DESIGN AND CONTROL OF CHEMICAL GROUTING: VOL. 1

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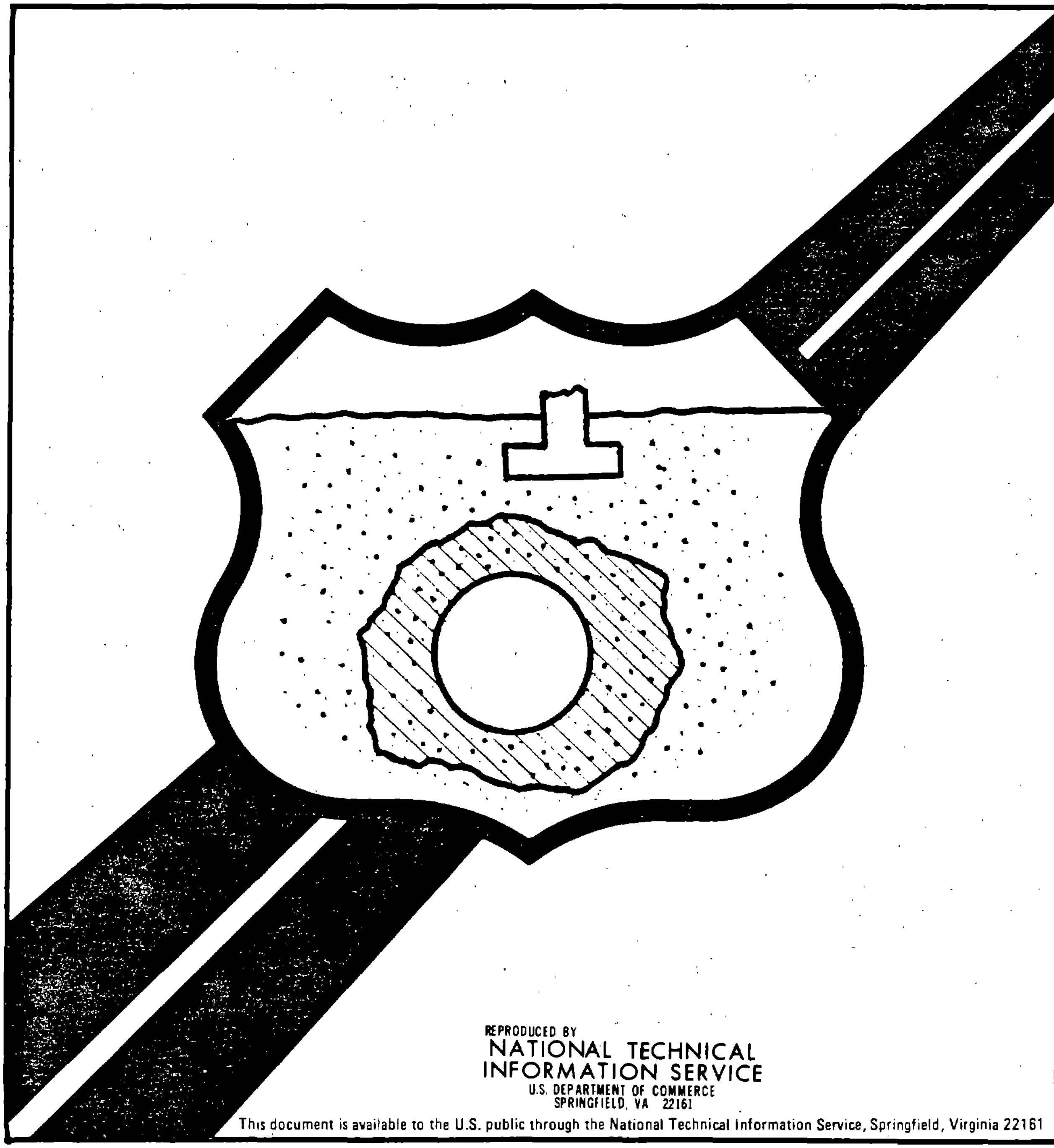

FOREWORD

After reviewing problems associated with the use of chemical grout injection to strengthen or render impermeable in situ soil masses that are to be excavated for transportation structures, the researchers addressed their efforts to improving concepts, controls and the resulting effectiveness of subsurface chemical grouting. The research included both laboratory and field work in order to make the results of the study most meaningful. The four volume report is being distributed as follows:

Volumes 1, 2 and 3 to other researchers in this field,
Volumes 3 and 4 to State Highway Agencies and to FHWA Regional and Division offices.

Copies of any or all volumes of the report are available to the public from the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, Virginia 22161. A fee is charged for reports furnished by NTIS.

Richard E. Hay

Richard E. Hay, Director
Office of Engineering
and Highway Operations
Research and Development
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NOTICE

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16. Abstract

This report presents the results of a laboratory and field research program investigating innovative methods for design and control of chemical grouting in soils. Chemical grouting practice is reviewed and standard evaluation and measurement techniques are presented. Problems particular to chemical grouting design and construction control are also discussed, and interviews with experts in the grouting industry are abstracted. Results of laboratory and field efforts to remotely evaluate grouting procedures and processes, including geophysical grout evaluation systems such as electrical resistivity, acoustic velocity, borehole radar, geotomography and acoustic emission techniques to monitor hydraulic fracture, are also presented. Finally, proof-of-concept tests were carried out on several on-going grouting jobs to evaluate the use of procedural controls to monitor grouting progress, and the application of acoustic emission techniques to detect and control hydraulic fracture.

This report is the first in a four-volume series. The others in the series are:

Vol. No.	FHWA No.	Title
2	FHWA/RD-82/037	Materials Description Concepts
3	FHWA/RD-82/038	Engineering Practice
4	FHWA/RD-82/039	Executive Summary

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APPLICABLE SI TO ENGLISH CONVERSIONS

0.3048 m	=	1 ft
3.28 ft	=	1 m
6.89 kPa	=	1 psi
0.146 psi	=	1 kPa
146 psi	=	1 MPa
1.97 ft/min	=	1 cm/second
1 U.S. quart	=	1.06 l = 9.464×10^{-4} m^3
1 U.S. gallon	=	4.24 l = 3.785×10^{-3} m^3
1 litre	=	1×10^{-3} m^3
0.265 l	=	1 U.S. gallon
1 in	=	25.4 mm
0.04 in	=	1 mm

CHAPTER 1—INTRODUCTION

Chemical soil grouting involves the injection of a chemical fluid into the soil interstices to bring about specific changes in subsurface soil properties, either to consolidate or strengthen the soil or to reduce or stop the flow of water. Chemical grouting of soils has been an accepted construction practice for nearly thirty years since the development of modern, single-shot chemical grouts in the 1950's. The use of chemical grouting in the United States has increased significantly over the past decade in the construction of several major urban mass transit systems. A variety of grouting systems and applications are now available, indicating a vigorous technology. However, certain characteristics peculiar to chemical grouting have tended to be a barrier to the complete acceptance of grouting technology. The correct application of chemical grouting results in no visible change at the ground surface. There is no accessible product to measure, weigh, or test for adequacy. Evaluation of a grouting contractor's work must be based on indirect measurements and inferences concerning conditions underground. Improved methods for the evaluation and control of chemical grouting are needed to verify that the grout has been placed where needed. In some cases, this may involve improvement in grouting technique, but more frequently the target technology is an improvement in the ability to measure grouting quality and performance.

This report summarizes the findings of a research study, to improve design and control techniques for chemical grouting in soils, with particular emphasis upon the correct selection of control and evaluation methods, tailored to the needs of specific grouting projects. It should provide a basis for continued improvements in the development of innovative Quality Assurance/Quality Control (QA/QC) methods. The application of the procedural controls and evaluation techniques to actual grouting practice, will do much to dispel the uncertainties regarding the location and reliability of chemical grout in treated soils.

This report is directed to the civil engineering community and should be of special interest to the design engineer, who may not be fully familiar with the grouting process. To select chemical grouting as a viable and economic construction alternative from among the many available soil support methods, the design engineer must be familiar with the design and conduct of grouting programs and have confidence in the evaluation methods used to provide quality control and quality assurance.

The control and evaluation techniques presented here are flexible and designed to be implemented to the extent justified by the work at hand. Thus, for major projects where the consequences of grouting failure would be severe, the full array of evaluation methods presented here may be used to insure proper execution of the work. For less important projects, or those in which the consequences of grout failure are relatively minor, the designer may elect to employ a less extensive quality assurance/quality control program.

This report begins with a brief review of current grouting technology and grouting applications. It then presents an overview of the research program, and then discusses the results obtained with each control and evaluation method, and discusses problems and weaknesses in chemical grouting design and construction control. The results of a case history review and consultations with experts in the field are distilled herein, and issues in acceptance of chemical grouting by the civil engineering community are also discussed.

The experimental plan follows. Here the research effort is discussed and the objectives and work plan are delineated. Detailed descriptions of the laboratory tests and the field sites where the research took place are given. Technical findings, including the grout distribution theory as it currently exists, and pertinent field data, are presented. A discussion of each control and evaluation method is then presented, showing the results obtained with each method, and its evaluation through the development of the experimental work. Equipment and operational recommendations are provided, where appropriate.

Finally, technical results and conclusions are summarized, and the methodology developed during the research program is described, giving the capabilities of each method and pointing to conditions under which its use is appropriate. Technical questions remaining to be solved are also identified, as well as recommendations for future research to improve the efficacy and increase the application of chemical soil grouting.

CHAPTER 2—BACKGROUND INFORMATION

CURRENT GROUTING TECHNOLOGY AND APPLICATIONS

A wide variety of materials and methods for chemical grouting are now available in the United States. A brief review of current grouting technology, applications and evaluation methods will serve to provide a framework for the information that follows.

Soil grouting generally refers to several techniques which include chemical grouting in soils, cement or clay grouting, and compaction or displacement grouting. This report is concerned primarily with chemical grouting in soils. Chemical grout is characterized by the use of a grout which is a true solution or a colloidal solution. In chemical grouting, the grout is injected into the soil so that it permeates into the soil interstices without causing gross movements or rearrangement of the soil fabric. The water or air in the soil void space is largely replaced by the grout. Fracturing of the soil may occur, but significant changes in dry density or displacements of the soil are avoided.

GROUTING APPLICATIONS

Chemical soil grouting is most often used to either improve the structural properties of a soil mass, or reduce its permeability. It is possible to do either without the other, or both together. For example, waterproofing a zone of potential running sand prior to tunneling may be done with a weak grout that will not impede tunneling progress. Conversely, a structural grout that will both waterproof and provide a high modulus zone around the tunnel to reduce lost ground and surface settlement may be used in the same situation.

Structural Grouting

Structural grouting is used when it is desirable to increase the strength or modulus of a soil mass. Structural grouts may be used in sand or silty sand containing up to 20 percent fines. Soil consisting of very coarse sand and gravel is typically grouted with a particulate grout, which is less costly than chemical grout, while soils having high clay contents cannot be permeated at all. The effect of chemical grout on sand depends somewhat upon the sand itself. The primary effect of chemical grouting is to add cohesion to the sand. Typically unconfined strengths between 0.7 and 3.5 MPa (100 to 500 psi) are obtained, depending upon the soil and the grout. Creep strengths are generally only one-third to one-half of the conventional unconfined strength. If a grouted mass must support long-term loads, the allowable stress must be reduced to the creep strength. Increases in modulus are moderate if measured by the tangent modulus, but dramatic if measured indirectly by acoustic velocity. Clough et al (5)* reports moduli increases of several hundred percent, while Krizek's group (15) reports that changes in tangent modulus depend largely upon soil density. Dense sands display relatively little increase in tangent moduli, while loose sands become as stiff as dense sands upon grouting. Our own data using acoustic velocity indicate significant increases in stiffness in grouted sands, which is substantiated by pressuremeter tests (8). Clearly, the observed increase in modulus depends both on the soil that is grouted, and on the strain level that is used. Both the acoustic determination and the pressuremeter test are small strain systems.

The information concerning properties of grouted soils is still somewhat fragmentary, but is nevertheless adequate for the design of civil engineering structures. It is less variable than the natural variability from point to point in a given soil mass which can cause greater uncertainty in predicting soil properties than the lack of data on the characteristics of grouted sand. The approach for the designer to take is to recover soil samples from his site and have them injected with appropriate chemical grout and tested. This process is simple, inexpensive, and provides much better data than "typical" data obtained by tests on soils not representative of the site in question.

In recent years, structural grouting has been used to protect fragile or important existing structures from movements during soft ground tunneling. Dynamically loaded foundation soils are grouted to eliminate settlement due to densification under vibration, and liquefaction prone soils are grouted to protect against earthquake. The treatment may be applied either before or after construction, and may replace more traditional systems such as underpinning.

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*Numbers in parenthesis identify references given at the end of this report.

Waterproofing

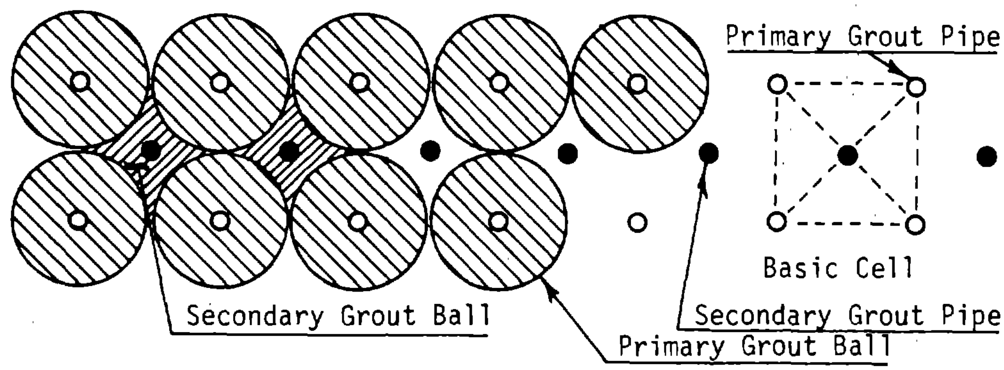
Waterproofing is accomplished by injecting either a triple line grout curtain, as shown in Figure 1 (a), or by laying down a blanket by injecting three sleeves deep in an areal array of grout pipes, as shown in Figure 1 (b). Subjects appropriate for waterproofing are any condition requiring the cessation of groundwater flow, such as excavations, hazardous waste disposal sites, and leachate ponds. Typically, if groundwater flow is a problem, the soils may be considered to be groutable. When using chemical grout for waterproofing, it is essential that full and complete coverage be obtained. If even small "windows" are left ungrouted, the high pressure gradients across the grout curtain will develop significant flows of groundwater through the small ungrouted "windows." If allowed to grow, these may be critical leading to piping and progressive failure of waterproofing effectiveness.

Grouts appropriate for waterproofing may be softer than structural grouts with no loss in waterproofing ability. This is often desirable if the treated zone will be excavated at some later date, since a strong grouted mass can make tunneling difficult for a tunneling machine designed for soft ground.

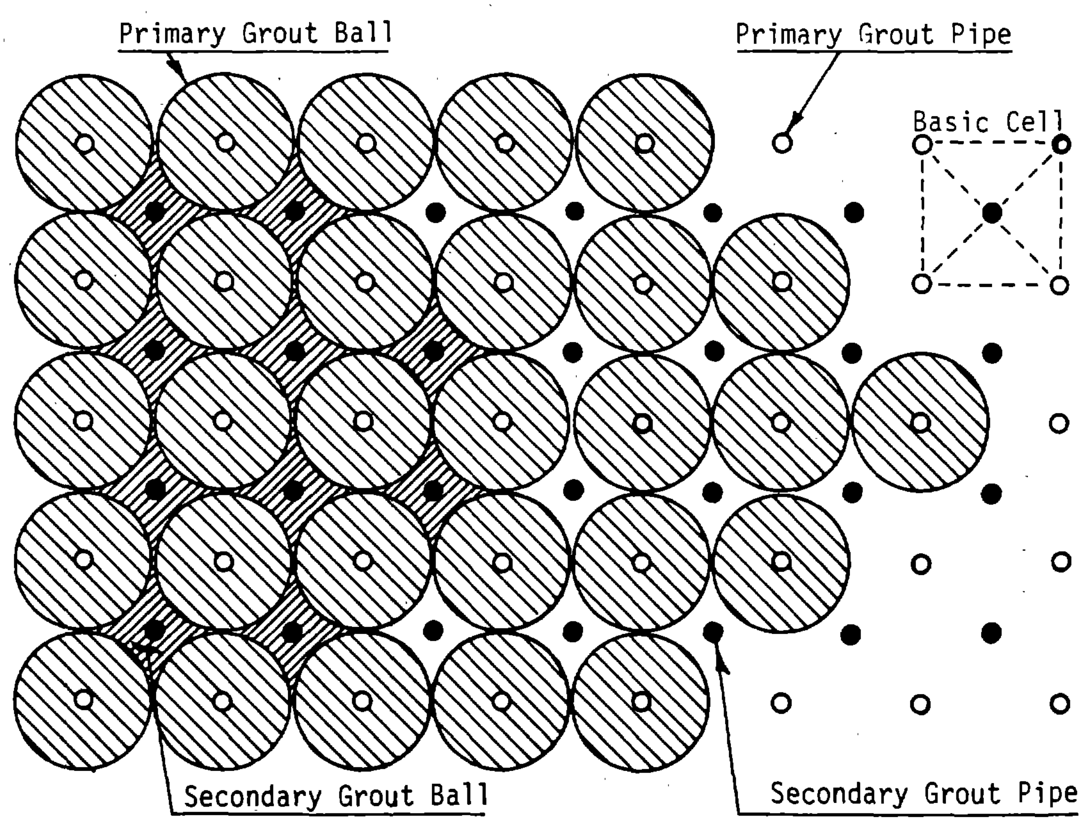
CHEMICAL GROUT MATERIALS

Materials used for modern chemical grouts are typically low viscosity chemical solutions which undergo gellation after injection into the ground. The most common chemical grout is sodium silicate cut with water and caused to gel by the addition of any of several reactants. Sodium silicate, once called waterglass, is a heavy, syrupy liquid having a pH of 11.3. Upon reduction of pH, by acidification or saponification, a gel of silicon dioxide and hydroxide is precipitated. Neat sodium silicate has a viscosity of several hundred centipoise, but upon the addition of water, the viscosity is drastically reduced. Structural grouts typically have 40 to 60 percent by volume of sodium silicate, and display viscosities between two and eight centipoise, enabling them to be injected into sand and silty sand, but not clay. Gel time, on the order of several minutes to several hours, is controlled by the type and amount of reactant.

Acrylamide (AM-9) grouts consist of monomer solutions that are catalyzed into cross-lineal polymous. Typical viscosity versus time curves for silicate and AM-9 grouts are shown in Figure 2.



(a) Grout Pipe Layout for Waterproofing Curtain Wall



(b) Grout Pipe Layout for Areal Grouting

Figure 1. Injection Patterns for Grouting

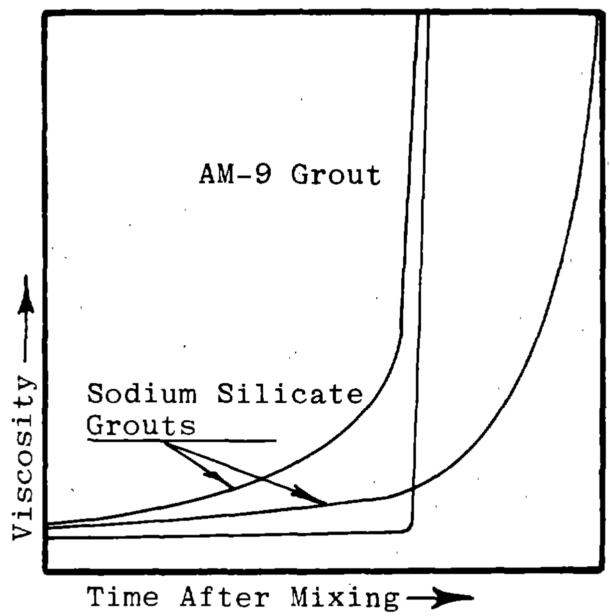


Figure 2. Viscosity of Chemical Grouts

The original Jooston process involved separate injections of neat sodium silicate and calcium chloride into the ground because of the instant reaction between the two. It is worth noting that this out-dated process required the injection of neat sodium silicate due to the large amount of water needed to dissolve the salt. The high viscosities and incomplete mixing obtained in the ground made this process less than satisfactory. Considerable detail on the various chemical grouts available on the market may be found in the FHWA-sponsored report by Tallard and Caron (21).

GROUTING METHODS

Grouting methods may be distinguished by mixing equipment and injection methods. Typically, a grouting contractor will be limited by his equipment and experience to one or two of the four combinations available. Because control and evaluation depend upon the particular grouting method used, these factors should be considered when establishing the quality control/assurance plan.

Mixing Methods

Chemical grouts are prepared either by batch mixing or by continuous mixing systems based on metering or proportioning pumps. Continuous mixing systems permit better control over the injection process since short gel times can be used. Typical gel times used with batch systems are several hours, whereas gel times used with the continuously mixed systems are usually ten to twenty minutes. Thus, the formation of large pools of ungelled grout in the ground is avoided. The importance of short gel time can be illustrated by a project that required grouting in loose material behind a tunnel lining. Grouting was used to stabilize the loose material to permit removing the existing lining. The configuration is given in Figure 3. The sandy material

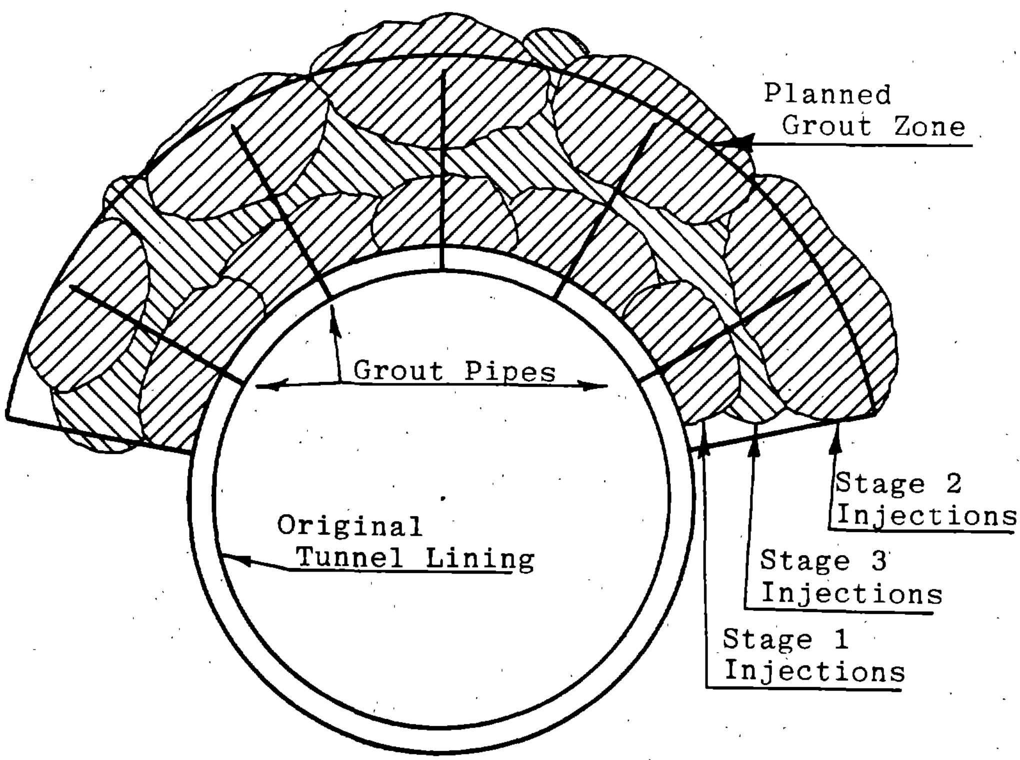


Figure 3. Grouting Sequence for Tunnel Relining Project

immediately behind the lining was injected with grout having gel times of 15 to 30 seconds. Despite these short gel times, liquid grout rained through the existing lining, but the job was successful in sealing the lining and stabilizing the loose material. The tunnel lining was subsequently removed, and the grouted soil was self-supporting during relining. Further argument for short gel time grout is provided by Karol (14), who conducted laboratory tests in samples subjected to lateral flow of groundwater. Tests using short gel times produced balls of stabilized soil around the injection point, but long gel time grout was diluted and washed away before it could gel. It is probable that batch mixing can be used without difficulty in most soil grouting projects, but greater control is afforded by the use of continuous mixing and short gel times.

INJECTION METHODS

Grout may be injected from the bottom of an open borehole, or from a grout pipe mounting a series of grout ports. The open borehole methods may use either stage-up or stage-down systems, while the grout port methods include the tube-a'-manchette or sleeve-port pipe systems (see Figure 4).

In stage-up grouting, the borehole is drilled full depth prior to injecting any grout. The drill is then withdrawn one "stage," leaving a length of borehole exposed. Grout is pumped into this length of open borehole until the desired volume has been injected. When injection of this stage is completed, the drill is withdrawn an additional stage, which is then injected. The hazard with stage-up grouting is that once a permeable horizon has been injected, there is a direct passage down the open borehole from subseqent stages. Grout may travel down the borehole to the more permeable layer in preference to the later stages. The permeable layer receives too much grout, which travels beyond the intended grout zone, while less permeable layers in the grout zone remain ungrouted. Both Karol (14) and Perez et al (19) report "Christmas tree" formations resulting from unequal grout distribution obtained using stage-up grouting.

Stage-down grouting is conducted in much the same way as stage-up grouting, except that grout is injected starting at the top and working down. The borehole is drilled into the top of the intended grout zone, withdrawn one stage, and the grout is injected. Upon completion of the first stage injection, the drill is advanced an additional stage, drilling through the first grouted stage, and the next stage is injected in the same manner. Stage-down grouting is more time consuming than stage-up grouting, but better control of the injected grout is obtained. Both stage-down and stage-up methods may be conducted using a variety of drills, driven grout pipes, or needle pipes. Care must be used in injecting two adjacent locations, and neither method is well adapted to subsequent injections in the same hole. Drilling is necessarily closely tied to injection, since a borehole cannot be left open during injection of an adjacent hole. If a grout connection to an open hole forms, an immediate leak to the surface results. Thus, in the open borehole method, drilling and injection generally proceed together.

The grout-port method (shown in Figure 5) employs a plastic grout pipe which is sealed into the borehole with a brittle portland cement/clay mortar jacket. The grout pipe has grout ports at intervals of one-half to one meter. These grout ports consist simply of several holes in the grout pipe covered on the outside by a short section of rubber sleeve. The rubber sleeve acts as a check valve, permitting grout to flow out. The grout from an adjacent grout port cannot flow into the pipe against the sleeve. A grout-rod with a double-packer passes inside the grout pipe, and is used to isolate a single grout port for injection. Injection proceeds by rupturing the mortar jacket, by a brief pulse of high pressure water. Grout is then pumped into the double packer, passes through the holes in the grout pipe, under the rubber sleeve, and out through the cracked mortar jacket into the soil. Grout ports are injected in sequence.

Several advantages result from the use of the grout port method. Generally, all grout pipes in an area are drilled and mortared-in prior to grouting. The drilling and grouting operations are separated, so that each may proceed at its own best rate. The grout ports are reusable, so that multiple injections can be made at a point without the necessity of redrilling. This is often desirable if the grouting evaluation shows that there is some question regarding the adequacy of grouting in an area. The grout pipes also provide excellent instrumentation points for the geophysical sensing methods that will be presented in Section 4 of this report. Finally, with the grout port method, it is known that the grout has entered the soil at a particular elevation with no risk that it has instead traveled along the borehole as it might when the open borehole injection method is used. Perez et al report that no "christmas tree" formations were found in

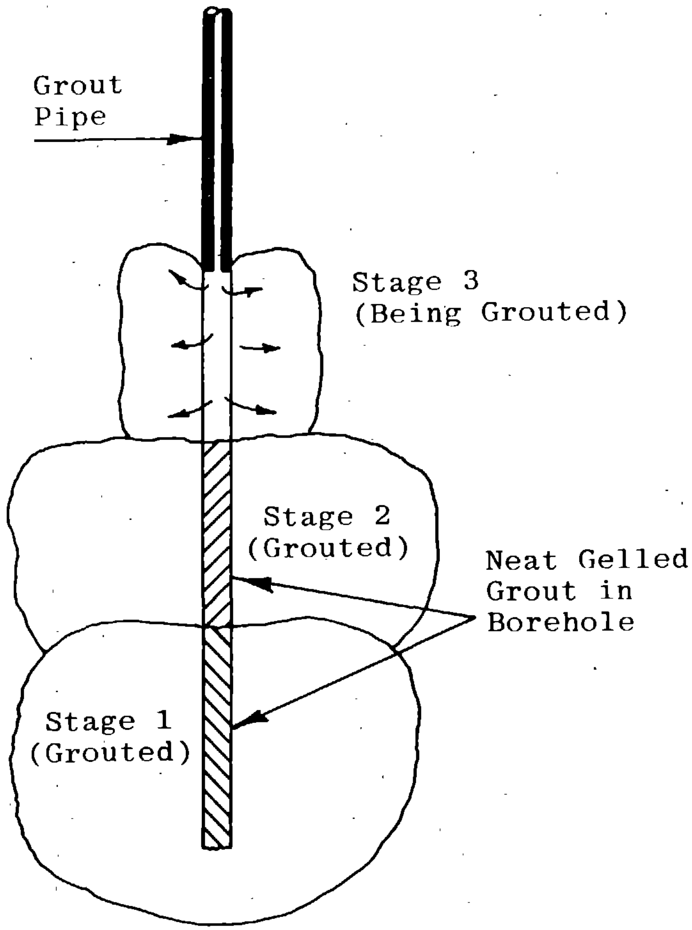


Figure 4. Open Borehole Injection System

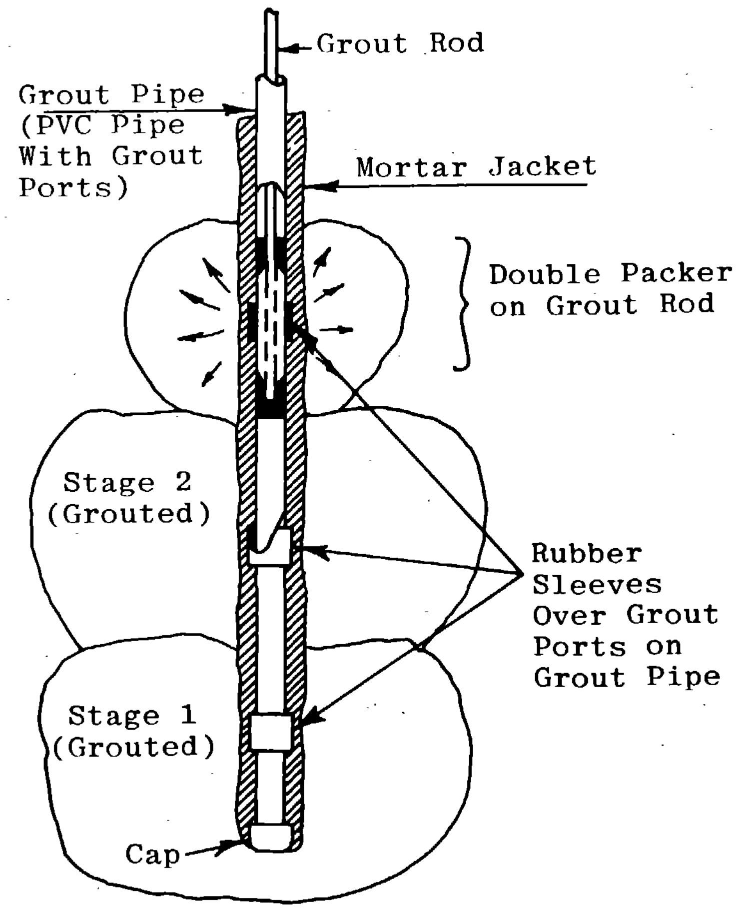


Figure 5. Grout Port Injection System

those sections of their test site that were grouted using grout port injection methods (19).

Injection Patterns

Chemical grout is almost universally injected using some form of primary/secondary grouting pattern. Typical layouts for a waterproofing curtain and for areal grouting were shown in Figure 1. In both cases, the primary holes are injected first. The grout takes are established so that each primary hole receives at least twice as much grout as each secondary hole. Typically, the primary holes are pumped to the designed grout take, without any indication that the soil mass is refusing additional grout. After the primary holes in an area are injected, and have had time to gel, the secondary holes are injected. As each secondary is surrounded by completed primaries, these are expected to refuse grout at takes at or near the design quantity of grout. If a secondary does not display grout refusal, the adjacent primary holes may not have been well grouted, and tertiary grouting--reinjection of already used grout ports--may be desirable. The secondary grout pipes are intended to fill in any gaps left by the primary grout injections, and to test the adequacy of primary grouting. In particularly critical areas, three and four stages of grouting may be used until the behavior during injection indicates that the void space in the soil is filled with grout. This must be done with care, since it can also lead to over grouting, which may cause destructive surface heave.

GROUT EVALUATION USING SITE EXPLORATION TOOLS

Attempts are frequently made to evaluate chemical grouting effectiveness using conventional site exploration tools. Tools that may be applied include the standard penetration test, borehole pressuremeter, undisturbed sampling, cone penetration resistance and the excavation of test pits. These systems have applicability under some conditions, but under others, are ineffective or even misleading.

Standard Penetration Test

The standard penetration test (SPT) is a favorite site exploration tool in spite of its liabilities and inadequacies. The primary advantage of the SPT is the general familiarity soils engineers have with its use. Unfortunately, it is a dynamic test and grouted soil is an easily shattered brittle material. These factors combine to increase the already large variability of the SPT when it is applied to grouted soil. While it is usually apparent in the blow counts that there has been an increase in soil strength after grouting, the increase in blow count is not commensurate with the anticipated increase in resistance to static loads.

Cone Penetrometer

A cone penetrometer driven hydraulically is an appropriate test system for grouted soils if sufficient reaction can be provided. Hydraulic cones are typically used in softer soils, particularly clays, and are not designed for the high strengths found in grouted sand at 10 or 20 meters depth (30 to 60 feet). A viable cone penetrometer system involving a conventional cone deployed from the bottom of a drilled borehole was used at Locks and Dam 26, (Perez, et al, 1979). Such a test method would probably be no more expensive in operation than the SPT, and would develop more meaningful data.

Borehole Pressuremeter

The borehole pressuremeter can be used effectively in grouted soils under limited conditions. A smooth clean borehole must be obtained, without loosening the material in the sides of the hole. This is virtually impossible in grouted gravelly sand. A gravel particle left protruding from the borehole wall in a grouted soil is an excellent membrane rupturing device. The borehole pressuremeter can be used in clean fine sands having no gravel if the hole is drilled using a "fishtail" or drag bit rotated under tight pressure and with a heavy drilling mud. Having obtained a smooth clean hole, the pressuremeter tangent modulus can be measured. Often, the strength of the grouted soil is greater than the pressure capacity of the pressuremeter.

Undisturbed Sampling

Occasional attempts are made to obtain undisturbed samples by core drilling. Shelby tube or split spoon sampling is out of the question in grouted soil. Rotary drilling with a core barrel is seldom successful. Small gravel particles or broken pieces of grouted sand work their way into the core barrel, abrade the side of the sample and usually break it in flexure. Even if a sample is recovered intact, it is of questionable value because of the rough handling it undergoes during drilling. It is difficult to trim an undisturbed specimen from a block sample in the lab, much less to drill one in the field. Undisturbed samples can, however, be obtained by trimming block samples recovered at tunnel headings or from test pits.

Test Pits

The most effective traditional grout evaluation method is the excavation of test pits. One can then enter the grouted zone and recover undisturbed samples, conduct plate bearing, CBR or wall reaction tests in-situ, and generally evaluate the grouted soil by personal inspection. If it is difficult to detect the grouted soil either by odor or color, an acid/base indicator such as phenothalene can be sprayed on the soil to detect the high pH grout. While a test pit is both destructive and expensive, it is among the most effective conventional grout evaluation methods.

Conventional site exploration tools can be used to obtain a qualitative idea of the grout location and condition, but cannot be used to obtain quantitative data. In most cases, even intuitive judgments based on extensive experience with the particular tool in question can be quite misleading because of the sensitive nature of grouted sand.

Over the last decade there has been a steady improvement in equipment and injection procedures, permitting high quality, efficient chemical grouting. A wide choice of grouts has become available, and field applications of grouting have shown a steady increase.

PROBLEMS IN CHEMICAL GROUTING DESIGN AND CONSTRUCTION CONTROL

The use of structural chemical grouting has been gaining acceptance in civil engineering construction in the United States in the last decade. The civil engineering community is now more aware of the chemical grouting process, but still not familiar with recently developed methods for reducing the uncertainty regarding the location and reliability of grout in treated soils.

In an effort to better understand the dynamics involved in the acceptance or rejection of available chemical grouting technology, a case history review was conducted of grouting projects carried out in the United States. The purpose of the review was a definition of problem areas in grouting, both institutional and technical, gained through a study of civil engineering experiences in the design, conduct, and evaluation of grouting projects. Questions addressed by this study included the following:

- What problems have been experienced in the application of chemical grouting?
- To what degree are these problems design related?
- What are the sources of these problems in the civil engineering industry?
- What growth has occurred in acceptance of grouting technology over the past decade?
- What attitude changes have been fostered by improved technology?
- What steps can be taken to solve these problems?

The results of the review were used to define areas of uncertainty in the technology and to aid in structuring the research program.

In the presentation of case histories in civil engineering, it is common to present a descriptive, case-by-case narrative of the projects reviewed, concentrating upon the physical aspects and technical details of the work undertaken. Selected illustrative case histories are presented in Volume 3, Engineering Practice. For the purpose of this review, the standard method of case history presentation was thought to limit candid discussion and analysis of problems that have occurred in the conduct of grouting projects. In addition, questions of professional liability and reputation, and the ever present possibility of litigation may hinge upon the presentation of poor job performance or problems, and failures that may have occurred. Thus, in an effort to present material ordinarily bypassed in case history reviews, the problems and trends abstracted from many case histories are presented herein, rather than the individual cases themselves. However, supporting data from the over 200 cases reviewed will be cited as necessary in the body of the report. In addition, a distillation has been sought of recurring problems in the institutional arenas and attitudes,which tend to hamper maximum use of available technology.

