigned to the project for its duration.

At the pre-proposal stage, 17 different prospective offerors submitted expressions of interest in the project. Of these 17 groups (all prospective offerors were comprised of teaming arrangements), nine groups submitted technical proposals for evaluation, and only four were judged to prequalify based on their technical submittals.

The unit-price grouting subcontract package was issued "certified for construction" on March 14, 1996. The grouting subcontract was awarded on May 24, 1996. A pre-construction meeting was held on July 12, 1996.

V. SUPPLY OF SLEEVE PIPES AND WELL PIPES

A. Intent

Grout was to be injected through sleeve pipes (*or tube-a-manchettes*). With sleeve pipes, grout can be injected through individual sleeves or groups of sleeves using single and multiple packer systems attached to the grout hoses. The sleeves can be injected in any order and repeated injections through the same sleeve or groups of sleeves are possible. A significant advantage of using sleeve pipes is that they are left in the ground after the completion of grouting and may be re-accessed in the future to assess hydrological changes and/or re-grout the formation, if deemed necessary. Also, in situ hydraulic conductivity tests can be performed through the sleeve pipes.

Groundwater monitoring wells were to be installed in the trenches for analytical sampling of groundwater, measurement of groundwater levels, and removal of displaced groundwater during grouting.

B. Requirements

The grouting subcontractor was to have complete responsibility for design and fabrication of sleeve pipes and groundwater monitoring wells, based on strict guidelines and performance criteria contained in the subcontract documents.[5]
Requirements were as follows:

1. Sleeve Pipes

Sleeve pipes were to be made of steel pipe, minimum 2-inch inside diameter, lengths as per contract drawings (i.e. 15- and 20-ft lengths), with sets of sleeve-protected grout ports at maximum spacings of 20 inches on center along the pipe. The first set of grout ports was to be within 6 inches from the bottom of the pipe and there was to be a minimum of four grout ports per set. The grout ports were to have a minimum diameter of 5/16 inches. Each set of ports was to be covered by a rubber sleeve. The rubber sleeves would act as one-way check valves, allowing grouts to flow from the inside of the pipes to the outside and preventing flow from the outside to the inside, which greatly reduces the potential for worker exposure to contamination. The rubber sleeves were required to be protected, by whatever means necessary, against damage during driving and against displacement that would leave any grout port exposed to entry of groundwater or back-flushing of grouts. As a minimum requirement, a slotted or extensively perforated, steel protector was to be welded to the sleeve pipe to protect each rubber sleeve. The drawings showed the sleeve protector to extend the full length of the sleeve pipe. Extension of the sleeve protector over the entire length of the sleeve pipe would also help prevent the pipe from buckling, bending, or failing during driving. The welds were to be far enough away from the rubber sleeves so that the sleeves would not be damaged during welding.

Rubber sleeve material was to be sufficiently supple to act as a one-way check valve during grouting, yet, be pliable enough that head losses to open the sleeves would not exceed 30 psi at a flow rate of 5.0 gpm at each sleeve. Due to the potentially hazardous nature of the buried wastes, the grouting subcontractor was required to furnish rubber sleeves that were chemically resistant to contaminants known to be present.

Each sleeve pipe was required to withstand an energy transfer to the drive point of 3500 ft-lbs without bending or buckling with the pipe support apparatus in place.

The grouting subcontractor was required to test all sleeve pipes prior to shipment to the site to verify that the sleeves did not permit back-flow, before, during, or after driving. As a minimum requirement, the grouting subcontractor was to verify that no water entered the pipe from the outside after the pipe was fully submerged in water, and after a number of sleeves had been pressurized with a double or single packer. The testing procedure was not specified, but left up to the grouting subcontractor.

2. Groundwater Monitoring Wells

Groundwater monitoring well pipes were to be made of steel pipe, minimum 2-inch inside diameter, and 10 feet long, excluding the drive point. The bottom 7 feet of the pipe was to be perforated or slotted, beginning 6 inches up from the bottom of the pipe. Sufficient pipe area was to be removed within the perforated zone to allow groundwater to flow freely into the pipe. Openings were to be sized such that the washed pea gravel, specified for backfill around the well pipes, would not pass through the openings. The pipe section was to be sufficiently strong to withstand driving energy without buckling or bending.

3. Drive Points

Drive points for the sleeve pipes and groundwater monitoring wells were to be made of solid steel. The massive point, with a diameter greater than the outside diameter of any part of the sleeve pipe or well pipe, including the sleeve protector, was to be securely attached to the end of the sleeve pipe or well pipe.

C. Actual Design

Figures 8 and 9 show the sleeve pipes used on this project. The grouting subcontractor elected to fabricate the sleeve pipes locally. The only component that came from outside the U.S. was the natural rubber sleeve, which was obtained from Canada. The sleeve pipes were made of 2-inch-diameter, schedule 80 steel pipe, with three, half-inch-diameter holes drilled through the pipe wall at spacings of 20 inches on center along the lengths of the pipes. The holes were covered with heavy, snug-fitting, natural rubber sleeves. Each rubber sleeve was protected from damage during installation by a small individual steel cage welded to the outside of the pipe. The grouting subcontractor favoured this system over a continuous sleeve protector. During the submittal review process, the design team raised concern

Figure 8. Shown are sleeve pipes prior to driving. The massive metal point, the rubber sleeves, sleeve protectors and the couplings are all visible in the picture.

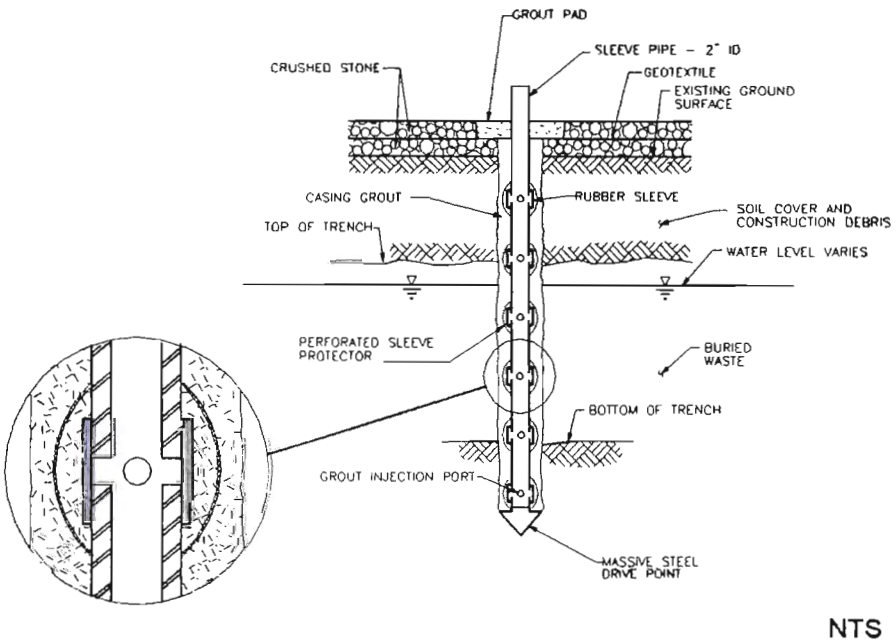


Figure 9. Sleeve pipe detail.

that the individual sleeve protector cages might prove to be inadequate if hard driving was encountered. After meeting with much opposition from the grouting subcontractor about changing his design, the design team agreed to compromise, letting the grouting subcontractor proceed with his design at his own risk.

The bottom sections of sleeve pipes were manufactured in 10 foot lengths with welded couplings at the tops of the pipes (refer to Figure 8). Massive steel drive points were welded to the bottoms of the sleeve pipes to enhance penetration of hard objects or layers. During driving, 5- or 10-ft extension pieces were added to the 10-ft bottom sections, as required, and welded together at the couplings, to produce the final 15- or 20-ft required lengths. The end product generally met the performance requirements set forth in the specifications.

Groundwater monitoring wells were made similarly to the sleeve pipes, except they did not have rubber sleeves and sleeve protectors. Additional 1/2-inch-diameter holes were drilled throughout the bottom 7 feet of the pipes to allow free access to groundwater. One half-inch-dia polyethylene well screens with 0.010-inch slots, manufactured by Aardvark, were placed inside the well pipes after driving. Due to the open nature of the trench materials, pea gravel was not placed outside the well pipes as detailed on the drawings.

There was disagreement between the design team and the grouting subcontractor as to the rigor at which sleeve pipes should be tested to verify resistance of the sleeves to back-flow. The tests performed by the grouting subcontractor involved applying a vacuum pressure equal to about 25 feet of water to the inside of each section of pipe and making a visual inspection for drafts. The tests were performed in air and not in a water tank. In our opinion, the tests performed were not adequate. A more rigorous test would involve constructing a chamber around the pipe to allow a positive pressure to be applied outside the pipe, as was successfully performed on a recent project ECO was involved in.

VI. MANAGEMENT OF WASTES

A. Intent

The grouting subcontractor was responsible for management of wastes generated from cleaning his equipment and flushing sleeve pipes after daily grouting operations, and management of groundwater removed during dewatering operations. LMES was responsible to treat and dispose radioactively contaminated wastes. It was originally intended to inject excess grout remaining in the mixers, holding tanks, and lines at the end of a stage of grouting into the grout disposal holes identified on the drawings. Also, it was intended that the rubber sleeves would prevent back flushing of trench fluids into the sleeve pipes. Thus, uncontaminated flush water would be decanted and recycled in future grout injections.

B. Requirements

The grouting subcontractor was required to develop a system to minimize, control, and recycle (to the extent possible), wastes generated from daily grouting operations, as described above. Also, his system had to be capable of pumping groundwater from observation wells in a safe, controlled fashion at a sufficient capacity to prevent groundwater from surfacing onto the working pads.

C. Actual Execution

As described in later chapters, several of the sleeve pipes contained sleeves that leaked (allowed back flushing) after driving, causing the insides and contents of the sleeve pipes to become contaminated. Since it was not possible to predict when and if a sleeve pipe might leak, all flush water was considered to be potentially contaminated and was not recycled. Instead, flush water and groundwater were pumped into polyethylene tanks and stored on site prior to disposal.

The grouting subcontractor developed an efficient flushing and collection system as shown in Figure 10.

Figure 10. After grout injection it was necessary to flush residual grout from the sleeve pipes to permit future re-injection. On the right, a "top-hat" is being used to flush casing grout from the sleeve pipes during Trench III grouting operations. The top hat extender on the left has not been attached.

With this system it was possible to flush the sleeve pipes at the completion of the daily grouting operations and pump out groundwater and/or grout in a safe and controlled fashion. The risk of worker exposure to contamination was small. The grouting subcontractor's system was called the "top-hat" system. The "top-hat" consisted of a steel cylindrical tube section approximately 8 to12 inches high by 4 inches in diameter seated in the grout pad. The grout pads were poured around the sleeve pipes after driving. The top hat was extended 2 feet higher with a leak free "quick-connection," from whence the flush fluids were pumped via a side valve into the polyethylene tanks. Also, as it turned out, it was possible to inject excess grout at the end of a grouting stage within the production grouting sleeve pipes. Thus, excess grout was not injected in the grout disposal sleeve pipes as originally designed.

For the "wash" water generated at the grout plant, the grouting subcontractor installed a sedimentation system (refer to Figure 11) which performed in accordance with the specifications.

A total of 20,000 gallons of contaminated water were generated and transported by tanker truck to the Process Waste Treatment Plant at ORNL for treatment and disposal. The majority of this water was flush water. A total of 872 cubic feet of solid low-level waste (SLLW) was generated.

Figure 11. The sedimentation pond used to treat wash water generated during cleaning of the mixers, quality control equipment, grout storage tanks, pumps, etc.

VII. PIPE DRIVING

A. Intent

All pipes, including sleeve pipes, check pipes (check pipes were sleeve pipes used for verification purposes at any stage of the grouting process, but not used for production grouting), and groundwater monitoring wells were to be installed by driving to reduce the potential exposure of workers to contaminants and classified materials contained in the trenches. Furthermore, driving was to be conducted semi-remotely (from a minimum 25-ft distance) to avert potential safety hazards associated with breaching containers of metallic sodium and potassium wastes that might be present in the trenches. Drilling into the waste materials was not allowed.

With drilling into the waste materials forbidden, installation of secondary holes after grouting would not be possible because of the difficulty of penetrating the hardened grouted mass. Hence, all pipes had to be installed prior to grouting and the spacing between grout pipes had to be adequate to allow permeation of the full breadth of the trench. Also, once driven into the ground, a pipe could not be removed because of the potential for spreading contamination. In cases where pipes might not penetrate completely to the bottoms of the trenches, additional sleeve pipes were to be added to surround the impenetrable object.

Since there was "no room for error" in the grout hole layout, a two-phase approach to grouting was undertaken. The intent was to divide the sleeve pipe installation and grouting operations into two phases as follows:

PHASE I: The initial phase (or test section) would consist of driving 10 pipes in the first trench section to be grouted and grouting. The grout spread, hole spacing, filling ratio, etc. would be evaluated. The initial 10 pipes would consist of nine production grouting sleeve pipes and one check pipe. The design (grout spread, hole spacing, filling ratio, etc.) would be optimized based on the results from Phase I.

PHASE II: The second phase would consist of driving pipes in the remainder of the first trench section and in the three other trench sections followed by grouting. The hole spacing and grouting methodology would be based on experience gained during Phase I.

Prior to completion of Phase I grouting, the grouting subcontractor would be allowed to install sleeve pipes in not more than two trenches. The specifications also provided for installation of five additional sleeve pipes per trench to aid in delineation of the trench boundaries.

B. Requirements

The grouting subcontractor was responsible for installation of sleeve pipes, check pipes, and well pipes, based on a performance specification. Responsibilities included selecting and providing equipment and apparatus necessary for safe installation.

As a minimum requirement, the driving equipment was to meet the following criteria:

a. The driving equipment had to be capable of transferring a kinetic energy of 3500 ft-lbs to the tip of the sleeve pipes and well pipes. The grouting subcontractor was given the option of transferring the driving energy to the drive point via the sleeve pipe or by use of a mandrel placed inside the sleeve pipe in conjunction with the sleeve pipe. Because of the potential for contamination, mandrels could not be used to install well pipes.

b. The support apparatus was to include a lateral support frame or other means of providing adequate support to the sleeve pipes and well pipes so that the pipes would not bend, buckle, slope, or drift.

The driving equipment was to be controlled by a semi-remote system, which would assure reliable operation with no personnel within 25 feet of the pipe while driving. Personnel were not allowed to re-enter the work area until re-entry was granted by health and safety and radiological control personnel.

The grouting subcontractor was required to keep driving logs for each sleeve pipe, check pipe, and well pipe showing the blow count for each foot of penetration and the depth to tip of the pipe. Where driving resistance was greater than five blows per inch, the blows and time required to advance each inch were to be recorded. The grouting subcontractor was required to submit driving logs on a daily basis at the end of each day.