Methodology and Data Collection

The data base explored has included the job files of Hayward Baker Company and cases reported in the literature and obtained from the Board of Consultants.* Cases reviewed included projects dating as far back as thirty years ago, to as recently as 1978, and run the gamut from small, remedial waterproofing jobs to major structural grouting projects, designed as part of major urban tunneling projects. Data collection was conducted through an exploratory field study, rather than a more formalized test of a specific set of research hypotheses. Though such field studies lack the precision of structured lab or field experiments, their greater flexibility and generality make them appropriate for the exploration of complex situations. The field study has sought to identify the issues involved in technology acceptance as it relates to chemical grouting and has discovered significant relationships between the variables encountered herein. This study can also serve as ground work for a later, more systematic and rigorous testing of the relationships between professional practices, group values, and technology acceptance.

The major instrument utilized in data collection in this study was a type of open-end interview schedule referred to as a funnel. The funnel started with a set of broad questions on construction problems in general and narrowed to a single set of closed questions directed specifically to chemical grouting. Such a funnel technique is designed to prevent early questions in a sequence from affecting answers to later questions; it is also useful in determining a respondent's particular attitudes, biases, and frame of reference. An effort was made to interview a cross section of the industry from field foremen, to contractors, to design engineers.

Problem Overview

A definite growth of technology and technology acceptance can be measured over the thirty-year span of time covered in this review, with the greatest growth in technology occurring in the last ten years. An important factor in the expansion of

* Members of Board of consultants included Prof. Reuben H. Karol, Rutgers University; John P. Gnaedinger, Chairman, Soil Testing Services; Edward Graf, President, Pressure Grout Company; Dr. Raymond J. Krizek, Northwestern University.

the technology has been the major urban tunneling projects for urban mass transit systems in Washington, D.C., and Baltimore, MD. The efforts of FHWA and UMTA to study and make available the results of this work have been a major factor in the growth of grouting technology and its increasing acceptance on the part of the civil engineering community. Prior to this time, according to several designers and engineers who have been involved with grouting over a long time period, large scale chemical grouting jobs were seldom seen except when it was necessary to remedy a serious field situation, or when the potential cost-savings were so great that it justified taking what was then viewed as the risk of grouting.

Major grouting projects, designed for the Washington, D.C., subway system were intentionally over-designed to guard against failure, since it was felt that the adequacy of grout treatment could not be evaluated. Since that time structural grouting has been successfully employed on at least twelve major urban tunneling projects, and is beginning to be considered for nuclear power plants as well. Enough is now known about grouting to design for specific parameters, and to integrate chemical grouting into a total design program. Numerous methods are available to evaluate system effectiveness, and the cost effectiveness of chemical grouting versus other technologies can readily be quantified.

Grouting projects on the major mass transit systems have also brought to light several technical problems specific to grouting for tunnel construction. Particular problems occurred from the strong ammonia odor typical of grouts that make use of certain formamide type reactants. This ammonia odor in confined spaces can become strong enough to be very unpleasant to workers. Recently introduced grout reactants have solved this problem.

Despite this, the majority of cases reviewed in this study were remedial in nature. That is, grouting was employed only after on-the-job problems, such as subsidence, loss of soils, etc., often grave enough to jeopardize the final success of the project, were encountered. Design for grouting, which incorporates grouting into the initial specification for construction work in the United States, though more frequent since the construction of the major urban mass transit systems on the East Coast, is still relatively rare. This review has indicated that grouting projects tend to fall into four categories: (1) Jobs that performed well; (2) Jobs that were designed only as insurance and were never performance tested; (3) Jobs that performed poorly either technically or institutionally; and (4) Jobs that actually performed well, but failed to meet design specifications or acceptance criteria due either to overdesign or a misunderstanding of the grouting process.

Problems Experienced in the Application of Chemical Grouting

Need for greater understanding of grouting on the part of designers, prime contractors and resident engineers was expressed by each individual interviewed in this study, from consulting engineers to field foremen, in the public as well as the private sectors. When grouting is the technique of choice, problems in design and specifications occur, based on a misunderstanding of its capabilities, interrelationship with other technologies, and its cost effectiveness. This also leads to problems in contractor selection and performance evaluation.

Design engineers in a major underground construction company expressed concern over every aspect of chemical grouting. The only member of the group with experience in chemical grouting had not been exposed to chemical grouting techniques in almost twenty years. At that time AM-9 was being used for water cut-off in limestone terrain in a zinc mine 240 m (700 feet) below ground level. Memories of mixing the grout bucket by bucket, only to have it gel in the pipes, left what was described as a "strong mental block" against ever trying chemical grout again. In addition, the cost of the AM-9, coupled with the inefficiencies of the process made grouting economically non-competitive with other solutions. At this date, chemical grouting has never been recommended or designed for by this firm. Though valid concerns as to permanence and durability of grout over time, and the cost-effectiveness of grouting in contrast to other technologies were expressed, much of the reservation against use of this technology was based on problems that have been resolved for one or two decades.

In another case, a consulting firm evaluating a water cut-off for a waste water retention pond rejected chemical grouting as an alternative, stating that "there is no way to determine where or how far the chemical grout flows or if the sealing is effective. This method therefore can't be used for this application." Anther example of chemical grouting to create an impervious curtain surrounding a pump plant is cited in the literature (3). A single row of grout holes, at 1 m (3-foot) intervals, was used. Sodium silicate grout was injected using a three shot method with primary injections at 2 m (6-foot) centers, and with secondary injections on 3-foot centers. Though failure to achieve a tight grout curtain was attributed to poor mixing of the grout and the fact that packers were not used, it is rare to achieve a tight grout curtain for water cut-off with a single row of grout holes, even under optimum conditions.

Poor communications in the field are a recurrent problem throughout the construction industry. Unrealistic expectations by inspectors and resident engineers as to grouting performance are frequent occurrences in the field and may seriously impact the performance of work, as well as the final result of the project. It is not unheard of for such cases to result in a contractor pulling off a potentially successful job or in the case ending up in litigation, even though the job itself is an engineering success. One such case was a grouting project for water cut-off in an existing tunnel. The inspector had had no previous experience with grouting. His frequent intervention in grouting methods and work procedures slowed progress to such an extent that the grouting contractor pulled off the job in order to prevent severe economic losses to his firm.

In a similar case involving a major grouting contractor grouting to seal a cofferdam in a lake, the inspector, a geologist, had extensive experience with conventional cement grouting, coupled with little understanding of the procedural differences between chemical and cement grouting technologies. At the insistence of the inspector, too long gel times were used, along with inconsistent pumping pressures. In addition, initial site evaluation was sketchy and consequently resulted in inadequate design for grouting. The experienced grouting contractor is accustomed to adapting procedures to particular site conditions, but in this case such innovation was prevented by the inspector and the job failed. The contractor had insisted on a signed release from liability when problems developed. The design engineer has not specified grouting since this experience.

Though the above problems probably resulted from suspicion and uncertainty as to the reliability of an unknown technology, it is not uncommon for overconfidence and unrealistically high expectations as to grouting performance to create equally difficult field situations. In one such case an addition to a building was being constructed and grouting was specified to underpin the existing structure. Waterproofing was not specified and was therefore not an integral part of the design. Though the construction job was completed satisfactorily, the inspector and design engineer had expected to see a completely grouted mass, without water leaks or discontinuities and the case went into lengthy and costly litigation.

Overconfidence, too, can lead to potential hazards. Grouting was performed to stabilize foundation support soils below an existing four-story hospital adjacent to an excavation for a building addition. The general contractor expected the grouted soils to provide full structural support, and without any bracing, protection or post grouting evaluation allowed it to be excavated using less caution than normally employed in ungrouted soils. Though the grout did not fail and excavation proceeded without incident, the potential for failure was great.

The private sector is not alone in its failure to understand and use available grouting technology in lieu of less innovative construction techniques. Historically, the development of chemical grouting in this country and grouting expertise have resided with the grouting contractor in the private sector and with the Bureau of Reclamation and the Army Corps of Engineers in the public sector. Both the Corps and Bu/Rec have concentrated on cement grouting techniques and retained chemical grouting techniques for limited use in dam grouting applications. Since their experience in this area has been largely successful, neither group has undertaken research into instrumentation for dam grouting, permanence and durability of grouted structures, or into a more thorough understanding of the effects of loading upon grouted soils, or even into the comparative economics of this technology.

Design for Grouting Projects

A rational procedure for the design of grouting projects is needed, according to many grouting specialists contacted in the course of this research. As design for grouting has become somewhat more common in the past years, specialist designers are developing to meet the needs of the market; their numbers are limited and guidelines to aid the non-specialist designer are a priority need. In addition, uncertainties as to the stability of grout in treated soils and questions as to the behavior of grout under permanent conditions, or under cyclic or repetitive loadings, are unanswered and tend to mitigate against selection of grouting as an alternative by the professional designer.

Civil engineering designers tend to be conservative and reluctant to innovate design procedures. In grouting design, the degree of permanence will have a strong effect on the design course taken. Since the permanence of grouts has never been satisfactorily established, the same designer may recommend relatively innovative procedures for a temporary or low-cost project, while taking a very conservative approach to major long-term, high-cost projects.

In any design project it is the goal of the designer to have access to all available design alternatives in order to produce the desired product for his client, weighed against performance and budgetary parameters. Furthermore, today's heightened awareness as to the social and environmental impacts of any large-scale construction project put additional burdens upon the designer. No longer are the least expensive construction solutions always acceptable, environmentally or politically. This may have special significance in design for grouting, where less sophisticated, but more cost-effective techniques may affect the homes and properties of citizens. As yet, grout toxicity assumes special importance to the designer. A shutdown of a grouting project on a municipal sewer system within the last year because of possible contamination of the water supply highlights this problem.

Current grouting designs and specifications do not always appear rational. It is not uncommon, for example, to find "designed for grouting" parameters for water cut-off or prevention of loss of soils during dewatering, stated in terms of minimum strength requirements, meaningless in terms of waterproofing. Significant increases in project costs, which limits marketability of grouting solutions, are sometimes the result of overly conservative design which extends the grout zones well beyond the areas necessary to insure project success, or requires strength well beyond those necessary to insure project success, or requires strength well beyond that necessary for a rationally engineered design. In a recent grouting project to stabilize an existing railroad tunnel as twin subway tunnels passed below, the grout zone extended well beyond the boundaries of the tunnel. It was estimated by the designer and the contractor that this resulted in increased costs to the project somewhere between 50 percent to 100 percent.

Design of grouting for "insurance" purposes, to prevent possible risks, in which the grouted zone is never tested also occur and tend to increase the client's total construction costs. This is particularly common in high risk projects such as nuclear power plants (where it can be argued that the cost of insurance is never too high). It is debatable whether the presence of the grouted zone in such cases gives the appearance that its only purpose was to increase the factor of safety or whether it actually did so.

Guidelines to the design process are presented in Volume 3 of this report. Adequate reviews of site data and job-site geometry on the part of the designer are mandatory to prevent frequent problems in the field. Failure to identify utilities or other hazards on site is a recurring problem that has resulted in liability to the contractor when grout migrates into sewerlines or when a drill knocks out a utility. Laboratory tests to insure compatibility of grout with soil chemistry are also often overlooked. In a grouting job for soil stabilization beneath a hospital for example, grouted laboratory samples were not prepared. Later, the soils were found to be contaminated with sewage which prevented the grout from gelling. The presence of salt, or a high lime content can also play havoc with gel times. In grouting beneath an old school structure for earthquake proofing, high concentration of lime in the soil, probably as a result of the concrete foundation, caused a layer of grout directly beneath the building to remain viscous.

An additional major problem for the designer is the separation between the design and construction phases of a project. The design engineer rarely, if ever, has feedback from the field as to problems that occurred in the conduct of the job. Unless the design fails to the point of litigation, the designer usually operates in an information vacuum, and therefore is denied the opportunity to learn from one project to the next.

Certain steps can be taken to alleviate these problems, with anticipated positive results. Project goals should be stated in terms of critical parameters (stand-up time, surface deflection, water inflow). These objectives should be translated into appropriate contractual requirements through the process of rational design based on specific properties, and clearly delineated in terms of:

- Soil modulus
- Soil strength
- Soil permeability
- Modification of soil properties (strength and/or modulus) to resist imposed or anticipated loading
- Reduction of soil permeability to control groundwater flow.

The degree to which grouting is critical to the satisfactory completion of the project should be reflected in all phases of contract preparation.

The designer should be involved in decision making at the field level. It is very difficult to write a set of specifications that can explain all instructions and foresee all difficulties for both the contractor and the inspector. Not only would the involvement of the designer at this level assist him in future designs, but could have positive benefits to on-going projects.

Specifications for Grouting

A set of written specifications is the end product of the design process. Unfortunately, in the case of most specifications for grouting, program objectives are seldom well defined. Though it is usually apparent that grouting has been proposed with a general purpose in mind (i.e., to reduce settlement, protect a building, provide water cut-off, etc.) detailed requirements relating to ground strength or stiffness, or whether the grouted zone is designed to withstand adjacent water pressures are seldom given.

Just as grouting jobs are either the result of the original design process or the result of a need for remedial action to correct an urgent situation, specifications themselves fall into two categories. In emergency situations, the client must rely on the grouting contractor to write his own specifications. In such cases a successful solution to the problem presents the ultimate in acceptace criteria, and if the problem is solved the client is usually satisfied. In cases where grouting is designed into the total construction program, specifications assume a much greater importance. Unfortunately, it is in these cases that specifications are most often a cut and paste job, put together from parts of many other projects and left very general with the hope of covering all possibilities.

It is the unanimous consensus of every grouting designer and contractor interviewed here that job specification should never be completely standardized, though certain standard areas should be treated in most cases. Without a specific statement of desired accomplishments in terms of subsurface modification, an adequate evaluation of the effectiveness of the grouting is not really possible prior to other construction in the area. Therefore, specifications should state in detail the scope, dimension, and final objective to be accomplished by chemical grouting, based on the engineered project design. Consideration in preparing specifications should be given to the following:

- Engineering parameters, including level of strength, or modulus, or permeability within the volume of soil; degree of allowable variation.
- Guidelines for measuring results, including specific test methods, (i.e., ASTM or similar designations for measuring soil strengths).
- Testing frequency, daily, weekly, or monthly, depending upon and reflecting the critical nature of the job.
- Record-keeping and data presentation, including mechanized as opposed to manual data aquisition; graphical display of field data; on-going review to allow for corrective action prior to completion of the project.

Record-keeping is sufficiently critical to the conduct of a grouting job to require further comment. Since there is currently no standard for data accumulation and presentation, improvement in this kind of record-keeping in the United States is essential to the development and acceptance of the technology. Requirements for record-keeping should be tailored to some degree to the job at hand since it is economically impractical to require the same record-keeping from a major structural grouting job and a small remedial project. In almost every case reviewed here, record-keeping and data presentation have been minimal. In cases where thorough records are maintained it may be impossible to analyze the volumes of data until after completion of the project, when the value of the information is minimal except for historical or instructional purposes. This suggests a possible need for computerized data systems. Whatever systems are devised should be accurate, accessible, and standardized industry wide and from job to job. A graphic display system, accessible even to the uninformed would have great usefulness in field situations, where inspectors may have limited experience with grouting or lack the ability to visualize what is occurring underground.

If grouting is performed in proper sequence and detailed records are maintained, the data indicates when the pores of the mass are filled with grout. Ultimately, it may be possible to require the engineer to accept or reject a job within a short time of completion, based almost solely upon records kept and reviewed regularly during grout injection. Development of such a system would afford greater protection to both client and contractor than is obtainable through current acceptance procedures.

Quality assurance programs are unanimously recognized as essential to the effective use of grouting as a tool in the construction industry and should be routinely included in the specifications. Without adequate quality assurance, the client is forced to rely upon his confidence in the expert's skill and competence, a degree of faith in the technology, and the courage to take any risk that might appear to be involved. Judging from the absence of quality assurance programs in almost 98 percent of the cases reviewed here, quality assurance in the chemical grouting industry is still in its infancy, despite quality assurance programs that have been applied successfully in test programs such as Locks and Dam 26, (Perez, et al, 1979), and in grouting for high risk projects such as nuclear power plants. It is a general consensus that test methods, injection procedures, and post-grouting evaluation tests are needed to obtain adequate quality assurance and must be developed accordingly.

The case history review resulted in a prioritized list of problems to be addressed by this research (see Table 1). Each area of concern was assigned a priority based on objective, degree of importance, method of attack, level of effort, probability of successful achievement of objectives, and resulting benefit to the overall program. Highest priority items were the technical problems most readily attacked by this program. Successful treatment of these problems is expected to result in wider acceptance of grouting as a construction alternative, as well as alleviation of many of the attendant institutional problems.

The major Quality Control/Quality Assurance questions identified during the case history review, included:

- Where does the grout end up after injection?
- What is the condition of the grouted zone?
- How may the injection process best be controlled to insure the desired results?

Table 1. Prioritized List of Problems

NATURE OF PROBLEM	Problem Severity	Probability of successful treatment	Priority
Acquaint Civil Engineering designers with chemical grouting capabilities	H	L	M
Rational design for chemical grouting	H	M	M
Specifications Reflect design requirements	H	M	M
Incorporate evaluation technology	H	M	H
Reflect limits, tolerances appropriate to project needs	M	M	M
Useful records & review mechanisms	M	H	H
Location of Grout - Geophysical	H	M	H
Quality Assurance - Injection Procedures	M	H	H
Grout Q. A. tests & Procedures	M	H	H

Note: Ranking Code

L Low
M Medium
H High

- How may the completed project be evaluated to determine what results were obtained?
- How much injection pressure can be safely used?
- What can be done using chemical soil grouting, and how should it be approached?

RELATED RESEARCH

The research reported herein is the third in a series of efforts funded by the Federal Highway Administration on chemical grouting. The previous efforts were by Tallard and Caron (21) on chemical grouts for soils, and Herndon and Lenahan (11) on the state-of-the-art of grouting technology.

A review of grouting applications and bibliography were assembled by Einstein and Barvenik (7) in their work for Oak Ridge, while bibliographies with abstracts on grouting have been compiled by the Waterways Experiment Station (1) and by Habercom (9) of the Department of Commerce, National Technical Information Service (NTIS). Design procedures for grouting to stabilize soft ground for tunneling are presented by Clough et. al. (5) and European practice is described in Grouting Design and Practice, Anon. (2). In addition, investigators such as Mitchell (12) and Karol (14) have published excellent reviews of ground modification techniques. Finally, the subject of hydraulic fracture is treated by Haimson (10), Davidson (6) and Leach (16). Major reserach programs such as conducted at Locks and Dam 26, sponsored by the Army Corps of Engineers have added to the body knowledge on chemical groting. The thoroughness of the research program at Locks and Dam 26 deserves special note.

The program was designed to evaluate the efficacy of chemical grouting for repair of erosion around driven piles at Locks and Dam 26 on the Mississippi River (19). Extensive site investigation prior to the test grouting phase was conducted by Woodward-Clyde Consultants to thoroughly document initial subsurface conditions. The physical properties of the soils were analyzed to determine characteristics and extensive in-situ tests were conducted to determine grain size, in-situ stress, density, strength and deformation properties and permeability. Lab tests simulating field conditions were also conducted in order to evaluate their predictive capabilities. Eight different chemical grouts were injected and the site later excavated to determine the effects of the grout treatments. Monitoring during grouting included the use of inclinometers, Soundex to monitor lateral as well as vertical movement, piezometers for pore pressure measurements, surface reference points, and Borros points for settlement and heave. In addition, an extensive quality control program was carried out during and after grouting to monitor and evaluate the effects of pore pressure and the quantities of grout injected.

The research at Locks and Dam 26 has left many questions unanswered as to what is actually going on underground during the grouting process. Unanswered questions concerning the 1 psi/ft of depth rule of thumb for injection pressure, and the dissipation of pore pressure away from the grout pipe during pumping need further research. Although the research at Locks and Dam 26 did not answer all the questions that might be raised about grouting, it is the most extensive and the most heavily instrumented grouting test program yet conducted in this country. The thorough site evaluation, procedural controls and quality assurance programs used there should be incorporated into future designs for grouting. This would contribute to wider acceptance and increased confidence of grouting technology industry-wide. Certain of the above reports can be purchased through National Technical Information Service (NTIS). The interested reader is advised to check the References at the end of this report for Agencies of origin and document numbers.

CHAPTER 3—EXPERIMENTAL WORK

PROGRAM OBJECTIVES

The overall objective motivating the program was the development of methods to control and evaluate the work of chemical grouting contractors. Concentration was in the area of quality control and quality assurance rather than the development of technical improvements in the grouting process itself. The project will have met its goals if designers can specify chemical grouting with improved confidence that (a) their designs are appropriate, and (b) that these designs will be properly executed in the field.

This broad objective was to be met by developing and delivering the following items:

- Improved theoretical model of the grout injection process.
- Techniques and procedures usable for control of the grouting process and for evaluation of the finished products.
- Guidelines for application by designers to assist in design and control of grouting projects.
- Demonstration of the control and evaluation techniques on a production grouting project.

These objectives would provide the theoretical basis for the injection process, evaluation and control methods necessary to monitor chemical grouting, and guidelines to explain how they should be applied. Finally, the demonstration project would display their operation in a real-world construction project. It was suggested that these questions might best be answered by a combination of geophysical remote sensing tools and a program of grouting procedures and data acquisition. These should be applied in a QC/QA program that is well integrated into the overall design/construction process. The specific methods selected were:

- Earth probing radar
- Electrical resistivity

- Acoustic cross-hole shooting
- Monitoring of injection pressure and flowrate
- Acoustic emission monitoring
- A specific format for recording and filing injection data.

PROGRAM HISTORY

The research effort included (a) bench scale laboratory tests (b) large-scale laboratory injection tests and (c) full-scale pilot field injection tests. In addition, two small test programs were conducted on production grouting contracts to answer specific questions raised by this program. These were separately funded programs, but are also reported herein. The first study was concerned with the effects of elevated injection pressures, and is of importance in soil grouting. The results of this effort are discussed under the heading "Laurens Street Site." The second study was concerned with the transfer of the results of this program to cement grouting in rock and is reported here under the heading "Reston Dam Site." The research program was completed by the demonstration program, which is discussed in detail in Volume 3. The chronology of these efforts is shown in Figure 6.

BENCH SCALE LABORATORY TESTS

The bench scale testing included breadboard geophysical testing to develop appropriate operational parameters for the remote sensing systems, bench tests for grout viscosity and permeability in sand, and development of an improved mathematical model of the grout injection process. Incidental to this process, a number of laboratory tests were developed and standardized for use with chemical grouts. The ASTM committee D18.16 on Grouting is now working on the problem of establishing standard test methods which will alleviate some of the current difficulties in trying to compare published test data from different laboratories.

Meter-cubed Laboratory Injections

Nine laboratory injections were conducted in the apparatus shown in Figure 7. This is a one meter-cubed container in which silty-sand from the pilot test site was compacted. The particle size analysis for this sand, shown in Figure 8, indicates that

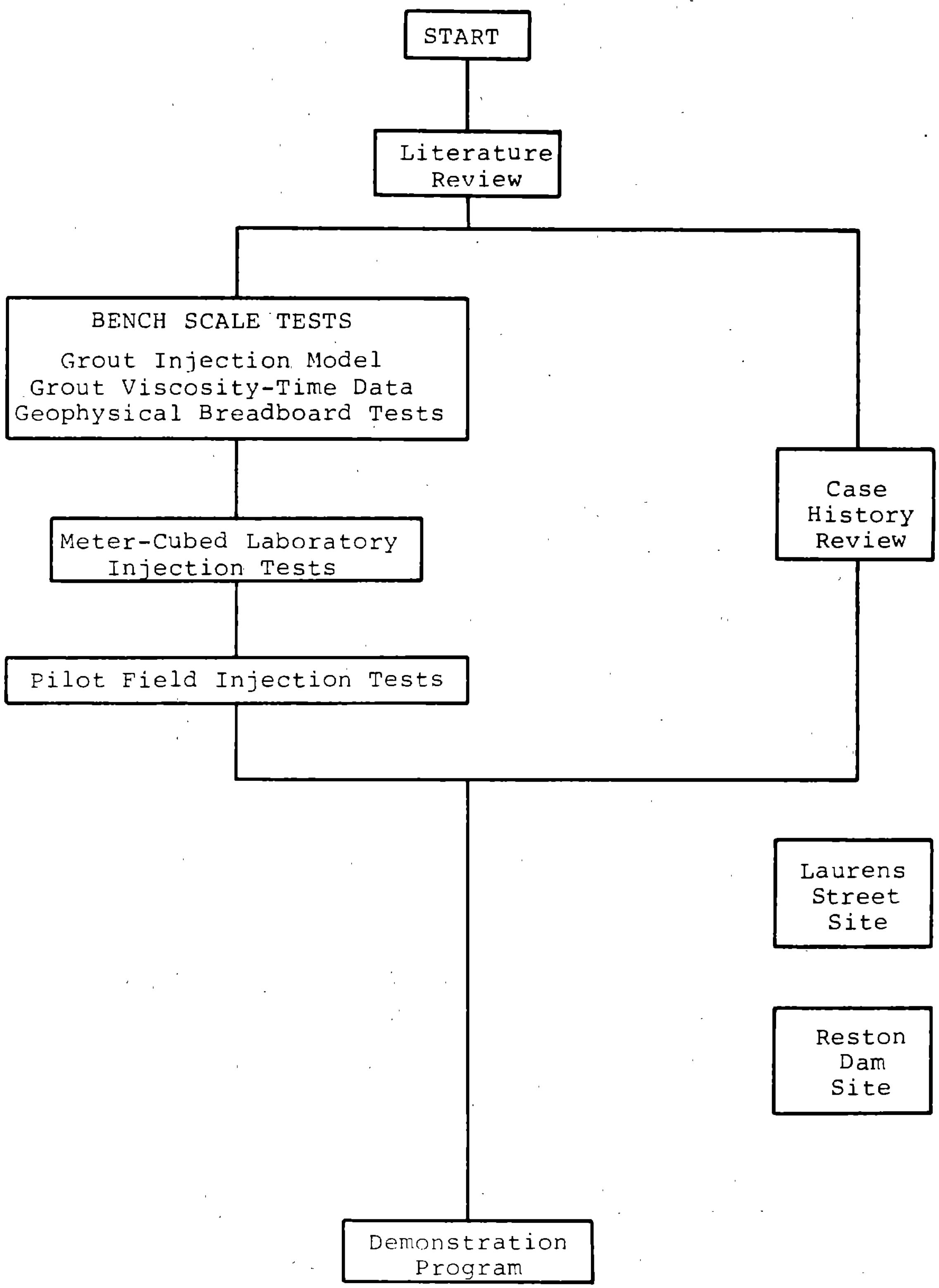
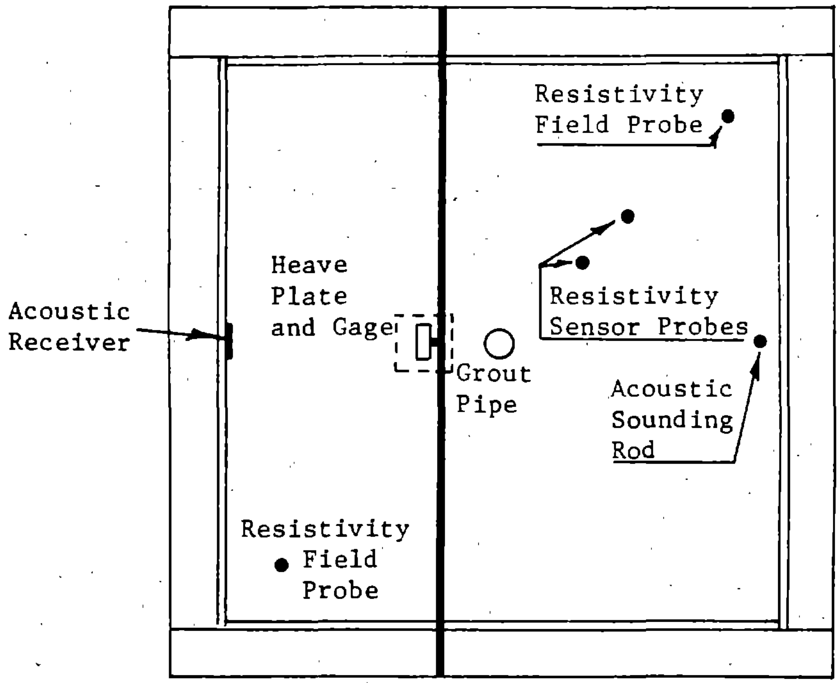
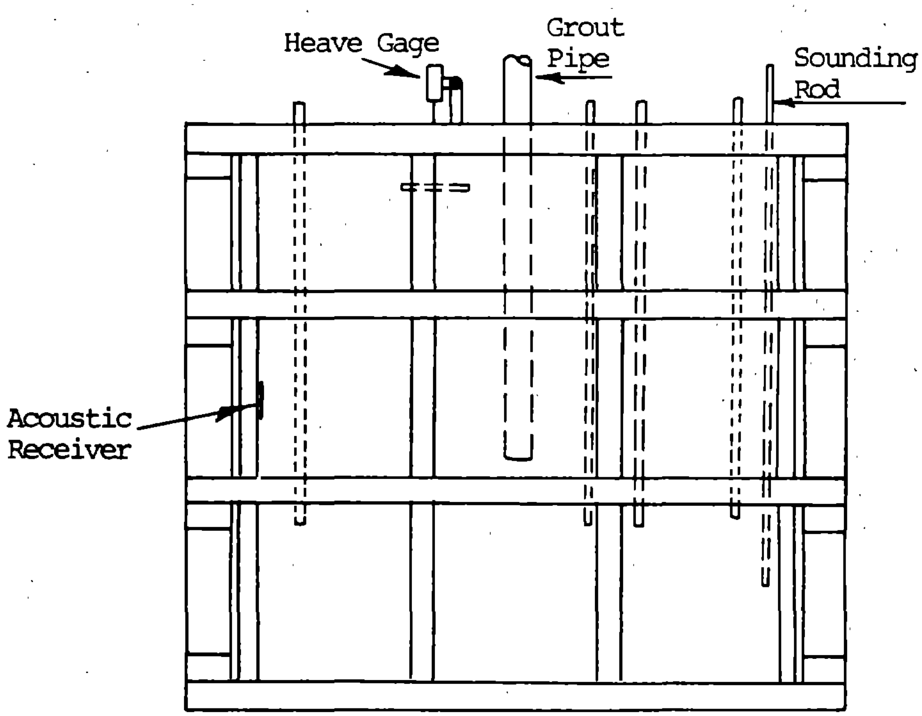


Figure 6. Chronology of Research Program



Top View of Specimen Container



Side View of Specimen Container

Figure 7. Meter-Cubed Injection Apparatus and Instumentation

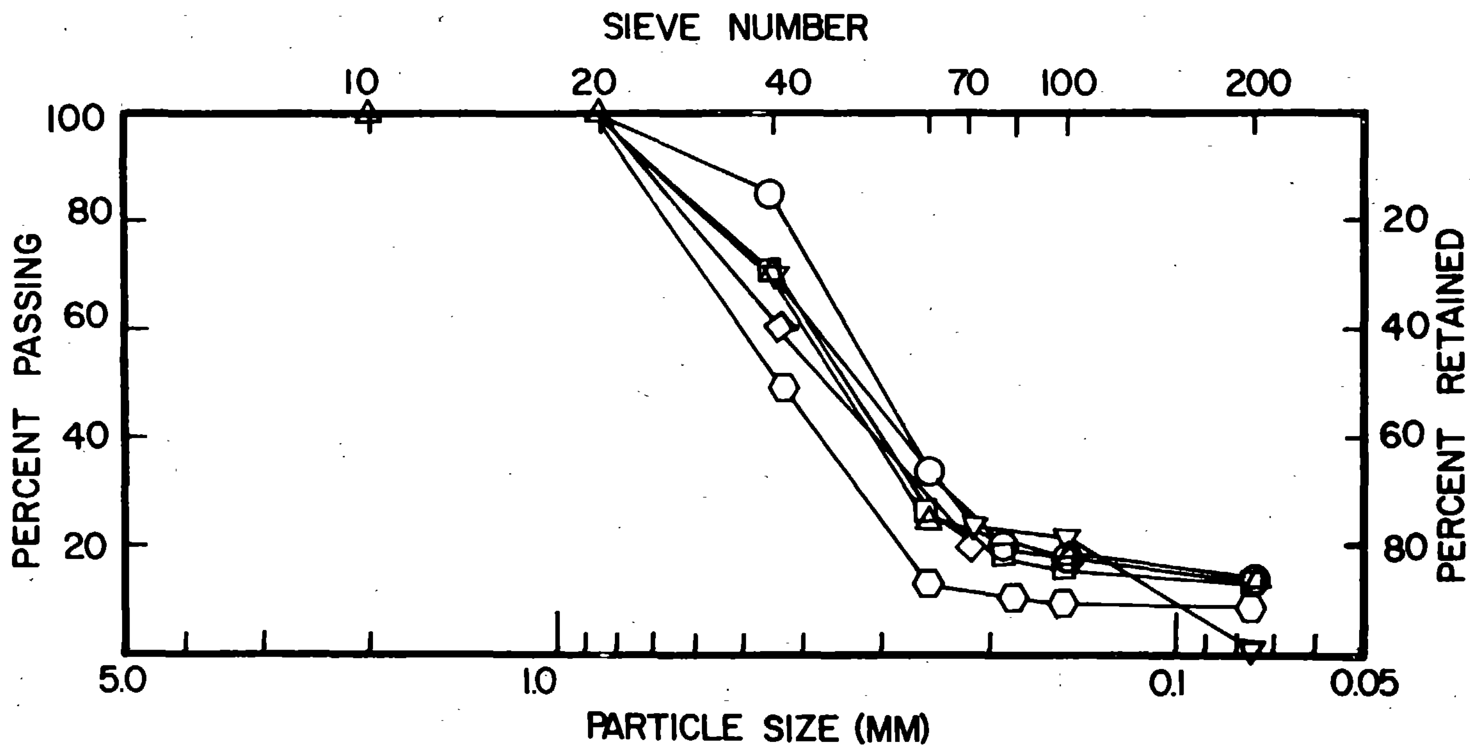


Figure 8. Grain Size Analyses-Red Silty Sand

12 to 20 percent passes the #200 U. S. (0.074 mm) sieve. Although the traditional rule of thumb holds that soils containing more than 10 percent fines cannot be grouted, it has been our experience that modern low viscosity grouts can be used in soils with up to 20 or 25 percent silt and clay sizes (passing the #200 sieve).

Each meter-cubed test involved the injection of 32 to 40 litres (8½ to 10½ gallons) of either structural or waterproofing grout. This was typically enough to develop a grout ball in the sand on the order of 0.6 to 0.75m (24 to 30 inches) in diameter. Some of the grout balls recovered from this test series are shown in Figure 9. Those having longer gel times tended to be flatter than those having short gel times, which were nearly spherical. Grout balls resulting from multiple injections were irregular. Typically, the first injection would produce a small spherical grout ball, and subsequent injections would form hemispherical caps at the top and bottom of this ball. This is shown in Figure 10, and the sequence for formation of these features was confirmed by dying the second injection. The experimental variables in the meter-cubed tests included grout type and strength, gel time, soil moisture content and injection sequence, single or multiple. High injection pressures were used in these tests. The conventional one psi/ft depth rule would have limited the injection pressures to 10 kPa (1½ psi). Although surface heave was measured with a precision of 0.1 mm (0.003 inches), no heave was observed during any of these tests.

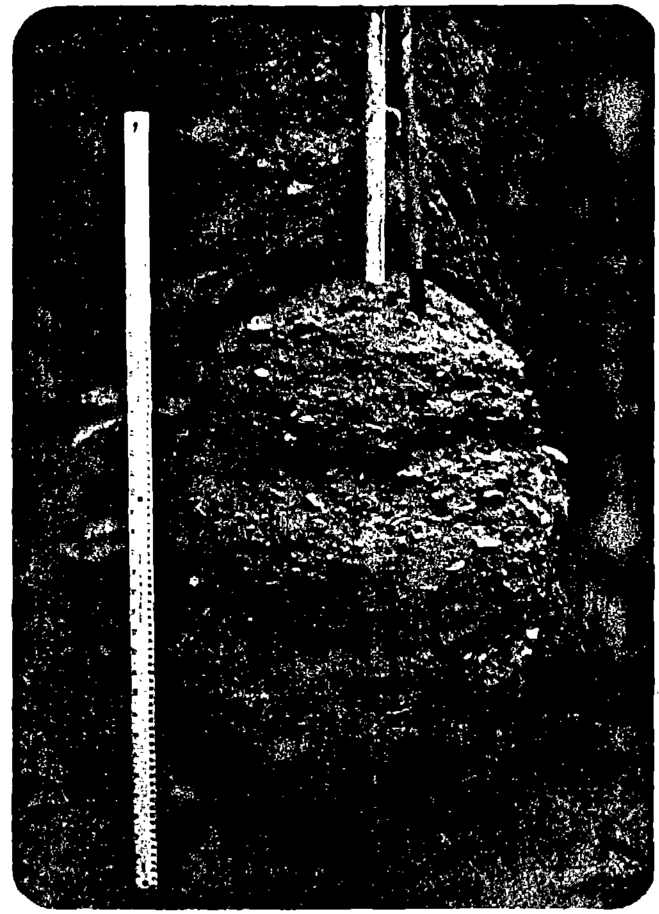
The geophysical systems selected for use in the meter-cubed laboratory tests included:

- Acoustic velocity
- Swept frequency electrical resistivity
- Earth probing radar.

The system assembled for the bench testing and subsequent meter-cubed tests were designed for placement in the soil mass.