If the sleeve pipe or well pipe could only be driven part-way because of obstacles, and in spite of the pipe and driving mechanism meeting all performance criteria, the grouting subcontractor was to terminate driving when progress did not exceed 1 inch after driving for 10 minutes at full driving capacity. Where applicable, and depending on the depth to which the sleeve pipe had been driven, it could be used as a grout hole, as a check pipe, or simply abandoned. The grouting subcontractor would be compensated for pipes driven to full depth and for pipes driven to partial depth in accordance with the specifications.

If any sleeve pipe failed to prevent water, grout, or trench material from entering the sleeve pipe from the outside, the grouting subcontractor was required to do the following: (1) pump 50 gallons of water via top-of-hole packer and (2) temporarily cap the pipe leaving the packers and packer rods in place. In each instance, the grouting subcontractor was required to install a replacement pipe. At an appropriate time during the grouting operation, failed sleeve pipes would be pressure grouted until refusal via top-of-hole packer, after a grout pad had been installed around the sleeve pipe. No casing grout was required for abandoned holes.

The grouting subcontractor would not be compensated for failed pipes, grouting of failed pipes, installation of grout pads around failed pipes, or packers and packer rods lost due to contamination therein.

C. Actual Execution

A pipe driving demonstration was performed at Advanced Construction Technique's yard in Maple, Ontario during mid-June 1996 to verify that the sleeve pipes and driving apparatus met the minimum performance criteria prior to mobilization. Representatives of LMES and ECO observed the driving demonstration.

The grouting subcontractor installed grout pipes, check pipes, and groundwater monitoring wells semi-remotely (from a 25-ft minimum distance) using a small track-mounted pile driver (see Figures 12 and 13).

Figure 12. Pipe driving apparatus. Note the shield used to protect the operator in case a sodium drum was penetrated.

The pile hammer was a DELMAG - D2, variable-energy, diesel-actuated hammer rated at 900 to 1800 ft-lbs energy per blow. The pile hammer was mounted on a Gardiner Denver 3100 air track drill carrier equipped with a 20-ft DELMAG lead. The hydraulic and throttle lines of the pile hammer were extended so that the hammer could be operated from a 25-ft distance. A mandrel was not used during driving of sleeve pipes.

A total of 250 sleeve pipes (including check pipes) and 31 groundwater monitoring wells were installed. Of this total, only 7 additional pipes were installed due to the original pipes encountering obstacles and not penetrating to full depth. The pile hammer was rated at about half of the driving energy specified in the contract documents. However, the pile hammer proved to have ample driving energy to install the pipes designed by the grouting subcontractor under the prevailing subsurface conditions in the trench sections targeted for grouting. The reader should understand, however, that the driving conditions encountered are specific to the contents of the particular trench sections penetrated

Figure 13. Sleeve pipes being driven semi-remotely into Trench III.

and the age of the trench contents. Typically, there were noticeable differences in driving resistance in the cover above the trenches, in the waste materials, and at the bottoms of the trenches. For the most part, it appeared that the trenches had been excavated down to hard weathered shale or limestone. There appeared to be little, if any, residual soil below the waste materials. The pipes typically met refusal shortly after encountering the hard weathered shale or limestone layer. Hard layers were encountered in the waste materials; however, typically, the pipes were easily driven into the waste materials, indicating that the materials in the trenches had deteriorated substantially after being buried for 30 to 40 years. In some cases, large open voids were encountered. Typical blow counts in the waste materials were in the 2 to 6 blows per foot (bpf) range. The recorded blow counts should be regarded as very approximate and should be used to provide a qualitative measure of penetration resistance only, since the driving energy was varied during driving to prevent buckling the pipes when hard layers were encountered or to prevent launching the pipe down into the wastes when very soft layers were encountered. Also, blow counts were typically higher and more uniform, and refusal depths were typically more shallow for pipes driven outside the trenches.

Driving records were kept for each pipe. The results were analyzed in light of historic topographic information for the site and knowledge gained from the site investigations conducted during 1994 and 1995. Daily results were tracked by Dr. Dale Huff of LMER using a geographical information system (GIS). Pipe driving logs are located in Volume 2, Appendix A. A section from the Trench IV driving logs is included in this report for discussion purposes (see Figure 14).

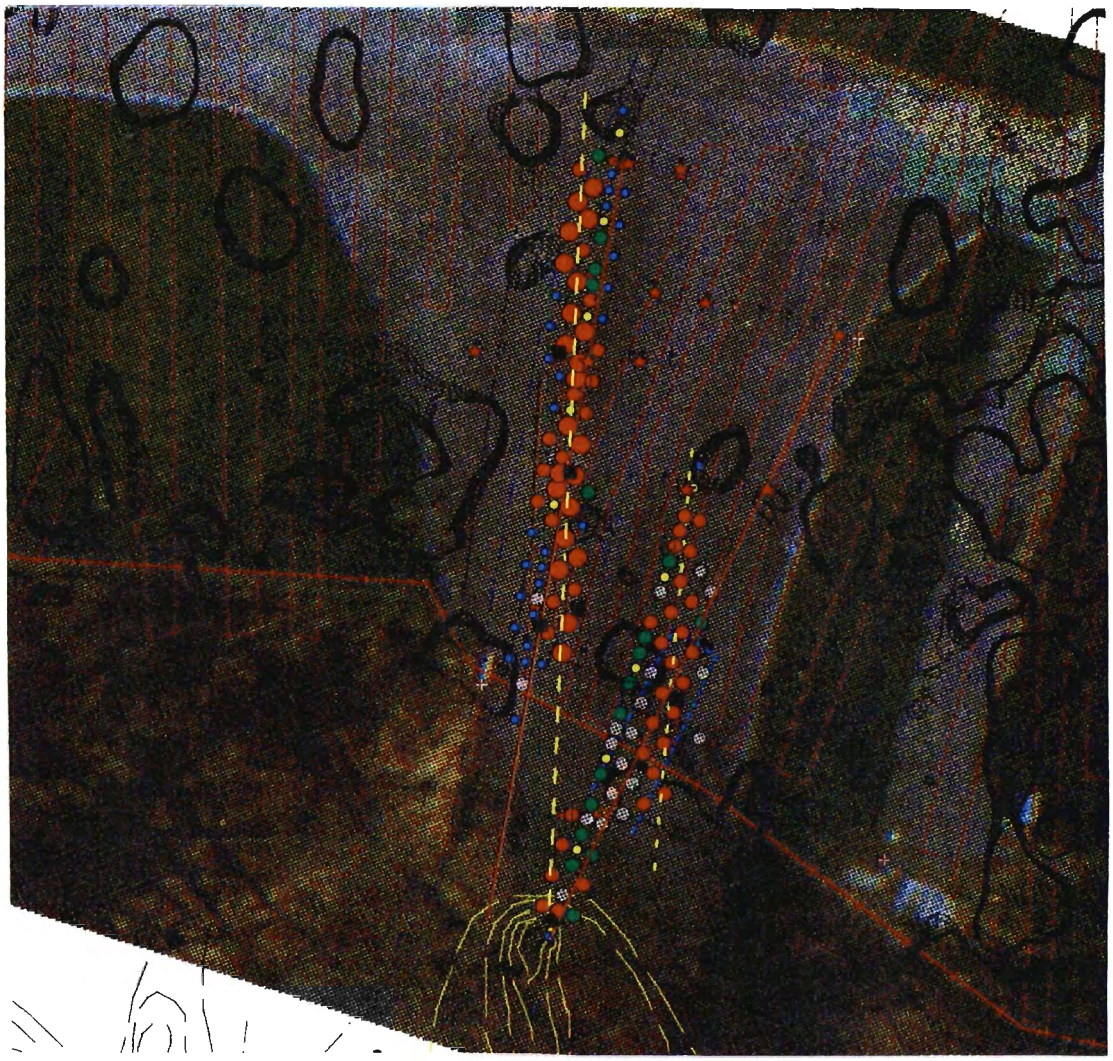
New Hole No.	Old Hole No.	Depth Driven	Date Driven	Full Depth	Partial Depth	BLOWS PER FOOT PER SPECIFIC MEASURE 0 - 1'	1 - 2'	2 - 3'	3 - 4'	4 - 5'	5 - 6'	6 - 7'	7 - 8'	8 - 9'	9 - 10'	10 - 11'	11 - 12'	12- 13'	13 - 14'	14 - 15'	Total Blows	Comments
4001	4002	14.6	16 August 1996	X		6	6	6	8	8	9	9	12	12	12	12	12	10	10	10	142	Poss soft rock. Refusal at 14'7"
4002	4003	14.4	16 August 1996	X		5	5	5	5	8	9	10	10	13	20	26	28	50	30	26	250	Poss soft rock. Refusal at 14'5"
4003	4004	13.3	16 August 1996	X																		
4004	4005	12.3	16 August 1996	X		6	6	5	4	3	3	3	3	2	5	6	12	13			71	Refusal at 12'4"
4005	4006	12.2	16 August 1996	X		5	4	4	3	3	2	2	2	6	8	6	8	18			71	Refusal at 12'2"
4006	4007	10.8	16 August 1996	X		4	4	4	4	5	5	5	4	4	18						57	Refusal at 10'0"
4007	4008	11.3	16 August 1996	X		20	10	8	8	8	6	4	3	1	1	1	12				82	Refusal at 11'4"
4008	4009	9.6	16 August 1996	X		5	5	5	4	4	4	3	4	4	15						53	Refusal at 9'7"
4009	4010	9.7	16 August 1996	X		4	4	4	4	5	5	5	6	6	18						61	Refusal at 9'8"
4010	4011	11.5	16 August 1996	X		12	2	2	2	1	1	1	1	1	1	8	35				67	Refusal at 11'6"
4011	4012	8.7	16 August 1996	X		6	4	4	3	3	3	5	5	16							49	Refusal at 8'8"
4012	4013	8.7	16 August 1996	X		5	4	4	4	4	4	4	6	18							53	Refusal at 8'8"
4013	4014	10.0	16 August 1996	X		6	4	4	4	2	2	2	20	48	90						182	Refusal at 10'0"
4014	4A	8.0	19 August 1996	X																	0	
4015	4016	9.3	16 August 1996	X		4	4	4	6	6	6	8	10	18	20						86	Refusal at 9'4"
4016	4B	11.5	19 August 1996	X																		
4017	4017	9.7	16 August 1996	X		4	2	2	4	4	8	8	12	9	19						72	Refusal at 9'8"
4018	4C	9.0	19 August 1996	X																		
4019	4E	11.3	20 August 1996	X																		
4020	4019	9.2	16 August 1996	X		4	4	5	6	6	6	8	8	12	20						79	Refusal at 9'2"
4021	4D	12.2	19 August 1996	X																		
4022	4020	9.2	16 August 1996	X		4	4	4	4	4	4	4	4	6	18						56	Refusal at 9'7"
4023	4047	12.8	19 August 1996	X		20	15	10	6	6	6	6	10	6	6	6	7	17			121	Refusal at 12'9"
4024	4022	9.2	16 August 1996	X		4	4	4	4	5	5	6	8	8	12						60	Refusal at 9'2"
4025	4023	9.1	16 August 1996	X		5	5	6	6	6	6	8	8	6	16						72	Refusal at 9'1"
4026	4048	12.3	19 August 1996	X		6	4	4	4	2	2	2	2	3	2	4	7	17			59	Refusal at 12'4"
4027	4024	9.7	16 August 1996	X		4	4	4	6	6	8	12	12	15	16						87	Refusal at 9'8"
4028	4049	12.3	19 August 1996	X		20	18	6	5	4	3	4	4	4	4	4	6	16			98	Refusal at 12'4"

Figure 14. Part of the pipe driving logs for Trench IV. Based on the depth to which the pipe was driven and the penetration resistance (blows per foot), it was possible to determine if a pipe was in the trench.

The first and second columns of the table shown in Figure 14 were used to keep pipe numbering consistent when and if pipes were renumbered in the field. The third column indicates the depth (from surface) to which the pipe was driven (refused). Other information such as date driven, whether the pipe was driven to full depth or partial depth, penetration resistance, total blows applied to the pipe, and comments were recorded. Based on the pipe refusal depth and penetration resistance, it was possible to assess strata breaks and whether a pipe was in the trench. For example, Hole #4027 (new hole number) was driven to a depth of 9.7 ft; it penetrated the first 5 feet easily and then encountered greater resistance. Hole #4028 was driven to a depth of 12.3 feet; it encountered stiff resistance over the first two feet, but then penetrated easily until coming to refusal at a depth of 12.3 ft. It was concluded that Hole #4028 was in the trench and Hole #4027 was not.

Figure 15 shows pipe driving information being tracked with the GIS system.

The hole pattern was modified in Trenches III and IV during pipe driving operations, because the orientations of the trenches were different than expected. Several pipes were



Legend

Large red circle = 16' - 20'	**Yellow circle = groundwater monitor well**
Small red circle = 14' - 15.99'	**Blue circle = sleeve pipe not driven**
Green circle = 11.01' - 13.99'	**Red star = drive point/well from previous investigations**
Grey circle = 8' - 11'	**Yellow line = inferred approximate centerline of trench**

Figure 15. This figure shows the pipe locations superimposed on an aerial photograph of the site. The size of circle is proportional to the depth the pipe was driven.

driven outside these trenches during the process of trench delineation. ("As built" locations of pipes are shown later in this report. See Figures 27 and 30.) Trenches III and IV proved to be narrower than anticipated. Fortunately, the number of required sleeve pipes and well pipes did not exceed the subcontract amount.

After completion of driving pipes in a given trench, samples of groundwater were obtained and analyzed in the laboratory for potential chemical and radiological changes that could occur by breaching containers. In Trench I, sulfate concentrations increased as a result of driving pipes (concentrations were as high as 1100 mg/l). This raised the issue of resistance of the grouts in a moderately aggressive sulfate environment. The grout mixes for that trench were reviewed to confirm sulfate resistance at the measured sulfate concentration levels based on published information. No significant chemical or radiological changes were observed in groundwater samples taken after driving pipes in the three other trenches.

The first pipes installed for the project were driven into the concrete-capped area of Trench II. Two problems arose during installation of pipes at Trench II. Thirty-one pipes in Trench II were situated in the concrete covered area. In an effort to expedite and simplify the grouting subcontract, holes were augered down through the concrete cap using a 9-inch-diameter auger under a service subcontract to LMES. (As stated earlier, this work was done during site preparation.) The holes were cased using 6-inch-diameter casings. Three holes (#2005, #2010, and #2015) were cased down into the concrete. The remaining holes were cased only down through the crushed stone working platform to mark the holes. Although the concrete cap was only 2 feet thick, the depth from surface to the bottom of the concrete, on average, was about 6 feet. After drilling through the concrete cap, the majority of the holes were completely filled with pea gravel to the surface. This was an oversight, since the statement of work required the driller to leave the insides of the casings open to facilitate pipe driving. Pea gravel was to be placed outside and below the casings to prevent the surrounding soft, saturated ground from collapsing. During driving into the gravel-filled casings, the pea gravel densified and wedged inside the casings. Consequently, the casings and grout pads were driven down into the ground with the sleeve pipes. This caused the pipes to shift out of plumb. In some cases, the pipes missed the augered holes in the concrete cap and were bent. Also, not related to the presence of the concrete cap, some of the pipes were over-driven at refusal, which resulted in buckling the pipes. The degree of bending in some pipes was such that it was impossible to insert double packers to the bottoms of the pipes. Sleeve pipe driving in this trench was temporarily suspended until the problems could be corrected. In the meantime, the driving operation was moved over to Trench I. The pea gravel inside the casings was removed using a "hepa-vac" vacuum cleaner. Also, the grouting subcontractor reduced the amount of driving energy when pipes neared the bottoms of the trenches so that the pipes would not buckle. However, ultimately, some of the pipes were bent anyway during driving.