The results of the laboratory work were the development of improved grouting procedures and geophysical grout sensing hardware. The theoretical grout injection model was found to predict bench scale grout injection tests quite well, but not the

Figure 9. Grout Ball Removed from Meter-Cubed System



(Note: This specimen is the result of two injections)

Figure 10. Irregular Grout Ball Being Removed from Meter-Cubed Injection System

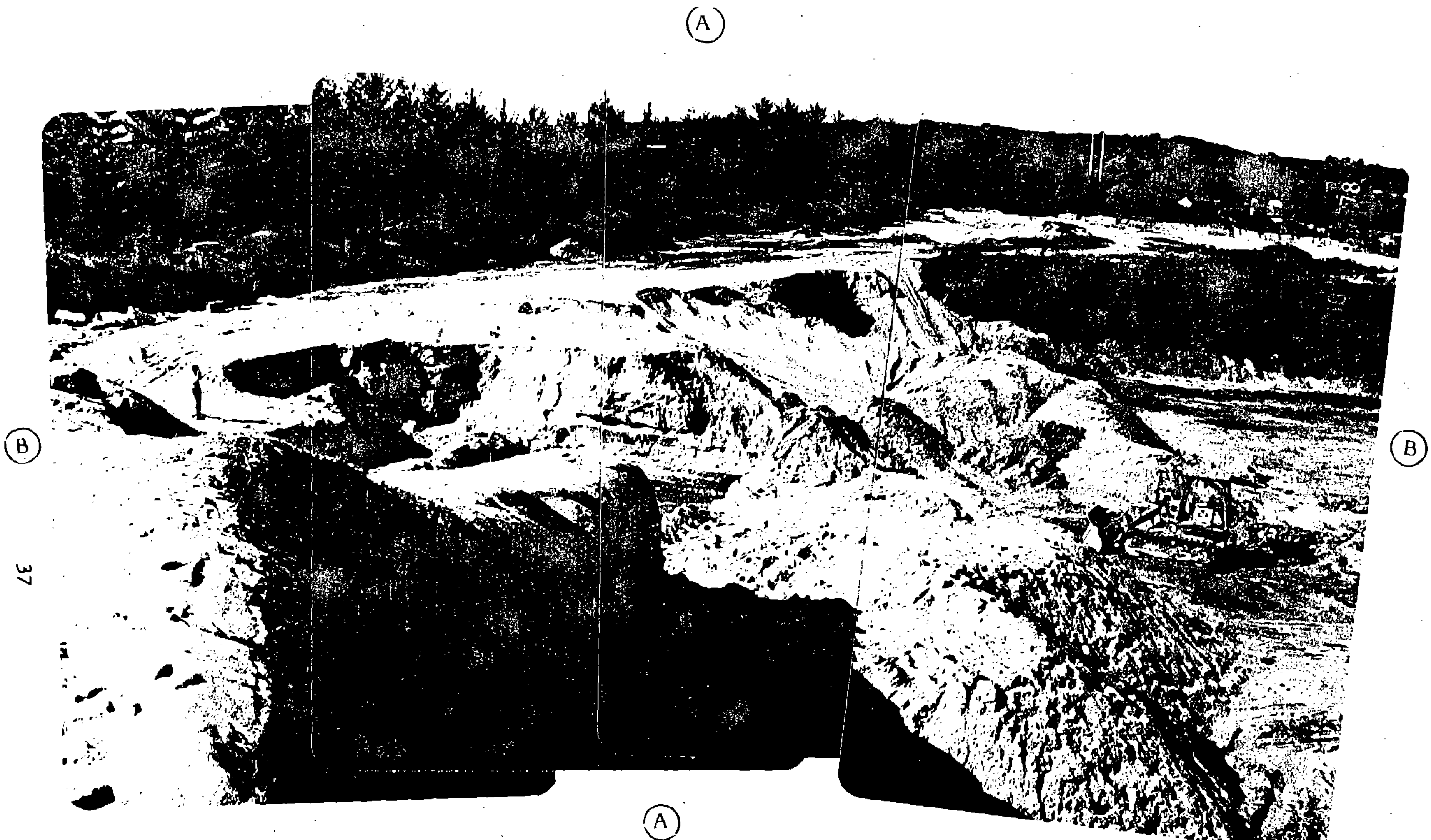
meter-cubed tests in which spherical flow geometry is permitted. The model should be revised to deal with the unstable flow conditions that occur when fresh grout is pushing behind more viscous grout. Studies with dyed AM-9 indicate inversion of the grout flow, with fresh grout pushing fingers through the older gelling grout. It was not clear how best to model this phenomenon; a minimum energy approach balancing flow area obstructed with gelled grout against viscosity of fresh grout passing through may give an approach.

The meter-cubed tests were very satisfying due to the results obtained with the remote sensing hardware. The electrical resistivity probes were excellent. In addition, the meter-cubed tests proved that the silty-sand at the pilot test site could be grouted, and indicated that the geophysical remote sensing systems could be used to detect injected grout. The continuously recording instrumentation used to monitor injection pressure in the meter-cubed tests proved quite useful in interpretation of the results, while the lack of surface heave suggested that the implications in the results of the theoretical model that high injection pressure need not be damaging were valid. High pressures were subsequently used in the pilot field injection tests with no ill effect.

FIELD TESTS

Field tests were conducted at a sand pit near the Baltimore/Washington Airport. This pit is shown in Figure 11. The purpose of these tests was to evaluate the geophysical remote sensing systems and the procedural control techniques for grout evaluation. The field tests included full scale injections using commercial grouting equipment, but with the test sequence constrained to the needs of the experimental program rather than a production grouting schedule.

The site stratigraphy, as deduced from preliminary borings, is shown in Figure 12. The zone to be grouted is a red silty sand, the grain-size analysis of which was previously shown in Figure 8. The underlying grey clay is an overconsolidated clay which formed the lower limit of excavation in the adjacent sand quarrying operations, so that the grout zone was above the level of the nearby excavation. Commercial sand quarrying was inactive during our test program. The layer of white clay above the grout zone was not a continuous layer as indicated by the initial borings, but a



Note: Tests were conducted on upper level (AA-BB). Subsequently, the grouted zone was excavated to examine grouted material.

Figure 11. Panorama of Pilot Field Test Site During Excavation

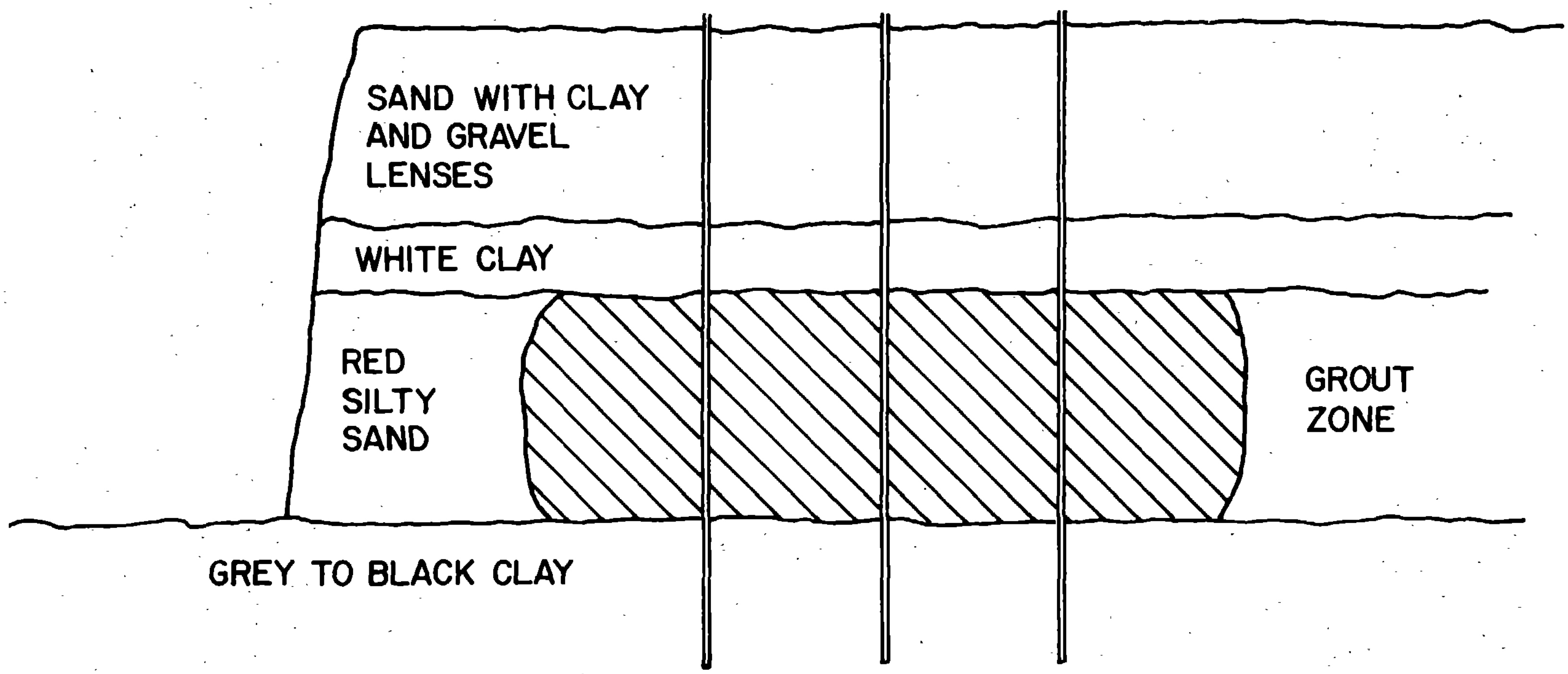


Figure 12. Pilot Field Test Soil Conditions

discontinuous layer of clay and gravel pockets which gradually graded into the clean white sand at the surface. The grout was injected at two levels in each hole. The first injection was to be at 3.35 m (11 feet) while the second was to be about 2.13 m (7 feet).

Geophysical instrumentation used on this site included:

- Earth probing radar
- Surface radar
- Cross-hole acoustic testing
- In-situ electrical resistivity
- Surface resistivity

Procedural controls employed included monitoring of pressure and flowrate and acoustic emission (AE) monitoring during hydraulic fracturing tests. The instrumentation layout is shown in plan in Figure 13, and in profile in Figure 14. Seven grout pipes designated G-1 to G-7 were placed, five in a conventional five-spot primary/secondary cell, and two off to the side for special tests. Four instrumentation holes designated D-1 to D-4 were placed about the primary/secondary cell as shown in Figure 13. Both the grout holes and the instrumentation holes were used for radar and acoustic tests. Sixteen electrical resistivity probes were placed within the grout zone as indicated by the + marks shown in Figure 13. Each probe carried three electrodes as shown in Figure 14. This permitted the synthesis of three conventional four-pole resistivity sensors adjacent to each grout pipe and in horizontal orientation.

The first series of tests was conducted before the grout was injected. The object of these tests was to provide a comparison for the tests conducted after the grouting. The acoustic tests were conducted in a crosshole mode. A 1000 Joule sparker was located in one hole and the receiver in another hole. Each pair of holes were tested at various vertical levels. Each of the holes was lined with PVC and was filled with water; however, the sand was basically dry.

The other technique used in mapping the area before grouting was ground probing radar. A series of borehole transillumination tests were run. The transmit antenna was located in one hole while the receiver was located in another hole. The two antennas were then raised up their holes at a constant rate.

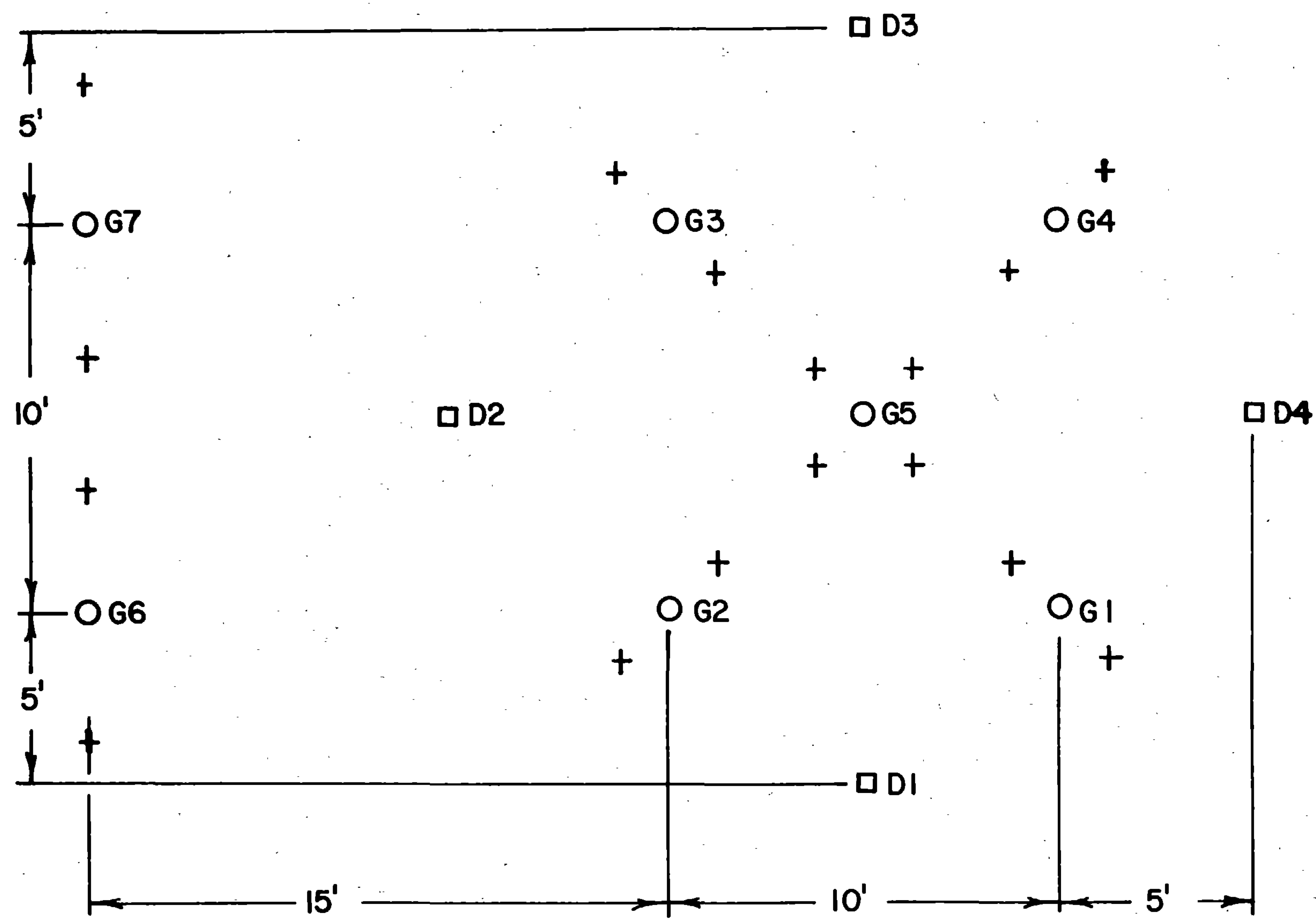


Figure 13. Pilot Test Site - Plan

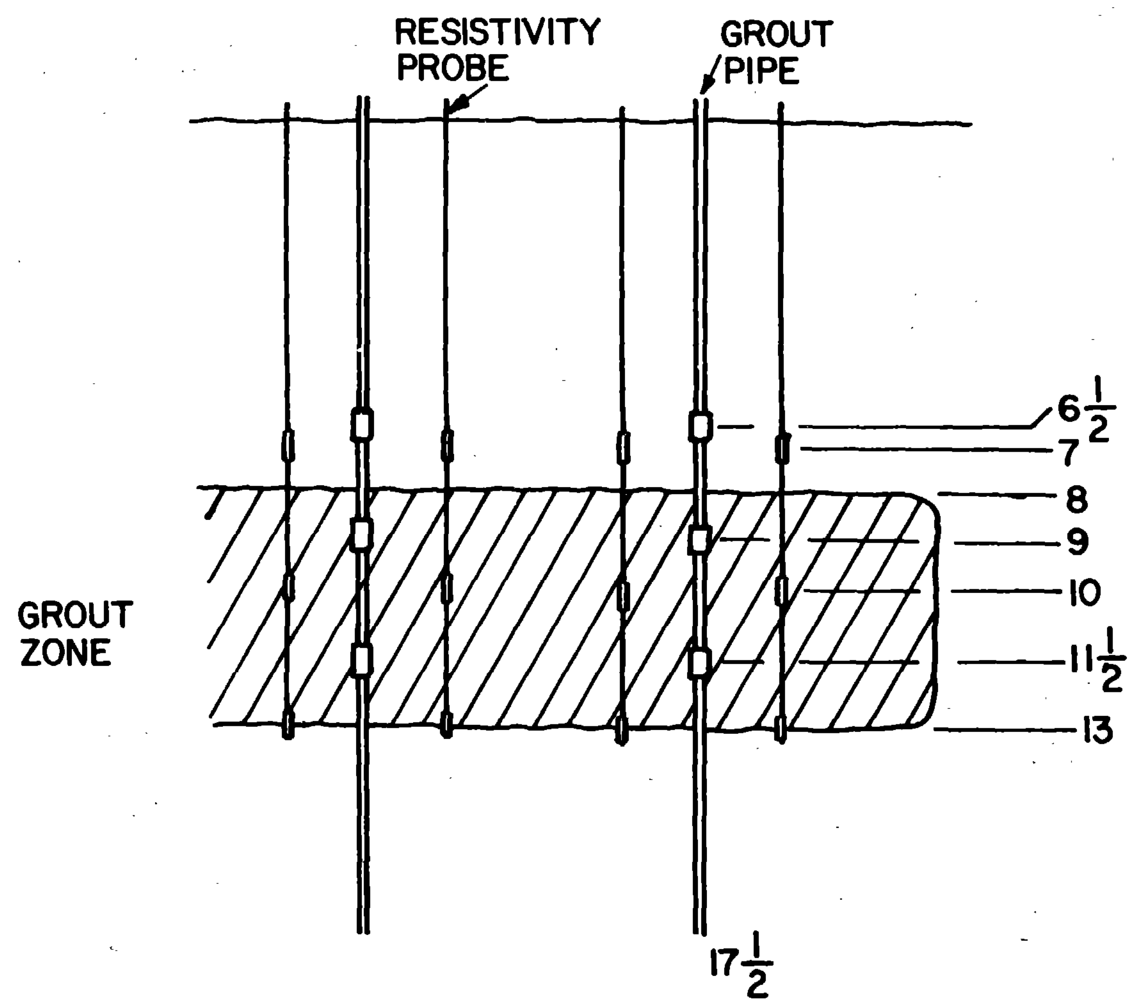


Figure 14. Resistivity Probe Elevation

The acoustic and radar tests gave a good picture of the area before the grout was injected. The next problem involved the mapping of the grout as it was injected. This was done using electrical methods. In each case measurements were made on a regular time basis as the grout was pumped.

A final series of tests were run several days after the last grout hole was pumped, to try to measure the extent of the grout and to outline the grout balls.

Grout was mixed by a plant parked on the lower level of the sand pit, and pumped up the small bluff to the grout pipes. Only one sleeve was injected at a time to provide better definition of the grouting process. After completion of the pilot field tests, standard penetration and borehole pressuremeter tests were conducted by a cooperating FHWA contractor. The site was then excavated and block samples of the grouted soil were recovered for triaxial tests.

During the pilot field tests, a total of 5,530 gallons (23,500 litres) of grout were injected into seven grout pipes. A variety of pressures and flowrates were used, all exceeding the one psi per foot of overburden rule, sometimes by an order of magnitude. Only a small amount of hydraulic fracturing occurred in the main grout zone, none caused measurable movement, even in the presence of a nearby vertical face extending below the grout zone. This would indicate that the fears of excessive movement due to high injection pressures are not justified when injecting primary holes, but should be of concern when injecting secondary holes.

Electrical resistivity was found to be an effective method of monitoring grout flow during injection, the only disadvantage being the need for probes within the grouted zone. It is likely that driven probes can be developed to reduce the need for extensive drilling. For those instances where grout must not migrate outside the intended grout zone, delineation of the boundary with resistivity sensors provide an economical and effective treatment.

The acoustic changes were also significant. In this experiment the acoustic velocity changed by a factor of 10 to 1. Although the system used required 3-inch (7.62 cm) holes, it could be configured to fit in a 1.5-inch (3.81 cm) hole. These probes could then be used to measure crosshole and uphole velocities quickly and easily from the grout injection holes. A simple facsimile or film display would be easy to interpret in the field. In some cases a simple spectra analysis of crosshole arrivals may also be useful.

The first-look radar data appeared very useful in mapping the grout, and identifying grout in zones outside the intended location. This was later confirmed by excavation. Earth probing radar must be considered a strong contender for post-grout evaluation. In summary, field tests indicated that it is possible to map soil that has been grouted using several tools. The geophysical methods tested during these field tests demonstrated significant promise for tracking the grout injection and measuring the grout afterward. Each one, with further development, is potentially capable of being implemented in a cost-effective manner in the field.

PROOF-OF-CONCEPT TESTS

Several techniques and theoretical concepts developed in the course of the laboratory and field research reported herein were eventually applied on actual field grouting projects to answer certain technical questions not addressed in the laboratory and field studies, and to evaluate and refine their applicability to real field situations. These proof-of-concept tests included an evaluation of the effect of injection pressure levels during chemical soil grouting and the development of quality control and quality assurance tools for clay and cement grouting, based on the development of similar controls for chemical soil grouting. Inter-agency funds, through the Urban Mass Transit Administration (UMTA), Water and Power Resource Service (WPRS), and Federal Highway Administration (FHWA), made possible this additional research.

The first study entitled "The Effects of Injection Pressure Levels on Chemical Soil Grouting", (12), was done in conjunction with an on-going chemical grouting project intended to protect an existing masonry railroad tunnel during subway construction. The second study, entitled "Monitoring and Control of Particulate Grouting in Rock", (13), was performed in the course of a remedial grouting program to prevent seepage in an earth dam founded over rock. The results of these two studies are summarized herein.

Laurens Street Site

The research program on improved design and evaluation of soil grouting exposed a lack of consistent criteria for the selection of injection pressure limits. Common cement grouting practice in the United States recommends that injection pressure be no greater than one psi per foot of overburden depth (23 kPa/m). This practice is often inappropriately applied to chemical grouting. British literature recommends a starting pressure of 45 kPa/m (2 psi/ft) with possible changes based on experience at each site. French practice ignores pressure and overburden depth altogether and sets limitations on flowrate dependent upon on-site experience. Since sound theoretical models of the injection process from which soil stability might be assessed at various phases of grouting do not currently exist, injection pressure is often specified without consideration for the grouting method to be used. In order to provide input data to theoretical injection models now under development, and to test suspected injection mechanisms experimentally, an instrumentation effort was added to a conventional chemical grouting contract.

The site selected was an ongoing chemical grouting project being performed by the contractor to protect an existing railroad tunnel during construction of the Laurens Street Section of the Baltimore Subway. Primary and secondary grout pipes were selected as test subjects, the primary approximately 3m (10 feet) and the secondary about 1.5m (5 feet) away from the existing tunnel.

Experimental Program

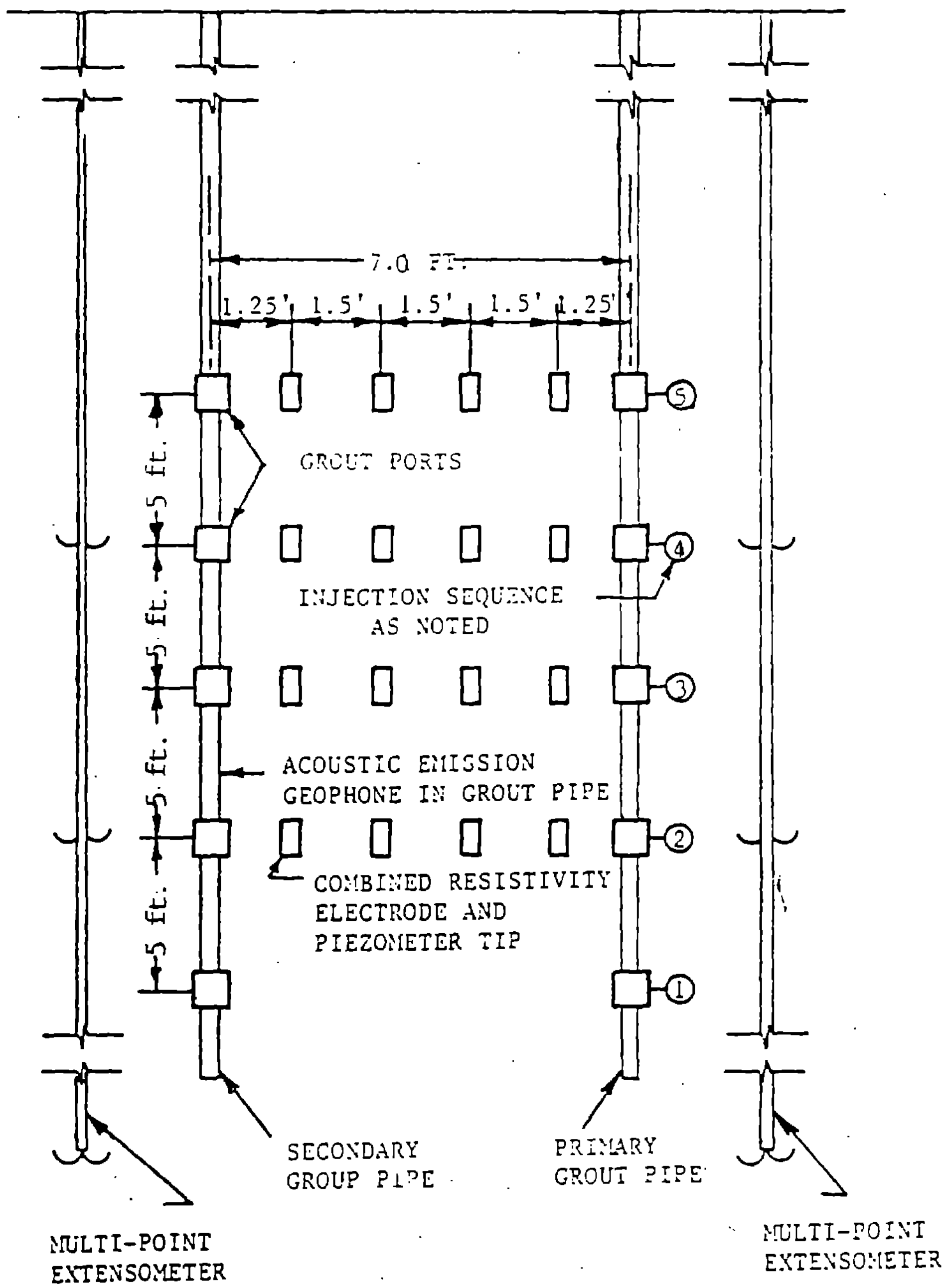
It was determined that the parameters that should be measured included the grout pore pressure in the soil near the injection point, the location of the grout after injection, vertical soil displacement in the grout zone and acoustic emission level to warn of impending hydraulic fracture. These data could then be compared to injection pressure and flowrate records to evaulate the effect of various injection pressures. Grout was injected using the sleeve-pipe method, which requires that grout pass beneath a rubber sleeve and through a mortar seal or jacket which is cracked at the start of injection.

The instrumentation was deployed as shown in Figure 15 and 16. Pore pressure was measured at 16 piezometer tips (Norton stones), using coaxial tubing that permitted de-airing the porous stones. Pore pressure was read on individual vacuum-pressure bourdon tube gauges. Water was periodically injected at each tip to flush out any grout that might have entered. The subsequent relaxation of pressure indicated whether the piezometer was still responding, or had been grouted shut. Each piezometer tip incorporated an electrode, so that four-point resistivity arrays could be generated in eight locations (four horizontal and four vertical). The resistivity system was driven by a constant current AC source. DC resistivity cannot be used effectively in grout due to electro-osmosis. Adjacent to each grout pipe a Multi-Point Extensometer (MPX) was installed to monitor vertical ground displacements. Acoustic emissions were monitored by two accelerometers, one at the ground surface and one down an adjacent inactive grout pipe. Finally, grout injection pressure was monitored at the top of the packer tube. Measurement of flowrate was attempted using acoustic transducers, but the flowrates involved were below the effective range of the two systems available, and only marginal data were obtained. Pressure and flowrate were measured independently by the conventional manually read gauges included in the normal grouting system, so flowrate data were recovered.

Summary of Results

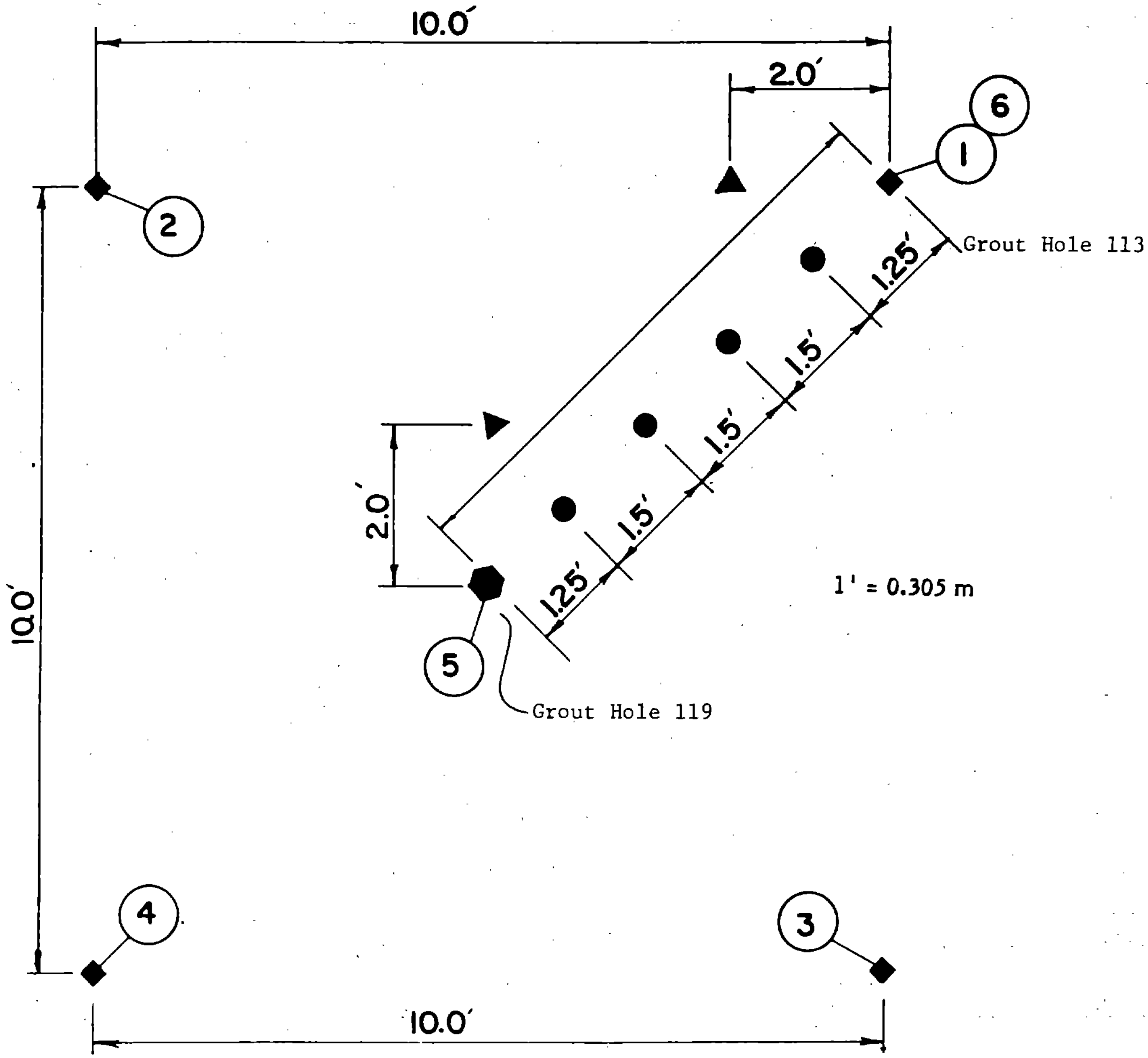
At the Laurens Street site, injection presure and flowrate, pore pressure, acoustic emission rate, electrical resistivity and ground motion were monitored during injection of one primary and one secondary grout pipe.

Pore pressures were measured to develop a preliminary correlation between injection pressure, and the pore pressures actually developed in the soil voids. The greatest pore pressure measured during the six days of injection was 45 percent of the injection pressure. Three-quarters of the pore pressure data were less than 20 percent of the injection pressure, the overall average pore pressure being only 13 percent of the injection pressure. A large part of this reduction in pressure probably occurs within the mortar jacket. From these data it was concluded that injection pressures may be twice the overburden stress, and usually higher, as long as the soil around the injection port is not completely confined by previous grouting. If a system is used to continuously monitor incipient ground distress, even higher injection pressures and



Note: 1 ft = 0.305 m

Figure 15. Grout Pressure Instrumentation Cross Section



◆ PRIMARY GROUT HOLE
⬢ SECONDARY GROUT HOLE
● PIEZOMETER HOLE (4 PIEZOMETER TIPS PER HOLE)
▲ MULTI POINT MECHANICAL EXTENSOMETER
(3) GROUT SEQUENCE AS NOTED

NO SCALE

Figure 16. Plan of Laurens Street Instrumentation

flowrates can be used with complete safety at the majority of the injection ports since those few ports at which reduced pressures are necessary would be identified. The savings in labor and time would permit significant reductions in the cost of grouting while the ground monitoring would provide positive assurance that no damage is being done.

Continuous recording of injection pressure and flowrate provided a much clearer visualization of events underground than can be obtained from manually recorded data. Production grouting should employ robust strip chart or circular recorders. When such data are available, the grouting operation can be controlled more precisely, and the field recorder charts provide an important tool for post-grouting evaluation.

Acoustic emission monitoring has already been shown to be capable of monitoring ground distress and hydraulic fracture. This research demonstrated that it can be effective on a noisy urban construction site. If acoustic emission monitoring is appropriately applied, it is an effective tool for monitoring ground distress, allowing elevated injection pressures to be used in many cases without compromising safety.

Electrical resistivity detected the build-up of grout in the test zone, but was somewhat hampered by the fact that some grout was already in the area from adjacent production grouting. Resistivity is a sensitive method for detecting the intrusion of grout into the sensor volume, if in-situ probes and an AC source are used. Vertical ground motion was measured using two multi-point extensometers having a total of eight anchor points. The pattern of motion observed was extension of the soil volume near the active grout port during pumping followed by rebound in the hours after the end of the day's injection. Motions during pumping were sometimes as large as 5 mm (0.2 inches), of which about 75 percent would rebound after pumping stopped. Permanent deflections after all primary and secondary injections were completed also ranged from near zero to about 5 mm (0.2 inches).

Comparison of heave rate and injection parameters indicated that the elastic heave observed during injection depends upon the total volume of grout pumped rather than the pressure, for the range of conditions observed at this site. By implication, it appears that the use of increased injection pressures may not result in increased heave. Correspondingly, the injection of excessive amounts of grout must result in either increased soil void ratio and surface heave, or migration of grout beyond the grout zone, even if low pressures are used.

Reston Dam Site

During the course of the study, the writers were able to test the geophysical techniques and grouting procedural controls on a rock grouting job. Grouting was employed to halt seepage through the rock mass underlying an earth dam.

The test site was a production rock grouting project conducted by the contractor. This is a homogeneous earthfill dam 18 m (60 feet) high and 260 m (850 feet) long. Plan and profile are shown in Figures 17 and 18. The embankment soil may be classified as sandy clayey silt to sandy silt, type ML according to ASTM-2487. The underlying foundation is phyllite rock of Wissahickon formation, generally highly fractured and weathered. The rock contains a prominent foliation which strikes nearly perpendicular to the dam axis, with a dip in the range of 80 degrees. Field permeability tests in the rock indicated a coefficient of permeability of 8×10^{-3} cm/sec, while the embankment permeability ranged around 1.5×10^{-5} (cm/sec). Grouting in the rock was undertaken to reduce suspected underseepage through the foundation rock.

The grout plan was the traditional split-spacing method, shown schematically in Figure 19. In this process, primary holes are drilled and grouted, and then the intervening spaces are grouted from holes placed midway between those of the preceding stage. In each stage of grouting, the distance between holes is halved. Grouting proceeds until the injection pressure and grout take indicate that the area is tight from previous injections. In this case primary holes were spaced at 9.1 m (30 feet) and the subsequent stages were at 4.5 m (15 feet) and 2.3 m (7½ feet). Grouting locations are indicated on the plan shown in Figure 17. Two stages of grouting were sufficient from station 3 + 40 to 4 + 00, then three stages were used for the remainder of the grout curtain.

The grout used was a bentonite clay/portland cement slurry with small amounts of sodium silicate added to accelerate setting upon need. If a hole was taking excessive amounts of grout, the silicate was added to limit the travel of the grout in

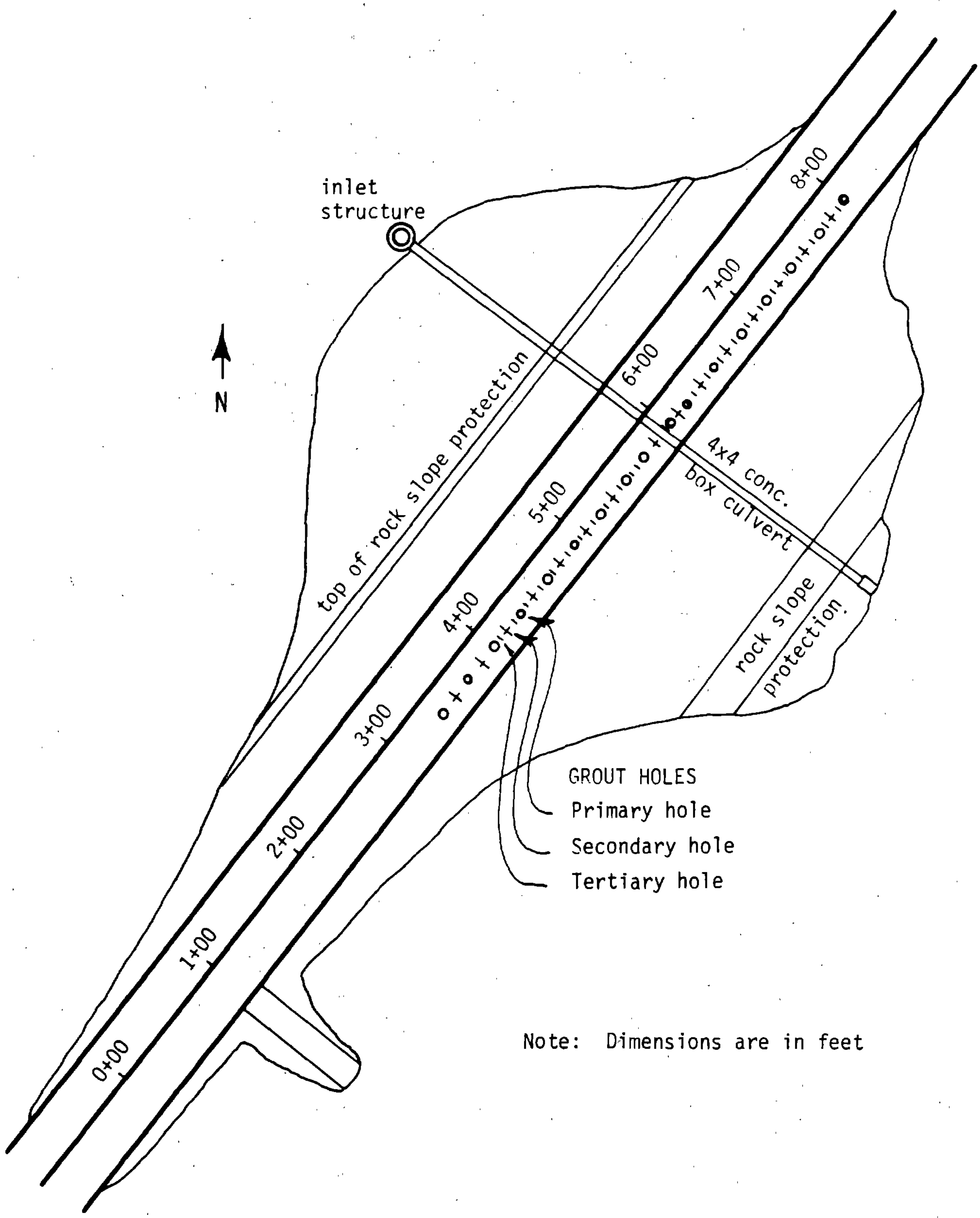


Figure 17. Plan of Earth Dam Test Site

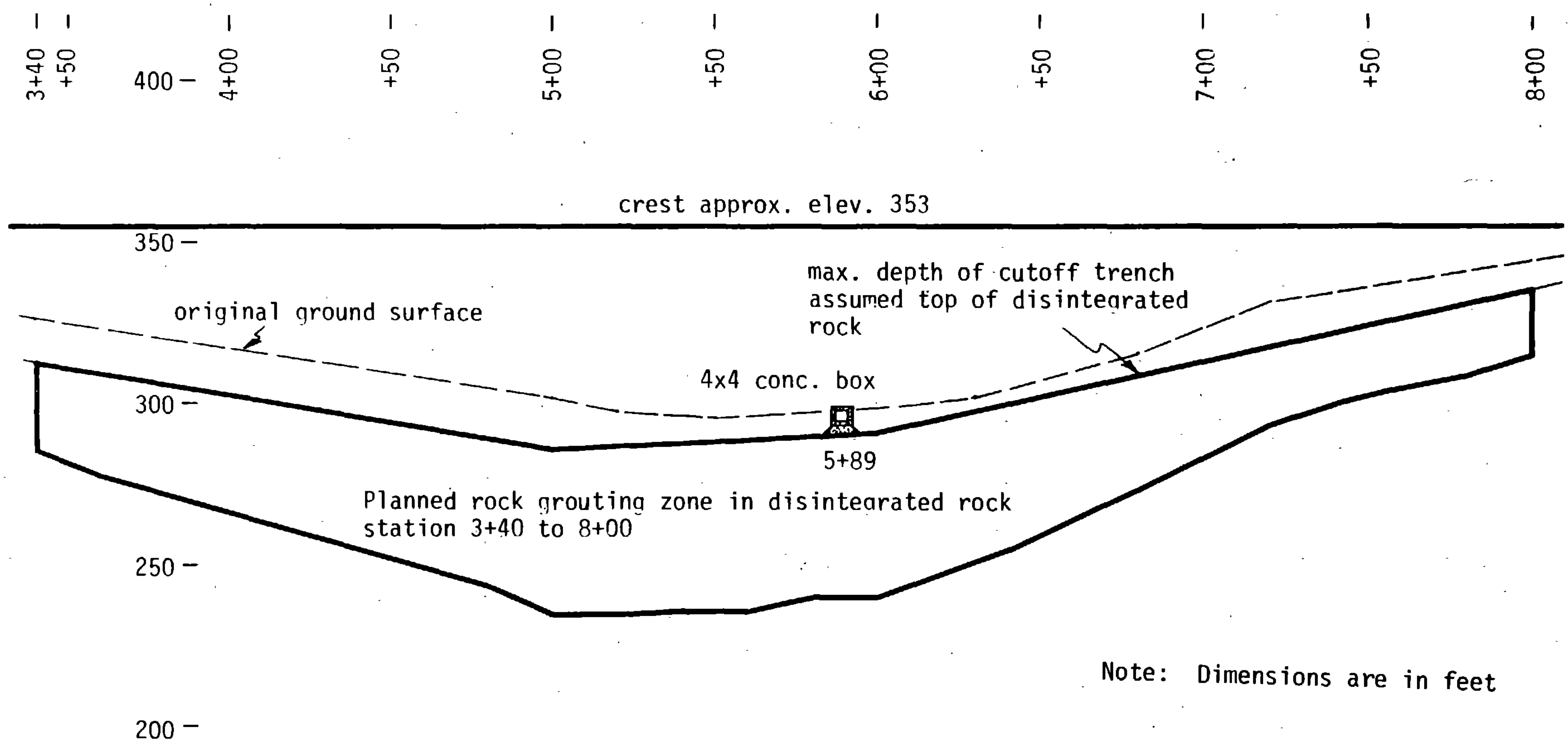


Figure 18. Elevation of Earth Dam Test Site

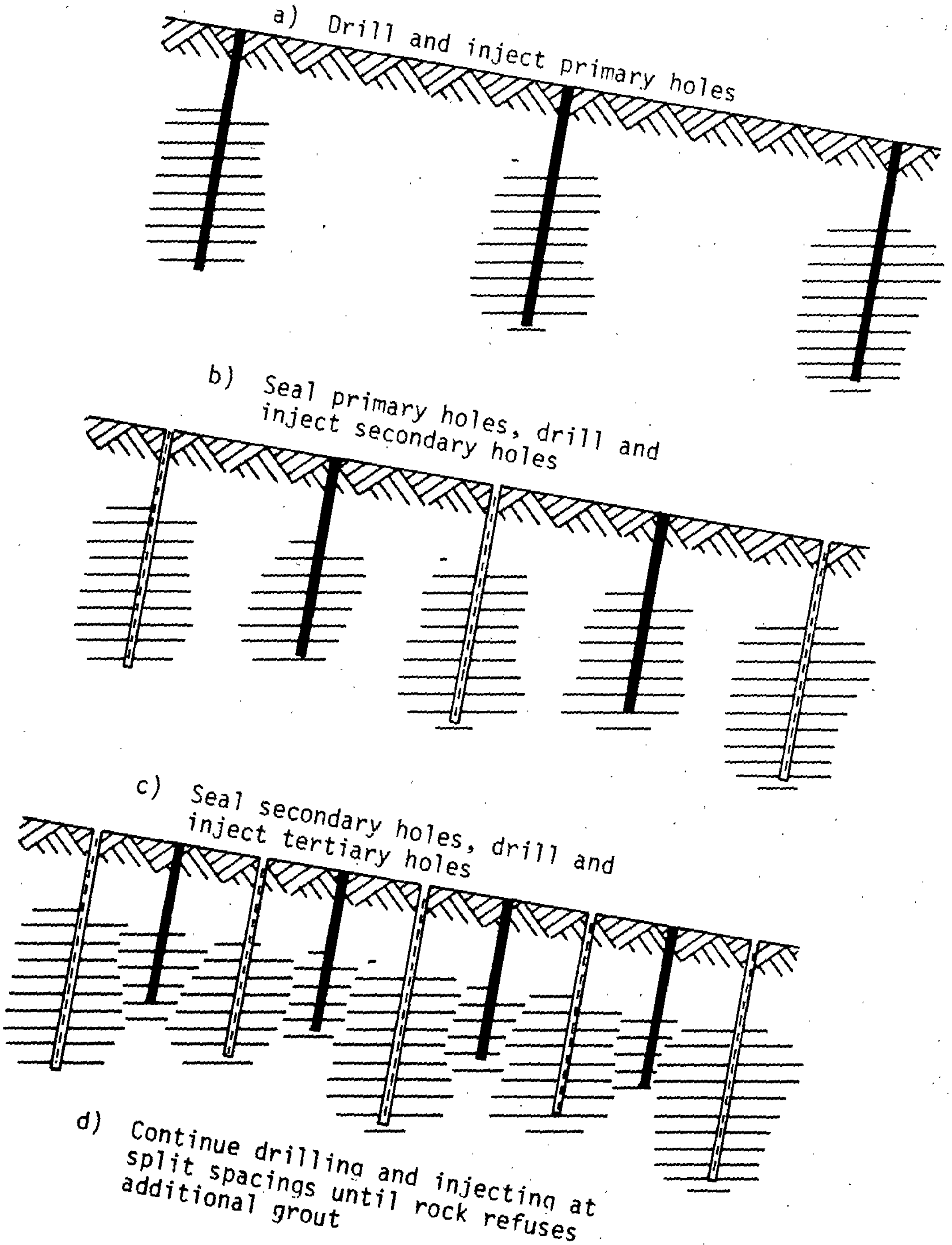


Figure 19. Conventional Split-Spacing Grouting Sequence

the rock. Following injection of this grout into the coarser fissures and voids, a conventional sodium silicate chemical grout was injected to seal the finer voids. This dual grouting process, using the less expensive particlate grout to seal the major portion of the void volume, and the chemical to complete impregnation of the mass, is frequently found to be cost effective.

During drilling and initial injections it was found that the foundation rock was quite open. During drilling, the drill water would on occasion be lost. It would flow into the rock so fast that flow back to the surface ceased completely. Water injection tests developed very high flow values, particularly in the section selected for the monitoring tests. The lack of competence in the rock was also seen in the difficulty experienced in maintaining open boreholes. In several cases, holes would collapse and require redrilling. The acoustic instrumentation could clearly distnguish the sound of small particles sloughing off and falling down the hole.

Experimental Program

The experimental program included both procedural tests and geophysical surveys. The procedural tests used at this site included injection pressure, flowrate, grout take, and acoustic emission to identify structural distress in the rock during injection, and to search for zones of high groundwater flow before and after injection. Water injection tests were conducted as part of the grouting contract. In this procedure, a 3 m (10 feet) section of borehole is isolated by packers, and the flowrate and pressure when an injecting water is determined. The test data are expressed in units Lugeon (litres/minute/meter of a borehole at an injection pressure of 10 kgf/cm^2).* The geophysical tests used included cross-hole acoustic shooting and transillumination borehole radar. These geophysical methods are well suited to defining fractures in rock, and are probably the most viable geophysical systems in the light of our experience in monitoring soil grouting.

*Water injection tests may be expressed in Units Lugeon, i.e. litres/minute/meter of borehole length at an injection pressure of 10 kgf/cm^2. Because the borehole diameter is not included in the calculation, a unit Lugeon is not a true measure of permeability.

Summary of Results

An unexpected feature of the site that significantly affected the result throughout this program were the high permeability and low strength of the basement rock. As a result of this, more grout was required than had been expected. Drilling was more difficult, and holes left open for any period of time tended to collapse. It was necessary to redrill several holes that collapsed before they could be grouted. The unevenness of the boreholes in the rock made the geophysical measurements difficult and limited the depth to which the instruments could be lowered in several cases. The acoustic emission measurements were also affected, for small particles would frequently slough off the borehole walls, causing spurious noise counts. The high permeability made water injection tests difficult, for only low pressures could be obtained at the highest pump flowrates available. Finally, the higher permeability made it impossible to cause hydraulic fracturing even in a tertiary hole. The major pressure losses in the hydraulic fracturing test were in the grout line, and it was impossible to obtain pressures at the grout hole higher than three psi/ft of overburden depth. This was insufficient to fracture the rock.

Despite difficulties in implementing some aspects of the instrumentation program, useful results were obtained. The program has resulted in a set of guidelines to aid in structuring quality assurance and quality control programs for future rock grouting projects.

Positive results obtained during this effort include:

- The use of continuously recorded injection pressure and flowrate was shown to identify grout refusal clearly, in addition to leaving a permanent document from which the activities of the grouting contractor may be reviewed at any future time.
- The use of plotted water injection test data was shown to be useful in showing the degree to which the rock is sealed by sequential stages of grouting. Once plotted, these data may be reviewed at a glance. If, however, they are presented as a sheaf of individual test results, they are very difficult to evaluate, unless the reviewer himself plots the data.

- The grout take log was not only shown to be an effective presentation of overall grouting progress, anomalies, and trends in underground behavior, but was found to be a focal point for project discussions and decisions.

CHAPTER 4—TECHNICAL FINDINGS

This section presents the detailed technical findings relating to each grout control and elevation procedure considered during the program. Discussion begins with the theoretical grout distribution model, including discussion of the laboratory grout viscosity tests, and the pore-pressure data contained at the Laurens Street site. The discussion then focuses on the geophysical systems and the procedural controls in order. These two categories divide conveniently into activities that require the specialized skills of a geophysical consultant, and those that can be applied by the grouting contractor or the civil engineer for the rather trivial cost of instrumentation and familiarization of personnel with its operation.

GROUT DISTRIBUTION THEORY

The motivation for developing a mathematical model of the grout injection process is primarily concerned with the stability of the soil being grouted. The strength of frictional materials can be characterized by the Mohr-Coulomb failure criteria as

$$T_f = (\sigma - u) \tan \phi + c$$

where T_f is the shear resistance at failure, ϕ is the friction angle, σ is the normal stress, and c is the cohesion. T_f is clearly sensitive to the distribution of pore-pressure, u, especially if as is often the case in groutable soils, the cohesion, c, is zero. Wherever the pore-pressure equals or exceeds the normal stress, the shear strength will be zero and a quick condition will prevail. Grout injection models previously published have considered only the case of a constant viscosity grout. These are not particularly useful even for a batch mixed system, where the grout in the ground varies in age and viscosity from point to point, a constant viscosity model is totally unrealistic. In addition, certain models found in the literature are unrealistic even in terms of a simple constant viscosity fluid, and can easily be improved.

Grout Viscosity

Laboratory tests were conducted to develop viscosity/time curves for the grout to be used in this project. Grout viscosity was determined using two independent measurements. One system used a laboratory grout injection system as shown in Figure 20. Sand was compacted into a 3.8 cm Ø by 7.6 cm long (1.5 in Ø by 3 inches long) plastic mold and the apparatus assembled so that flow was in the same direction as the compaction. Water was pumped through the specimen at constant flowrate to develop specimen permeability data. A 600 ml. batch of grout was then mixed, introduced into the system and pumped through the specimen. Thus, the grout was of uniform viscosity everywhere in the system, and the viscosity at any given time after mixing could be determined from the instantaneous pressure and flowrate. These parameters were measured by pressure transducer, and by stopwatch and electronic balance respectively. A time tic was placed on the X-Y recorder every time the balance indicated that an additional 10 ml of grout had passed through the specimen. Typical data are shown in Figure 21. The increase in pressure when the grout is poured into the system can be seen, as well as the gradual increase in viscosity as the batch of grout gels and finally refuse to pump. Viscosity was back figured by comparison of pressure and flowrates for the water and the grout at various times.

The other grout viscosity measuring system was a laboratory rotating spindle viscometer (Brookfield model RVXO.25 with UI adapter). Because the original users of this system were primarily interested in the viscosity near the point of gellation, around 100 cP, little data were reported in the range below 10 cP which was of interest to this program. Data from the injection tests are shown in Figure 22 for waterproofing grouts, and in Figure 23 for structural grouts. Figure 23 also shows a low-viscosity curve obtained by Borden using the lab viscometer. These data are characterized by an initial period of nearly constant viscosity, followed by a rapid increase. The gradual increase in viscosity seen in Figure 23 for the 60 percent grout is an artifact of the tests; this particular specimen displayed a uniformly decreasing permeability during water injection, apparently caused by silt fines being swept into the porous stone end cap. These data are peculiar to the grout formulation and test conditions. Different reactants would not necessarily be expected to display exactly the same curves, although the trends observed should be similar. The time to gel, of course, depends on the reactant concentration, and can be varied from seconds to

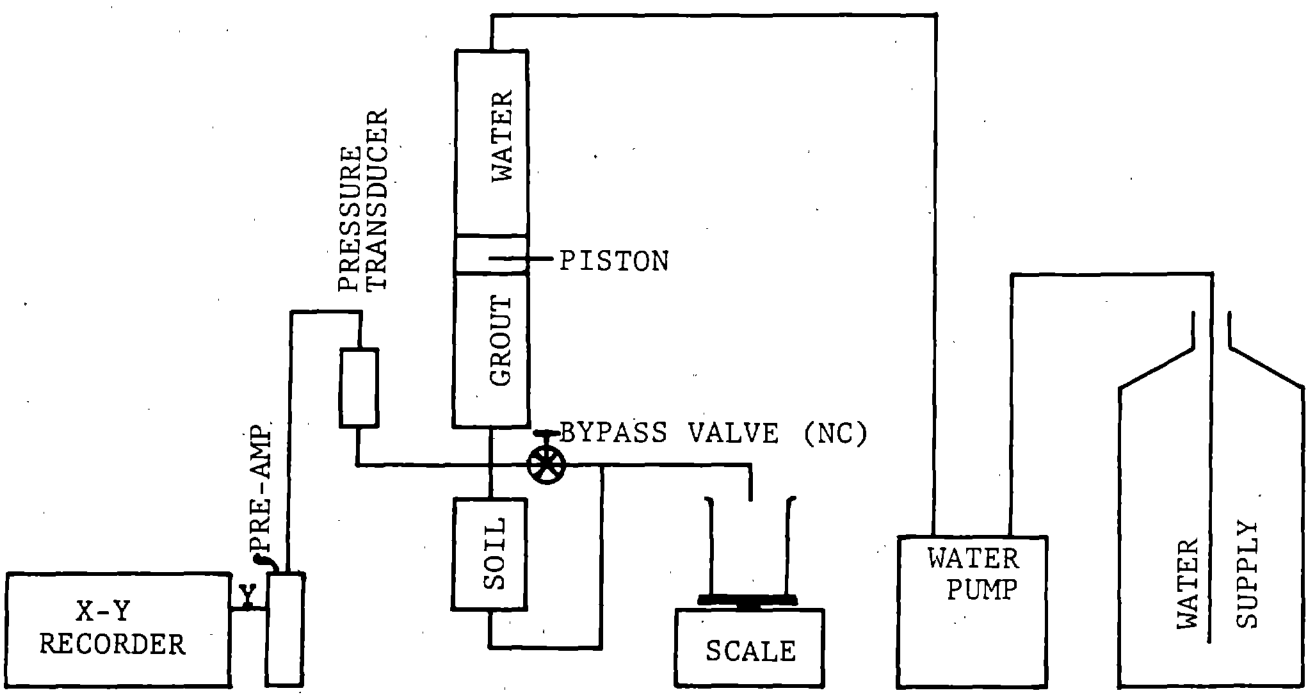


Figure 20. Laboratory Grout Injection System

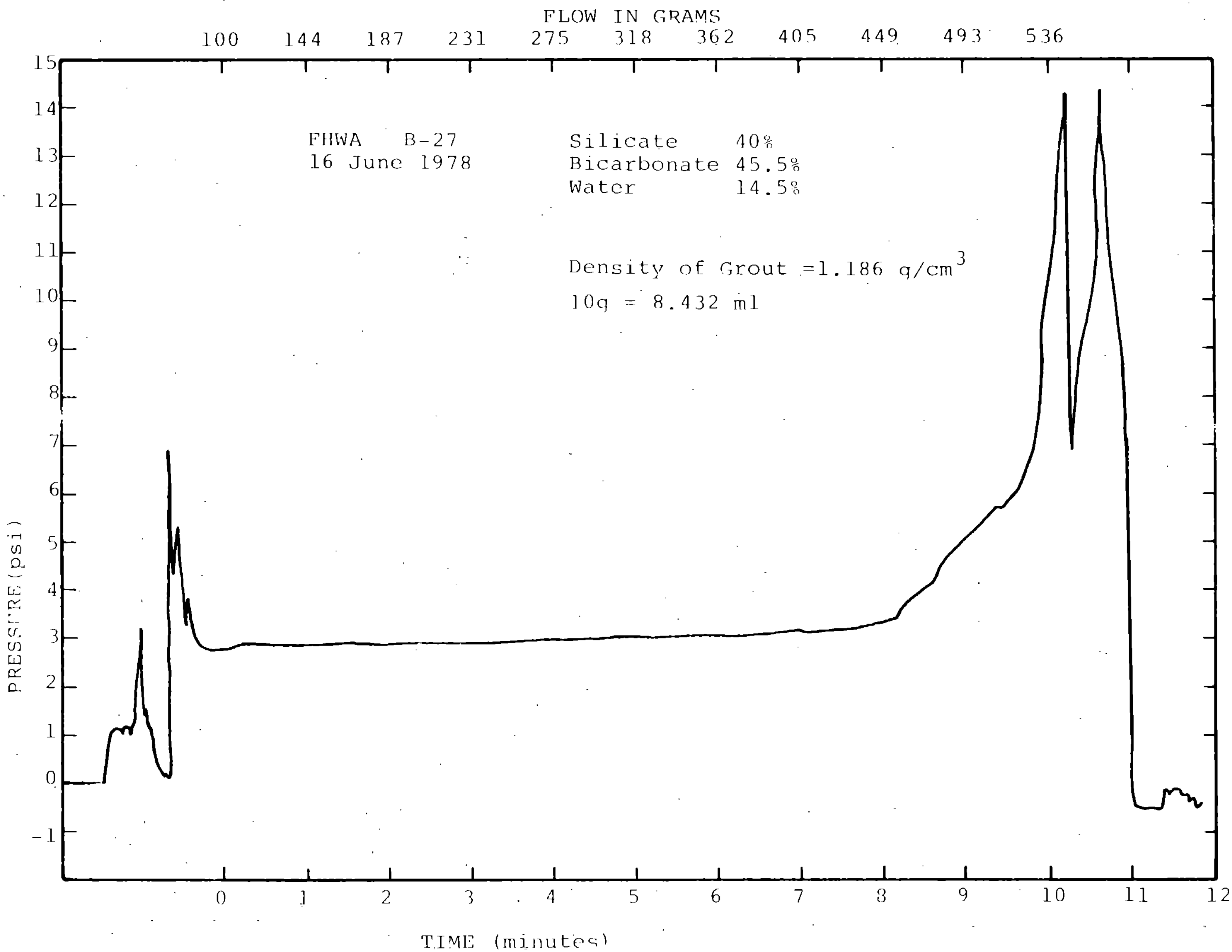


Figure 21. Pressure-Time Data - Grout Viscosity Test - Waterproofing Grout

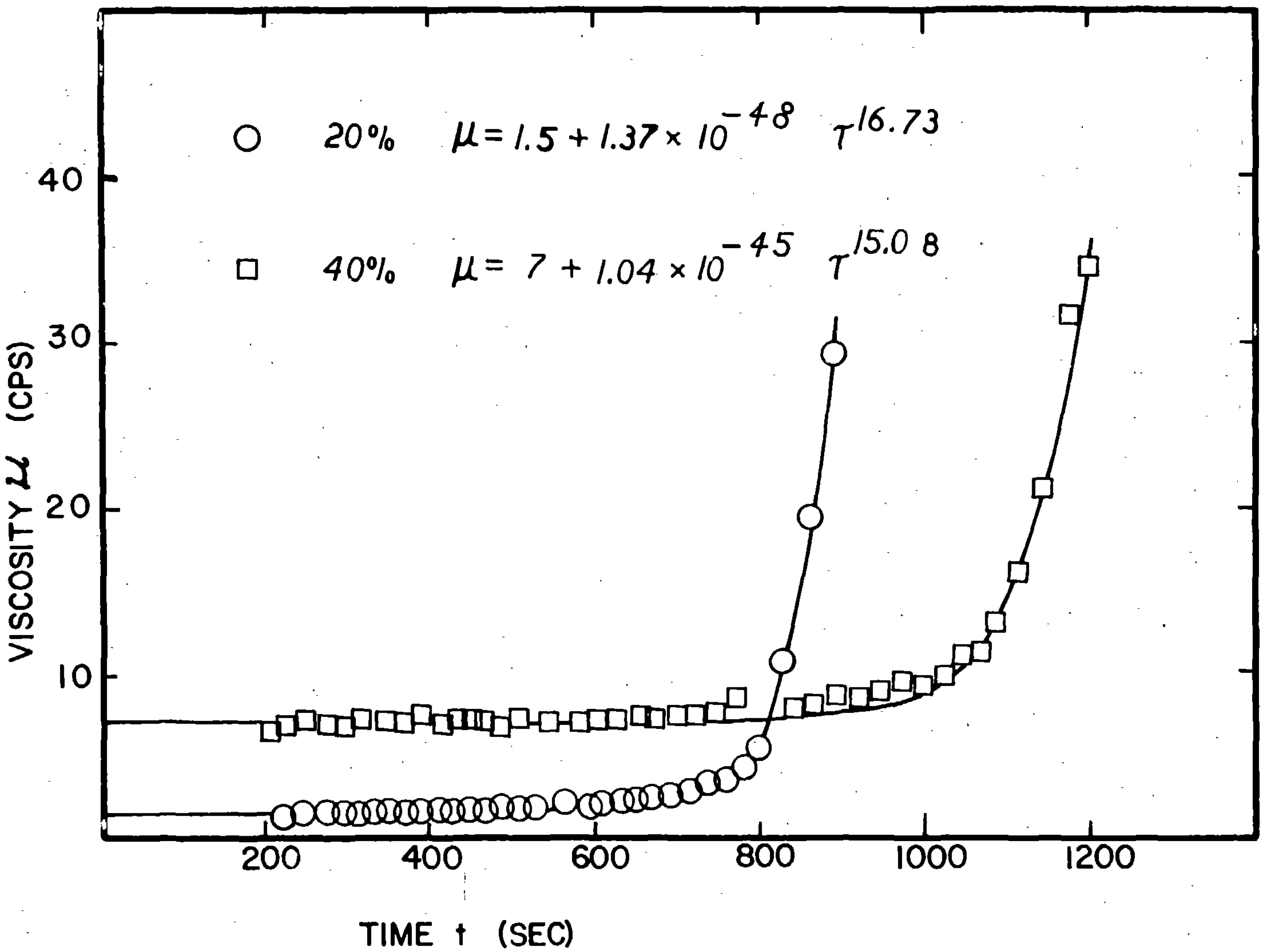


Figure 22. Viscosity of Waterproofing Grouts - Time Data

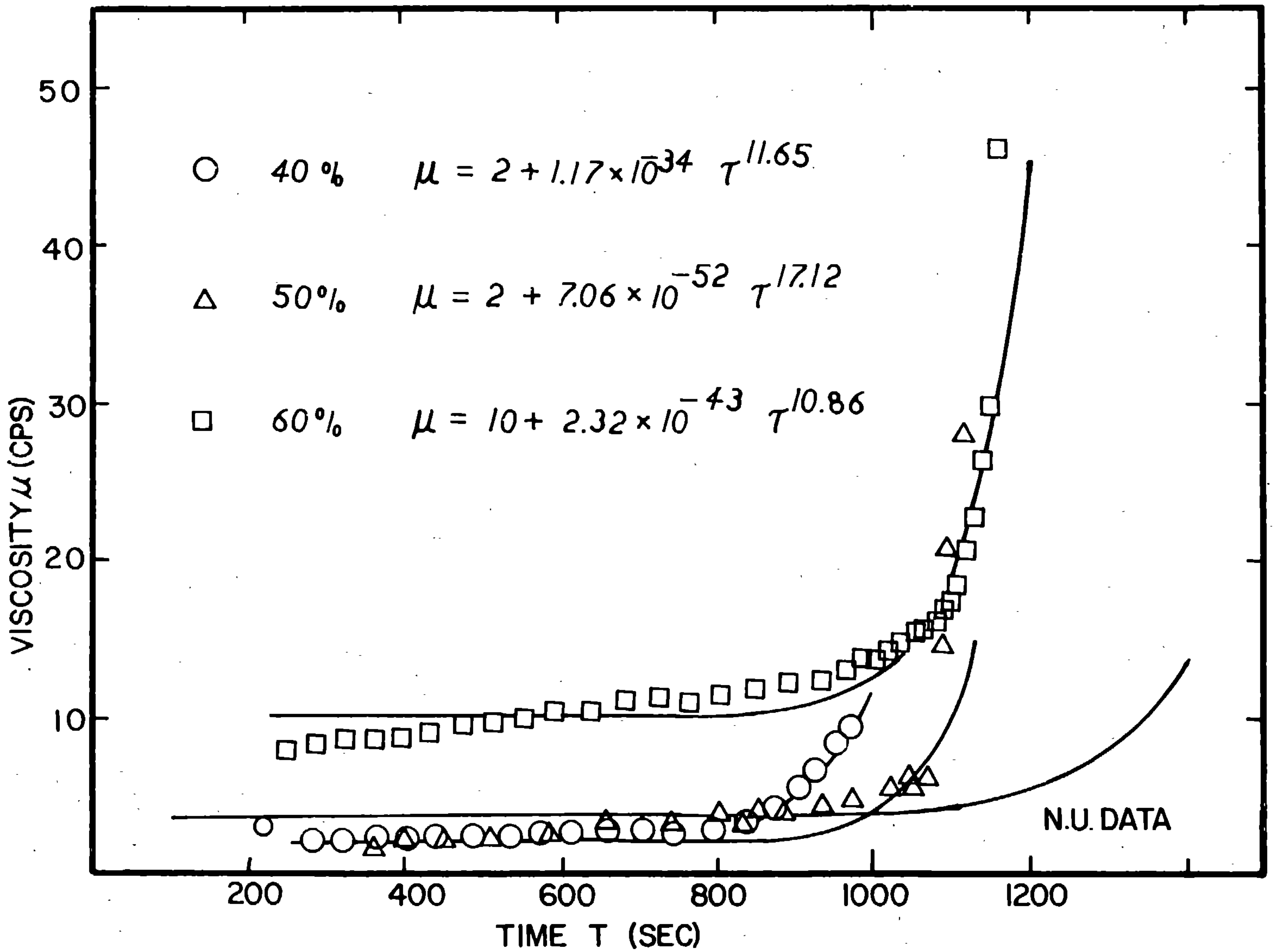


Figure 23. Viscosity of Structural Grouts - Time Data

hours with corresponding changes in the viscosity curves. These data were fit to a power law to provide easily integrated input into the mathematical model. The coefficients and exponents resulting from this curve-fitting are shown in the figures.

Injection Model Arithmetic

The theoretical grout injection model was developed applying the grout viscosity law to Darcian flow and integrating from the limit of the advancing grout front to a small radius taken as the radius of the injection port. Spherical geometry is employed.

Assumptions of Conditions

Consider a model corresponding to the following assumptions and conditions:

- Darcian flow holds in an isotropic homogeneous medium.
- Grout viscosity is represented by a power law appropriate to a continuous mixing injection system.
- Flow is radial (spherical) and away from a point.
- Source has a constant flowrate of q.
- Flow does not invert.

The last assumption limits the usefulness of the model to pumping times less than the gel time, since it is known that the flow of a less viscious fluid behind a more viscious fluid is unstable. The less viscous fluid tends to push through the more viscous fluid ahead of it, inverting the order of flow. This has been graphically displayed for chemical grouts by Karol (1968) using sequentially dyed grout injections. The grout injected first gels near the injection point, while the grout injected last pushes through and ends up farthest away. However, incorporating flow inversion into the model would complicate the problem.

Beginning with Darcy's Law

$$q = k'iA \qquad \text{Eq. (I)}$$

in which q = flowrate, k' is Darcy's coefficient of permeability, i is the hydraulic gradient, and A is the flow area at any given point, flow area $A = 4\pi r^2$

hydraulic gradient $i = \frac{\partial h}{\partial r}$

physical coefficient of permeability $k' = \frac{k\rho}{\mu}$

we substitute the following relations into eq. (I):

$$\gamma = \frac{k\rho}{\mu} \frac{\partial h}{\partial r} + \pi\gamma^2$$

Separating variables gives:

$$\partial h = \frac{q\mu}{k\rho\, 4\pi r^2} \partial r \qquad \text{Eq. (II)}$$

in which h = head, dimensionless

$$\frac{\partial rq}{k\rho\, 4\pi r^2} = \partial r$$

q = flowrate in cm^3/sec

γ = viscosity in centipoise

r = radius in cm

k = physical permeability in cm-sec (note: not cm/sec)

μ = fluid density in gm/cm^3

Assuming a constant q, we find that $\mu = f\left[t\right] = f\left[g(r)\right]$
an empirical power law for viscosity as a function $f\left[t\right]$,

$$\mu = \mu_o + \alpha t^\beta \qquad \text{Eq. (III)}$$

and computing the travel time from r = o to r = r from the pore volume of a sphere containing soil of porosity n as:

$$t = \frac{4\pi r^3 n}{3q}$$

we find that our $f\left[g(r)\right]$ takes the form

$$\mu = \mu_o + \alpha \left[\frac{4\pi n}{3q}\right]^{\beta} r^{3\beta} \qquad \text{Eq. (IV)}$$

This may be inserted into eq. (II), integrated from r_f to r, taking r_f as the radius of the grout front and performing the integration toward the source. Knowing that pressure P is given by ρh, we have:

$$P = \rho h = \frac{q}{4\pi k} - \frac{\mu_o}{r} + \frac{\alpha}{3\beta - 1}\left(\frac{4\pi n}{3q}\right)^{\beta} r^{3\beta - 1} \Big|_{r_f}^{r} + C \qquad \text{Eq. (V)}$$

The pressure at the grout front ($r = r_f$) is taken as zero for a dry soil, and may be easily computed for a saturated soil from conventional well pumping analyses. We restate r_f in terms of pumping time

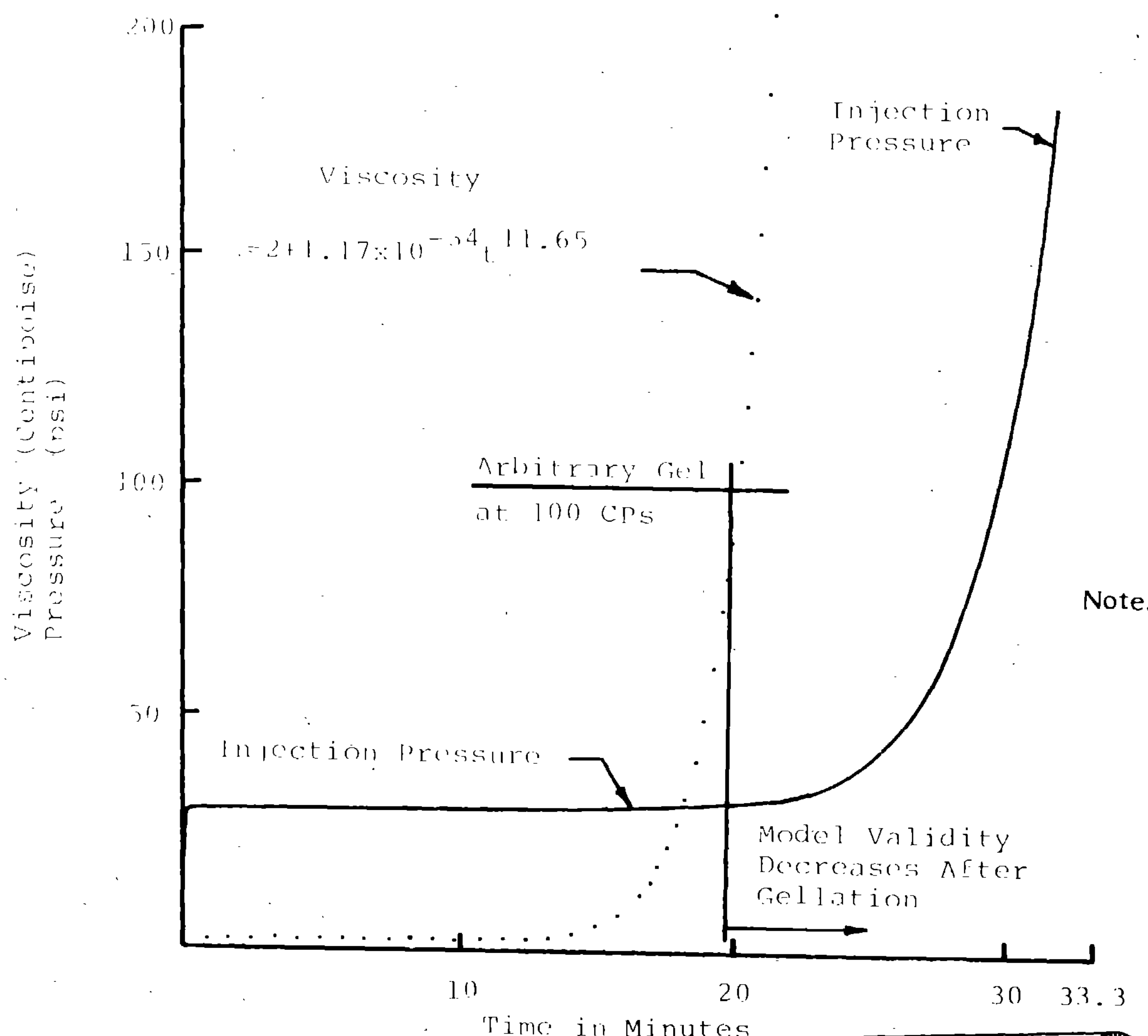
$$r_f = \left(\frac{3qt}{4\pi n}\right)^{1/3}$$

and evaluate eq. (V) yielding:

$$P = \left[\frac{8}{4\pi k} \quad \mu_o \left[\frac{4\pi n}{3qt}\right]^{-1/3} - \frac{1}{r} + \frac{\alpha}{3\beta - 1}\left(\frac{4\pi n}{3q}\right)^{\beta} - \left(\frac{3qt}{4\pi n}\right)^{\frac{3\beta - 1}{3}}\right] \qquad \text{Eq. (VI)}$$

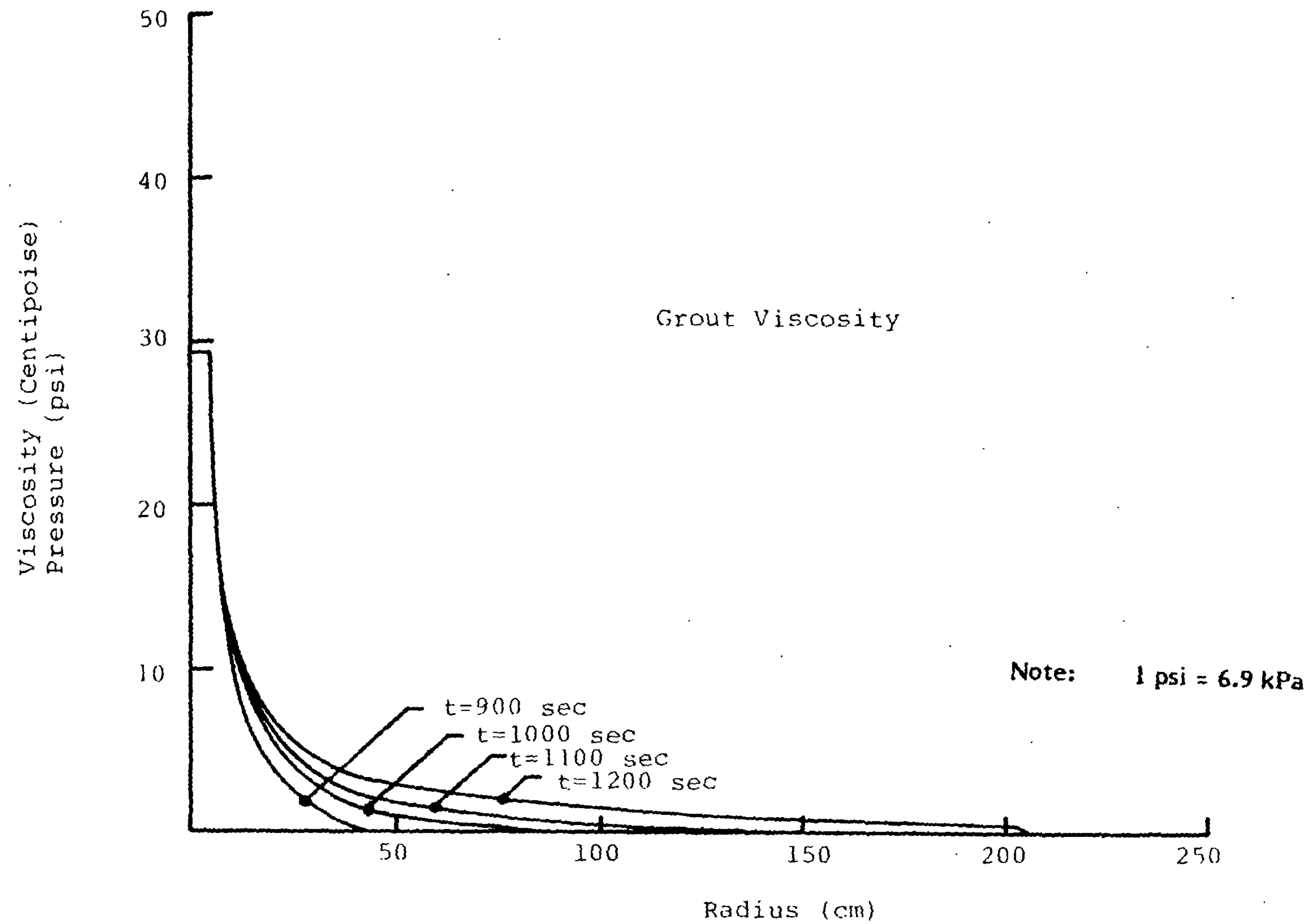
Using experimental values for a 40 percent sodium silicate structural grout, the model can be used to compute pressure at a radius of 3.8 cm (1.5 inches) to develop a predicted injection pressure time history. This is shown in Figure 24. This particular grout has a gel time of 20 minutes, as indicated on the figure. The model indicates that injection pressure begins increasing rapidly at 25 minutes. In the real world, the grout at the leading edge would be pushed aside by younger, less viscous grout in the process referred to as flow inversion. The model unrealistically assumes that the gelling grout will form a thin membrane around the advancing edge of the growing grout ball. The model predicts a rather gradual increase in injection pressure because it is believed that this skin of gelling grout is very thin, and being at a large radius, is advancing very slowly through the soil.