During driving, a "driving foot" was used to stabilize the pile hammer. The clearance between the "driving foot" and rubber sleeve protector cages was only a few inches. The presence of large open voids and interspersed hard layers or objects caused misalignment of the sleeve pipes from time-to-time. The cages occasionally caught the "driving foot." This condition may have resulted in the sleeve protector cages being ripped off, and hence, damage to the rubber sleeves. Since nobody was allowed within 25 feet of the pile driver during driving, the above assessment could not be positively confirmed by visual observation.

A number of sleeve pipes were not acceptable according to the letter of the specifications[5], because groundwater and grout containing radioactive contaminants back-flowed through the

rubber sleeves. It was not possible to remove the sleeve pipes from the ground and determine the cause(s) of failure. As to possible causes of failure, it is speculated that the rubber sleeves may have been damaged during driving from contact with the "driving foot" (as described above), from contact with hard or sharp objects in the trenches, from poor sealing of the rubber sleeves, or possibly from deterioration of the rubber sleeves from chemical attack. Occasionally, in some areas of the trenches, strong organic solvent vapors were detected after the pipes had been driven, suggesting higher localized concentrations of contaminants than measured at the groundwater monitoring wells.

In the spirit of partnering and in view of the schedule constraints, a satisfactory solution was developed to allow the use of pipes with failed sleeves (containing radioactive contamination) without jeopardising quality or safety. It was noted that after the first grout injection, the radiation levels measured in sleeve pipes and groundwater dropped considerably. The grout was able to flush contaminated fluids out of the sleeve pipes and induce precipitation and sorption of certain radionuclides. The actual grouting operation was modified to utilize the positive effect of the injection of the first grouting stage (with reference to Chapter XV. Grouting, Section C - Actual Execution). Since it was possible to still use the sleeve pipes in question for grouting, the grouting subcontractor was compensated for both the supply and installation of the sleeve pipes with failed sleeves.

Overall, pipe driving went smoother and faster than anticipated. After the driving crew gained experience, production rates rose to about 20 pipes per day. The grouting subcontractor had a very experienced foreman who sorted out the few problems and perfected the system. Adjustments to the hammer energy were made, depending on the resistance of the materials contained in trenches. The operation was conducted very safely:

- a shield separated the rig from the operator, who maintained the specified distance while driving, and
- health physicists and industrial hygienists checked radiation levels and organic vapors before allowing anyone to conduct actions around the pipes.

VIII. GROUT PADS AND CASING GROUT

A. Intent

Grout pads and casing grout were intended to prevent production grout and groundwater from surfacing adjacent to the pipes during grouting operations. First, grout pads were to be cast around each pipe to seal the opening made at the ground surface. Next, casing grout would be injected below the grout pad to seal the annulus between the sleeve pipe and trench material over the entire length of the pipe. Casing grout is weak, bentonite-rich, non-sag grout, which provides an avenue for repeated access of production grouts through the same sleeve(s) to the

subsurface materials. Casing grout is hydro-fractured horizontally by water or grout to open access to the subsurface materials.

B. Requirements

Grout and/or concrete to be used for construction of grout pads was to be sufficiently competent to seal the opening between the pipe and ground surface from upflowing grout and groundwater. A guideline casing grout formula, consisting of water, bentonite, portland cement, and viscosity modifier was given in the specifications. The mix formulations (rheology and viscosity) were to be adjusted to correspond with in situ conditions.

Casing grout was required to be injected through each sleeve on each sleeve pipe via a single-fold, double packer, with the exception of the lowest sleeve, which could be accessed via an open-ended single packer. Injection of casing grout via sleeves located one foot below the groundwater level in the trench and higher was to be performed semi-remotely. Personnel were required to maintain a minimum distance of 25 feet from the point of injection during injection of casing grout until re-entry to the area was granted by health and safety and radiological control personnel. Injection of casing grout via sleeves located more than one foot below the groundwater level in the trench could be conducted in a normal fashion (without semi-remote control).

The grouting subcontractor was required to flush out each sleeve pipe after installation of casing grout. Flush water was to be prevented from exiting through the sleeves and was to be kept to an absolute minimum. The flush water, with left over casing grout residue, was to be channelled to a holding tank for later recycling, while providing protection from spilling onto the ground. The flush water was to be monitored for contamination by health and safety and radiological control personnel.

The grouting subcontractor was required to keep accurate records of casing grouting operations and submit them along with results of quality control testing for casing grouting at the end of each shift. Quality control testing included: specific gravity by mud balance, initial set time, cohesion by shear vane (initial, 4 hour, and 8 hour) and resistance to pressure filtration.

C. Actual Execution

Initially, before electrical power was available, grout for grout pads was batched off site by a local concrete supplier and delivered to the site in ready-mix trucks. During that time, grout pads were installed using wheelbarrows, shovels, and rakes. Figure 16 depicts early installation of grout pads. Later, grout for grout pads was batched on site and pumped into the ground via grout hoses.

Casing grout injection records are located in Volume 2, Appendix B. Typically, 8.5 gallons of casing grout were injected in each sleeve below a depth of 2 to 3 feet. Initially, each sleeve was injected separately. Later, this requirement was relaxed since the impact was deemed to be negligible and the grouting subcontractor's production could be increased. Injecting the bottom sleeve required the use a single-fold packer; to inject any of the other sleeves independently, required the use of a double-fold packer. At Trenches III and IV, rather than injecting the bottom sleeve separately, the grouting subcontractor was permitted to inject double the volume into the second sleeve. The "top hat" system made sleeve pipe flushing after casing grouting operations a simple, safe, and routine procedure.

The casing grout mix formulations utilized by the grouting subcontractor were likely not thixotropic enough to compensate for the large open void spaces present in some zones of the waste materials. It is likely that most of the casing grout injected into zones with large open voids sagged away from the outsides of the sleeve pipes. The grouting pressures recorded at the gauges and shown in Volume 2, Appendix C reflect the large head losses associated with pumping a pasty grout over a long distance. Higher viscosity casing grouts could have been utilized by the grouting subcontractor; however, this would have required the grouts be injected from a grouting station located closer to the point of injection to reduce head losses. Due to project time constraints, the switch to a higher viscosity casing grout formulation and associated equipment modifications was not made.

IX. AMENABILITY THEORY

The amenability theory[6] was utilized during production grouting injection. The amenability of a given soil formation or subsurface zone for a particular grout formulation is a measure of the ability of that grout to flow through apertures that water will flow through. The hydraulic conductivity (permeability) of a specific subsurface zone is directly related to the size of the combined area and distribution of flow paths accessible to the test fluid being used. By comparing the in situ permeability of a given zone using the Newtonian fluid "water" with the in situ permeability using the Binghamian fluid "grout" (and after applying a correction factor for the difference in apparent viscosity), the percentage of apertures in the formation intersected by the borehole and accessible to both water and grout can be calculated. The amenability is 100 percent if all apertures accessible to water are also accessible to grout.

Figure 16. Early installation of grout pads with wheelbarrow, shovels, and rakes.

With proper instrumentation that permits real-time monitoring of grouting pressure and take, the entire grouting process can be treated as an extended hydraulic conductivity test using grout as the test fluid. (A view of the real-time data acquisition system used by the grouting subcontractor is shown in Figure 17.) The apparent Lugeon values generated while grouting can then be used to judge the formation's response to the grout formulation currently being injected.

The data provided by initial in situ hydraulic conductivity testing and the real-time grouting data are compared to evaluate the amenability of the formation for each particular grout. To calculate the amenability coefficient[6] of a particular grout for a specific zone:

- the hydraulic conductivity value using water as the test fluid is determined;
- the hydraulic conductivity value using grout as the test fluid is determined;
- the hydraulic conductivity value determined using grout as the test fluid is then corrected for the difference in apparent viscosity between that of the grout and that of water; and finally
- the corrected hydraulic conductivity value determined with grout as the test fluid is then divided by the hydraulic conductivity established using water as the test fluid to yield the amenability coefficient.

X. INITIAL IN SITU HYDRAULIC CONDUCTIVITY TESTING

A. Intent

Since no other geotechnical data was available, determination of the initial (prior to grouting) hydraulic conductivity profile was a crucial step in the selection of the initial grout formulation for a specific subsurface zone. Prior to grouting, in situ hydraulic conductivities were to be measured through selected sleeves to develop a hydraulic conductivity profile of the various trench zones. Application of amenability theory[6] would then allow initial grouting formulations to be adjusted.

B. Requirements

Prior to in situ hydraulic conductivity testing and grouting, the grouting subcontractor was required to establish the head losses curve for the injection system by testing through the injection system, using single-fold, double packers straddling one sleeve of a sleeve pipe. A head losses curve correlates pressure losses in the lines to water or grout flow rates. To simulate the restriction caused by the casing grout, the head losses were to be measured through a sleeve pipe, which had been encapsulated in casing grout above ground.

To measure hydraulic conductivities, the grouting subcontractor was required to utilize real-time monitoring equipment. The grouting subcontractor was to provide complete and

accurately marked X-Y recorder charts and legible documentation of results including date, time, number of sleeve pipe, sleeve depth, duration of test, pressures (gauge and effective)

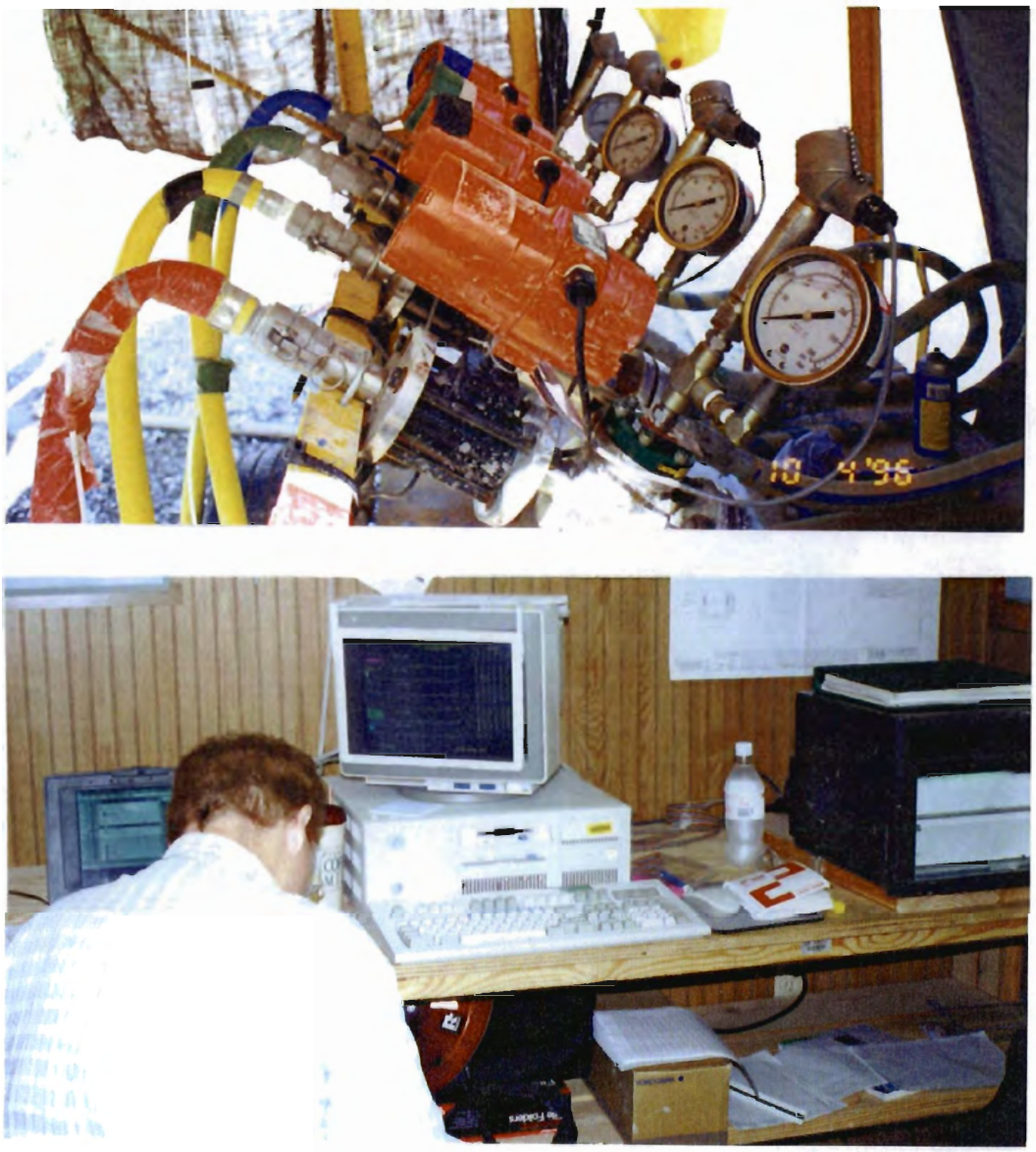


Figure 17. Real-time data acquisition and display is essential for the application of amenability theory. Magnetic flow meters and pressure gauges with transducers were used to monitor grouting. The information was immediately processed and displayed with an X-Y recorder and computer-apparent Lugeon values were then calculated.

and flow for each test, and relevant comments specific to each test. The information was to be clearly marked on the recording chart(s). A table, including the following information, was also required daily: sleeve number, sleeve depth, water take, effective test pressure, properly calculated hydraulic conductivity value, and date and time of test.

Initial in situ hydraulic conductivity was to be measured in each sleeve of 18 sleeve pipes of each trench. Each sleeve was to be tested individually to establish a detailed in situ hydraulic conductivity profile of the trench. In the first trench to be grouted, the initial in situ hydraulic conductivity was to be determined in all sleeves on each remaining sleeve pipes using a double-fold double packer testing two sleeves at a time. In the three remaining trenches, the initial in situ hydraulic conductivity was to be determined in 33 percent of the sleeves of each remaining sleeve pipe. Furthermore, all sleeves located above the water table were to be tested. Initial in situ hydraulic conductivity was to be tested at each check pipe. Sleeves selected for testing were to give a reasonably representative hydraulic conductivity profile of the trenches.

C. Actual Execution

The real-time monitoring equipment consisted of magnetic flow meters (Tiger-Mag) accurate to 0.1 gallons/minute, pressure gauges equipped with transducers, an X-Y recorder (Yokogawa), and a computer, on which the output was displayed. The quality and accuracy of the real-time data collection/display system implemented by the grouting subcontractor surpassed current grouting industry standards (see Figure 17). The head losses curve was determined as specified (see Figure 18). The casing grout was given approximately 40 hours to cure, depending on ambient and mixing temperatures, before in situ hydraulic conductivity tests were performed.