The distribution of pressure in the soil determines if the soil will become unstable under high neutral stress. The model prediction for injection times up to 25 minutes is shown in Figure 24. Recall that the assumption of constant flowrate allowed us to develop a function = f g(r) giving viscosity as a function of radius. This is shown in Figure 25 for gel times of 20 minutes (1200 sec) and less. The grout



Note: 1 psi = 6.9 kPa

Figure 24. Model Prediction of Injection Pressure History



Grout Viscosity
Viscosity (Centipoise)
Pressure (psi)
50
40
30
20
10
t=900 sec
t=1000 sec
t=1100 sec
t=1200 sec
50
100
150
200
250
Radius (cm)
Note: 1 psi = 6.9 kPa

pressure decreases with the inverse cube of radius as would be expected. At longer times, the skin of gelling grout tends to confine the still large pool of grout at elevated pressure over a spherical volume four meters in diameter. This is shown graphically in Figure 26, which shows the predicted pressure distribution for times between 20 (1200 sec) and 32 minutes (1900 sec). Note the change in the pressure scale. Very high pressure gradients are predicted in the outer regions where viscosity is high, with the result that a large pressurized region is predicted. This behavior would not be expected at a primary injection point due to the process of flow inversion, which prevents the formation of the confining membrane of gelling grout. On a secondary injection, however, the grout ball is confined by previously injected primary grout balls, and something like this may occur. The effect depends on the indigenous pore fluid, water or air, and the degree of confinement provided by the primary grout balls. At times less than one gel time, the injection pressure decays very rapidly as a radius increases, so that only a small volume of soil is subjected to high pore pressure. One normally would not be concerned if a spherical volume a few inches in diameter is subjected to high pore pressure.

The picture given by the model is very conservative for the grout port injection method. The grout must force its way under the grout sleeve and through cracks in the mortar jacket around the grout pipe, which contribute an unknown, but not insignificant pressure loss on the grout before it even enters the soil. The pressures shown in Figures 24 through 26 are pressures in the soil. Actual injection pressures measured in the grout pipe would be significantly higher.

General confirmation of this model within its limitations was obtained by pore-pressure measurements during the Laurens Street experimental program. Here, pore-pressure measurements were made during primary and secondary injections using the array of piezometer tips previously shown in Figures 18 and 19. The Laurens Street site had a complex geology of sand and clay layers and pockets, which, in combination with the changes continuously wrought by grouting activities, departed considerably from the isotropic homogeneous assumption of the model. A uniform inverse cube of radius pressure distribution was not seen. In two cases, the response at individual piezometer tips was rapid enough so that injection pressure could be plotted directly as a function of injection pressure in real time. These are shown in Figures 27 and 28. The slope of correlation coefficient gives the relative change in pore-pressure caused by the

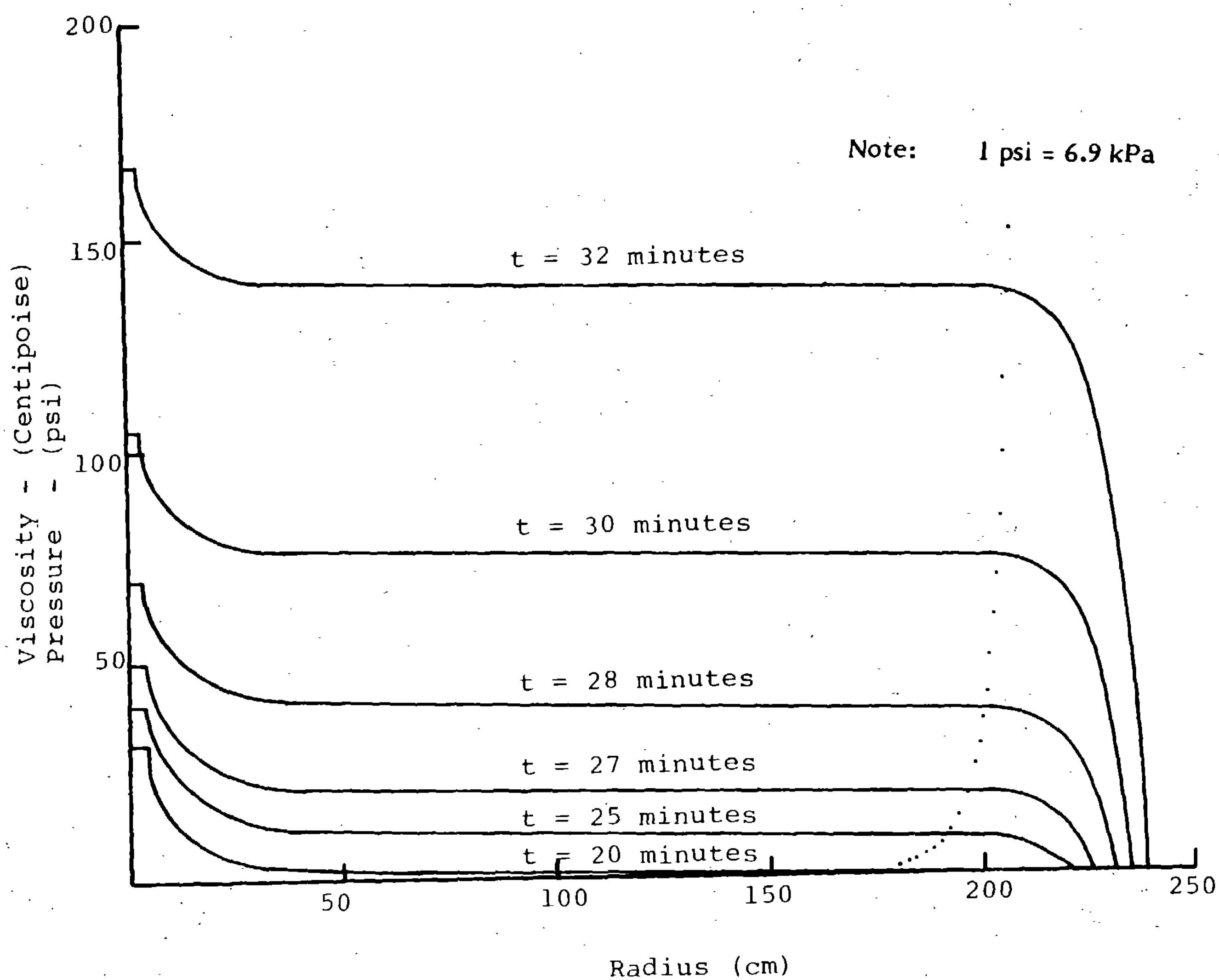


Figure 26. Model Prediction of Injection Pressure Distribution for Times Between 20 and 32 Minutes

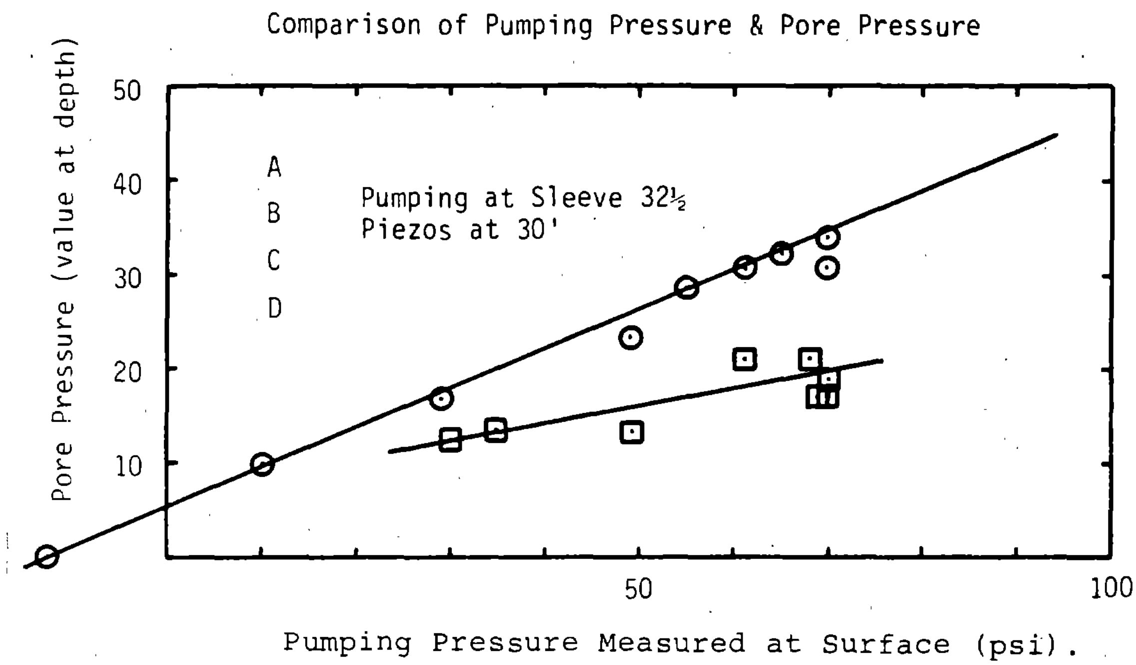


Figure 27. Pore Pressure Response 2 1/2 Feet Above Injection Point

Note: 1 psi = 6.9 kPa; 1' = 0.305 m

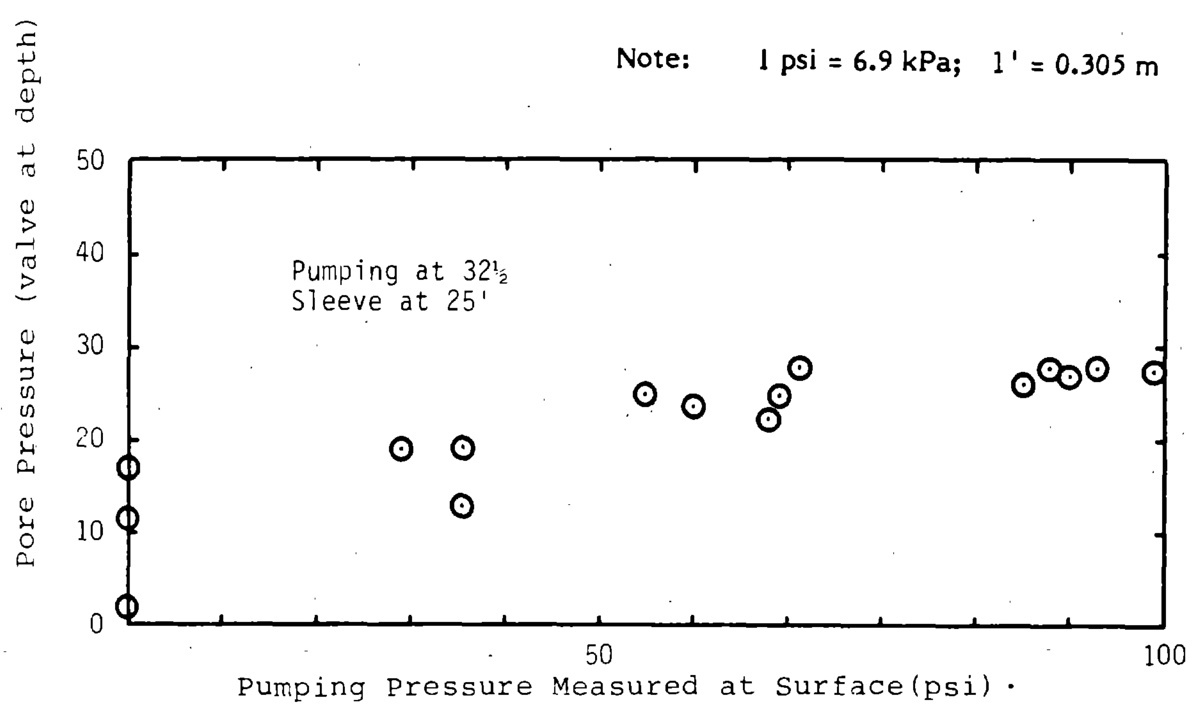


Figure 28. Pore Pressure Response 7 1/2 Feet Above Injection Point

injection pressure. The largest coefficient measured was 0.45. In other cases, an increase in pore-pressure was seen, but the delay was so long that a plot of pore-pressure versus injection pressure was not feasible. In these cases, the approximate increase in pore-pressure was compared to the injection pressure to obtain a ratio. In many cases, no increase in pore-pressure was seen. While some of these data were undoubtedly realistic, they were dropped from consideration, due to inconsistency.

The theoretical model, together with grouting experience and experimental measurements, suggest that the following items should be considered in planning an injection program.

a. Grout pore pressures may exceed in-situ effective stresses and cause soil yielding in a small region near the injection point in a reasonably homogeneous medium. When using the sleeve pipe system, large pressure drops would be expected across the grout port, rubber sleeve and mortar sleeve. In stage pipe grouting, the borehole is exposed directly to a full injection pressure. Normally, a quick condition within a small volume near the injection port would be of little concern.

b. In a medium that has been made non-permeable, as by prior grouting, or in a porous medium which is confined by non-permeable boundaries, grout pressure may build up over large areas with attendant risk of ground motion.

The theoretical model is more useful in understanding the phenomenon than in application to particular grout injection cases. The injection process in the field is typically dominated by variations and uncertainties in soil permeability. Because such a general understanding of the process is important, the further development of a model which incorporates the flow inversion process should be considered to extend the useful range of application to times greater than the gel time. The average increase in pore-pressure over all the cases was 13 percent of the applied injection pressure. In only one quarter of the cases was an increase in pore-pressure as large as 20 percent of the injection pressure measured. As can be seen, and as is predicted by the model within its range of validity, the pressures experienced in the soil are significantly less than the injection pressure. This finding was confirmed independently by the hydraulic fracturing data, which consistently required injection pressures much higher than the theoretical to induce soil distress.

GEOPHYSICAL GROUT EVALUATION SYSTEMS

The geophysical systems evaluated in this contract were electrical resistivity, acoustic velocity, and earth probing radar. To achieve maximum resolution, all geophysical systems were used in borehole or in-situ configurations. Attempts to use surface instruments in Europe have met with indifferent success, which was confirmed by the poor resolution in the surface measurements for this study. The borehole instrumentation, operating near or within the target grout mass provides much improved resolution and simple interpretation. With the abundance of grout holes to be found on a grouting project, the cost of using borehole instrumentation is trivial.

Electrical Resistivity

Chemical grout is highly ionic and an excellent conductor. Because previous trials reported by Goldberg/Zoino had shown that direct current (DC) methods are defeated by electrode coating and nearfield saturation, alternating current (AC) resistivity was used on this project. The initial system was a constant AC current multiple frequency instrument that would step through ten frequencies between one Hertz and one kHertz. At each frequency, both the voltage and phase lag were measured. It was anticipated that as the grout gelled, the gradually lengthening polymers would cause a phase shift at the higher frequencies, which could be used to diagnose the grout setting. This was not found to be the case, for the major effects were found in the amplitude signal, and little departure from zero phase angle was found. This resulted in the use of a much simpler constant frequency system when the project moved into the field tests.

The response seen in the meter-cubed laboratory tests, using the multiple frequency system, was typically that seen in Figure 29. When pumping began, the resistivity increased initially, and then decreased as grout began to infiltrate the volume between the sensor probes. Because of the constant current source, resistivity changes elsewhere in the system have no effect on the voltage registered by the sensor

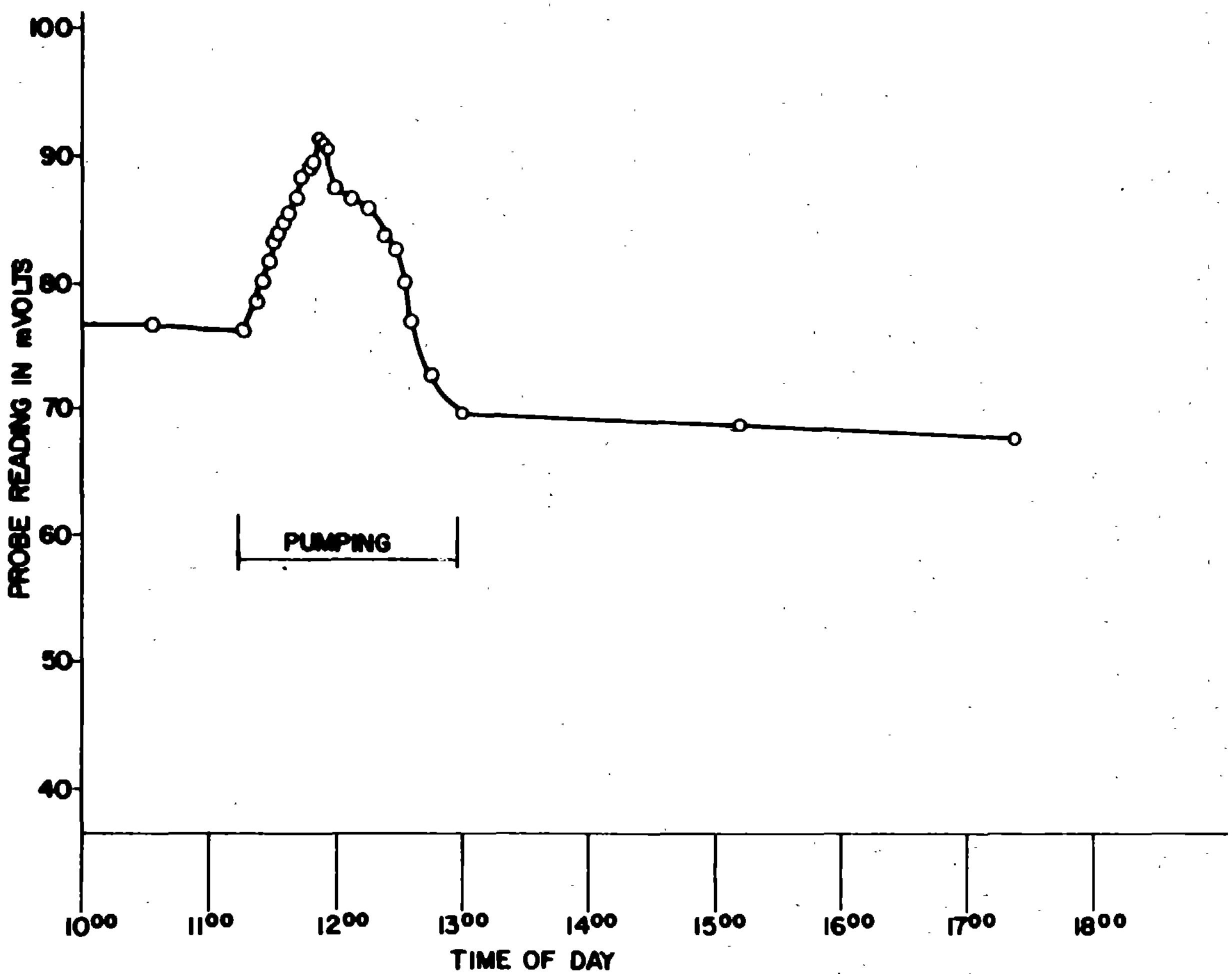


Figure 29. Resistivity Reading in Laboratory Test 8

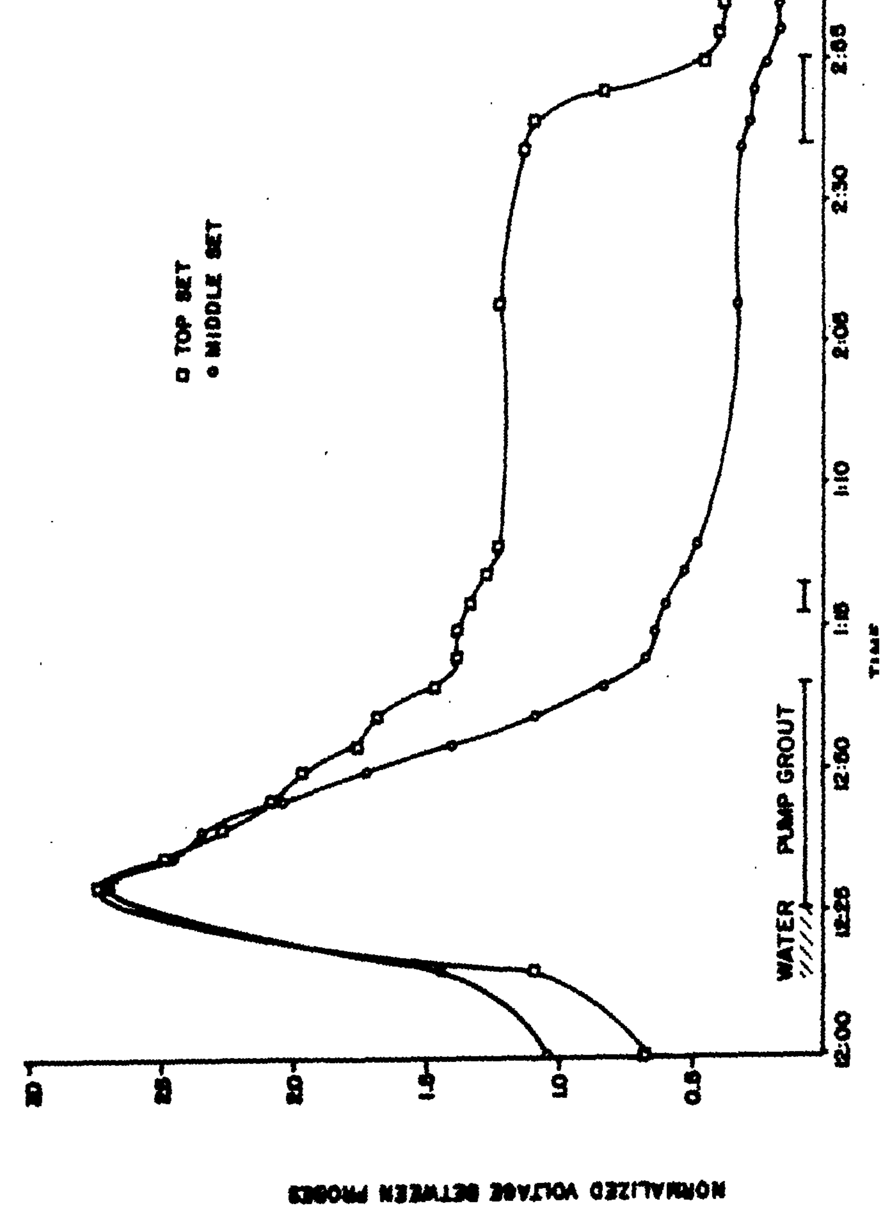
probes. The resistivity did not change when gellation occurred, but remained at the low value obtained by the end of pumping.

Resistivity data were taken at the field test site using a 200 Hz constant current source during primary hole injections. Data for primary hole injections, shown in Figures 30 to 33, followed the same pattern of initially increasing and then decreasing resistivity. Because the resistivities were near zero by the end of primary injection, little change was seen during secondary injection. Finally, resistivities were also measured at the Laurens Street site in Baltimore. Data over the four-day test period are shown in Figure 34. These data did not display the initial increase in resistivity, possibly because of low injection pressures or because a small amount of grout had invaded the test volume prior to placing the instrumentation. The resistivities measured at Laurens Street were very low, which accounts for the large amount of scatter seen in Figure 34. Even though the work was done near the lower range of the instrument, a continuous decrease in voltage was seen throughout the working hours at this site.

Electrical resistivity is a very sensitive grout detection system. It requires the added expense of in-situ probes, and the ultimate use will probably be to delineate the boundaries of the intended grout zone in cases when grout cannot be permitted to migrate outside the intended location. Automatic alarms for this purpose could easily be set so that the grout technician would not be required to monitor the system constantly. Any of the AC resistivity systems now on the market would be suitable for use in grout monitoring. Although a four-pole electrode array was used, greater experience with the use of resistivity may permit the use of fewer electrodes. If only one grout pipe is injected at a time, the active grout port would make an excellent field electrode. Dissimilar voltages could not be impressed on grout pipes that were connected by grout in the pumping system, so that all active grout ports would tend to carry the same signal.

Acoustic Velocity

Acoustic velocity was measured in the meter-cubed tests and the pilot field tests. The meter-cubed system employed a source that generated both distortional (s, or shear) waves and dilatational (p, or compressional) waves. A two-channel receiver



TOP SET
MIDDLE SET
NORMALIZED VOLTAGE BETWEEN PROBES
3.0
2.5
2.0
1.5
1.0
0.5
12:00
12:25
12:50
1:15
1:10
2:05
2:30
2:55
WATER
PUMP GROUT
TIME