Initial hydraulic conductivity values were determined for all but 5 check pipes. The grouting subcontractor used two-fold double packers (i.e. testing of two sleeves at a time) for the hydraulic conductivity testing. As the project advanced, the trench boundaries (especially in Trenches III and IV) changed. It became necessary to re-assign some of the sleeve pipes as check pipes in order to have meaningful check pipe locations, with respect to the trench boundaries. Also, since test results were generally similar, the frequency of testing was decreased.

Initially, the grouting subcontractor's equipment configuration did not allow for accurate measurement of head losses, and hence, effective pressures. This was due to a return line that was several hundred yards in length, running from the header back to the pump. The pressure required to push water back through the return line was significantly greater than the small pressures required to inject the water into the very open trenches. This made the effective pressure determinations extremely difficult. The grouting subcontractor later eliminated the by-pass line to resolve this problem.

In situ hydraulic conductivity testing records for tests performed by the grouting subcontractor are located in Appendix C of Volume 2. Initial hydraulic conductivity values

Figure 18. Establishing head losses curve. Prior to in situ hydraulic conductivity testing, the head losses curve was established. A sleeve pipe was placed in a geotextile sock and encapsulated with casing grout to simulate in situ conditions. After the casing grout cured, the head losses curve was established.

were generally very high. In most trench zones, it was not possible to pump fast enough to match the absorption rate, and therefore, accurate hydraulic conductivity values could not be calculated. Taking into account the accuracy of the measurement system and maximum possible pump rates, the initial hydraulic conductivities the grouting subcontractor lists as infinite in Appendix C of Volume 2 represent values greater than 1×10^{-1} cm/sec.

XI. REGULAR PORTLAND CEMENT BASED SUSPENSION GROUTS

In this section the formulations of regular portland cement based suspension grouts are discussed.

A. Intent

Type III portland cement based suspension grouts were to be used in trench zones where the initial in situ hydraulic conductivity was greater than 0.1 cm/sec. It was anticipated that during the first stage of grouting, certain trench zones would be amenable to regular cement based suspension grouts. The goal was to permeate these zones over several stages with type III portland cement based suspension grouts, and where possible, reduce the hydraulic conductivity of these zones to the targeted value of equal to or less than 1×10^{-6} cm/sec.

B. Requirements

Due to the possibility of a wide range in subsurface conditions, and variations in the performance characteristics imposed on a grout to make it suitable for injection during a specific stage of a multiple stage permeation grouting program, many regular and ultrafine cement based formulations were designed. Each formulation was designed for use in a specific stage or subsurface zone. The rheological properties of each grout formulation were to be optimized to make it suitable for injection into the zone it was designed to treat. The grouts were formulated to:

- generate minimal bleed water so that no residual flow paths are left behind in the grouted mass after injection;
- have high resistance against pressure filtration to permit grout injection into flow channels and pores without rapidly forming a filtercake and thereby losing access to that zone;
- be water repellent to displace (instead of mix) with groundwater present in the trenches;
- in the first grouting stage in very pervious zones, be thixotropic and non-sag to allow large open voids to be filled without the grout sagging away. Sagging grout may smear or coat the formation, resulting in loss of access to the formation during subsequent injections without true grout penetration;
- have a matrix permeability of less than 10^{-8} cm/sec to stop the leaching of radionuclides encapsulated in the grout;
- have no noticeable shrinkage to eliminate secondary flow paths;
- have the desired rheology to allow the grout to permeate the formation;
- have retarded set time to allow multiple injections through the same sleeve of the same sleeve pipe, over a period of several days, to permit wedging and densification of the grouts; and
- be as durable as possible to maximize the durability of the waste treatment process.

The mix formulations for the regular cement based suspension grouts were to contain some or all of the following ingredients at various proportions:

1) type III portland cement
2) bentonite
3) water
4) silica fume
5) natural pozzolan
6) dispersing agent/retarder
7) viscosity modifier
8) class "f" flyash

The grouting subcontractor was required to be able to batch a base formulation for the regular cement based grouts. The base formulation would then be modified by adding or subtracting predetermined increments of the different additives as determined by the grouting conditions. The construction specifications required the grouting subcontractor to be able to batch up to 20 different formulae of type III portland cement based grouts. The construction specifications clearly defined the requirements for grout mixing, storage, and pumping equipment. The performance criteria for the grouts were set out in the specifications and were to be tested regularly through a rigorous quality control program (refer to Chapter XIV. Quality Control).

C. Actual Formulations

The grouts were mixed in a high shear mixer (Colcrete colloidal mixer) providing high speed (2000 rpm), high shear mixing of the grouting ingredients. A water meter was used to measure the amount of water being introduced into the mixer. The ingredients were measured using pre-calibrated containers (refer to Figure 19). The ingredients were introduced in a specific order and allowed to mix for two minutes. Changing the order in which the ingredients are mixed can have a fundamental impact on the grout rheology. Typical batch sizes were about 40 gallons. The grout was then transferred to a holding tank containing a low speed (adjustable 40 to 200 rpm) paddle mixer (see Figure 20). The paddle mixer prevented flocculation and segregation of the mix. The grout was subsequently transferred from the bottom of the holding tank to a helicoidal screw-pump (mohno pump). When casing grouting and production grouting operations were executed concurrently, a separate grout plant was used for casing grouting.

Figure 19. The ingredients were measured using pre-calibrated containers. The high shear mixer and water meter are also visible in this picture.

Figure 20. Paddle mixer used as storage tank.

Grout was pumped directly to the hole being injected one hole at a time. The grout was pumped to a multiple-hole-grouting header, suitable to feed as many as six grout zones at the same time. The grouting header was nicknamed the "hot dog stand" and is shown in Figure 21. It was equipped with both pressure transducers and magnetic flow meters for each injection line.

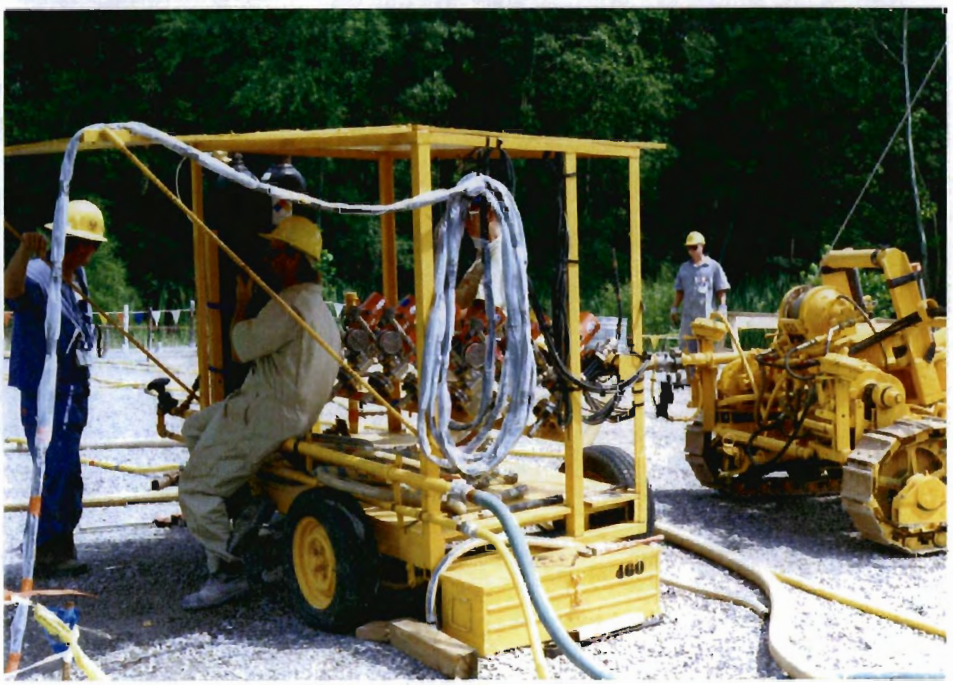


Figure 21. The multiple hole header nicknamed "the hot dog stand."

Grout batching operations are shown in Figure 22. High amenability was obtained and very high apparent Lugeon values were maintained with the most thixotropic formulations during all grouting stages, but especially during the first grouting stage. High viscosity and thixotropy of the grout was required so the grout would not sag, thereby smearing/coating the formation and leaving an unacceptable residual permeability. For this reason, thixotropic agents causing significant increase in viscosity at low shear rates were used. During the second grouting stage through the same sleeve, predominantly low viscosity grouts with high resistance against pressure filtration were used to permeate secondary pathways. The goal of the second grouting stage was to create densification and wedging of the grout. Retarders were used in all formulations to enable re-accessing the holes.

Portland and ultrafine cement based grouts offer an added treatment benefit for certain radionuclides, since the increased pH from grouting causes certain radionuclides to precipitate out of solution. During grouting, reductions in radiation levels measured within

Figure 22. The grout mixing area.

the sleeve pipes and in groundwater samples were observed after just one stage of grouting with portland cement based grouts.

XII. ULTRAFINE CEMENT BASED SUSPENSION GROUTS

A. Intent

The ultrafine cement based suspension grouts were to be used as a permeation grout in subsurface zones with initial or residual hydraulic conductivity values between 0.1 cm/sec and 0.005 cm/sec. It was known that over time, trenches were repeatedly "topped-up" with soil fill as the waste materials decayed and subsided. The types of soils were unknown; hence, the provision was made for possible use of ultrafine cement based grouts to penetrate low permeability zones such as the soil fill. Also, it was anticipated that at a certain stage of grouting, a particular trench zone might no longer be amenable to regular cement based grouts, but would be amenable to ultrafine cement based grouts. Thus, further grouting with ultrafine cement based grouts could be required to achieve the previously stated hydraulic conductivity goal for the project.

B. Requirements

Ultrafine cement based grout formulations with the ability to permeate the fine pore channels and cure without bleeding, while minimizing residual permeability were required. In addition to meeting the requirements for regular cement based suspension grouts (refer to Chapter XI. Regular Portland Cement Based Suspension Grouts, Section B. Requirements), ultrafine cement based grout formulations were required to have the ability to permeate low permeability materials in the trenches such as soil fill. The ability to permeate low permeability materials was assured by requiring a very low pressure filtration coefficient (high resistance against pressure filtration). After permeating soil pores, grouts should cure leaving the pores completely filled. Krizek and others [7] studied the characteristics of grouted sands with rather unstable ultrafine cement based suspension grouts. They discovered that the vertical permeability of the grouted soil mass was a factor of 100 lower than the horizontal permeability, because the pores were not completely filled. On this project, stable ultrafine cement based suspension grouts were utilized to minimize residual horizontal permeability. Also, a refusal grouting pressure criterion was stipulated to force the grout particles closer together during curing, further reducing hydraulic conductivity of the grouted mass.

The mix formulations for ultrafine cement based suspension grouts were to contain all of the following ingredients at various proportions:

1) ultrafine cement
2) bentonite
3) water
4) silica fume

5) natural pozzolan
6) dispersing agent/retarder
7) viscosity modifier

The grouting subcontractor was required to batch a base formulation for ultrafine cement based grouts. The base formulation would be modified by adding or subtracting predetermined increments of the different additives as dictated by grouting conditions. The construction specifications required the grouting subcontractor to be able to batch up to 5 different formulae of ultrafine cement based suspension grouts. The construction specifications clearly defined requirements for grout mixing, storage, and pumping equipment. Performance criteria for the grouts were set out in the specifications. Verification would be achieved by regular testing via a rigorous quality control program (refer to Chapter XIV. Quality Control).

C. Actual Formulations

The ultrafine cement based suspension grouts were prepared, stored, and pumped with the same equipment and general procedures as the regular cement based suspension grouts. For a description of the mixing, storage and pumping of the regular cement based suspension grouts refer to Chapter XI. Regular Portland Cement Based Suspension Grouts C. Actual Formulations.

XIII. ACRYLAMIDE SOLUTION GROUTS

In this section the batching process for the acrylamide solution grouts is described.

A. Intent

The acrylamide solution grouts were to be used as a permeation grout in subsurface zones with a hydraulic conductivity less than 0.005 cm/sec. It was anticipated that some trench zones would not be amenable, either prior to grouting or after executing several grouting phases, to regular or ultrafine cement based grouts, but would be amenable to acrylamide grouts. The goal was to penetrate these zones with acrylamide solution grouts and reduce the hydraulic conductivity of these zones to the target hydraulic conductivity value previously cited.

Due to the possibility of wide variations in subsurface conditions and outside temperatures, it was necessary to have acrylamide grouts with set times that could be varied between one and 40 minutes and monomer concentrations between 10 percent and 20 percent.

B. Requirements

The performance criteria for the acrylamide solution grouts were set out in the specifications. Verification would be achieved through regular testing via a rigorous quality control program (refer to Chapter XIV. Quality Control). The construction specifications[5] clearly defined the procedures and equipment required for safe grout mixing, storage, and injection.

C. Actual Formulations

Mono-acrylamide in its powder form (uncatalyzed) is a neurotoxin. It can enter the body through the nose, mouth, or skin. Contact by these means was avoided, and measures for prompt clean up in case of an accident were taken. All individuals required to work in the area where acrylamide grouting was taking place were required to attend a mandatory material safety and handling seminar specific to safe acrylamide handling.

Avanti International supplied the acrylamide grouting materials. The base ingredient of the acrylamide grout was the mono-acrylamide powder containing a cross-linker. The acrylamide monomer was AV-100. The batching of acrylamide was done on site. A pure 40 percent concentrated A-component of the acrylamide solution was prepared from the mono-acrylamide powder. From this, the acrylamide-based solution grout was ultimately injected at a concentration (of the monomer in the solution grout i.e. A+B) of 10 percent. The preparation and mixing of the various ingredients constitute two components, A and B. The other ingredients making up the acrylamide grout included:

- Activator - tri-ethanol-amine,
- Initiator - ammonium persulphate,
- Inhibitor - potassium ferri-cyanide,
- Buffer - sodium bicarbonate, and
- Dye - a dye was used for ease of detection of the component containing the acrylamide.

The different ingredients were added to their respected sides. The acrylamide solution grout was injected at a 1:1 ratio using a two component piston pump delivering the two components using separate grout lines to a static mixer. The acrylamide grout plant is shown in Figure 23. Each of the grout lines contained a one-way check valve just upstream of the static mixer. Downstream of the static mixer was a flow meter equipped with pressure transducers. A multiple hole header enabled the grouting subcontractor to inject up to six zones with similar Lugeon values simultaneously. It was necessary "to play the valves" to verify all zones were taking grout. Grout was taken to refusal in all holes (allowed to gel in the lines) and had to be extruded from the lines by applying high pressures.

Figure 23. Acrylamide grout plant. A and B sides are clearly marked.

XIV. QUALITY CONTROL

A. Intent

A quality control program is an important part of a professionally executed grouting project because the end product depends on the consistency and quality of the grouts being injected. For this reason, an extensive quality control program was included in the construction specifications. The purpose of the program was to ensure that the cement based grout formulations being injected met the performance criteria established prior to construction during the laboratory testing program, and that the acrylamide grouts had the appropriate chemistry and predictable gel times.

B. Requirements

The specifics of the quality control program were set out in the specifications. It was the grouting subcontractor's responsibility to perform quality control tests and forward information in a timely fashion to ECO.

For the cement based grout formulations, the grouting subcontractor was responsible for providing adequate and qualified staff to perform the following tests: specific gravity of each batch using a baroid mud balance, viscosity of every tenth batch using a Marsh cone or for very viscous mixes using a shear viscometer, and sedimentation (bleed) and pressure filtration coefficient (K_{pf}) using an API filter press at least once a week for each mix formulation used in a specific week.

For the acrylamide formulations, the grouting subcontractor was required to perform the following quality control functions:

- accurately measure the amount of water used for the preparation of the components of the acrylamide solution grouts;
- accurately measure the weight of the various components (which were prepared in small packages) with accurate weigh scales;
- accurately measure small volumes of activator using graduated cylinders;
- monitor the liquid levels in tanks A and B during grouting to ensure they were dropping at same rate; and
- sample each batch of grout in labelled plastic sampling bottles. The sample bottles then were to be marked with the date and batch number.

C. Actual Execution

Initially, the grouting subcontractor was understaffed and quality control testing of the cement based suspension grouts was not as rigorous, nor were the test results reported in a timely fashion, as required by the construction specifications. This deficiency was brought to the attention of the grouting subcontractor. ECO assisted where possible to achieve minimum requirements. Quality control personnel staffing were improved later in the grouting program and the acrylamide quality control process was excellent. During some of the hotter days, grout samples were also taken at the grouting header just past the in-line mixer. Equipment used to perform some of the quality control tests is shown in Figure 24.

D. Results

The quality control test results are included in Volume 2, Appendix D. Grout samples for testing purposes were taken after mixing before the grout was transferred to the holding tanks. The viscosity modifiers were selected to cause the viscosity of the grouts to be dependent on

Figure 24. Quality control tests: (Top Left) A static penetration cone was used to measure cohesion. (Top Right) Also, a shear vane was used to measure cohesion. (Bottom) API filter press was used to measure resistance against pressure filtration.

the shear rate. The impact of the shear rate was reflected by the substantial fluctuations in Marsh time (viscosity) which was dependent on the lapsed time between mixing and testing.

Throughout the grouting operations, ECO performed random quality control tests (not reported), checked the grouting subcontractor's testing methodology, and monitored initial setting of the grout samples. Based on these tests, it was concluded that the grouts injected met the specified performance criteria.

The pH, temperature, and set times for the acrylamide grouts were recorded. This information is contained in Appendix D of Volume 2. By referring to previous days' quality control data, it was possible to control the gel time of the grouts, even when site conditions (temperature and length of line exposed to sun) varied considerably.

XV. GROUTING

A. Intent

As stated previously, a goal of the grouting design was to reduce the average in situ hydraulic conductivity of the waste materials contained in the trenches to a value equal to or less than 1×10^{-6} cm/sec. Grouting was to be executed in two phases as described in Chapter VII. The grouting subcontractor was responsible for making technical decisions subject to approval by ECO. The intent of the contract was to have a Specialist Grouting Contractor perform a contract referred to as a "type B" contract by Houlsby[8], whereby technical input is provided by the Grouting Engineer, who makes the ultimate technical decisions by:

- either not interfering with the operation if conducted according to the Specialist Grouting Contractor's plan, or as per a mutually established plan; and
- directly ordering particular actions or calling mix designs. This was not to be interpreted that the Specialist Grouting Contractor could or should act incompetently. Under no circumstances should the Specialist Grouting Contractor claim to be the "dumb contractor who does only what he is told to do."

B. Requirements

The construction specifications[5] stipulated the requirements and criteria the grouting subcontractor must meet in execution of the grouting program. The grouting subcontractor was required to use low pressures to inject grout into the ground at flow rates that allowed the grout to permeate or flow into the void spaces of the subsurface, displacing any water which was present and sealing the void spaces.

Since the specific contents of any particular trench were unknown and since the contents could be highly variable, it was anticipated that the waste disposal trenches could vary greatly in terms of void size, permeability, amenability to particular grout formulations, etc., making it necessary to inject a wide range of different grout mixes. Various cement based suspension grout formulations were required:

(1) to place the most competent grout with the highest possible solids content, but with acceptable rheology to permeate the subsurface;

(2) so that during all grouting stages, but especially during the first grouting stage, it was desirable for the grout to not sag, thereby smearing/coating the formation and leaving an unacceptable, high residual permeability. For this reason, additives with thixotropic characteristics causing significant increase in viscosity at low shear rates were selected;

(3) for the second and additional grouting stages (through the same sleeves) predominantly low viscosity grouts with a high resistance against pressure filtration were needed to permeate secondary pathways. The actual formulation was dictated by the evolution of the apparent Lugeon value (see below); and

(4) to allow multiple injections of grout into the same zones of the subsurface by using grouts with delayed set times.

In order for permeation to take place, the particle size of the cement based suspension grouts had to be sufficiently small to penetrate the voids of the subsurface material. The grout was to be mixed in a colloidal mixer, which separated particles from each other at a speed of 2000 rpm. (Refer to Chapter XI. Regular Portland Cement Based Suspension Grouts.)

Due to the possibility of wide variations in subsurface conditions, it was necessary to perform final grouting with solution grouts. Acrylamide was the grout chosen because of its low viscosity, long durability and adjustability of set times.[3,4]

The grouting subcontractor was required to monitor grout injection using real time data collection/display equipment allowing apparent Lugeon values to be continuously calculated by treating the entire grouting operation as an extended hydraulic conductivity test using grout as the test fluid. By comparing the in-situ permeability of the formation with the apparent Lugeon values, grout formulae, were to be continually optimized to allow the injection of the most competent grout with the highest possible solids content, but with acceptable rheology to fully permeate the subsurface.

Grout injection quantities were to be continuously compared to estimated theoretical quantities of grout take. By monitoring the apparent Lugeon value trend, changes were to be made to the grout formulation being injected to ensure that the quantity of grout to fill the theoretical "grout cylinder" around the grout pipe could be injected without hydro-fracturing. If the apparent Lugeon value was dropping too fast to allow the installation of a desired grout volume, a reduction in viscosity, thixotropy, and/or solids content was required.

C. Actual Execution

Execution of grouting operations will be discussed in four parts:

- General Comments;
- Grouting Trench I;
- Grouting Trench II; and

- Grouting Trenches III and IV.

Appendix E of Volume 3 contains the daily grouting records, the grouting subcontractor's report, the X-Y strip chart printouts, and daily grouting reports, organized in a chronological order. The reader is also directed to Chapter XVII. Chronology of Events, which gives a time line for the project. A close-up view of the flow meters used on the project is shown in Figure 25.

Figure 25. The flow rates and pressures were monitored both at the X-Y recorder in the trailer and at the gauges in the field by the header man.

1. General Comments

A good working relationship existed between the grouting subcontractor and ECO throughout the execution of the project. Technical decisions were usually reached by mutual agreement through a suggestion/discussion process, which allowed everyone to contribute.

The grouting subcontractor mobilized later in the schedule than anticipated. The water supply line to the site was not installed until late July. By that time, a substantial portion of the construction period had passed. In order to bring the project back on schedule, the grouting subcontractor was permitted to drive pipes in Trenches III and IV before fully completing Phase I grouting. The two-phase approach was partially aborted in order to meet the October 31 grouting completion date.

Strict safety protocol was enforced throughout the project. As a result, the production rate accomplished over the duration of the project was significantly less than would have been achieved working in a non-radiological work place.

After evaluating the results of the initial hydraulic conductivity testing, the appropriate starting formulations were selected. The initial stages of the grouting operations in all four trench sections involved type III portland cement based suspension grouts. The later stages (i.e. third to fourth stages) of grouting were performed with ultrafine cement based suspension grouts. The final stage for all four trenches was performed using acrylamide solution grout (this predominantly occurred during the fourth stage for the majority of the sleeve pipes).

Grout injection pressures were kept low to reduce hydro fracturing (claquage or fragmentation) and to reduce the amount of ground heave. Although, some hydro fracturing was observed during all grouting stages, it was predominantly used to enhance grout spread during acrylamide grouting, and did not adversely affect the quality of the operation.

By continuously calculating the apparent Lugeon values it was possible to quickly react to the subsurface response to a specific grout formulation:

- if the apparent Lugeon value remained constant, the solids content was increased while proper changes were made to adjust its rheology;
- if there was a sudden drop in apparent Lugeon value, the solids content was decreased; and
- depending on the response of the formation to the adjustments, a change to ultrafine cement based suspension grouts or acrylamide solution grouts was made.

The predictability of set times for the acrylamide grouting enabled definitive control over grouting procedures even at times when the trend or mode of the grouting operation was indicating void filling phenomena (i.e. when apparent Lugeon values were not decreasing over a period of time).

A re-occurring event during grouting was that groundwater and/or grout was expelled up to the ground surface while grouting operations were on going. The different courses of action involved:

- "resting the hole" (i.e. stop the injection process to give the grout a chance to gel);
- moving away from the zone of influence where the leakage was evident; and
- returning to the leakage area holes after 24 to 48 hours of resting to re-attempt grouting.

It became apparent that 48 hours was required to obtain stiffening of the grout to a point where no more leaks would occur.

Essentially the entire grouting program was performed in modified level "D" protective clothing. Health and safety and radiological control personnel were on site during the entire grouting process and monitored the work areas over the trenches for potential health hazards.

2. Trench I

The first and second stages of grouting in all trenches were executed with type III portland cement based suspension grouts due to the high initial permeabilities encountered. Grouting progressed from the downhill end of the trench in an uphill direction. Figure 26 shows grouting operations in progress at Trench I.

Figure 26. Packer being inserted during Trench I grouting operations.

a. Trench I: Bulkhead

During installation of sleeve pipes, it was determined that the planned grout hole layout did not extend to the downhill end of the trench. Additional sleeve pipes were added along the southeastern (down-hill) end, but it was not possible to distinguish whether the end of the trench had actually been reached or whether the holes had crossed over into other trenches which ran perpendicular to Trench I along the southeastern side[a]. A grouted bulkhead, using predominantly type III portland cement based suspension grouts, was installed through the additional sleeve pipes to prevent grout in the targeted trench section from flowing freely down slope under gravity. Bulkhead grouting proceeded in the following sequence:

- first stage - grouted the middle zone (sleeves 3,4,5) - type III port. cement based grout;
- second stage - grouted the lower zone (sleeves 1,2) - type III port. cement based grout;
- third stage - grouted the upper zone (sleeves 6,7,8) - type III port. cement based grout;
- fourth stage - grouted all zones until refusal using top-of-hole packer - type III portland cement based grout; and

[a] Bulkheading area and extension of trench are shown in Figure 27.

- fifth stage - repeated fourth stage if fourth stage was terminated prior to refusal due to a grout leak.

During bulkhead grouting several grout leaks were observed exiting at both the southeastern end of the trench and at the surface of the gravel pads. The following statistics summarize bulkhead grouting (note that grout quantities are approximate):

- total volume of grout: 6,200 gal.;
- volume of type III portland cement based grouts: 15,000 gal. - 93 percent of total volume injected;
- volume of ultrafine cement based grouts: 400 gal. - 2.3 percent of total volume injected;
- volume of acrylamide grouts: 800 gal. - 4.6 percent of total volume injected;
- total theoretical trench volume: 24,000 gal.; and
- filling ratio: 68 percent.

Since it was difficult to quantify the distance grout traveled from the point of injection, the amount of grout that permeated the crushed stone working pad, and the amount of grout that leaked outside the trench and/or to the surface, the actual grout filling ratio of the trench materials was most likely about 50 to 55 percent. The average grout take per sleeve pipe was about 750 gallons.

Figure 27 shows the "as-built" locations of grout pipes for Trenches I and II.

b. Trench I: Production Grouting

For the first 12 twelve sleeve pipes in Trench I, production grouting proceeded in the following sequence:

- first stage - grouted the bottom 3 to 4 sleeves - type III portland cement based grout;
- second stage (18 hours later) - grouted all the sleeves below the construction debris - type III portland cement based grout;
- third stage (48 hours after the first stage) - grouted all sleeves below the construction debris with either type III portland cement based or ultrafine cement based suspension grout depending on residual in situ permeability value and the amenability;
- fourth stage - repeated third stage if necessary either because third stage was terminated before refusal due to grout leak or third stage was performed with type III portland cement based suspension grouts; and
- final stage (after either refusal was obtained with ultrafine cement based grouts or at least two stages with ultrafine cement based grouts had been completed) - grouted all sleeves below the construction debris with acrylamide solution grout to complete refusal.

A large number of grout leaks, out the sides of the pads and to surface occurred. A variety of stage grouting methodologies were undertaken over the course of the grouting activities reflecting continuously evolving site specific conditions and experiences gained from previous days of grouting. The grouting pressure and pumping rates were also varied. It

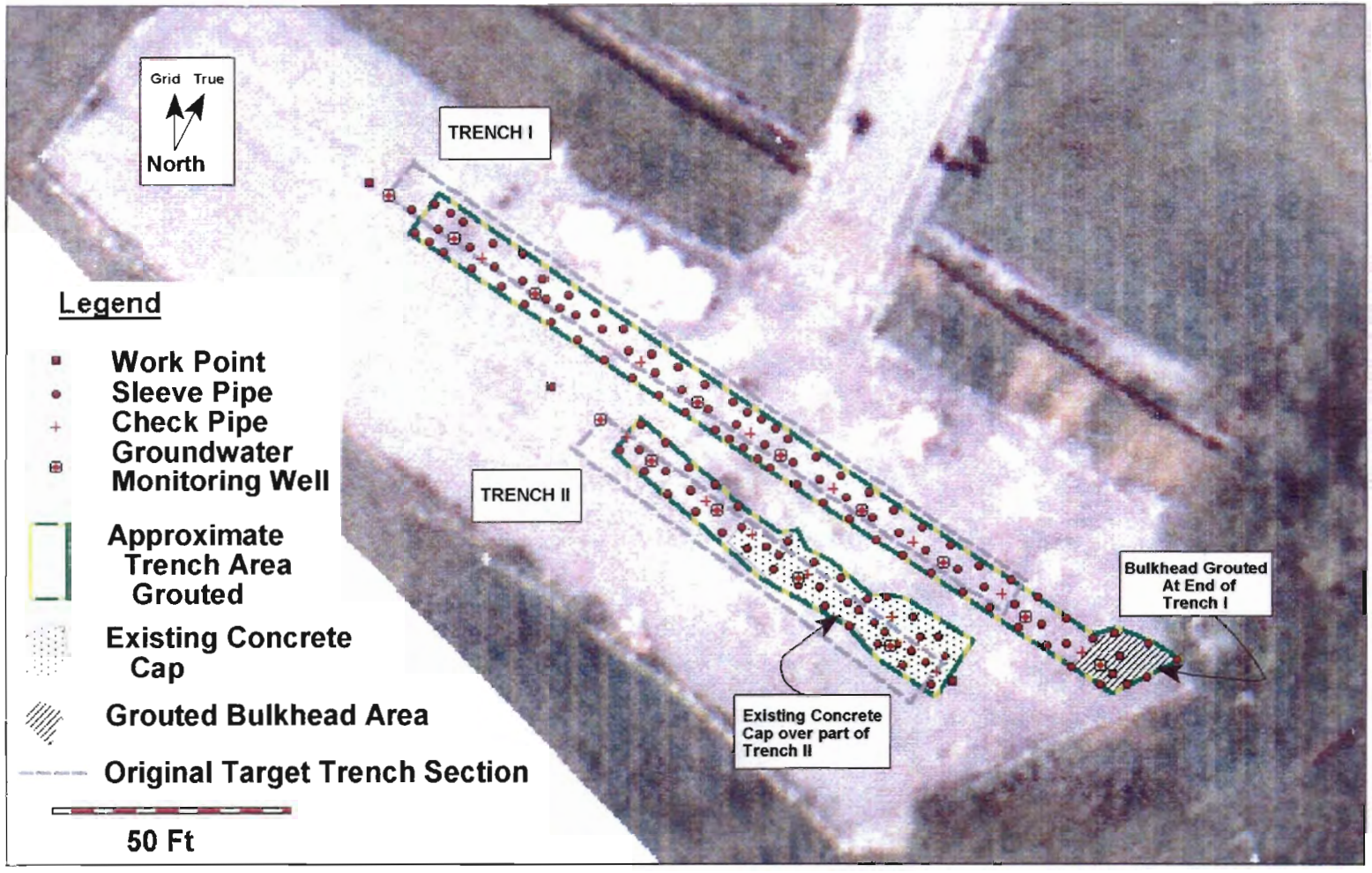


Figure 27. As-built locations of grout pipes near Seep 6.

was necessary to determine the rate/pressure at which the grout could permeate the trench material without extensive hydro fracturing.