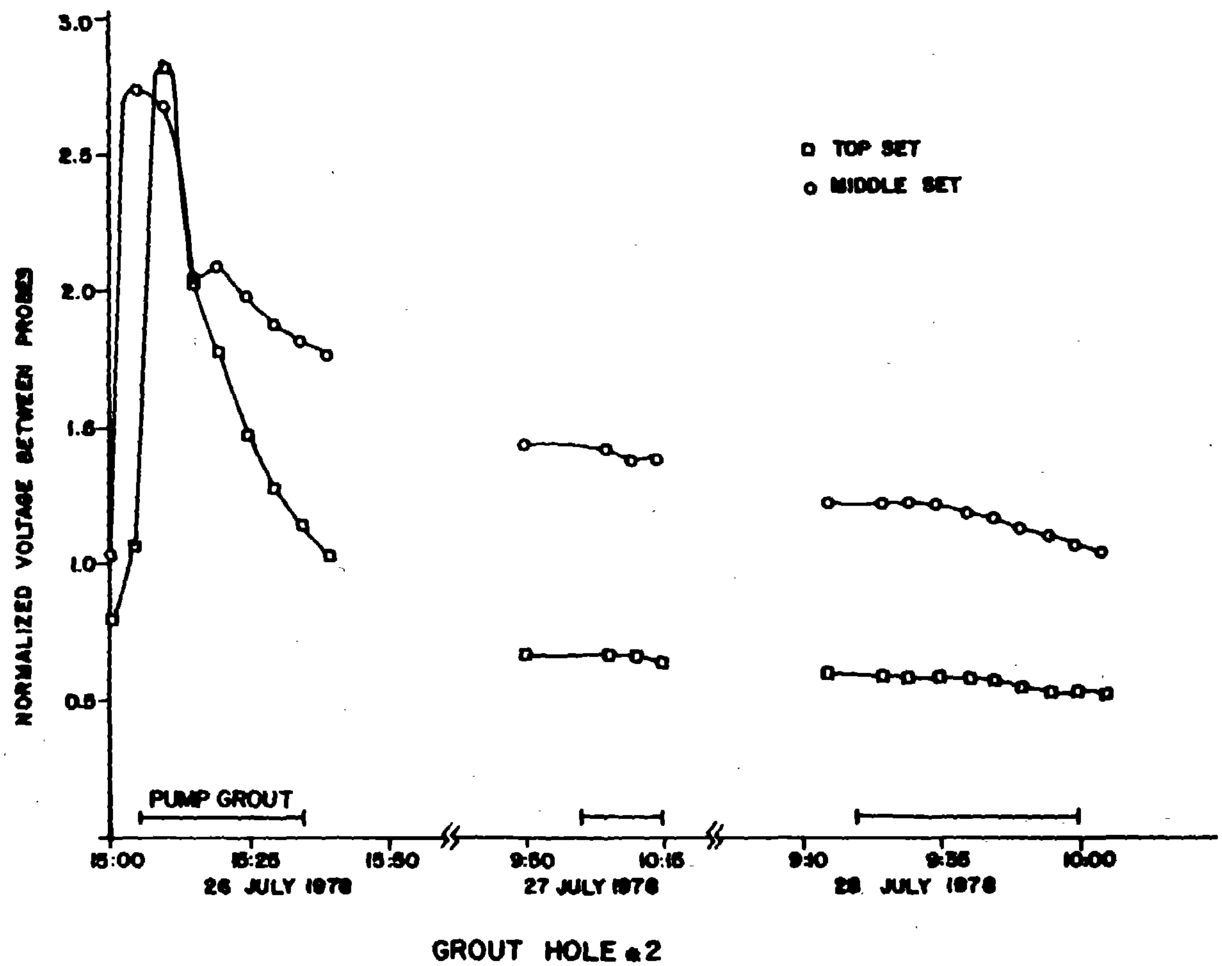


Figure 31. Grout Hole # 2

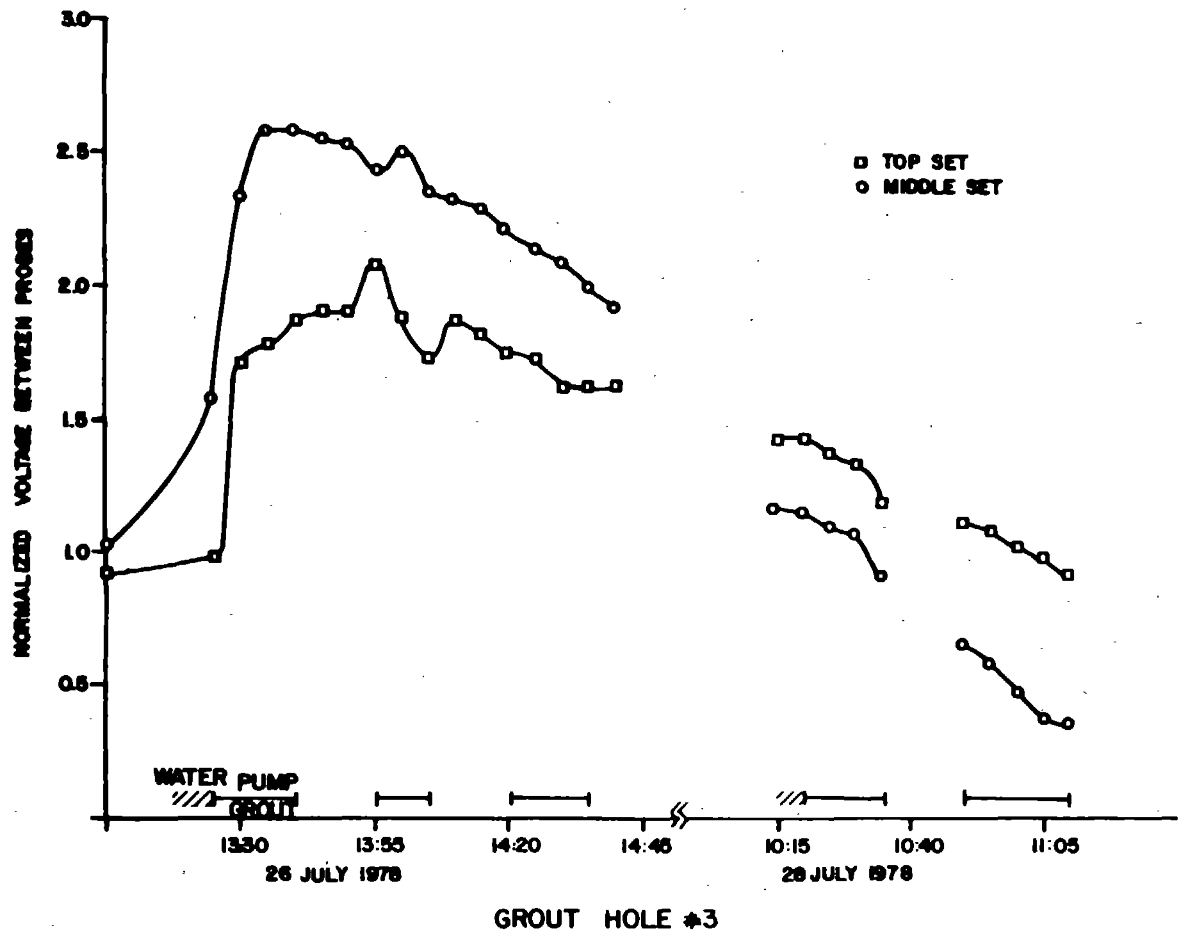


Figure 32. Grout Hole # 3

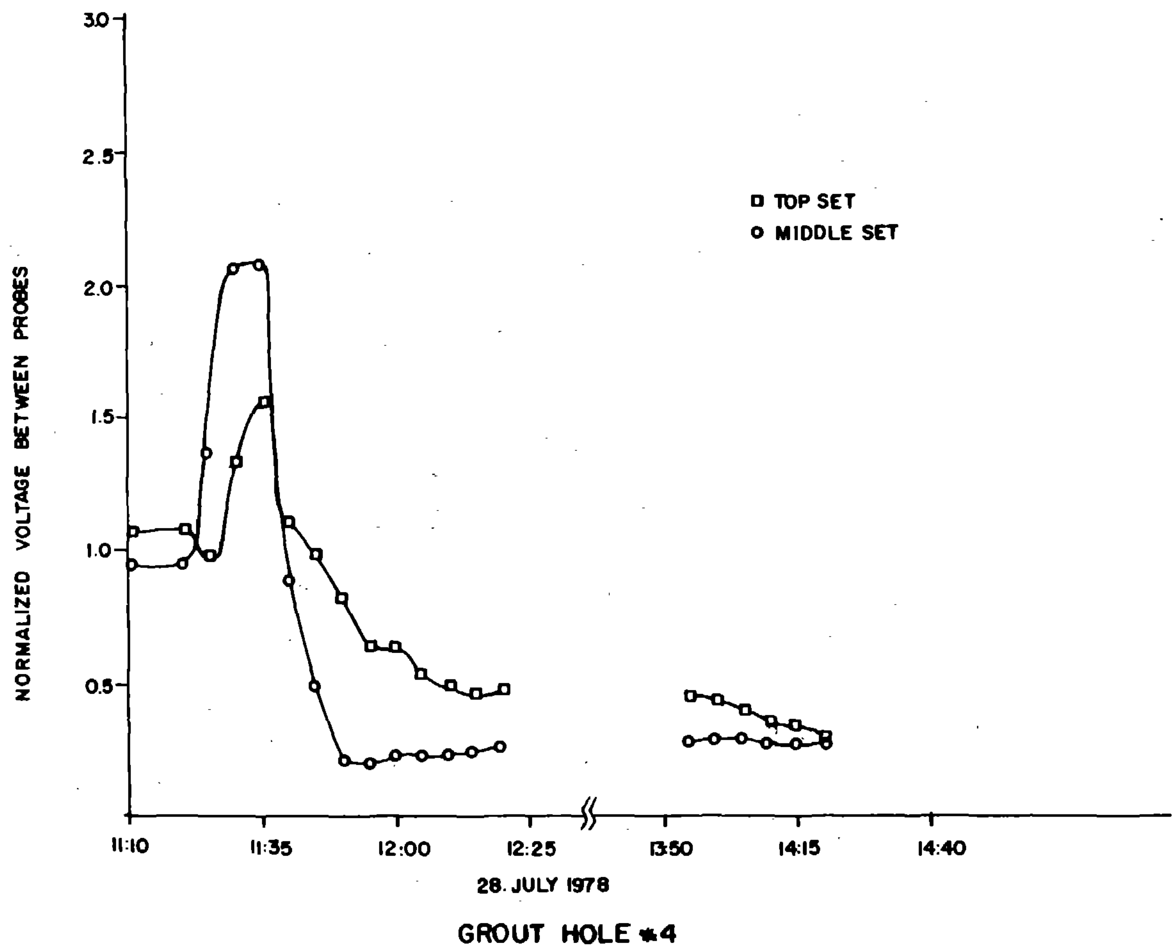


Figure 33. Grout Hole # 4

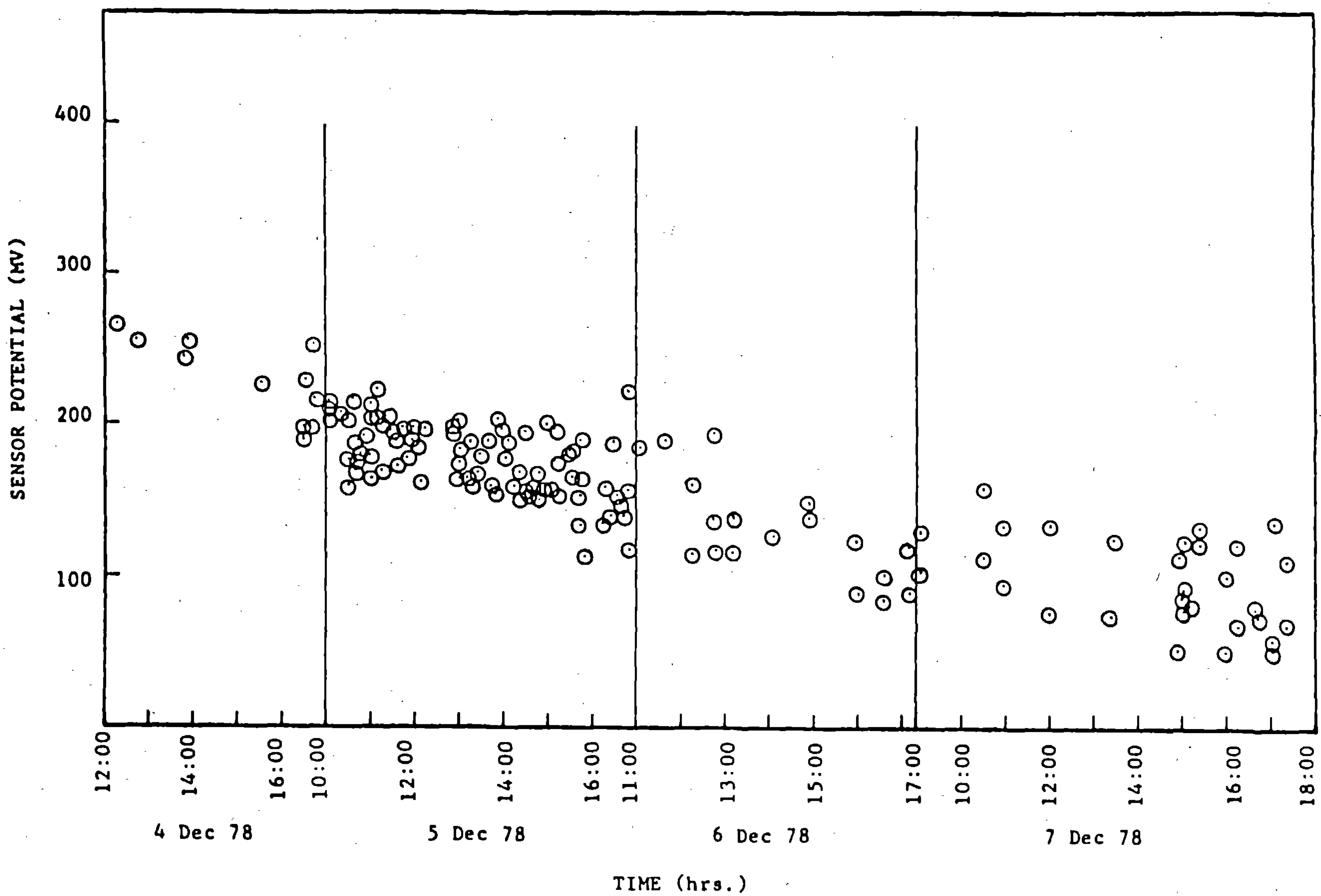


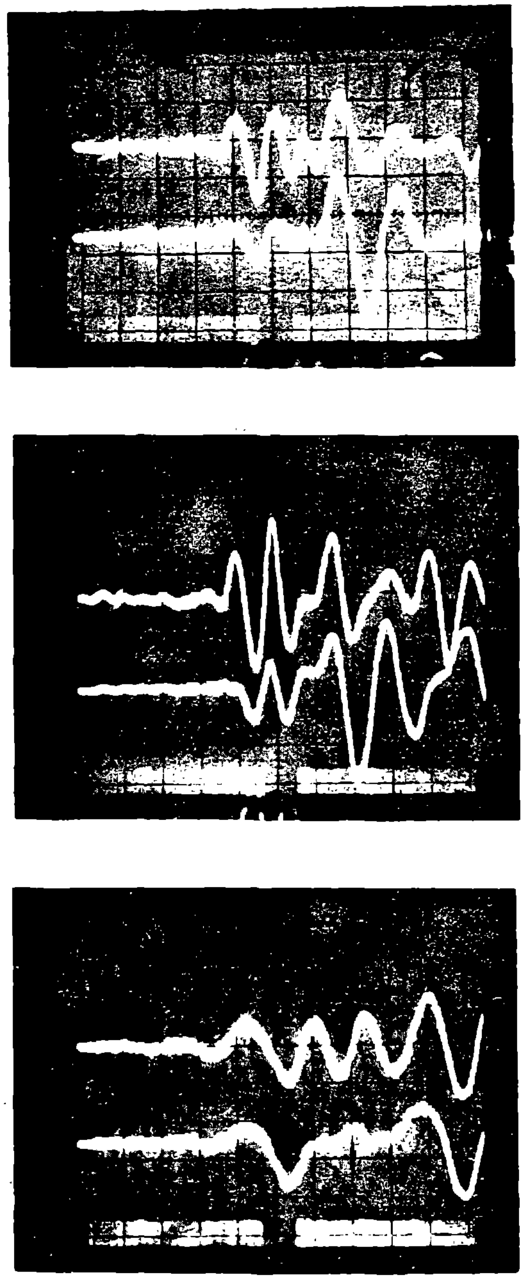
Figure 34. Resistivity History - Sensor Pair 64-74 (Depth 40 ft)

was used to separate the two components. Figure 35 shows the traces from the tests, both before and after grouting. Immediately after grouting, the signal is attentuated or weakened and the velocity is little changed. The velocity is increased significantly after the grout is set. The initiation of the shear arrivals in each case is not easy to identify, but it can be seen that it changes about the same as the P-wave velocity. Both velocities increase by a factor of about 2 to 1 in this particular case.

Table 2 shows a table of average P-wave velocities before and after grouting for the various meter-cubed tests. With the exception of test number 8, the P-wave velocity shows a very diagnostic change after grout gellation. It is also possible to note that the higher average velocities also occur for the larger grout balls and the slower final velocities occur for the smaller and the irregular balls. This indicates the P-wave velocity is diagnostic of grout. This is emphasized by plotting the relative increase in p velocity against the silicate percentage for those tests using structural grouts. As shown in Figure 36, the grouts containing higher concentrations of sodium silicate, that is, the stronger grouts, also display greater changes in p velocity. Two of the tests employed multiple injections, from which very irregular grout balls resulted. The determination of velocity in these two cases is suspect.

Acoustic tests for the pilot field tests were conducted in cross-hole mode. A 100 Joule sparker was located in one hole and a receiver was placed at the same level in another hole. Tests were conducted at various levels in each pair of holes to produce an acoustic velocity profile. The PVC grout pipes and instrumentation holes were filled with water to provide acoustic coupling. Coupling and multiple path transmission made it very difficult to measure the velocity in the intended grout zone before grouting. The acoustic signal traveled down the water filled boreholes and across the high velocity clay below the grout zone faster than directly across from pipe to pipe. The best estimate for the pregrouting velocity in the intended grout zone is 200 m/sec. (600 ft/sec).

The final series of tests were run several days after the last grout hole was pumped. The acoustic tests for this series were conducted in the same fashion as those run before the grout injection. Figure 37 shows the crosshole signals between Holes G2 and G5. After grouting, the acoustic signals were strong and had a velocity of about 6,000 ft/sec (2 km/sec). Another indication of the types of changes the grout



P

BEFORE PUMPING

S

TIME=2ms/div

P

AFTER PUMPING

S

TIME=2ms/div

P

AFTER GROUT SETS

S

TIME=1ms/div

Figure 35. Acoustic Signals from Test 5

Table 2. Average Velocity for P-waves in Test Box

TEST	% SILICATE	v_b BEFORE GROUT FT/SEC (m/SEC)	v_a AFTER GROUT FT/SEC (m/SEC)	v_a / v_b	DIAMETER OF GROUT BALL FT (m)
1	40 (1)	----	----	------	2.5 (.761)
2	50 (1)	630 (192)	1583 (482)	2.51	3.0 (.914)
3	60 (1)	704 (214)	1056 (322)	1.50	Irregular
4	40 (1)	609 (186)	1267 (386)	2.08	3.3 (1.01)
5	50 (1)	487 (148)	1091 (332)	2.24	2.5 (.761)
6	60 (1)	527 (161)	1667 (508)	3.16	3.3 (1.01)
7	20 (2)	609 (186)	792 (241)	1.30	Broken
8	40 (2)	528 (161)	528 (161)	1.00	2.0 (.609)
9	60 (1)	608 (185)	1056 (322)	1.74	Irregular

(1) Geloc 3 Used As A Silicate

(2) Sodium Bicarbonate Used As A Silicate

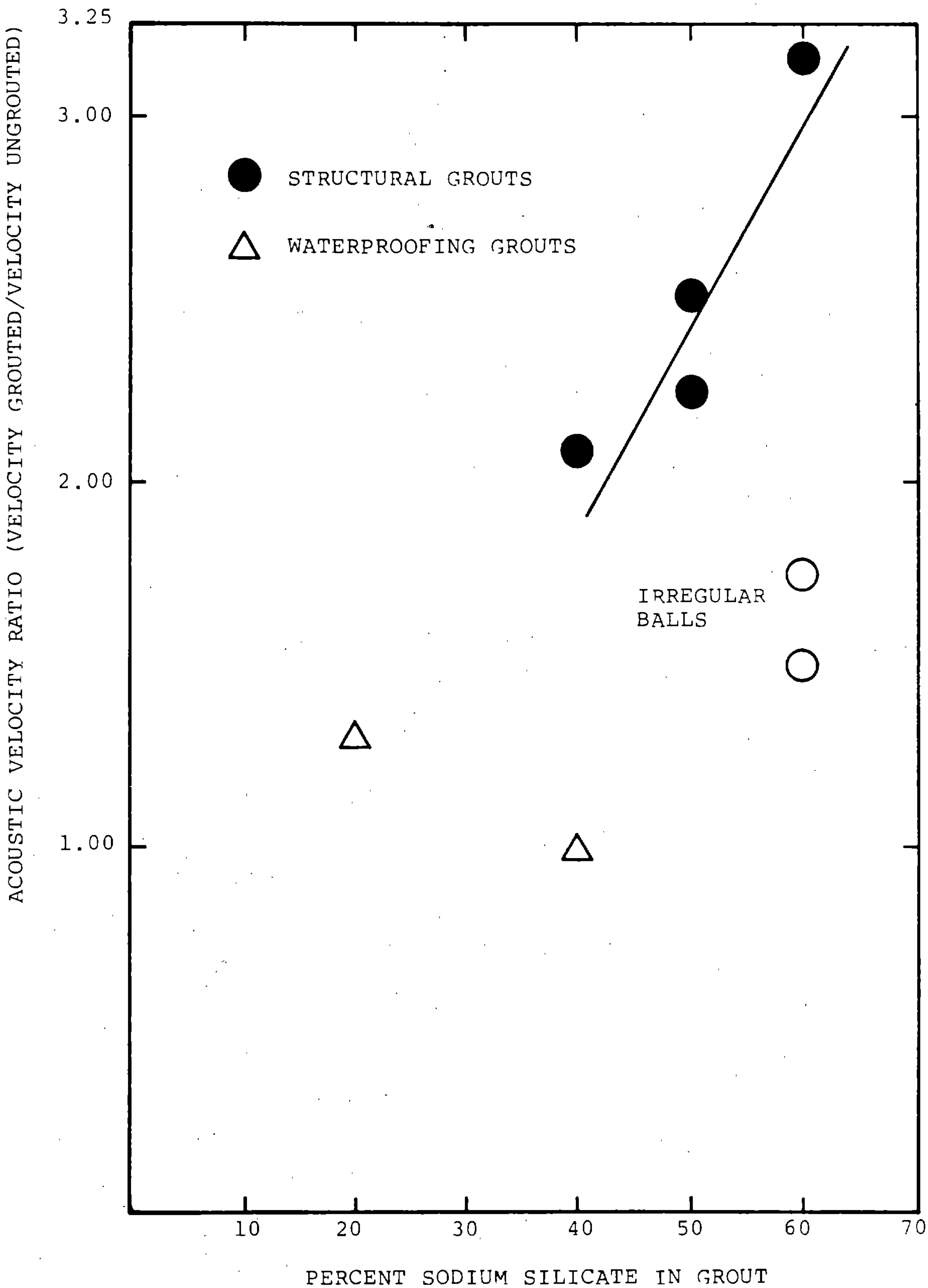
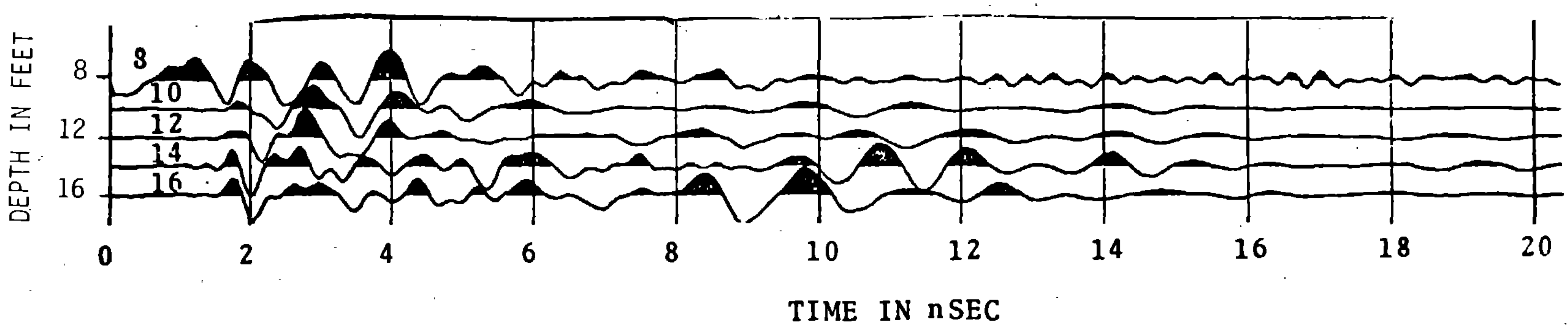


Figure 36. Increase in Acoustic Velocity vs. Silicate Percentage



1 ft = .305 m

Figure 37. Acoustic Crosshole Data - G2 to G5 - After Grouting

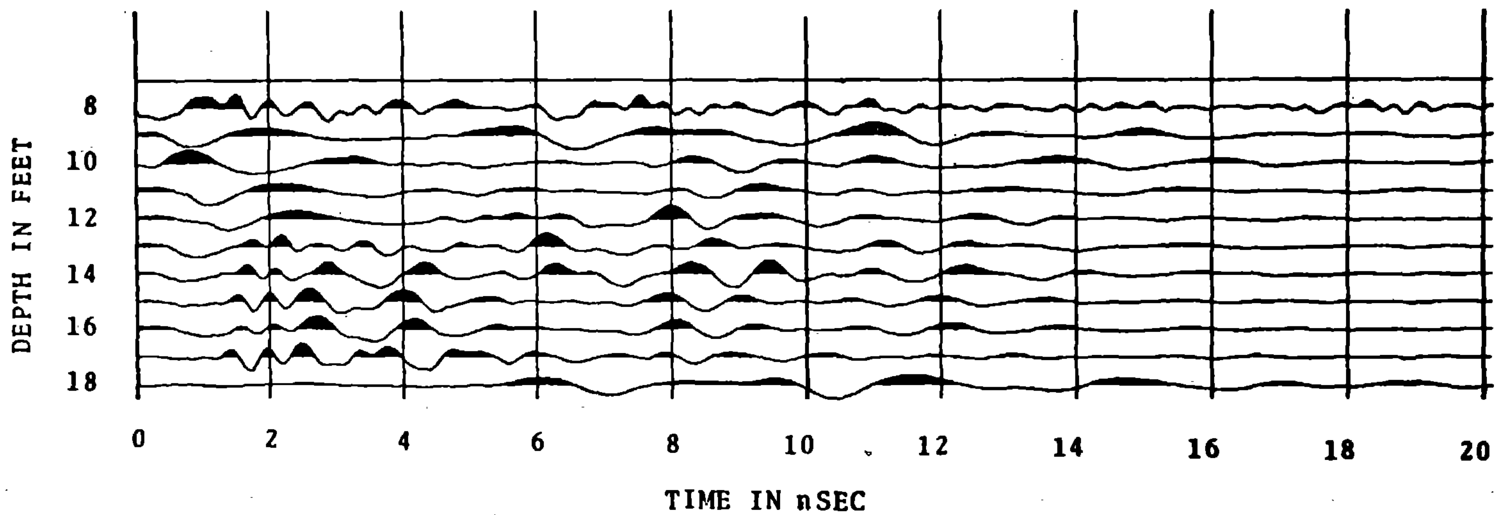
induces is shown in Figure 38 and 39. Figure 38 shows the results of firing the source at the bottom of Hole D3 with the receiver uphole in G3. Note the high frequency content of the arrivals. Figure 39 shows the same sort of test between Holes G3 and D4. Note the slow first arrival and the low frequency content. D4, an instrumentation hole, was the only hole that was not encased with grout. This lack of grout is shown very well in the acoustic data. Spectrum analyses were conducted on signals before and after grouting, as shown in Figure 40. The post grout spectrum extends to higher frequencies, indicating a more competent material.

If groundwater is present, the before-grouting survey may be confounded by the high velocity water P-wave. Shear wave velocity measurements are not subject to this distortion, and have recently been used for P-wave measurements for quality control of chemical grouting on the Pittsburgh Subway (22). Several acoustic velocity systems, appropriate for use in grout monitoring, are commercially available. The critical point is that the transducers should go down-hole. With grout pipes available before and after grouting, little expense is involved in making a cross-hole survey. Thus, this system provides an excellent mode for determining the condition of the grouted zone before and after grouting. Changes that result from the grout are easily seen, and most civil engineers can judge almost intuitively the character of a soil by the acoustic velocity. In the case of these tests, the after grouting velocity of 2 kM/sec are indicative of sandstone rather than sand, an unmistakable indicator that the grout is between the pipes surveyed, and of good structural quality. Because the acoustic signal will follow high velocity paths around corners, the cross-hole method does not give a precise indication of the grout location.

Borehole Radar

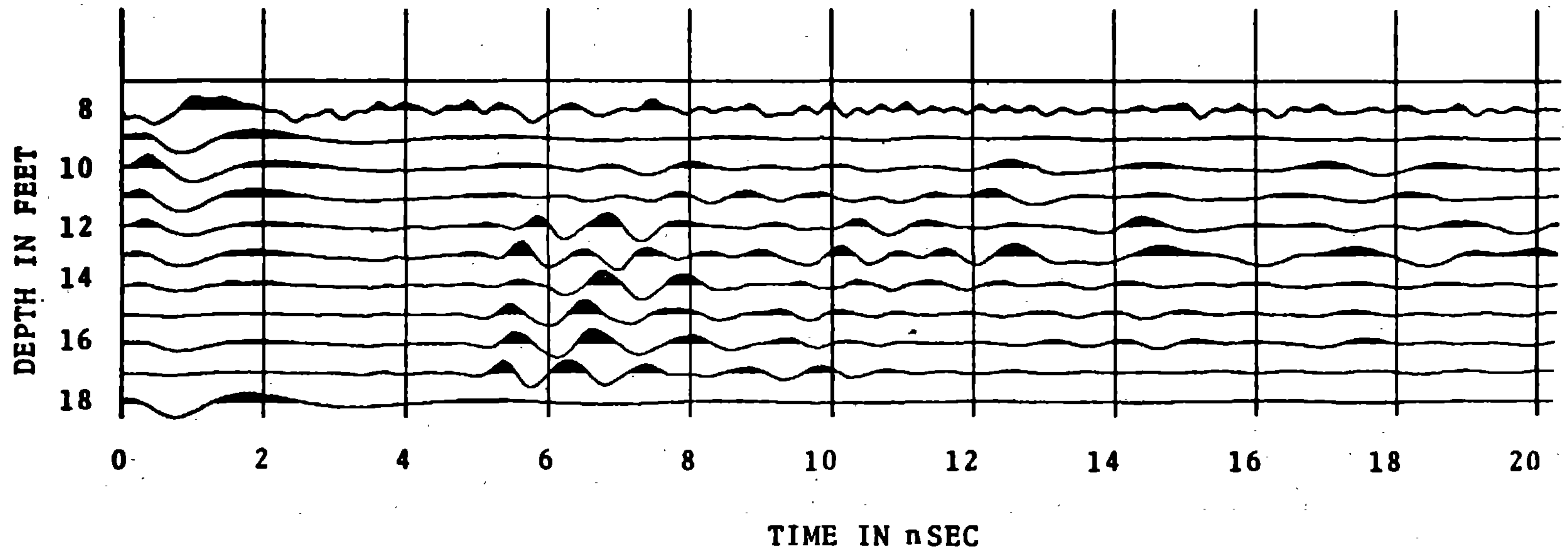
Borehole radar was used in both pulse and continuous wave (CW, also referred to as microwave) configurations. The laboratory meter-cubed tests were conducted only to the extent of showing that the radar could see through the specimen container prior to grout injection, but not after. The grout severely attentuates the radar signal, but is not itself a good reflector.

Pulse radar may be used in the transillumination or the transmit/receive (T/R) mode. Transillumination requires a separate transmitter and receiver which operate



1 ft = .305 m

Figure 38. Acoustic Crosshole Data - D3 to G3



1 ft = .305 m

Figure 39. Acoustic Crosshole Data D4 to G3

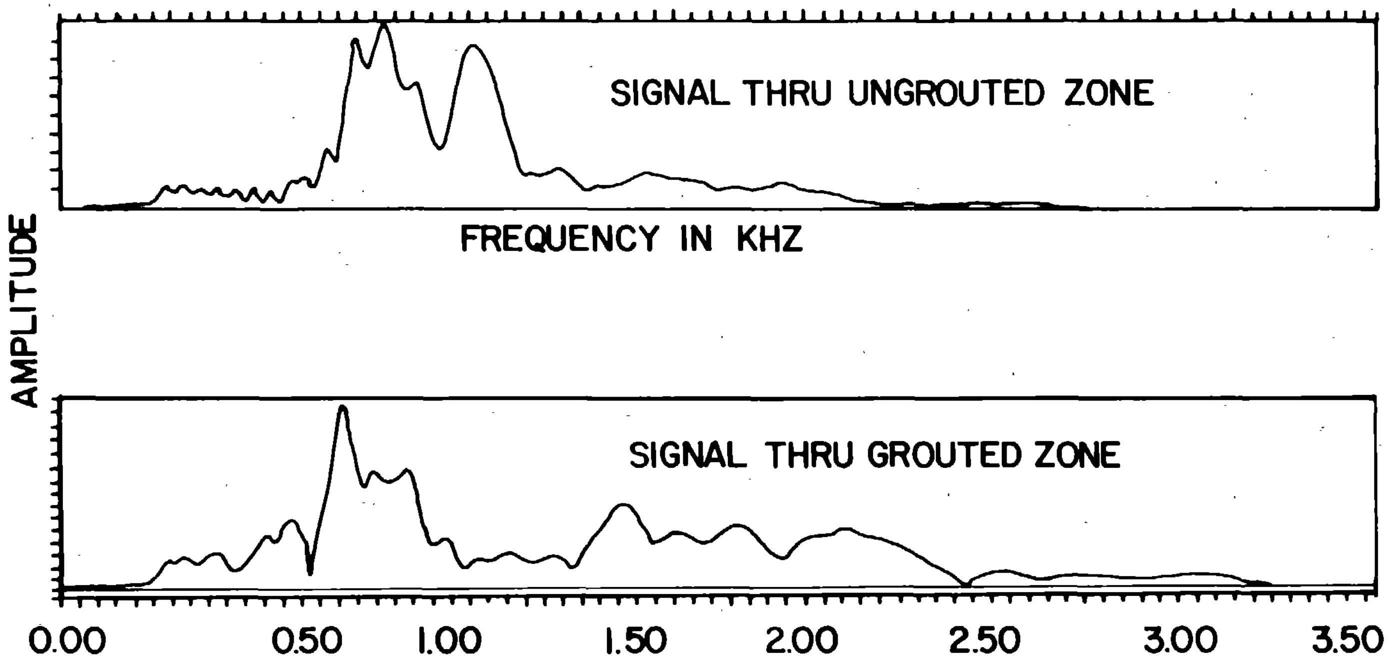
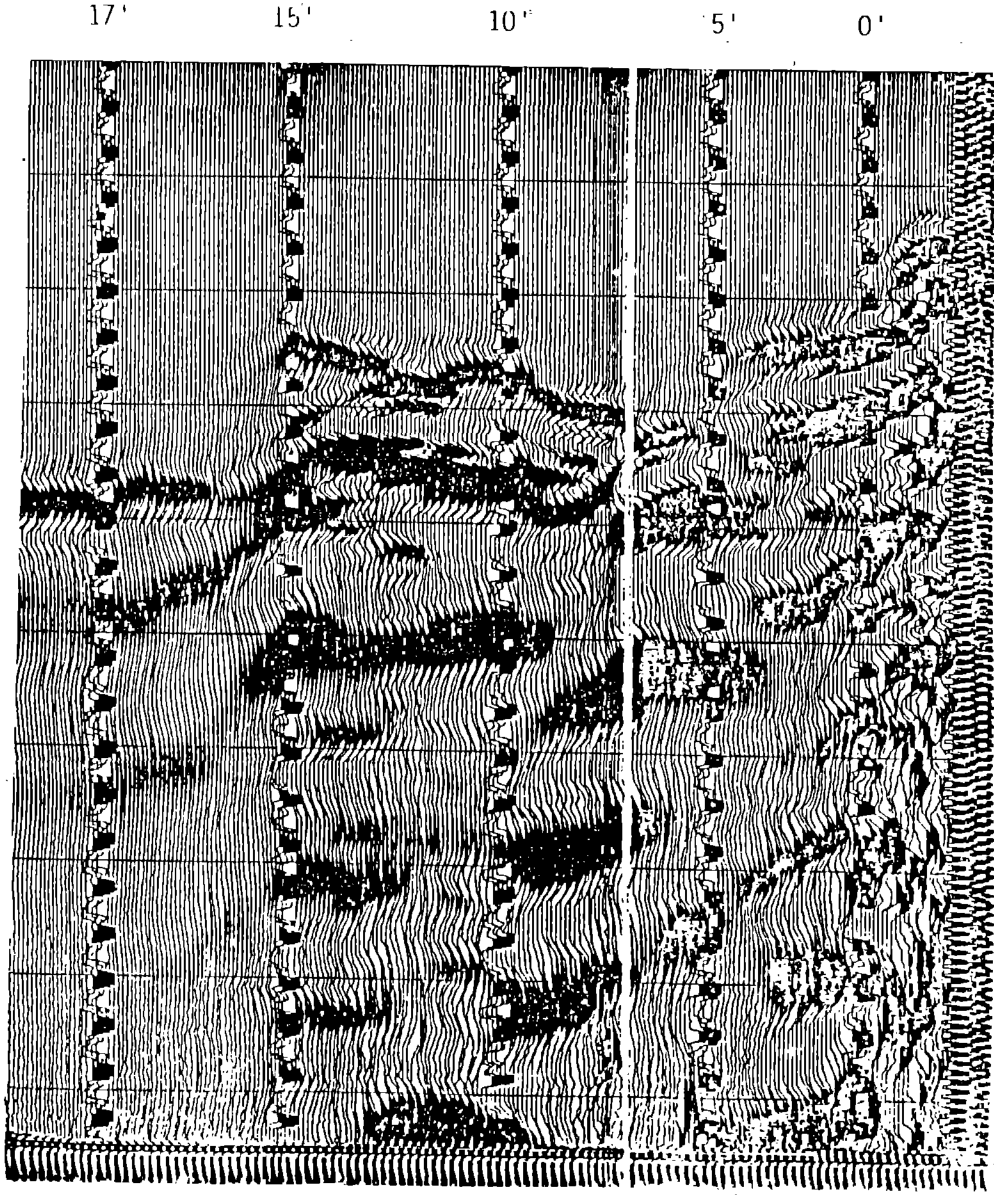


Figure 40. Amplitude Spectra Before and After Grouting

in two boreholes much like crosshole acoustic shooting. By placing both instruments at the bottom of adjacent boreholes, and gradually raising them at the same rate, so that the signal path between them is horizontal, a transillumination borehole survey is obtained. The T/R mode uses a single instrument with both transmit and receive capabilities. A pulse is emitted, and the reflected signal that returns to the instrument is detected. T/R mode may be used in either borehole or surface instrument configurations. Because of the poor reflective qualities of chemical grout, this project concentrated on transillumination.

The pulse radar with GSSI equipment was used in mapping the pilot field test site before grouting. A series of borehole transillumination profiles were run. The two antennas were then raised up the hole. Figure 41 shows the signal received between holes G5 and G1 before grouting. The slow arrival below 5 m (16 feet) is through the deeper clayey layer, then the dip in the first arrival at 1.5 m (5 feet) is due to the small clay layer at that level. The transillumination survey was repeated after grouting. Figure 42 shows the run between the same holes after grouting. The signal is completely attentuated in the grout zone, in contrast to the detail that appears in Figure 42. Note also the abrupt cut-off at the lower boundary, and the more gradual transition at the upper boundary. Excavation after grouting indeed showed that the lower surface of the grouted zone was a sharp flat surface, but the upper surface was irregular, with grout having intruded into the upper sand layer in several locations. The pulse radar was successful in identifying this unexpected condition.

The continuous wave (CW) radar displayed the sharp lower boundary and gradual upper boundary in the first look data. In addition to horizontal transillumination, the CW system was operated across all possible paths between points in the boreholes, as shown schematically in Figure 43. This provides the data necessary for tomography, similar to the much publicized C.A.T. (computer-aided tomography) scan X-ray systems used in modern medical facilities. A grid is established over the plane defined by the two grout holes, and a transmissivity value assigned to each grid element based upon the strength of those signals which intersected in the element. This first trial array of values is then relaxed to a best-fit solution by successive iterations. The resulting map, displayed by color values on a CRT provides an image of the plan in question. Typical output is shown in Figure 44. The tomographic technique has both advantages and disadvantages. Used from vertical boreholes, it resolves horizontally



Note: 1 ft = 0.305 m

Figure 41. Radar Transillumination Before Grouting

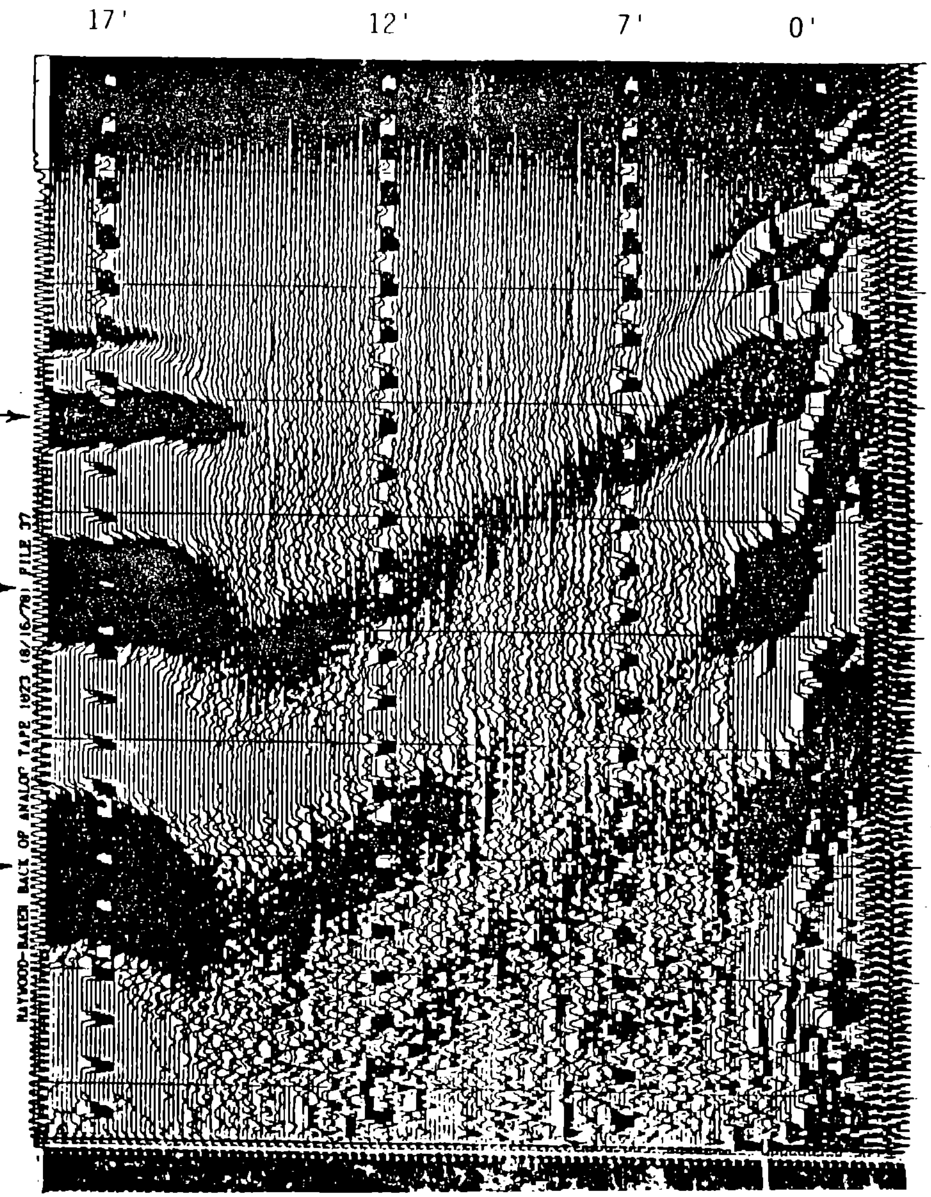


Figure 42. Radar Transillumination After Grouting

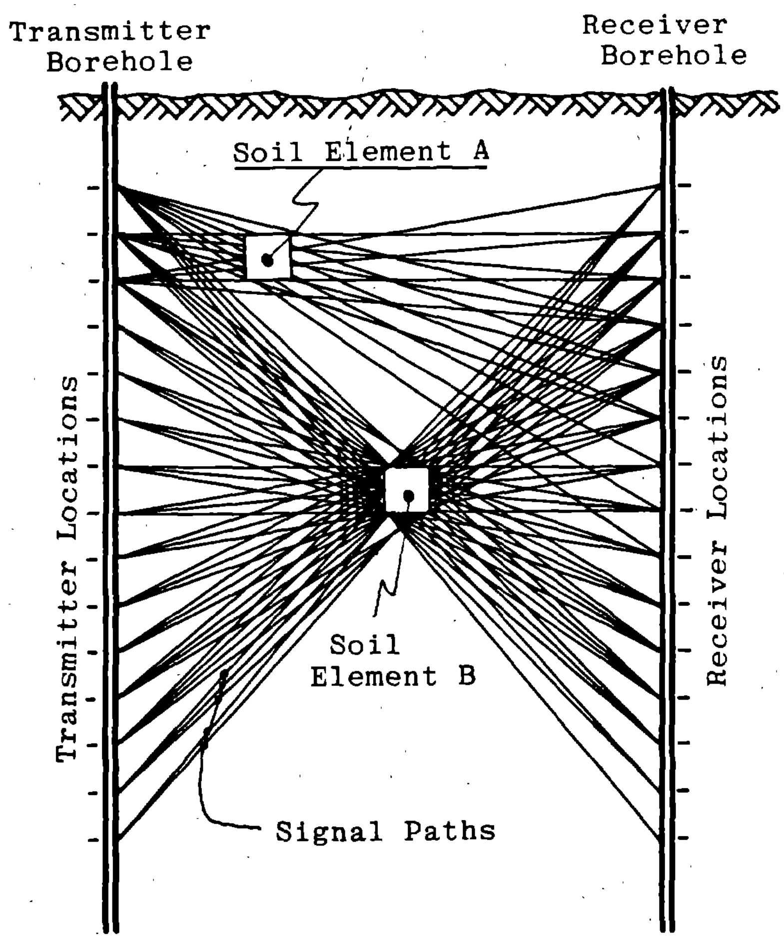


Figure 43. Radar Geotomography Layout Showing Signal Paths Through Two Typical Soil Elements

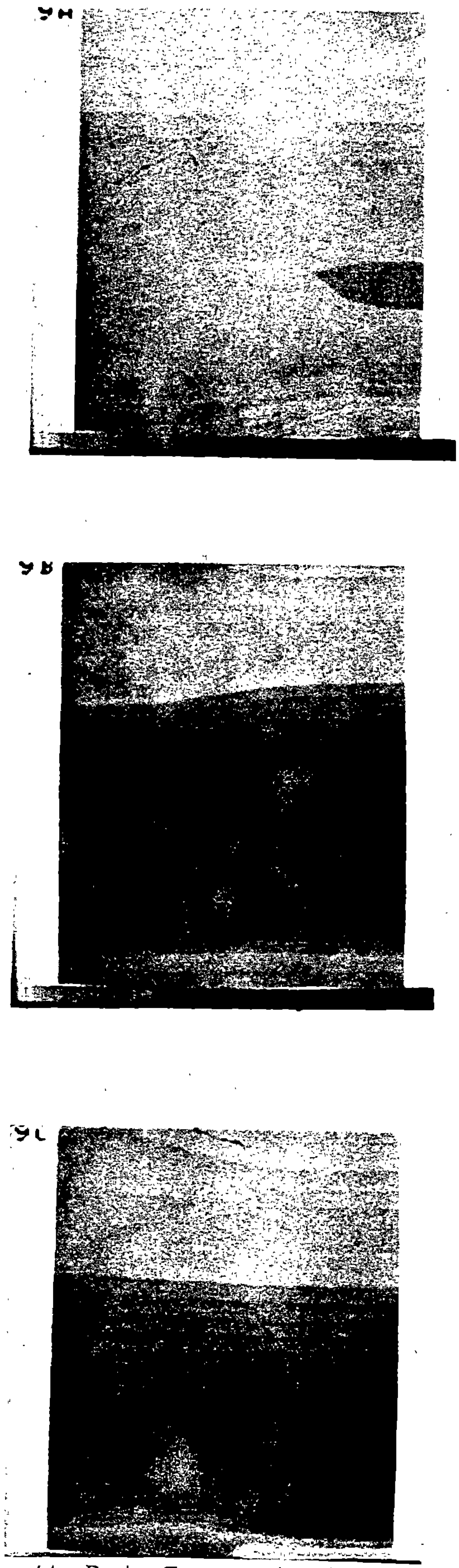


Figure 44. Radar Tomography Photograph

layered media poorly, because high angle near vertical paths cannot be obtained. An enormous advantage lies in the dramatic presentation mode, which gives the uninitiated the impression of an actual underground image.