Eventually, production grouting was executed in the following sequence in an attempt to minimize the number of grout leaks to surface:

- first stage - grouted the bottom 3 to 4 sleeves, injecting a theoretical volume of type III portland cement based grout;
- second stage (18 hours later) - grouted all the sleeves below the construction debris with a theoretical volume of type III portland cement based grout;
- third stage (48 hours after the first stage) - grouted all the sleeves below the construction debris with a theoretical volume of ultrafine cement based suspension grout depending on residual in situ permeability value and amenability; and
- fourth stage (within two weeks of third stage) grouted all sleeves below the construction debris with acrylamide solution grout to complete refusal.

Over the duration of the initial grouting operations, it was observed that the "grout front" was progressing ahead of the most up-slope sleeve pipes that were being grouted. This situation caused concern, since it resulted in elevated water levels ahead of the grouting operation. The grouting subcontractor was required to pump water from groundwater monitoring wells, as well as from leaking sleeve pipes. The groundwater monitoring wells tended to clog quickly, but it was possible to use defective sleeve pipes (sleeve pipes with leaking sleeves) as dewatering wells. In some respects, the leaking sleeve pipes were a blessing in disguise. Because of the slow curing grout, it was possible to inject sections where the grout had prematurely permeated before the grout had fully cured. Proper planning was required to avoid curing of the fringe grout before being regrouted. For this reason, only second or third phase grouting was performed on Fridays.

During Trench I grouting, a rise in Trench II water levels was observed. Although it is somewhat speculative, we believe that a significant volume of grout may have passed through the deteriorated berms between the trenches and into Trench II. It is also possible that the two trenches were connected when constructed. Grout breakouts in the slope east of Trench I leading down towards the drainage ditch were indicative of horizontal fracturing of the (weak) berms between the trenches or movement of grout along open joints in the saprolitic subsurface formation.

Inevitably, small amounts of contaminated grout broke through to surface or simply travelled to the sides of the pads, as a result of routine grouting operations. These conditions gave rise to stricter health and safety protocol: radiological work permits (RWP) were required to enter and work in the exclusion zone. The remainder of the grouting operations were conducted under RWP.

A few pipes were contaminated and designated radiologically contaminated (RAD) holes. The RAD holes were grouted in one stage with a top-of-hole packer.

The final stage of grouting using acrylamide solution grout was executed within 2 weeks after completing the last cement based suspension grouting stage. A higher than expected residual hydraulic conductivity was noted during acrylamide grouting. All of the sleeve pipes were amenable to acrylamide grout and were grouted until refusal. The refusal pressure for acrylamide was adjusted to 40 psi to avoid hydro fracturing through the acrylamide, either to the trench boundary or surface. Acrylamide grouting operations are shown in Figure 28.

Figure 28. Multiple hole grouting during acrylamide injection. It was difficult to maintain even flow in more than two lines, so the number of zones being injected simultaneously was reduced to two.

The following statistics summarize Trench I grouting, including bulkhead grouting (note that grout quantities are approximate):

- total volume of grout: 65,000 gal.;
- volume of type III portland cement based grouts: 53,000 gal. - 82 percent of total volume injected;
- volume of ultrafine cement based grouts: 5,000 gal. - 8 percent of total volume injected;
- volume of acrylamide grouts: 7,000 gal. - 10 percent of total volume injected;
- total theoretical trench volume: 130,000 gal.; and
- filling ratio: 50 percent.

The average grout take per sleeve pipe was approximately 750 gallons, including:

- 615 gal. of type III portland cement based grouts,
- 60 gal. of ultrafine cement based grouts, and
- 75 gal. of acrylamide grouts.

The void filling ratio of 50 percent was higher than the 30 to 40 percent that was predicted. Grout takes predicted prior to construction were as follows:

- 70 percent of type III portland cement based grouts,
- 8 percent of ultrafine cement based grouts, and
- 22 percent of acrylamide grouts.

Bulkhead grouting increased the relative percentage of type III portland cement based grouts that was injected. It is suspected that a significant volume of grout permeated the crushed stone pads and possibly travelled to other trenches. The actual void filling ratio is probably slightly lower than 50 percent. The discrepancies between expected and actual percentages are not surprising since it was impossible to estimate both the expected total accessible void space and the expected size distribution of the accessible void space, given there was no geotechnical information available during the design phase of the project.

3. Trench II

Grouting of Trench II benefited from the experiences gained at Trench I. The general approach remained consistent with that developed in Trench I. During initial hydraulic conductivity testing, 17 of the first 33 sleeve pipes were labeled by the health physics technicians as "high radiation" holes due to high readings on their radiation detectors. Implications were that a higher level of personal protective equipment (PPE) was required to work around these high radiation levels. However, based on findings during similar events in Trench I, it was learned that the cement grout precipitated certain radionuclides out of solution, reducing the radiation level. This knowledge raised the notion that by injecting a pre-determined volume of grout into the designated "high radiation" holes; the radiation levels could be lowered to limits consistent with the current degree of PPE worn. This method was, indeed, successfully implemented. Additionally, some of the "high radiation" holes were grouted using a single top-of-hole packer to reduce exposure of workers to radiation hazards.

Cement based suspension grouts were being injected in Trench II concurrently with acrylamide solution grouting in Trench I. Due to higher than expected residual hydraulic conductivity noted during acrylamide grouting (after regular and ultrafine cement grouting) in Trench I, a modified approach was developed for cement based grouting in Trench II. During grouting of Trench I and initial stages of Trench II, several important observations made a different grouting methodology possible:

- regardless of the methodology used, grout leaks to surface were inevitable;
- these leaks could be contained and handled in a safe manner (i.e. the contamination was controlled); and
- the crushed stone pads worked well to direct surfacing groundwater and grout away from the work area without it coming out on top of the pads.

Grouting proceeded in a stage up fashion using a single-fold packer to inject two sleeves at a time. Once a stage was refused, or such a large volume of grout was injected that it was obvious that grout was being pushed a long way from the injection point, the packer was moved up to the next stage. The aim was to produce a slowly advancing grout front that

would displace water in the trenches. If refusal was not obtained in a specific zone, that zone was re-injected the following day. All zones were grouted until refusal, until grout leaked to surface, or until hydro fracturing occurred. Zones which did not refuse were typically given 24 to 48 hours to allow the subsurface to relax and/or the grout previously injected to gel enough to close off the easy pathways to surface, before re-injection. It was important not to allow the regular cement based grout front to advance too far ahead of the injection point. With the grout formulations being used, a sleeve pipe encapsulated in grout would remain re-injectable for two to three days. The grouting schedule and sequence was adjusted to prevent the grout front from moving too far ahead. The aim was to eliminate the possibility that some sleeves would be prematurely refused from grout flowing around them and setting.

Typically, two to three stages were injected with the regular cement based grouts, one to two stages were injected with ultrafine cement based grouts, and finally one stage was injected with the acrylamide grouts. Usually, each sleeve was injected between four and six times.

The following statistics summarize Trench II grouting (note grout quantities are approximate):

- total volume of grout: 27,000 gal.;
- volume of type III portland cement based grouts: 20,000 gal.- 74 percent of total volume injected;
- volume of ultrafine cement based grouts: 5,000 gal.- 9 percent of total volume injected;
- volume of acrylamide grouts: 2,000 gal. - 7 percent of total volume injected;
- total theoretical trench volume: 56,000 gal.; and
- filling ratio: 48 percent.

The average grout take per sleeve pipe was approximately 730 gallons, including:

- 540 gallons of type III portland cement based grouts,
- 140 gallons of ultrafine cement grouts, and
- 50 gallons of acrylamide grouts.

The void filling ratio of 48 percent was higher than the 30 to 40 percent that was predicted prior to construction. It was expected that the total grout quantities would have consisted of:

- 70 percent of type III portland cement based grouts,
- 8 percent of ultrafine cement based grouts, and
- 22 percent of acrylamide grouts.

The decrease in relative percentage of injected regular cement based suspension grouts from 80 percent for Trench I to 74 percent for Trench II is due to there being no bulkhead required at the down-hill end of Trench II.

During acrylamide grouting, it was noted that about 50 percent of the sleeve pipes did not take any acrylamide grout because of an improved grouting program in Trench II. Hence, the relative percentage of acrylamide grout injected dropped from 10 percent for Trench I to 7 percent for Trench II. The modified methodology (pertaining to the injection of suspension

grouts) reduced the in situ hydraulic conductivity remaining after the cement based grouting stages and was followed for Trench III and IV grouting.

4. Trenches III and IV

Grouting of Trenches III and IV followed the same methodology as used for Trench II. At this stage of the project, the concern heightened for meeting schedule because of anticipated inclement changes in weather conditions. Production rates were increased by grouting in both Trenches III and IV simultaneously. The same total number of lines (and packers) was used, but grout was injected at higher flow rates. Figure 29 shows multiple hole grouting during cement based grouting operations at Trench IV.

Figure 29. Multiple hole grouting during cement based grouting at Trench IV. Typically four zones were injected simultaneously. Notice each line is color-coded and distances are marked so packer could be easily inserted to the proper depth to straddle a specific zone.

Grout leaks were more frequent during grouting of Trenches III and IV than during grouting of Trench II. It has been speculated that this may have been due to the increased production rate. We are of the opinion that this is only partly true because the total grout flow rate was being divided between two trenches. It is more plausible that the increased leakage rate was due to a combination of increased slope of the trenches and more narrow width of the trenches relative to the hole pattern. Also, Trench IV was considerably more shallow than the other trenches, and probably filled more quickly with grout.

As discussed in an earlier section, the actual boundaries and orientations of Trenches III and IV were different than predicted (refer to Figure 30). Review of the pipe driving logs by Dr. Dale Huff, aided by a GIS program, showed that the downhill region of Trench IV was actually part of Trench III. Pipe access was limited along the northern-most side of Trench III because of the presence of a culvert. Bulkhead grouting was performed through pipes located at the downhill ends of Trenches III and IV (refer to Figure 30). The bulkhead was grouted with a methodology similar to that used for production grouting, except a more thixotropic grout was used. When possible, a larger volume of grout was injected during initial stages, and grouting was performed in fewer stages. The aim was to produce a steep grout front with high angle of natural repose.

The two trenches were divided into three distinct sections for grouting purposes. Grouting continued as per the same grouting methodology as used in Trench II.

The following statistics summarize Trench III grouting (note grout quantities are approximate):

- total volume of grout: 18,600 gal.;
- volume of type III portland cement based grouts: 14,300 gal. - 76 percent of total volume injected;
- volume of ultrafine cement based grouts: 2,300 gal. - 12 percent of total volume injected;
- volume of acrylamide grouts: 2,000 gal. - 12 percent of total volume injected;
- total theoretical trench volume: 41,000 gal.; and
- filling ratio: 45 percent.

Note that in the above statistics, the two pipes at the bottom of the Trench III hole pattern used for bulkheading are included. The average grout take per sleeve pipe was approximately 465 gallons, including:

- 354 gal. of type III portland cement based grouts,
- 56 gal. of ultrafine cement based grouts, and
- 56 gal. of acrylamide grouts.

The following statistics summarize Trench IV grouting (note grout quantities are approximate):

- total volume of grout: 7,800 gal.;
- volume of type III portland cement based grouts: 4,600 gal. - 60 percent of total volume injected;
- volume of ultrafine cement based grouts: 2,500 gal. - 32 percent of total volume injected;
- volume of acrylamide grouts: 700 gal. - 8 percent of total volume injected;
- total theoretical trench volume: 16,000 gal.; and
- filling ratio: 48 percent.

The average grout take per sleeve pipe was approximately 520 gallons, including:

- 312 gal. of type III portland cement based grouts,

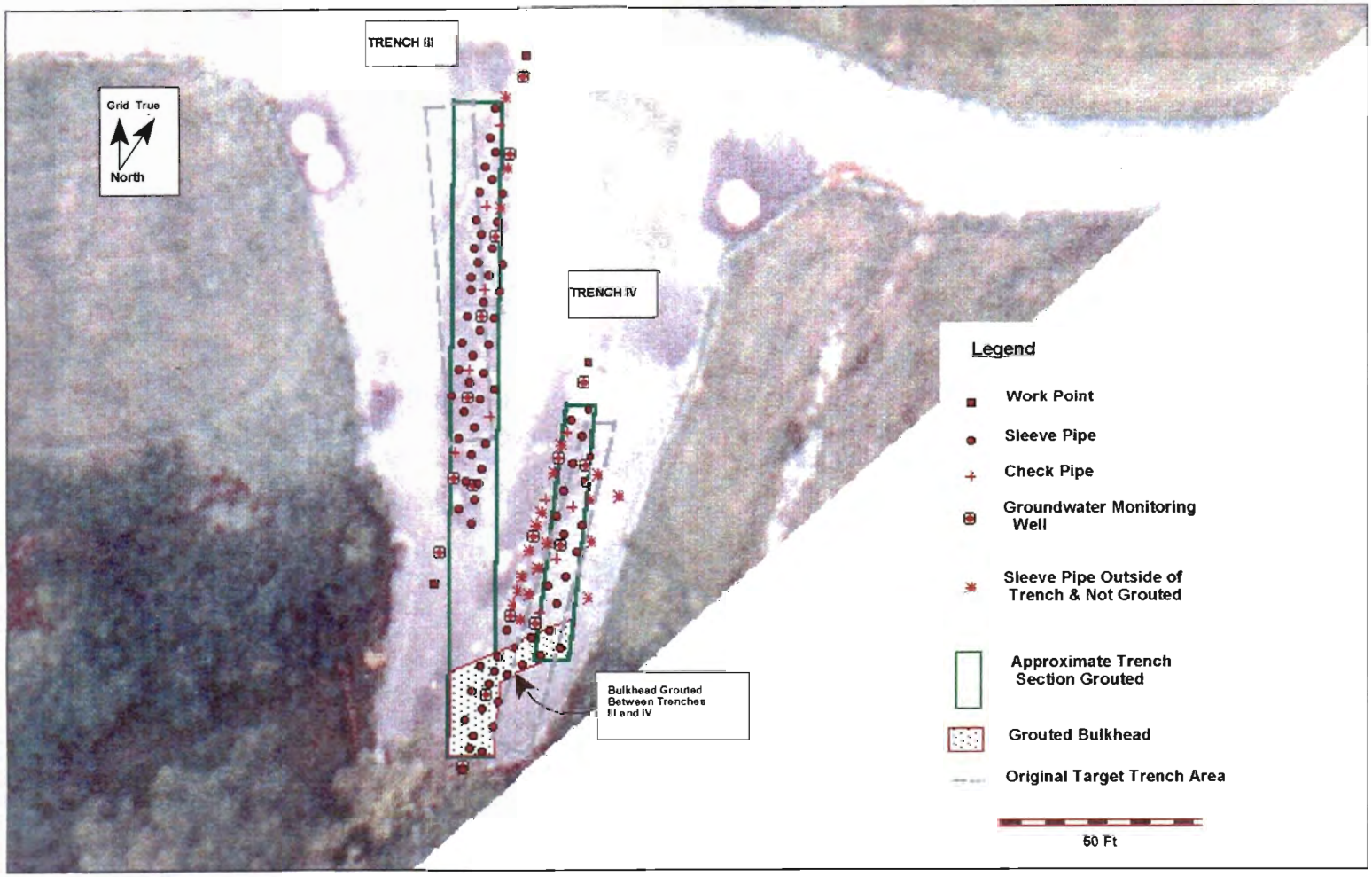


Figure 30. As-built locations of grout pipes near Seep 4.

- 166 gal. of ultrafine cement based grouts, and
- 42 gal. of acrylamide grouts.

The following statistics summarize bulkhead grouting at the southern end of Trenches III and IV (note grout quantities are approximate):
- total volume of grout: 8,500 gal.;
- volume of type III portland cement based grouts: 6,000 gal. - 71 percent of total volume injected;
- volume of ultrafine cement based grouts: 1,700 gal. - 20 percent of total volume injected;
- volume of acrylamide grouts: 800 gal. - 9 percent of total volume injected;
- total theoretical trench volume: 15,000 gal.; and
- filling ratio: 56 percent.

The average grout take per sleeve pipe was approximately 470 gallons, including:
- 330 gal. of type III portland cement based grouts,
- 95 gal. of ultrafine cement based grouts, and
- 35 gal. of acrylamide grouts.

The filling ratio was close to 50 percent for all three trench sections.

XVI. RESIDUAL IN SITU HYDRAULIC CONDUCTIVITY TESTING

A. Intent

The purpose of the residual (after grouting) in situ hydraulic conductivity testing was to assess the effectiveness of grout spread and residual in situ hydraulic conductivity of the grouted waste materials. It was originally intended to measure residual in situ hydraulic conductivity in both randomly selected sleeve pipes and all check pipes for calculation of the average residual in situ hydraulic conductivity. However, time constraints did not allow for measurement in the sleeve pipes.

B. Requirements

The grouting subcontractor was required to conduct the hydraulic conductivity tests with the same procedures outlined in Chapter X. Initial In Situ Hydraulic Conductivity Testing. The residual in situ hydraulic conductivity was to be tested at each check pipe and at each sleeve pipe as directed by the Grouting Engineer (ECO). Sleeves selected for testing were to give a reasonably representative hydraulic conductivity profile of the trenches.

C. Actual Execution

After grouting was completed, residual in situ hydraulic conductivity tests were performed. Initially, attempts were made to perform the tests using the grouting subcontractor's equipment; however, the flow meters used for grouting and routine hydraulic conductivity testing could not quantify the very low flow rates associated with measuring the residual in situ hydraulic conductivities. A "no flow" reading (zero total flow) was recorded in all but two check pipes at a gauge pressure of 10 psi while isolating the lower portion of the check pipe corresponding to the full depth of the waste disposal trench that was grouted. Each zone was isolated using a single packer and the tests were continued for a duration of 10 to 15 minutes. The two holes that did not register a "no flow" condition were actually located in the bulkhead at the downhill end of Trench IV. It was not intended for the bulkhead to meet the same hydraulic conductivity criteria as the targeted trench sections and the bulkheads were not grouted as rigorously as the targeted trench sections. Note that the lower limit of measurement with the grouting subcontractor's flow meters was about 0.5 liters per minute (lpm). To measure hydraulic conductivity criteria in the range of 1×10^{-6} cm/sec at pressures that would not damage the grouted mass, the "permeameter" shown in Figure 31 was fabricated by LMES. The grouting subcontractor was permitted to commence demobilization at this point since performing hydraulic conductivity tests with the newly constructed permeameter were outside the scope of work outlined in the construction specifications.

1. Apparatus

A single pneumatic packer manufactured by Aardvark, having a bladder length of 30.48 cm (12 in.) was used to seal the top of the check pipe. The grouting subcontractor provided the packer. The packer was inflated using a separate system consisting of nitrogen supplied from a tank via 3/16-in. high-pressure nylon tubing. The packer and tubing utilized are shown in Figure 32. The volume of water flow was measured directly using a graduated flow tube. The apparatus had both a large, 4-inch diameter flow tube, and a small, 1-inch diameter flow tube, that could be used, depending on the rate of water take. Having two sizes of flow tubes increased the range of possible flows that could be measured. The test water was tap water stored at ambient temperature in the field. Although not measured, ambient testing water temperature is estimated to have ranged from about 65 to 75 degrees F. The permeameter was designed to be able to measure a range of hydraulic conductivities varying from about 10^{-8} cm/sec to 10^{-5} cm/sec for a 5-min test duration with an average stage length of 2 meters under an effective pressure head of 10 psi.

2. Procedure

Representatives of ECO performed testing during October 28 and 29, 1996. The procedure was an adaptation of the Lugeon test described by Houlsby[9]. Test intervals were selected based on grouting records. In each case, water testing was performed in the lower portion of the check pipe corresponding to the full depth of the waste disposal trench that was grouted. The test section was isolated with a single packer. It was the opinion of the

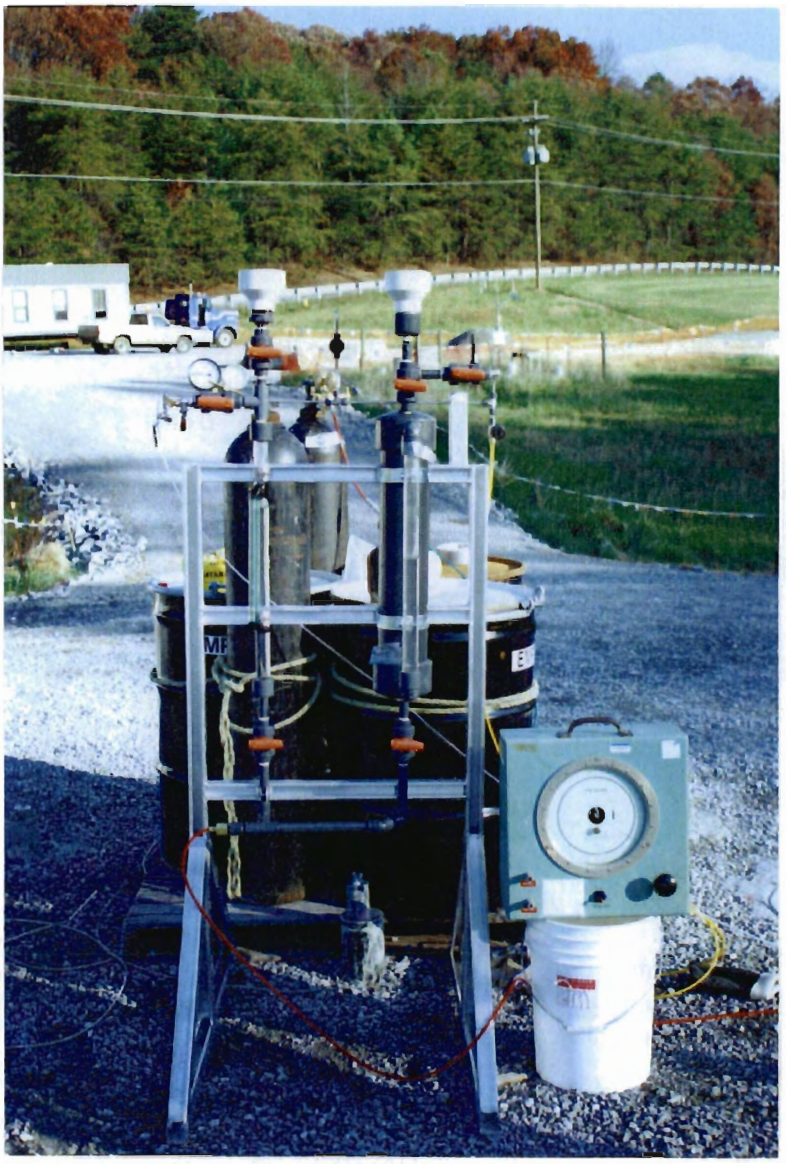


Figure 31. The permeameter built for final hydraulic conductivity testing. All hoses to packers were replaced with plastic tubing to avoid measuring volume changes related to hoses flexing. The two chambers were used to measure the flow rate - the large chamber for zones with higher permeablities, the small chamber for zones with lower permeablities. The chambers were pressurized with a high-pressure nitrogen bottle. Pressure was regulated and monitored with a large-dial pressure gauge requiring two revolutions to register 15 psi.

Figure 32. (Bottom) Initial tests in a sealed pipe showed that flexing in the hoses leading to the packer added a large uncertainty to the small flow rates we were trying to measure. (Top) The hoses were then replaced with the red plastic tubing . The clear plastic tubing was used to inflate the packers.

project team that testing the full grouted depth in one stage would give a better measurement of the average residual in situ hydraulic conductivity as compared to using smaller stages. With the use of smaller stages, it would be impossible to compute an average in situ hydraulic conductivity for the entire column of waste materials if one or more of the stages were less permeable than the practical limits of the equipment being used. Effective pressures at the test zone were maintained below about 10 psi to prevent hydro fracturing of the grouted mass. Since the casing grout had been "cracked" during initial in situ hydraulic conductivity testing and prior experience on site indicated that effective pressure head less than 10 psi would initiate flow out of the sleeve pipes, a high "cracking pressure" was not used during residual water testing for fear that the grouted mass would be hydro-fractured, as well.

First, both permeameter tubes were filled with water. Next, the check pipe was filled with water to reduce the amount of water that would have to be supplied from the permeameter to fill the test section. This helped to balance the gravity head difference across the packer inside the check pipe. Balancing the gravity head was important since it helped to reduce the leakage rate around the packer and back up through the inside of the check pipe. The packer assembly was lowered down into the check pipe to the desired test depth. The packer was inflated to 350 psi. The supply side of the permeameter was opened to allow pressure head of 5 psi to be placed on top of the permeameter tube. The outflow valve was opened to allow water to flow slowly down into the "packed-off" test section. Typically, testing was attempted first using the smaller permeameter tube. If the flow rate was too great, flow was switched over to the larger tube. The flow was given a few minutes to stabilize, then the volume of water flow was recorded for three successive time intervals, typically 5 minutes each. In some cases, the available volume of water contained in the permeameter tube was not sufficient to supply three tests. Thus, testing would be temporarily stopped after completion of the test increment and prior to depletion of the test water in the permeameter tube. The tube would be re-filled and the test would be resumed. The volume of flow was recorded for each test interval. At completion of the test, the water flow was turned off. The pressure head was released. The inflation pressure to the packer was turned off and vented to the atmosphere. The down-hole equipment was then removed from the check pipe. A total of twenty-three tests were performed.

Hydraulic conductivity tests were also performed in an un-ported section of sleeve pipe at the ground surface to assess water losses around the packer. One end of the pipe was welded shut. The pipe was clean (had not been smeared/coated with grout) on the inside. The testing procedure was similar to that described previously. Three 5-minute tests were performed.

3. Results and Analysis

A calculation summary is included as Appendix F of Volume 3. A more detailed report of the residual in situ hydraulic conductivity testing is being prepared under separate cover. Several manipulations of the raw data were made prior to calculating the residual in situ hydraulic conductivity using the following equation[5] which is a simplification of the

mathematical formulae cited by Naudts[6] based on his prior experience in soil grouting with sleeve pipes:

$$k = \frac{Q}{4 \times pi \times r \times L \times P_{eff}} \qquad \text{(Eqn. 1)}$$

where:

- k = in situ hydraulic conductivity in meters per second
- Q = water flow (water take) in cubic meters per second
- r = radius of grout hole in meters
- L = correction factor (dimensionless) which equals the length of the zone injected divided by 1 meter
- P_{eff} = effective injection pressure in meters of water column head (gage pressure + hydrostatic head - head losses).

First, for the majority of the data, the flow rate decreased slightly with time, suggesting that the flow rate had not completely stabilized while performing tests of such short duration. As discussed by Houlsby[8] the decrease in water take over time could have been a result of crack filling or consummation of the water by the hydrating Portland cement grouts. In cases where the flow rate decreased over each time increment, the flow rate for the third increment of flow measurement was taken as the "raw flow rate." In cases where the flow rate did not decrease with time, the flow rates for the second and third increments of flow measurement were averaged to compute the "raw flow rate."

As described earlier, hydraulic conductivity tests were performed with the packer placed in a clean (un-grouted) section of un-ported sleeve pipe with a closed end to check for leakage of water around the packer during testing. The test indicated that the packer leaked slightly. The packer leakage rate was calculated as described above. The packer leakage rate was subtracted from the measured "raw flow rate" to yield the "corrected flow rate." The "corrected flow rate" was used in Eqn. 1. In cases where the packer leakage rate equaled or exceeded the "raw flow rate," the flow rate was calculated as the lowest increment of flow that could be measured on the graduated permeameter tubes. For the small flow tube, the smallest increment of measure was about 1 cc/min; for the large flow tube, the smallest increment of measure was about 3 cc/min.

Head losses to overcome sleeve resistance were conservatively estimated to be 2 psi based on measurements made during initial water testing.

The calculated residual in situ hydraulic conductivities are summarized in Table 1.