PROCEDURAL CONTROLS

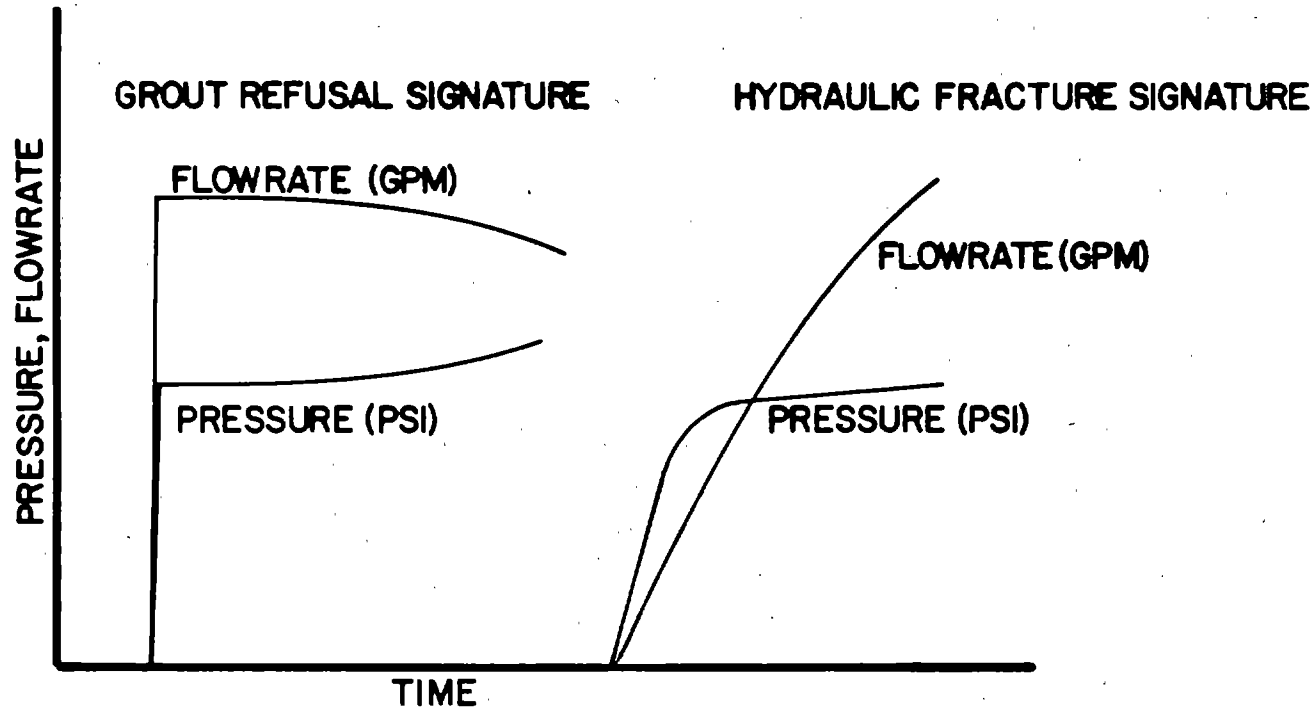
The grouting procedure itself may be modified and monitored to improve confidence in the final product. Controls that improve the grouting process include:

- Continuous monitoring of pressure and flowrate
- Acoustic emission monitoring to control injection pressure
- Gel time control
- Improved data collection/reporting methods and hardware
- Grout take logging
- Injection procedures to limit surface heave or undesirable deflections in-situ
- Use of rational specifications as opposed to rule of thumb and empirical rules.

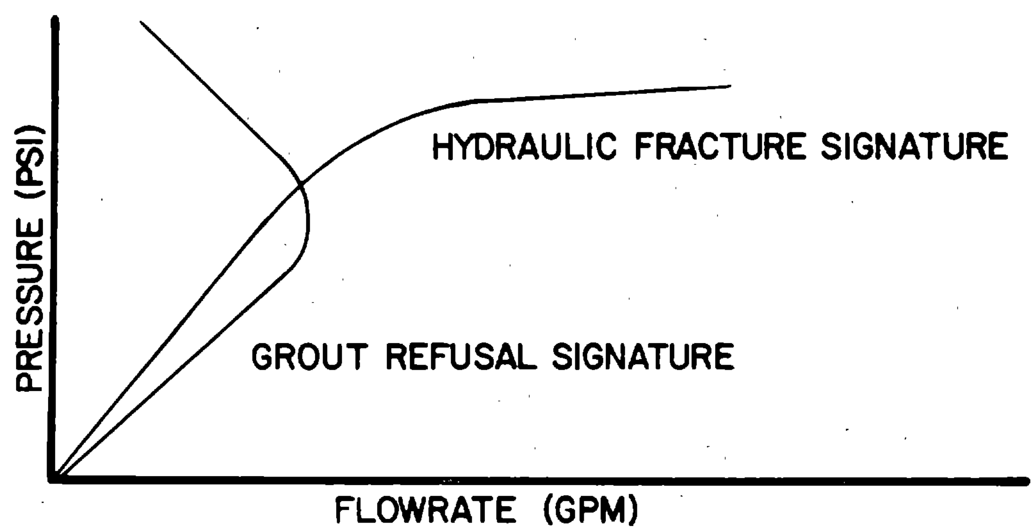
Because of the concern with both quality control and quality assurance, it is emphasized that the most perfect control over the grouting operation will not enhance the confidence that can be placed in the product unless it can be proven that the desired objectives have been met. Documentation of tests and evaluation data in an easily understood format is critical to quality assurance.

Injection Pressure and Flowrate

The effectiveness of the injection process and the behavior of the grout after injection can be deduced from observation of injection pressure and flowrate. An experienced grouting technician develops a "feel" for underground conditions, comparing the information he receives from pressure and flowrate gauges with the response he has learned to expect under various previous conditions. Two conditions of interest are grout refusal and hydraulic fracturing. Characteristic curves diagnostic of these responses are displayed schematically in Figure 45. Hydraulic fracture is indicated by large increases in flowrates with only small pressure increases. That is, the pressure/



SCHEMATIC - DIAGNOSTIC PRESSURE & FLOWRATE HISTORIES



SCHEMATIC - DIAGNOSTIC PRESSURE vs FLOWRATE RESPONSES

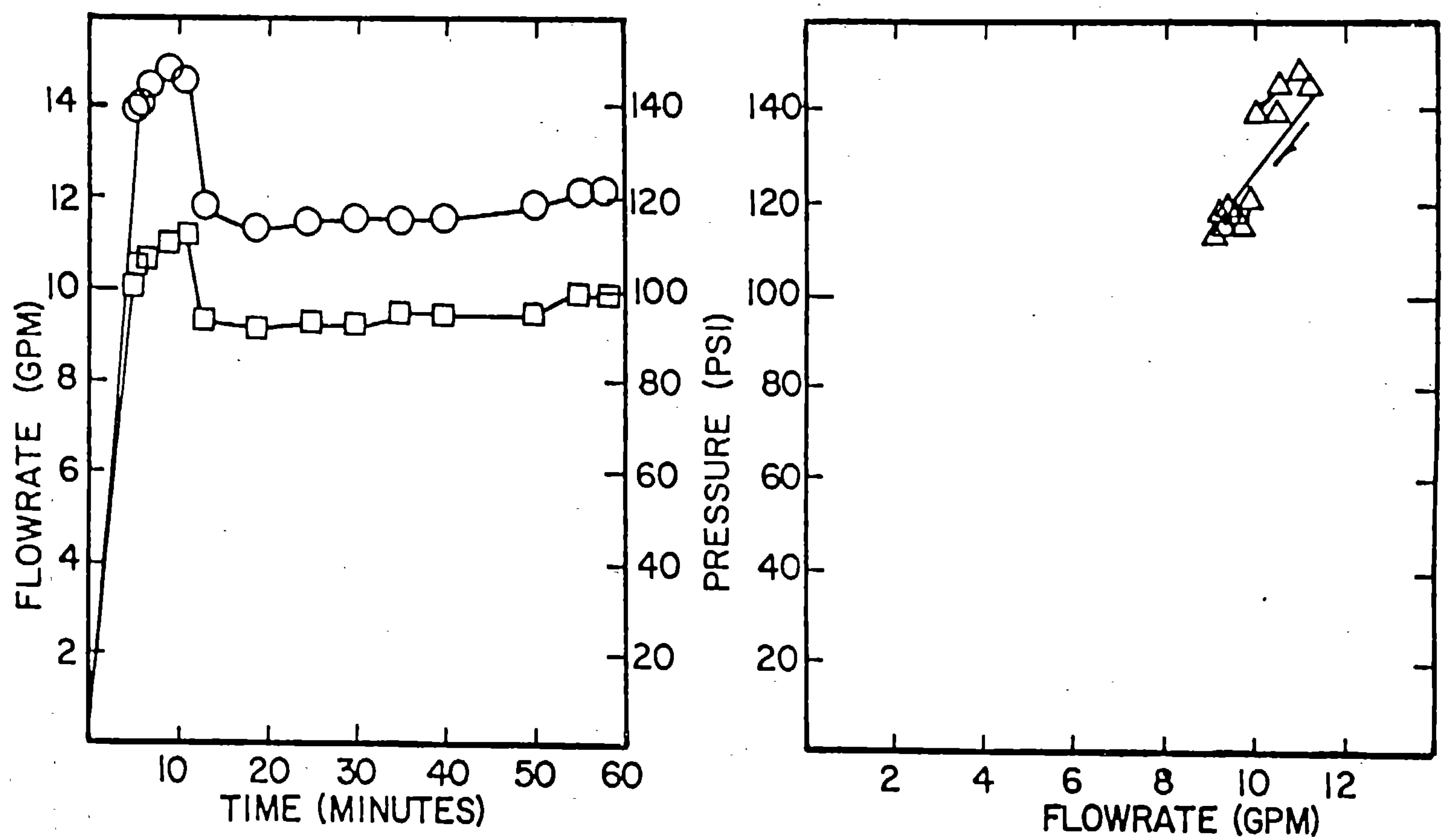
Figure 45. Signature Diagnostic of Grout Refusal and Hydraulic Fracture

flowrate curve is concave toward the flowrate axis. Hydraulic fracture is easily diagnosed during injection into impermeable materials, but it is difficult to identify in the permeable soils in which chemical grouting takes place. Grout refusal is the condition obtained when the soil in the vicinity of the injection port becomes saturated with gelling grout, and the adjacent soil is impervious and confined. The soil generally displays decreasing permeability, forcing injection pressure to increase and flowrate to decrease. This is termed an inverted pressure/flowrate response. The actual data from a high pressure injection test is shown in Figure 46. The time history curves are parallel, yielding the conventional linear pressure vs. flowrate curve shown in the cross-plot. Because this indicates no grout refusal, the full planned volume was injected at this grout sleeve. In Figure 47, a gradually increasing pressure and decreasing flowrate pattern begins 20 to 30 minutes after start of injection. The pressure-flowrate cross-plot shows that the sleeve is nearing rejection by the inverted response. Because the pressure was moderate and the pressure/flow inversion response was not strong, grouting continued to the design take. Both the sleeves were primary sleeves. The secondary sleeve shown in Figure 48 displayed a strongly inverted pressure/flowrate response. Injection was terminated when approximately one-quarter of the planned take had been injected.

The inverted response results from a gradual decrease of permeability as the soil becomes saturated with gelling grout. Brief pressure excursions result in normal response of the current permeability values as seen by the pressure spike and conventional proportional response pattern near the end of injection in Figure 48. Other conditions can also produce anomalous pressure-flowrate response. For example, a combination of low flowrate and fast gel time may result in the grout reaching the packer after its viscosity has begun to increase. Under these conditions, increased flowrate imply reduced residence time in the grout system and may actually result in decreased pressure. However, with a frequent monitoring of injection parameters and experience in the behavior of silicate grout, the injection process can be evaluated, and unusual conditions identified in time to take corrective action.

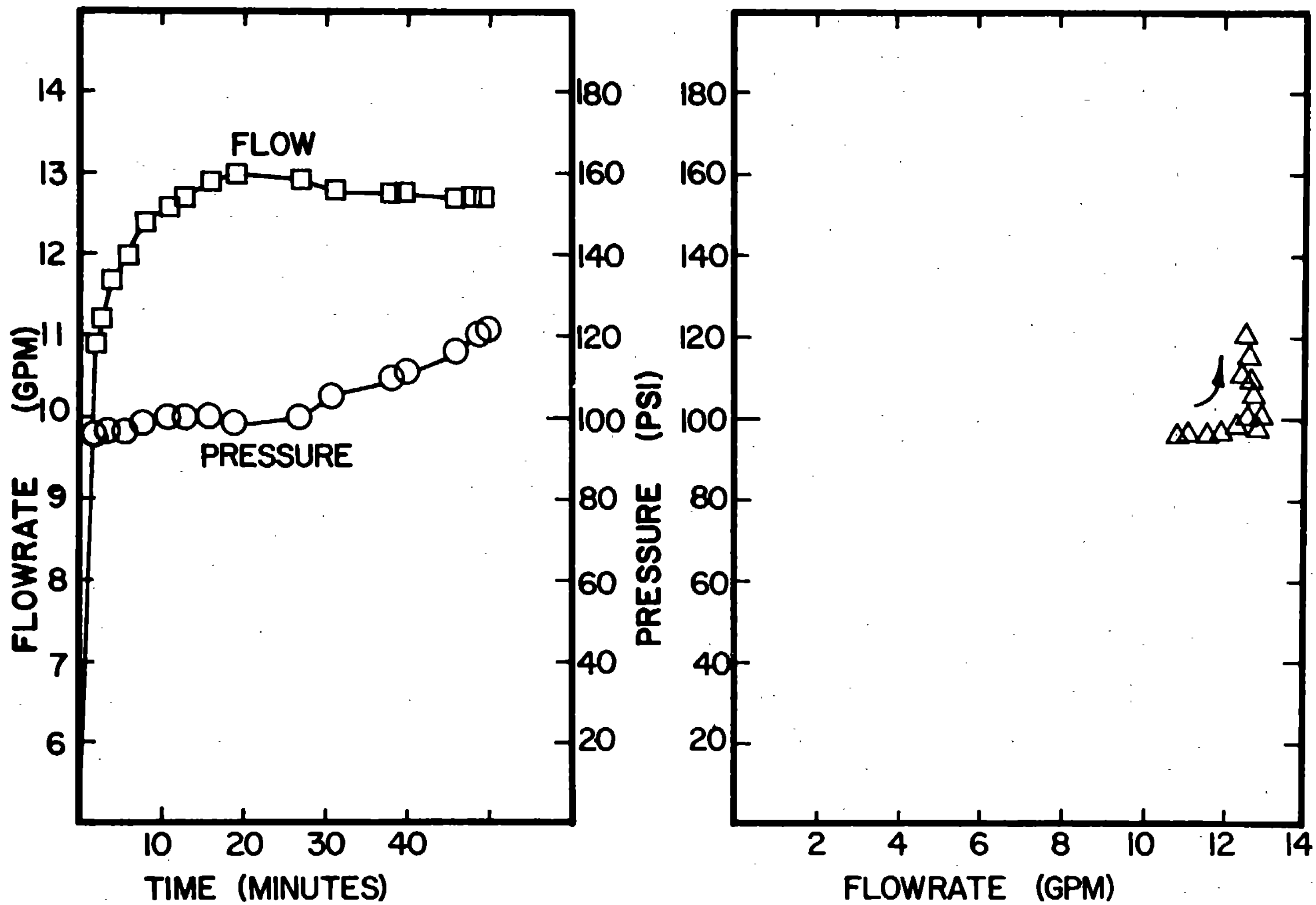
Permissible Injection Pressures

When establishing injection pressure limits, it is essential to consider the grouting method to be used. Stage pipe grouting exposes an open borehole to the full



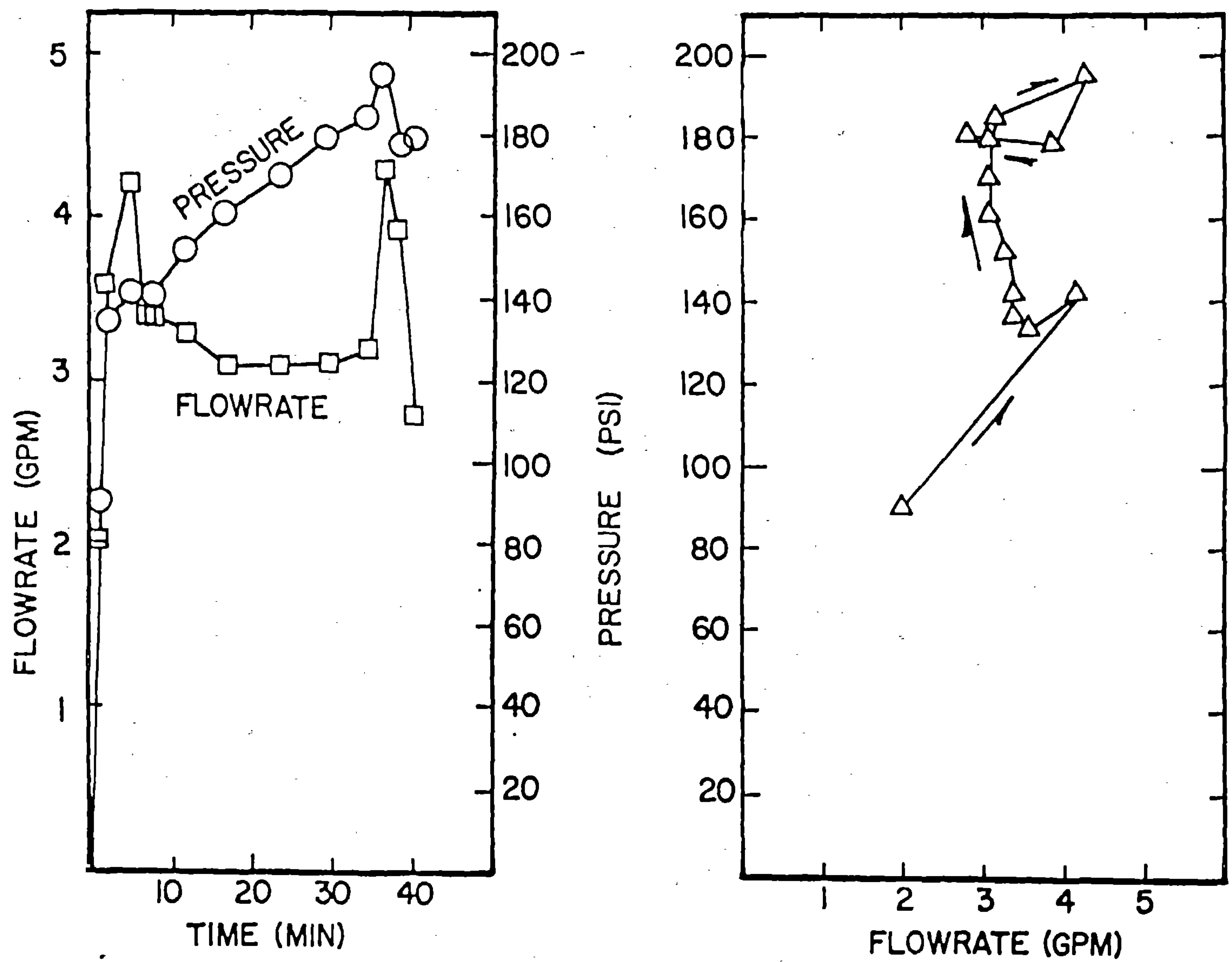
Note: 1 GPM = 6.31 x 10^{-5} m^3/s; 1 psi = 6.9 kPa

Figure 46. Injection Parameters for Grout Sleeve G2 - Middle



Note: 1 GPM = 6.31×10^{-5} m^3/s; 1 psi = 6.9 kPa

Figure 47. Injection Parameters Sleeve G3 - Middle



Note: 1 GPM = 6.31 x 10^{-5} m^3/s; 1 psi = 6.9 kPa

Figure 48. Injection Parameters for Grout Sleeve G5 - Bottom

injection pressure, while the sleeve pipe or tube-a'-manchette systems impose a significant pressure loss in the passage through the injection port and mortar jacket. The actual loss of pressure depends strongly upon the thickness and properties of the jacket mortar. Difficulties in predicting the actual head loss have led some European grout contractors to abandon pressure criteria in favor of flowrate determinations.

Pressure limitations are established to avoid hydraulic fracturing or heave. It is not clear, however, that moderate fracturing is either dangerous or detrimental. It may actually benefit the grout distribution. Experts are divided on the question of appropriate pressure limits. Field data are needed to resolve this matter.

Injections in the meter-cubed system in the laboratory employed pressures about 10 times the one psi per foot of overburden rule. Heave measurements with a precision of 0.1 mm (0.003 inches) disclosed no heave. This led to an experiment with high injection pressures in the pilot field tests, with similar results. Pressures as high as 1300 kPa (200 psi) did not result in surface heave. The grouting contractor must not use high pressures indiscriminately, however, since it is possible to cause moderate heave even when using low injection pressures, particularly if excessive volumes are pumped.

Preliminary field data on permissible injection pressures were obtained from the Laurens Street instrumentation effort on an actual grouting job (12). As was expected of field work in a complex geology, the data are noisy, yet trends do appear. Grout pore pressures were measured by the array of piezometers tips shown in Figures 16 and 17. In one case, pore pressures reached 45 percent of the injection pressure. The mean pore pressure, however, was only 13 percent of the injection pressure and fully three-quarters of the data were only 20 percent or less of the injection pressure. These comparisons were made between the injection pressure measured at the top of the grout pipe, and the actual pore pressures at the elevation of the piezometer tips. There was no consistent relationship between pore pressure and distance from the grout pipe, indicating that the test zone was not an isotropic homogeneous medium, but contained preferred flow paths which continually changed as grouting proceeded. In some cases, pore pressures were higher at greater distance from the injection point than pressures measured closer to the grout pipe. It is clear that the pore pressures in the grout are significantly less than the measured injection pressure when the grout-port injection method is used.

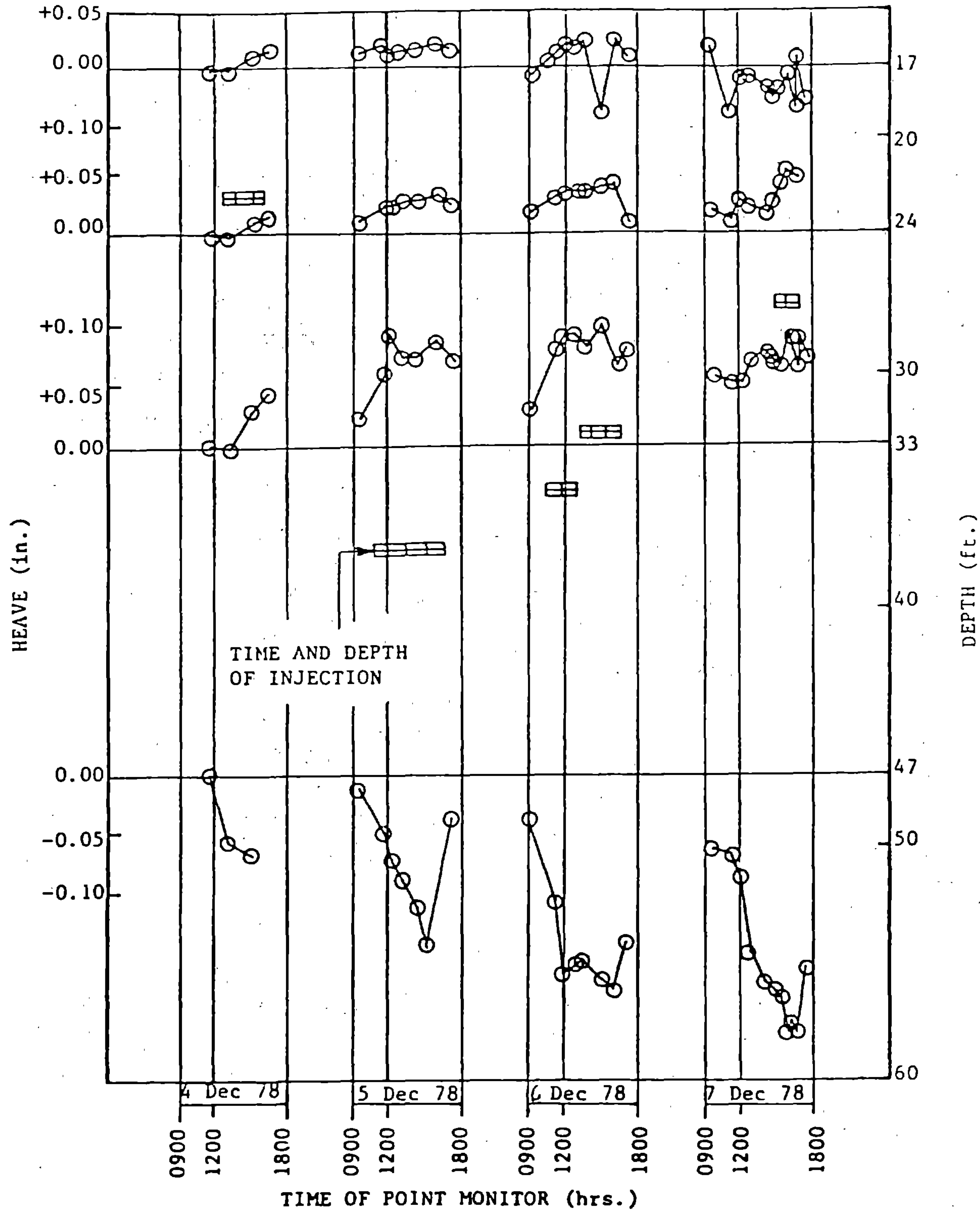
Surface Heave

Neither the meter-cubed laboratory injections nor the pilot field tests caused measurable heave, although high injection pressures and flowrates were used. The Laurens Street site incorporated two Multi-Point Extensometers (MPX) to measure extension in the grout zone. Because of the much larger scale of grouting on this project, strains summed over a grout zone depth of 6m (20 feet) were expected to develop measurable deflections. Heave data for each anchor point during primary injection (December 4-7, 1978) are shown in Figures 49 and 50. Motion data for MPX B during secondary injection (January 25-26, 1979) are shown in Figure 51. In the seven weeks between primary and secondary tests, MPX A was internally grouted by nearby injections and rendered inoperable. The heaves experienced during injection are on the order of several millimeters, the largest part of which rebounds or subsides overnight. From the extension data, a heave velocity can be roughly determined. The correlation between heave rate and injection pressure is shown in Figure 52. Because the range of flowrates was small, a meaningful correlation cannot be made directly to flowrate. The correlation with pressure indicates that, at this site, the heave rate tends toward zero near 200 kPa (30 psi). This was also the approximate pressure at which flowrate tended toward zero on this project. Based on the sparse data available, it appears that the total amount of heave experienced at a point depends only upon the number of gallons injected, regardless of the pressures and flowrates involved. The result is surprising, but the argument that surface heave is the result of overgrouting is compelling. A finite soil void volume is available within the grouted zone. If excess grout is injected, it must either pass beyond the intended grout zone or act to increase the void volume, and cause surface heave.

Since injection procedures tend to be uniform on any given project, it seems that such a mechanism might not be obvious purely on the basis of pragmatic experience. It is testable, however, and the implications of increased productivity in this labor intensive technology are of sufficient import that evaluation of high injection rates at separated grout pipes should be considered.

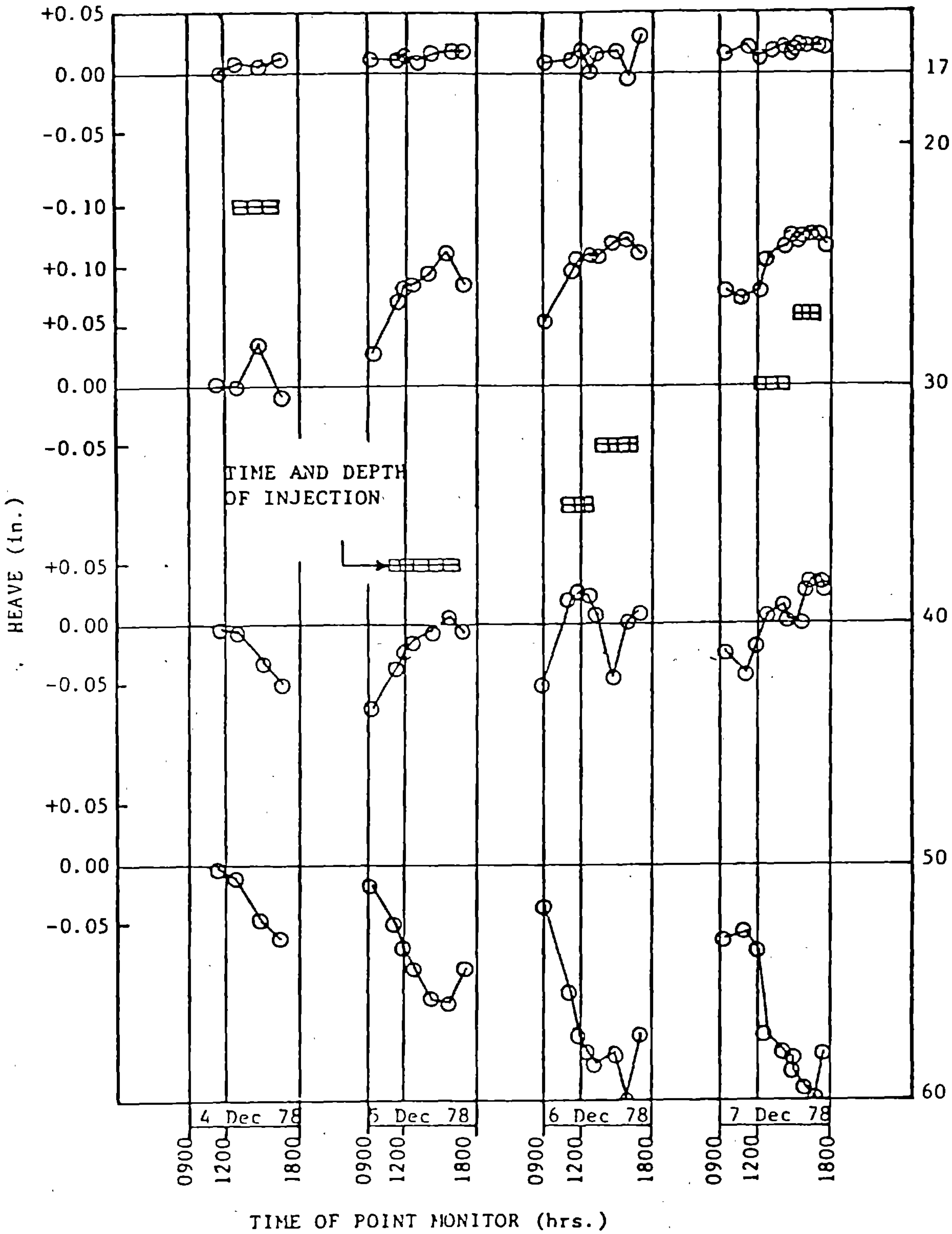
Acoustic Emission

Late in the program Professor R. W. Koerner of Drexel University was retained to conduct Acoustic Emission (AE) observations in conjunction with the hydraulic



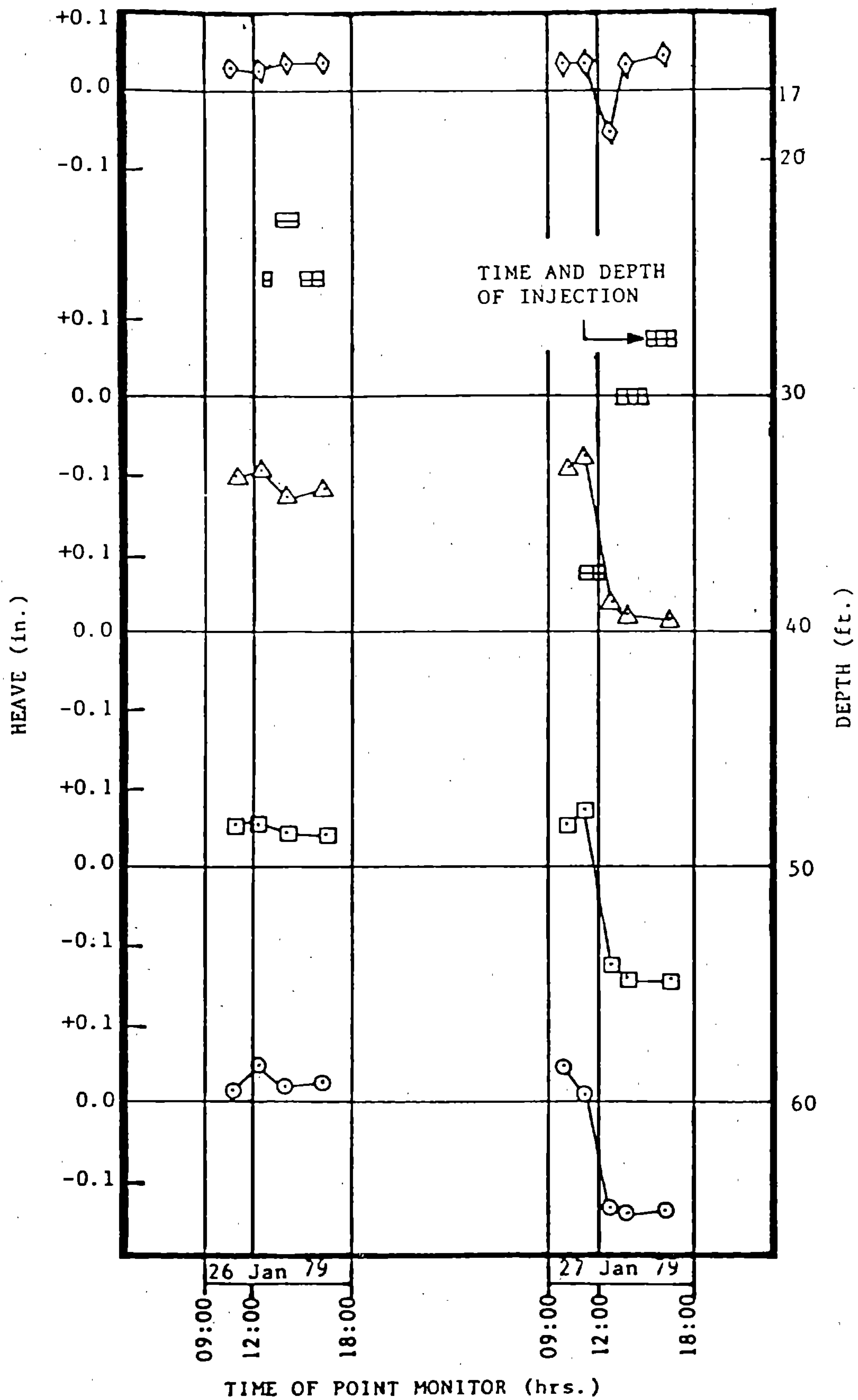
Note: 1 in = 25.4 mm; 1 ft = 0.305 m

Figure 49. Heave Motion Measured by MPX-A During Primary Injection



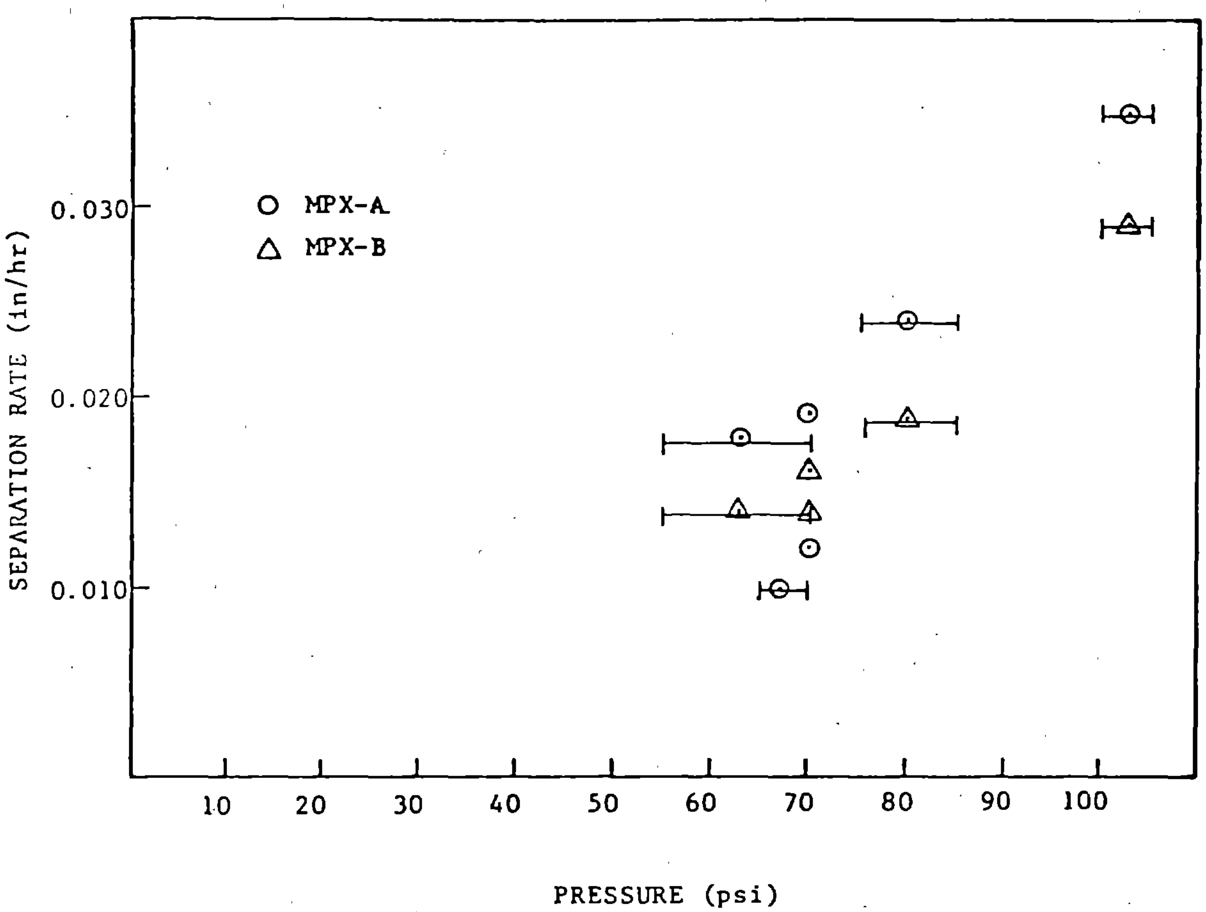
Note: 1 in = 25.4 mm; 1 ft = 0.305 m

Figure 50. Heave Motion Measured by MPX-B During Primary Injection



Note: 1 in = 25.4 mm; 1 ft = 0.305 m

Figure 51. Heave Motion Measured by MPX-B During Secondary Injection



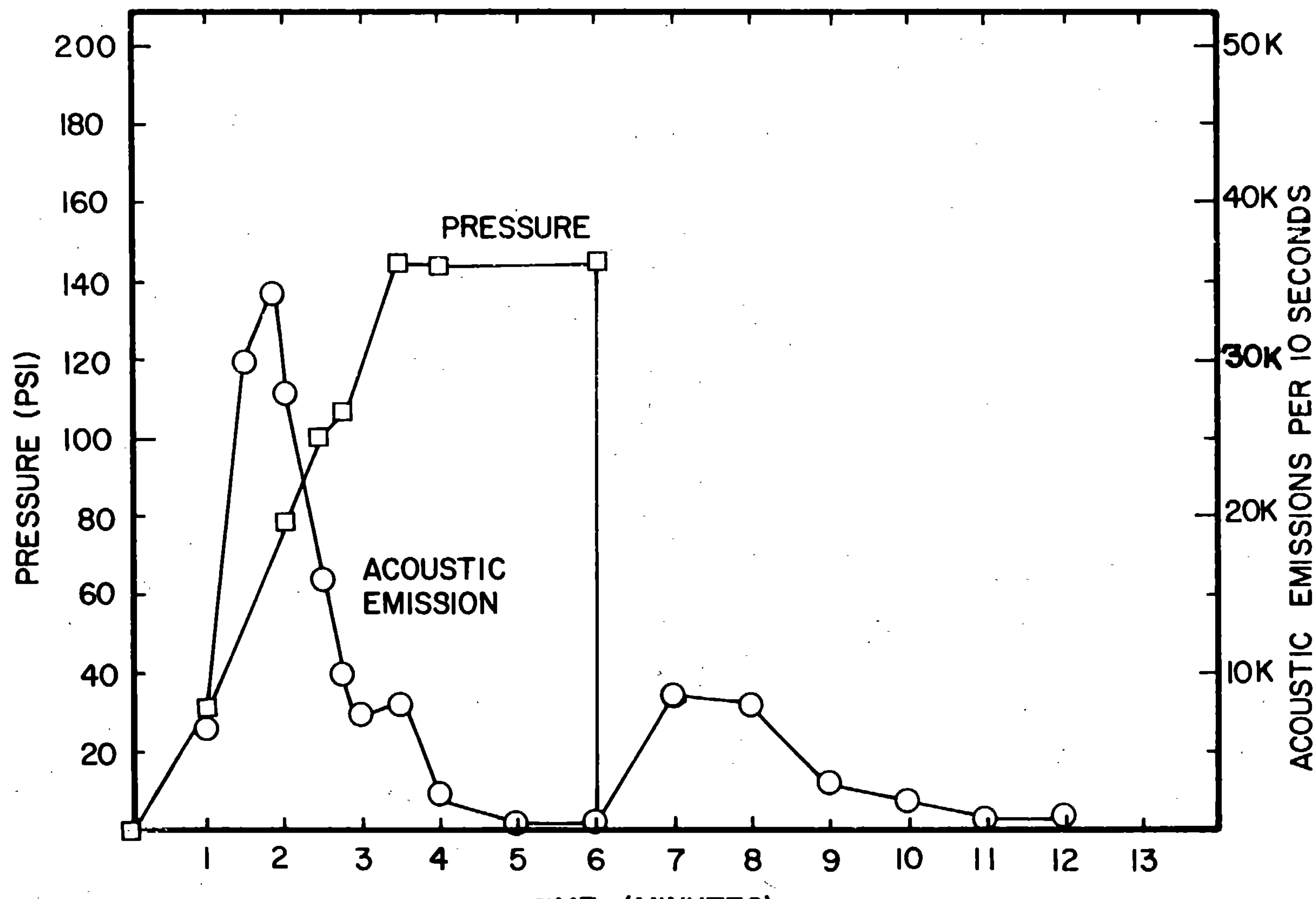
Note: in/hr = 0.706 x 10^{-5} m/s; 1 psi = 6.9 kPa