TABLE I
RESULTS OF RESIDUAL IN SITU HYDRAULIC CONDUCTIVITY TESTING

Check Pipe Number	Hydraulic Conductivity (cm/sec)
1002*	1.4×10^{-7}
1012	5.1×10^{-6}
1022	3.0×10^{-6}
1032	1.4×10^{-6}
1042*	4.4×10^{-7}
1055	9.6×10^{-7}
1075*	2.7×10^{-7}
2002	4.6×10^{-6}
2013*	2.9×10^{-7}
2024	1.1×10^{-7}
2031	1.5×10^{-5}
2036	2.3×10^{-6}
2046	1.3×10^{-6}
3007	2.2×10^{-6}
3013	7.1×10^{-7}
3019*	1.4×10^{-7}
3029	2.5×10^{-6}
3040	2.6×10^{-6}
3049*	1.3×10^{-7}
4023	5.2×10^{-7}
4033	8.6×10^{-7}
4042	4.4×10^{-6}
4053*	1.0×10^{-7}

Geometric Mean: 0.90×10^{-6} cm/sec
Simple Arithmetic Mean: 2.1×10^{-6} cm/sec
Range of Values: 1.0×10^{-7} cm/sec to 1.5×10^{-5} cm/sec

*Water take was less than packer leakage rate.

The average residual in situ hydraulic conductivity was calculated statistically as the geometric mean. Groundwater hydrologists at ORNL typically use this type of average when reporting hydraulic conductivity values for natural geologic formations. It is

believed that the geometric mean (or geometric average) provides a better statistical representation of the "average." When using the simple arithmetic mean, the mean or average value can be significantly skewed based on a few outlying data points.

Calculated residual in situ hydraulic conductivities ranged from 1.0×10^{-7} cm/sec to 1.5×10^{-5} cm/sec. The geometric average was calculated to be 0.90×10^{-6} cm/sec. Twenty-two of the twenty-three test results yielded hydraulic conductivities on the order of 10^{-6} cm/sec or less. The one test result greater than this was only slightly greater, being 1.5×10^{-5} cm/sec. Twelve of the twenty-three measured hydraulic conductivities were less than 1×10^{-6} cm/sec and seven of the twenty-two values were lower than could be measured with the testing equipment due to packer leakage.

XVII. CHRONOLOGY OF EVENTS

A breakdown of important project activities is summarized below:

- October 1995 - Water level readings were taken on WAG 4 site to base design information.
- December 1995 to March 1995 - Project scoping, design, and contract preparations were performed.
- February 14, 1996 - Pre-proposal meeting was held.
- March 1996 - Site preparation work was initiated.
- April 1996 - Tenders were released to four pre-qualified bidders.
- May 24, 1996 - Grouting subcontract award was made.
- May to July 11, 1996 - Pre-construction preparation was performed.
- July 12, 1996 - Pre-construction meeting was held.
- July 13 to July 26, 1996 - Grouting subcontractor mobilized.
- July 22, 1996 - Initial observation well pipes and sleeve pipes were driven in Trench II.
- July 23, 1996 - Driving operations were moved to Trench I until problems in Trench II could be resolved.
- July 27, 1996 - Water and electrical power were finally made available on site.
- July 29, 1996 - First casing grout was installed (Trench I holes).
- August 5, 1996 - First hydraulic conductivity tests were performed.
- August 8, 1996 - Production pipe driving began in Trench II.
- August 13, 1996 - Bulkhead grouting commenced downstream of Trench I (first day of grouting).
- August 16,1996 - Production pipe driving began in Trench IV.
- August 20, 1996 - Bulkhead grouting was completed in Trench I; approximately 16,000 gallons were placed.
- August 20, 1996 - Phase I grouting commenced in Trench I.
- August 27, 1996 - Production pipe driving began in Trench III.
- September 6, 1996 - First acrylamide grouting was performed in Trench I.

- September 13, 1996 - Cement based suspension grouting operations were completed in Trench I; 58,000 gallons of cement based suspension grouts were injected.
- September 17, 1996 - Production grouting began in Trench II.
- October 1, 1996 - Acrylamide grouting was completed in Trench I, marking completion of all grouting in Trench I. Total volume injected (including bulkhead): 65,000 gallons, including 7,000 gallons acrylamide, 5,000 gallons ultrafine cement based grouts, and 53,000 gallons type III portland cement based grouts.
- October 4, 1996 - Cement based suspension grouting was completed in Trench II; 25,000 gallons injected: 20,000 gallons type III portland cement based grouts and 5,000 gallons ultrafine cement based grouts.
- October 7, 1996 - Cement based suspension grouting started in Trenches III and IV.
- October 8, 1996 - Acrylamide grouting was completed in Trench II, marking completion of grouting in Trench II. Total volume injected: 27,000 gallons, including 2,000 gallons acrylamide, 5,000 gallons ultrafine cement based grouts, and 20,000 gallons type III portland cement based grouts.
- October 12, 1996 - Trench IV grouting was completed. Total volume injected: 7,800 gallons: 700 gallons acrylamide, 2,500 gallons ultrafine cement based grouts, and 4,600 gallons type III portland cement based grouts.
- October 21, 1996 - Trench III grouting was completed. Total volume injected: 18,600 gallons: 2,000 gallons acrylamide, 2,300 gallons ultrafine cement based grouts, and 14,300 gallons type III portland cement based grouts. Grouting in all trenches was completed.
- October 29, 1996 - Residual in situ hydraulic conductivity testing was completed.
- October 31, 1996- Grouting subcontractor fully demobilized.

XVIII. CONCLUSIONS

Professionally executed, low-pressure permeation grouting is a powerful method of source control for solid radioactive wastes in burial trenches. Although not investigated, it is possible to chemically treat the wastes, depending on the chemistry of the grouts injected. Hydrologic monitoring is ongoing to evaluate the overall effectiveness of grouting in reducing the off-site transport of ^{90}Sr.

The multi-phase, multi-stage, low-pressure sleeve pipe grouting process, utilizing a multitude of carefully formulated portland and ultrafine cement based suspension grouts, in conjunction with acrylamide solution grouts, was successful in dramatically reducing the in situ hydraulic conductivity of trenches filled with a myriad of radioactive and other wastes of an unknown nature. With proper engineering controls, the work can be performed safely without environmental insult and with generation of only small volumes of wastes.

In spite of the fact that no secondary grout pipes were placed, the average residual in situ hydraulic conductivity of the materials in the trenches, as measured through more than 20 check pipes, was reduced to less than 1×10^{-6} cm/sec. This value corresponds to a reduction of the in situ hydraulic conductivity (as measured before grouting) by a factor of 10,000 to 10,000,000. The ability to achieve this low target value is believed to be primarily due to the application of repeated injections of multiple grout formulae into the same zones of the trenches, application of advanced engineering principles such as the apparent Lugeon theory, tight monitoring/direction of the grouting program, and proper quality control. Also key to the success of the grouting program were the use of cement based suspension grouts with delayed set times which made it possible to inject the same zones of the waste disposal trenches multiple times before the grout reached initial cure.

Finally, the use of semi-remotely driven sleeve pipes reduced potential exposure of workers to contaminants and provided the vehicle for in situ hydraulic conductivity testing and multiple injections of grouts. Having been left in place, the sleeve pipes can be re-accessed in the future for hydrologic evaluation and remedial grouting, in the unlikely event that future grouting would become necessary.

XIX. LESSONS LEARNED

Some of the "lessons learned" from the project are described below:

1) Cement based suspension grouts have the ability to cause certain radionuclides to precipitate out of solution. Even where the highest radiation levels were detected in some contaminated pipes, radiation levels dropped off dramatically after an initial stage of grouting.

2) The "top-hat" system proved to be an invaluable tool for containing flush water and groundwater during flushing and pumping operations.

3) Specialized sensitive instrumentation (such as a permeameter) is required to measure the low target residual in situ hydraulic conductivities resulting from a grouting program such as this one. The monitoring equipment for permeation grouting operations utilized by the grouting subcontractor exceeded industry standards, but it was not capable of quantifying the very low flow rates required to verify the target residual hydraulic conductivities. On future projects, suitable instrumentation should be specified if the grouting subcontractor is required to perform residual (after grouting) in situ hydraulic conductivity tests.

4) Accurate trench delineation is a critical factor in the achievement of a quality end product. We were fortunate that the grouting subcontractor was cooperative. In future projects it is advisable to accurately determine trench boundaries prior to tender to avoid unforeseen delays and claims for extra costs by the grouting subcontractor. A

less desirable option would be to include provisions for exploratory work in the contract package and include more flexibility as to grout hole layout.

5) A minimum of four grout stages was required to reach the targeted permeability value. Ultimately, injection of solution grout was required to achieve the end result during the fourth or fifth stage of grouting. There was no "overkill" in the rather complicated grouting process selected for this project. Use of cement based grouts with delayed set times and use of repeated injections of a multiple of grout types and formulae were key to achieving low in situ hydraulic conductivity. Use of initial in situ hydraulic conductivity testing and application of the amenability theory provided a rational approach to performing the grouting operation. (The work was not done blindly!) The importance of proper quality control testing should not be overlooked, as well.

6) The fast-track production pace of the project did not allow for any "scientific experimentation" or "optimization" of grouting procedures which could yield considerable cost savings on future projects. This was a pilot project to demonstrate the viability of this technology. More could have been attempted, ultimately achieving the same target results. On future projects, more time should be allotted in the schedule to optimize grouting procedures, particularly if a large project is undertaken.

7) The crushed stone pad provided a clean and safe work area. An improvement would be to stitch seams in the geotextile, rather than just overlapping them. This would greatly reduce the potential for grout surfacing on the working pads. Also, consideration should be given to the use of a moving platform, without placement of crushed stone.

8) Depending on location and orientation of a trench to be grouted and its proximity to other site features, the creation of grout curtains or grout bulkheads at strategic locations can be a valuable method of containing future grout injections and controlling the movement of contaminated groundwater.

9) Although the sleeve pipes generally performed satisfactorily, some were nevertheless damaged (i.e. some sleeve pipes bent or buckled inside the trench when driven to refusal as a result of limited lateral support). Some pipes were bent to a point that the pneumatic packer could not be lowered to full depth. Also, some of the rubber sleeves leaked.

10) As a solution to no. 9 above, four recommendations are made:

a) In future projects, stiffer sleeve pipes should be specified or mandrels added - robust enough not to bend/kink when the above-mentioned condition is experienced (i.e. when no lateral support is present to full depth of pipe).

b) Smaller diameter packers should be used. The small packers work well in bent/kinked sleeve pipes. The smaller diameter packers were able to "sneak" by the bend in the sleeve pipe.

c) All sleeve pipes should be tested against negative pressure in a test chamber. This test should be conducted by the owner's representative and **NOT** by the grouting subcontractor.

d) When hard driving conditions are anticipated, a continuous sleeve protector should be used.

11) The grouting subcontractor should not perform quality control. The engineer, on behalf of the owner, should perform this task.

12) De-airing/anti foaming additives should be used in future cement based formulations. This is more critical for ultrafine cement based suspension grout formulations as compared to the type III portland cement based suspension grout formulations. The cause for the foaming is related to the retarder/deflocculator used in this project.

13) A more continuous grouting operation is warranted (i.e. either by automated batch plants, more colloidal mixers, better set-up, etc.). Higher production rates actually benefit the grouting from a technical standpoint. This requires more site supervision, but the time required to grout a trench would be reduced, as well as reducing the risk that the grout on the fringes sets before being re-grouted.

14) Prior to tender, more clear stipulation of training requirements for people working in specific areas is required (i.e. an individual working in the clean support zone does not require the same training required of an individual working in the exclusion zone).

15) The grouting subcontractor must have sufficient back up key personnel, properly trained, prior to the start of the operation.

16) The preparation of future technical specifications requires the expertise, experience and knowledge of specialists in the grouting field. However well intended, there should be less interference and second guessing regarding the technical aspects of the grouting program from outside groups during the preparation of the tender documents.

17) Any large contract should be set up in phases. If a grouting subcontractor does not perform to the specified standards, the owner should have the opportunity to enforce changes specified at the end of any phase. Also, a bonus/penalty system, as suggested for this project, should be considered further.

18) The prequalification of qualified grouting subcontractors is a must. A "weighted" bidding/tendering process (such as an evaluated procurement) should be considered for future tenders to help ensure that the most qualified group prevails. The technical package should carry similar, if not more weight, than the pricing package.

19) There is a direct relationship between knowledge of subsurface conditions in the trenches and the cost and probability for success of the grouting program. Little was known about subsurface conditions on this project. Consequently, potential bidders had no real basis for estimating pipe driving and grouting difficulty. This uncertainty tended to increase the cost of this project. Experience gained from this project should help to reduce costs of future similar grouting projects.

20) The grouting sequence greatly reduced dewatering requirements. For the most part, groundwater was displaced outward into the formation or uphill in the trench. Only groundwater that rose up in the groundwater monitoring wells and sleeve pipes with failed sleeves was pumped. It would have been very difficult and costly to have completely dewatered the trenches prior to and during grouting. The high cost is primarily associated with treatment cost. Trenches I and II were at the confluence of several trenches. Trenches in both seep areas were hydraulically connected to other trenches in the vicinity.

21) The groundwater monitoring wells clogged early in the grouting process. This may have been due to the fine screens used in the wells. The groundwater monitoring well design should be improved.

22) Approval should be sought from environmental regulators to allow spent grout flush water to be injected into the trench ahead of the grout front as a form of pre-treatment. This would help to reduce worker exposure to contamination and greatly reduce the volume of waste generated by the grouting operation.

23) Management of grout breakout to the surface and its impact on grouting procedures must be carefully planned and implemented. It is imperative that environmental regulators understand that some grout breakout is inevitable.

XX. DEFINITIONS OF TERMS

Stage of a grouting operation: A stage in a grouting operation in a particular zone/hole refers to the sequence in a series of grouting activities, conducted in the same zone through the same sleeves on the same sleeve pipe at different times.

Check pipe: A check pipe is identical to a production grouting sleeve pipe, except its purpose is primarily for observation/monitoring and evaluation. However, a check pipe can be used for grouting, if necessary, for more effective void filling.

Flush water: The water used for flushing of sleeve pipes (via open pipe or deflated packer) after grouting.

Contaminated water or grout: Water or grout that has been in contact with radioactive or hazardous substances, causing it to be contaminated.

Casing grout: A low strength, non-sag, slowly-curing bentonite/cement based suspension grout injected via each sleeve in each sleeve pipe to encase the sleeve pipe, preventing preferential pathways for the grout to travel along the outside of the sleeve pipe.

Hole/zone refusal: A zone or hole is brought to refusal once the grout flow rate to the hole/zone has been registered to be less than 0.5 litres/minute for a minimum of 20 minutes at the maximum allowable effective grouting pressure.

Head losses curve: A curve correlating various flow rates to pressure losses (line, elbow, sleeve and casing grout losses) head established with the same equipment, hose lengths, etc as for the in situ water testing operation/grouting operation.

In situ hydraulic conductivity test: refers to the permeability testing of a zone of a given thickness, via sleeve pipes using water (with or without precipitation enhancing agents) as the test fluid, conducted in one operation via a single or double packer.

Packer rods: Packer rods are pipes, connected to the packer and inserted inside the sleeve pipes, acting as the extension of the grout line.

Packer lines: Packer lines fulfil the same function as packer rods. The only difference is that instead of pipes, flexible lines or hoses of suitable diameter are connected to the packer, instead of a series of pipe(s).

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