Figure 52. Effect of Injection Pressure on Rate of Heave

fracturing tests. The grouting protocol was to make three injections into each of the two sleeves (six injections total) at grout pipe G6. Each injection would pump 200 litres (50 gallons) at the maximum flowrate obtainable. Time was permitted for gelation between injections. If hydraulic fracture occurred, a burst of noise would be detected by the AE system. Typical data are shown in Figure 53. Count rate can be taken as a rough indication of fracture propagation. A burst of noise began at relatively low pressure, increased to a maximum count rate, and then declined as the injection pressure was monotonically increased. This noise indicates that a fracture had initiated at one minute into the test, propagated rapidly, and then ceased extension by the fourth minute. The count rate had declined to a low level by the time the peak pressure was attained, and fell to zero when the pressure (and flowrate) stabilized at their maximum values. During the period of steady pumping, the fracture was held open, but did not extend significantly. However, when pumping was abruptly stopped at six minutes, a second burst of noise signaled the closing of the fracture. It appears that a steady flowrate establishes a stable fracture surface area, such that flow into the fracture at the injection point is balanced by permeation into the sand from the face of the fracture. Thus, the crack extension depends upon flowrate rather than total volume injected as would be the case in an impermeable medium.

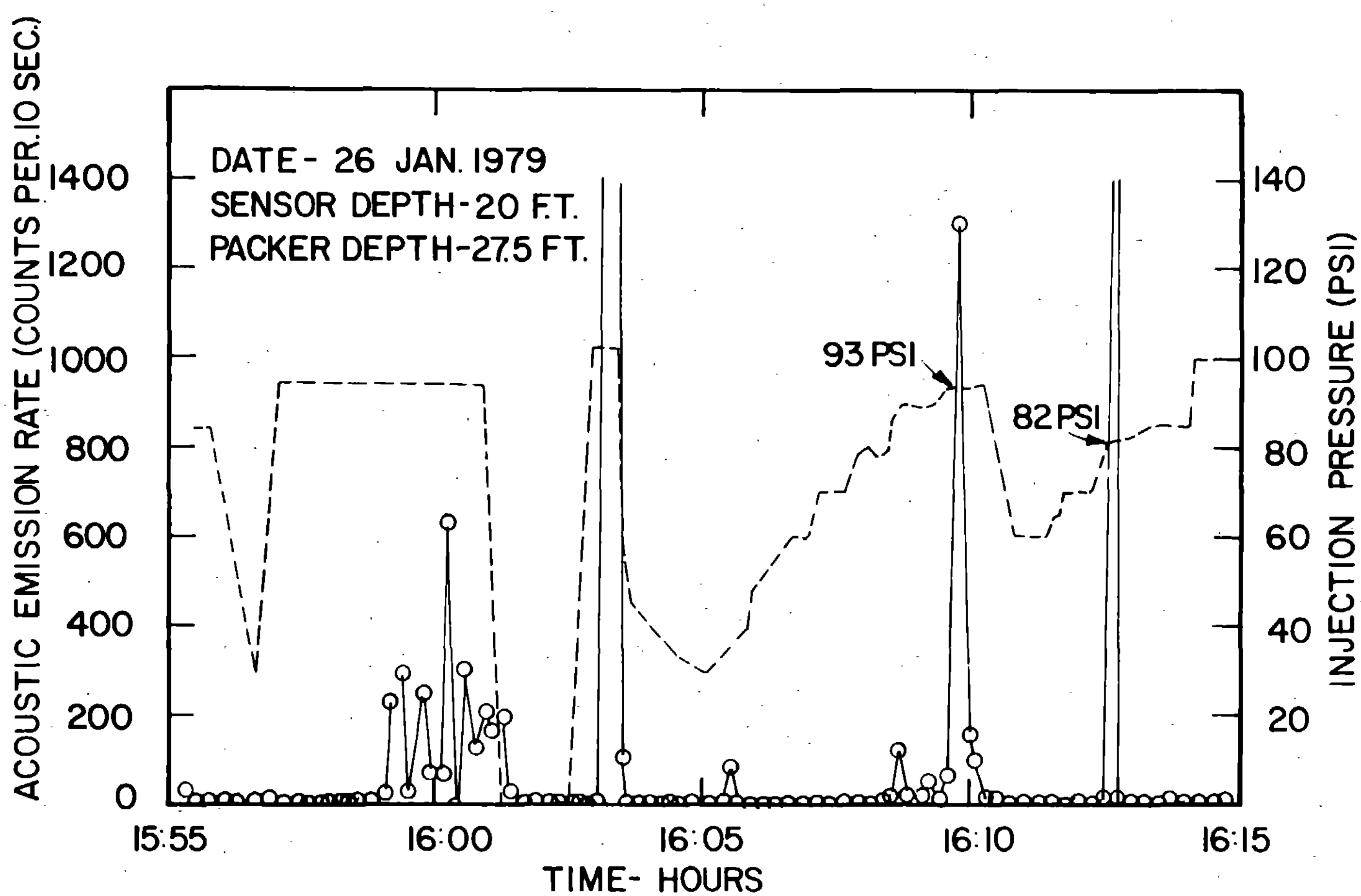
Because of the promise displayed by AE monitoring in detecting ground distress, it was scheduled for use in the Laurens Street tests. The primary concern was whether an acoustic system could be effective on a noisy real world construction site. Accelerometers were placed at the surface and down hole in an inactive grout pipe. Nearby, the surface instrument picked up considerable traffic and construction noise which was not picked up by the borehole instrument. Both channels heard acoustic emissions from fracturing tests, but data from the surface instrument would have been difficult to interpret because of the interfering surface noise. Typical data showing pressure levels that result in soil distress are shown in Figure 54. Acoustic emissions can be used effectively on construction sites if down hole systems are used.

The desirability of fracturing the ground by high pressure injection during grouting is a subject of some debate. Hydraulic fractures initiate in the axis of the borehole and tend to maintain their original vertical orientation during extension. Thus, they do not contribute to heave, but may exert horizontal force. The behavior of hydraulic fractures is different in an ungrouted porous medium, and a previously



Note: 1 psi = 6.9 kPa

Figure 53. Acoustic Emissions During Hydraulic Fracture



Note: 1 psi = 6.9 kPa

Figure 54. Acoustic Emission Test for Critical Injection Pressure

grouted, hence a low permeability medium. A fracture propagating through grouted soil will extend as necessary to create fracture volume to accommodate increasing flow volume. When it reaches ungrouted soil, however, the fracture stops extending and the grout permeates normally into the soil voids. In contrast, a fracture in a porous medium will extend only until the flowrate permeating into the faces of the fracture surface balances the injected flowrate. Because the crack surface area available for fluid losses increases roughly with the square of the distance the fracture is extended, a fracture in a porous medium tends to stabilize rapidly. The 1 psi/ft rule is often cited as a device to avoid the possibility of hydraulic fracture. Arguments for permitting fracturing are that it promotes complete coverage of the grout zone, and it results in more economical injection, to the benefit of all parties concerned. Concerning heave, the data from this project indicates that hydraulic fracture may not occur until injection pressures reach several times the pressure permitted by the 1 psi/ft rule. The proper approach is to monitor the soil distress (acoustic emission or surface movement) and control pumping accordingly. Because fractures initiate in the borehole axis, any resultant motion would be lateral. Because fractures will run through previously grouted soil, and be arrested when they encounter ungrouted permeable soil, they would tend to intersect any weak (ungrouted) volumes in the grouted mass and permeate grout into them.

The question of how much injection pressure is permissible can be summarized as follows:

(i) Hydraulic fracturing is not predictable by such simplistic means as the 1 psi/ft rule.

(ii) Potential ill effects such as heave should be monitored directly and frequently, regardless of pressure used.

(iii) The time and hence the cost of grouting can be much reduced by the use of high injection pressures in those cases where they are safe. The ends of both owner and grouting contractor are served by safe and economical grouting.

(iv) High pressures might safely be used during primary injections because the pressure is dissipated by head losses very close to the injection port. The use of high pressure during secondary injections should be approached more cautiously.

(v) Each project should be approached individually, with consideration for its unique hazards, and pumping pressures selected accordingly.

Acoustic Emission Monitoring of Seepage

Another application for AE monitoring was tested on the Reston Dam site (13). The detection of seepage through a dam using AE monitoring equipment was accomplished by the identification of anomalous zones of high acoustic emission rates. The seepage monitoring was conducted during off-shift and weekend hours on this study. Although the AE system used has the capability for filterizing both low and high frequencies in the bandpass 1 Hz to 50 kHz, any construction equipment in the near vicinity can make the background noises difficult to separate from the true AE signal.

Initial AE monitoring for detection of seepage was conducted in the primary holes between stations 7+10 and 8+00. Surface noise was a factor even on quiet days. In one case, anomalous noise was finally identified as the sound of truck traffic on a highway several miles distant. This airborne sound was barely audible to the naked ear, but was cured by covering the top of the casing in which the microphone was hanging. Most of the background noise level seemed to consist of the sound of small soil grains sloughing off the borehole walls and falling into the water near the microphone. The lack of borehole stability, found throughout the site, was most obvious when listening with the AE system.

Additional AE monitoring for seepage was conducted approximately three months later. The holes monitored in this case were at stations 4+53, 4+83, 5+64 and 7+03. The zone between 4+53 and 5+64 had not been fully grouted, while the region around 7+03 was well grouted at the time of this monitoring. An initial survey was conducted by monitoring AE rates in each of the four holes at a depth of 16.8 m (55 feet). The hole at 5+64 was then surveyed at depths between 13.7 and 21.3 m (45 to 70 feet). Data are listed in Table 3 below.

Table 3 Acoustic Emission Survey

24 August 1979

Station

	4+53	4+83	5+64	7+03
Depth			Count Rates	
13.7m (45 ft)			3130 ± 120 2360 ± 200	
15.2 m (50 ft)			1860 ± 330 3400 ± 320	
16.8 m (55 ft)	330 ± 19 370 ± 18	52 ± 2 21 ± 6	3460 ± 500 2930 ± 290 4940 ± 510	1 ± 0.6
18.3 m (60 ft)			5190 ± 420* 3380 ± 450	
19.8 m (65 ft)			2650 ± 550* 1230 ± 170	
21.3 m (70 ft)			420 ± 81* 26 ± 2	

*Small backhoe working nearby during these intervals.

Each entry in the table represents the average of at least eight and as many as 20 one-minute counts, and the standard deviation of the one-minute counts within each group of data. In Figure 55, showing the AE count rate for the three holes near station 5+00, the count rates are plotted on a logarithmic scale. Proceeding from 4+53 up-station, the AE count rate drops by a factor of ten at 4+83, and then increases by a factor of 100 at 5+64. Clearly, the areas near 4+53 and 5+64 are displaying significantly more AE activity than 4+83. At 7+03 the AE rate (not plotted) was near zero. Because these data were obtained in two sequential traverses of the four holes, the variation in AE rate is not some extraneous external noise source that varies in time, but something associated with the specific location of the dam itself. The most obvious candidate is the sound of seepage.

The individual counts for each one-minute interval as a function of depth at 5+64 are shown in Figure 56. Each band thus indicates the spread of data recovered during a period of up to 14 minutes of sounding. These data were taken during two traverses down the length of the borehole, so that the variation is clearly a function of location, and not external events. Note that the backhoe moving into the area did not seriously effect the distribution of count rates. This distribution indicates a noisy region three or four meters thick, with relative quiet above and below. This is at the approximate level of the soil/rock interface, and is thought to represent the sound of water seepage.

The possibility that this distribution of AE count rates could have been caused by the sound of water flowing in the overflow structure some 7.6 m (25 feet) distant is discounted because of the sharp bounding of the noisy region. A noise source at a distance would tend to produce a much broader range of elevated count rates unless the sounds had propagated preferentially along the soil/rock interface. If this were the case, one would expect to hear higher count rates at station 4+83. Essentially, if sound were radiating spherically from a source at station 5+89 (the overflow structure) one would see a much broader depth vs. noise distribution at station 5+64, in accord with the inverse cube law. This is not seen. In accordance with inverse cube radiation, one would expect the noise level at station 4+83 to be lower than at 5+64 by a factor of (25/106) cubed, or down by a factor of about 75. This is nearly the case. The overflow structure is, however, not a point source, but a line source. If radiation followed an inverse square relationship, say by being confined to

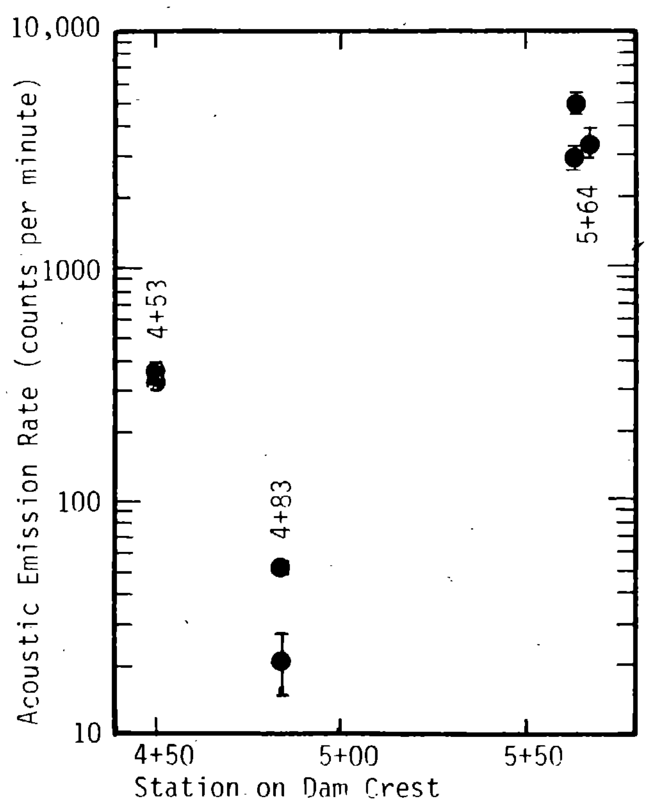


Figure 55. Acoustic Emission Monitoring for Water Seepage

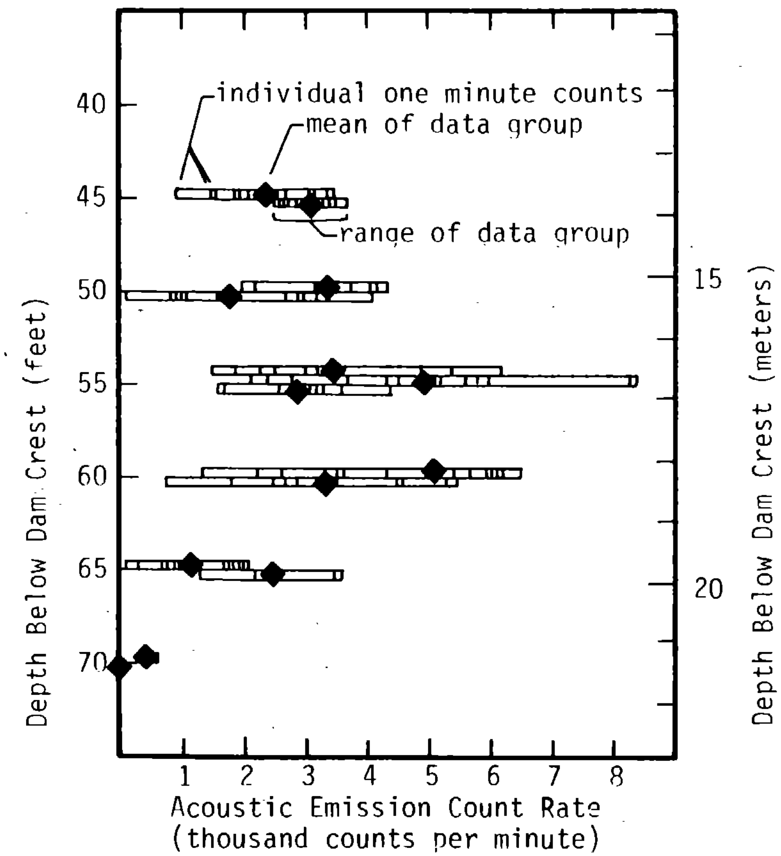


Figure 56. Acoustic Emission Depth Probe at Station 5+64

the interface, then the AE rate at station 4+83 should be down only by a factor of 20. Thus, we believe that the noise heard at 5+64 is being emitted from a source nearer than the overflow structure, and that the AE rate at 4+83 is not related to the same source. Note that station 7+03, only 34.7 m (114 feet) from the overflow structure, was down by a factor of more than 1000.

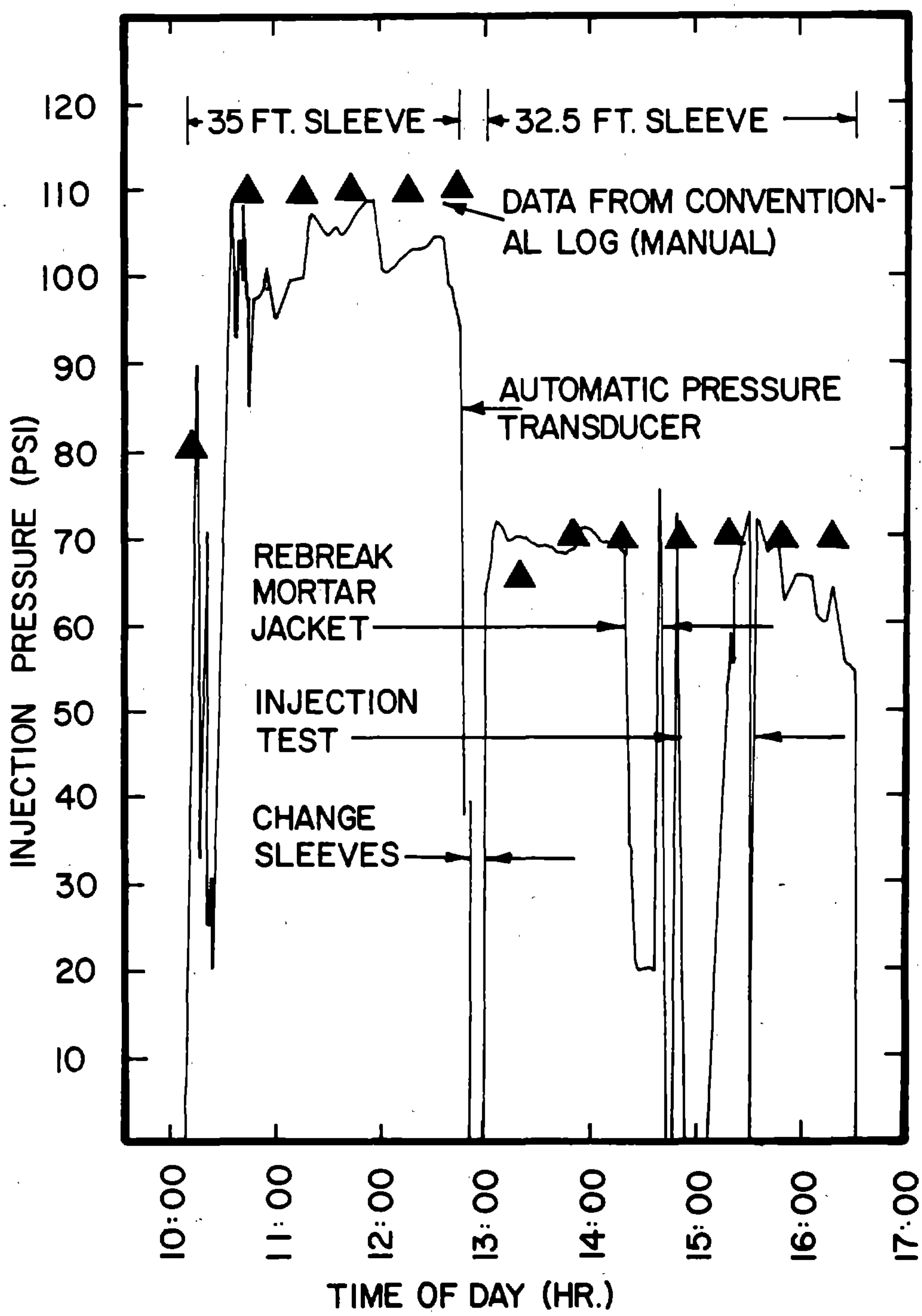
Secondary Grout Port Test

Grouting is traditionally conducted in stages, with primary, secondary and sometimes higher level injection points. The secondary stage of grouting serves to fill in any gaps left by the primary grouting, and to test the adequacy of primary grouting. Typically, primary grout pipes will be drilled in a 3 m (10 feet) grid pattern, with a secondary grout pipe at the center of each square. In some cases, alternate grout ports in the same grout pipe will be used as primary and secondary ports. The grout takes are best designed so that most of the grout is injected during primary grouting. If the secondary injections refuse grout (high pressure - low flowrate) soon after pumping begins, confidence in the primary grouting is increased. If refusal is not reached in a reasonable time, a third stage of grouting may be advisable. The different resposes expected of primary and secondary injections were shown in Figures 46 to 48. The early refusal of grout by the secondary hole, Figure 48, indicates that the primary grouting was effective.

Documentation

The procedural controls are best implemented through the use of continuously recording pressure and flowrate instrumentation. A comparison of pressure data recorded manually and automatically is shown in Figure 57. The continuous data displays much greater detail, and will alert the grouting engineer more rapidly when unusual behavior is experienced. The data should be recorded on robust industrial style strip chart recorders, annotated during injection, and properly filed for later review. The ability to review the contractor's decisions and performance at a later date will much improve the confidence that may be placed in a grouting job.

Data from several injection points can be logged on forms as shown in Figure 58. This is a display of grout take at each sleeve in a single line of grout pipes. A single



Note: 1 psi = 6.9 kPa; 1 ft = 0.305 m

Figure 57. Comparison of Data Recorded Manually and Automatically

row of grout pipes is not recommended unless absolutely dictated by site conditions, as was the case here, because the single row does not provide full confinement for the secondary injections. In this case, alternate sleeves in each grout pipe were designated as secondary injection points. Even with only partial confinement, the secondary sleeves attained grout refusals at lower volumes than did the primary sleeves. It is useful in some cases to plot injection pressures on one side of the line representing the grout pipe, with total volume injected on the other. The log conveys detailed information about conditions in-situ. In this case, it can be seen that the ground is slightly tighter at the upper portions of grout pipes six through eight than it is elsewhere. As injection proceeds through such an area, the grouting engineer will perceive such changes in time to determine their cause and take action if necessary. Finally, such a log makes it very easy to monitor progress on the project, and review the project after grouting.

CHAPTER 5—TECHNICAL RESULTS AND CONCLUSIONS

This section of the report summarizes the techniques that may be used to insure the proper execution of a chemical grouting project. Volume 3, Engineering Practice, details when and where these systems may best be used. An effective Quality Control/Quality Assurance (QC/QA) effort must be integrated into the design and specification of a project in order to apply these techniques during construction. Enhanced confidence in the quality of any grouting project will require that the location and condition of the grout be determined, and further, that grouting procedures that improve control over the process be used.

CONSIDERATIONS FOR CONFIDENCE IN GROUTING

The purpose of this research was to develop methods for monitoring and controlling chemical grouting, so that the designer might be able to specify its use under appropriate circumstances with confidence that the desired objectives would be met. From the engineer's point of view, there is little difference between a badly executed grouting project and one conducted impeccably, but without inadequate testing and documentation. Neither can be trusted, because neither is known to have been properly executed. To a large extent, the onus for confidence in grouting is shared by both the designer and the specialty contractor. The designer must know and state just what he needs done, specify appropriate tests and controls, structure the overall project to expedite application of these controls, and finally, review the progress of the project. The specialty contractor must provide personnel and technology capable to control and do the work as engineered construction.

CONTROL OF GROUT LOCATION

Chemical grout may be placed in the desired location by the use of short gel times, grout port injection methods, and AC electrical resistivity monitoring. The most precise injection method requires continuous mixing and injection by sleeve-port pipes. If long gel times are permitted, the grout may have time to migrate downward below the intended grout zone, or may be diluted and carried away by groundwater flow. It is desirable to have the grout flow dominated by the injection pressure, not by gravity, groundwater flow, or other factors that cannot be controlled. Precision in injection is further enchanced by avoiding the build-up of large volumes of ungelled grout underground. In general, gel times between 10 and 20 minutes are appropriate for most cases, although shorter gel times may be necessary to prevent migration of the grout.

If open borehole injection systems are used instead of grout port systems, the hazard exists that grout will not enter the soil directly, but will travel down the previously grouted borehole section to a more permeable horizon. The experience at Locks and Dam 26 showed that much less reliable results were obtained with the open borehole system than the grout port method. Of the open borehole methods, stage-down methods are more dependable than stage-up methods. Regardless of the injection system used, care is required to insure that the grout is injected in the proper location.

Electrical resistivity may be used to determine the location of the grout during injection. More specifically, resistivity will give a qualitative indication of the amount of grout that has intruded the soil volume sensed by previously emplaced probes. The system used must employ alternating current (AC) as opposed to direct current (DC), and must have sensor probes in the grout zone for best precision. A constant current (peak-to-peak) source is preferred. Because the system uses in-situ probes, the placement of which may be costly, the ultimate use of electrical resistivity will probably be to delineate the boundary of the intended grout zone in those cases where it is essential that the grout not be allowed to travel outside. Thus, it may be used primarily to protect utilities, or other zones that must not be grouted.

EVALUATION OF GROUT LOCATION AND CONDITION

The location of injected grout may be determined after grouting by the use of borehole radar, and whether it has set up or not determined by the use of acoustic cross-hole shooting. Both systems are applied before and after grouting, so that the changes caused by grouting may be easily determined. Both make use of plastic grout pipes already placed for the grouting operation itself, and so are quite economical in application.

Borehole radar is used in transillumination mode. A transmitter is placed in one borehole, a receiver in another, and the signal traversing the path between is diagnostic of the amount of grout intersected. Chemical grout is "lossy", that is, it absorbs the signal, and little radar signal can pass through a well grouted soil. Surface radar, or borehole instruments using transmit/receive (T/R) mode (a single instrument that transmits a pulse and then receives its own reflected signal) are of less use because grouted soil is not highly reflective. Little energy is reflected, it is simply absorbed in the grout.

Either pulse radar or continuous wave (CW) microwave may be used effectively in a borehole transillumination system. Geotomography, resulting in a schematic picture of the zone, was attempted, but the results obtained were not worth the additional effort required. This judgement is open to reevaluation, since geotomography is a less mature art than conventional radar profiling, and is advancing at a correspondingly higher rate. It is possible that advances in hardware and software will have sufficiently improved geotomography to make it the preferred choice in the next few years.

Acoustic cross-hole shooting was found to be quite diagnostic of the condition of the grouted mass. In the laboratory, a significant relationship was found between acoustic velocity increase and grout strength. In the pilot field tests, the high velocities measured after grouting gave clear indication that the grouted mass had been converted to a rock-like mass. Acoustic cross-hole shooting is less effective in showing the exact grout location, since the acoustic signal tends to follow high-velocity paths rather than strictly straight lines.

CONTROL OF INJECTION PRESSURE

It is common practice in this country to specify a maximum injection pressure without regard for the grouting method or variability of soil conditions. The competent designer will abjure this costly and counter-productive practice in favor of specifications directly relating to those factors which affect the quality of the finished product. Injection pressure and flowrate should be monitored to determine when grouting at a point should be discontinued to avoid overgrouting, and when regrouting should be required to assure adequate performance.

Grout refusal is easily identified by the gradually increasing pressure and decreasing flowrate signature that indicates reduced permeability in the soil volume being injected. A primary hole, through the process of flow inversion, typically may be pumped for periods much longer than the gel time without displaying refusal. Grouting in such cases should be terminated when the design volume has been injected, or grout may be pushed outside the grout zone. Secondary holes are used to grout the zones left ungrouted by the primary injections, and also as a test of primary grouting adequacy. If a secondary hole does not display refusal when the design volume has been injected, and the zone is critical, regrouting should be considered. Injection should stop long enough to permit complete curing of grout in the area, and then additional grout injected until refusal occurs. In areas less critical, regrouting may serve no purpose other than to increase the cost. Those zones which are critical to project performance should be identified in the contract documents, and the criterion by which grout refusal will be monitored and identified should be established.

The commonly specified limitation on injection pressure is often said to be intended to avoid the possibility of hydraulic fracture. It is not clear from available experience that hydraulic fracture should be avoided. If, after due consideration, the designer determines that hydraulic fracture is to be avoided, neither a flat limitation on injection pressure nor interpretation of the classical broken-back pressure-flowrate curve are effective for his purpose. Hydraulic fracture is easily distinguished by acoustic emission (AE) monitoring. Because the critical fracturing pressure varies widely from point to point in the ground, the AE instrumentation should be set up so as to be clearly visible from the grouting technician's operating station and closely monitored by him.

HEAVE LIMITATION

Overgrouting, the injection of grout in excess of the total soil void volume within the grout zone, must lead either to surface heave or to grout migrating outside the intended zone and no constructive purpose is served. Elastic heave, which occurs during injection, and then dissipates in off-shift hours as excess pore-water pressure dissipates, may be avoided by spacing grout injections each day to avoid mutual interaction between the pressurized zones. High pressures and flowrates may then be used without producing damaging heave if relatively small amounts of grout are injected within any small area on any given day. Primary injections should be staggered, with adjacent holes not being injected before pore water pressure dissipates. High flowrates with associated high productivity and reduced cost may be achieved without significant surface heave. Additional study and evaluation of this proposed technique will be recommended below.

INSTRUMENTATION AND QUALITY ASSURANCE

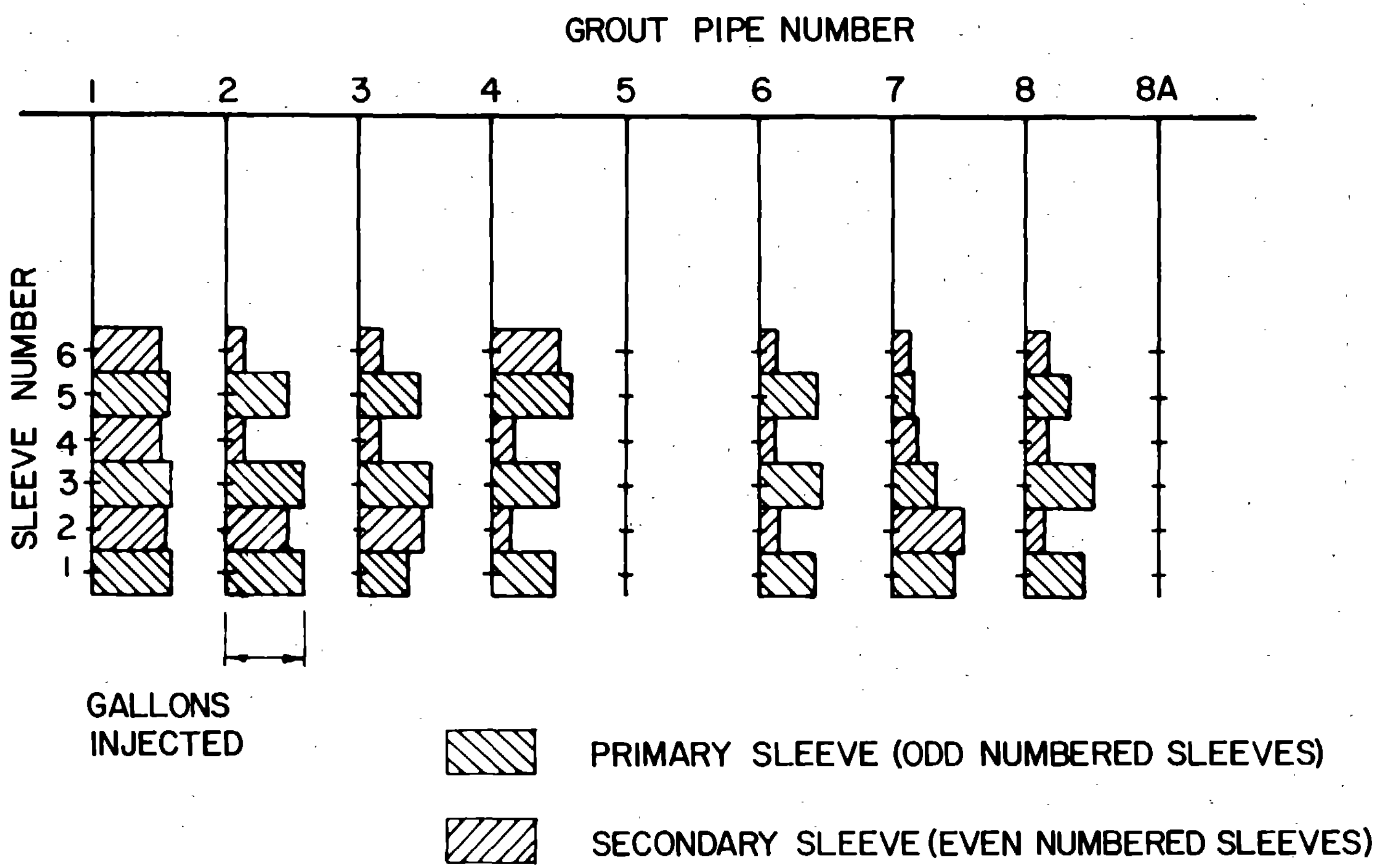
The techniques discussed above relate largely to quality control, and require certain hardware and software. Quality assurance, the documentation to prove that appropriate and adequate work was accomplished, makes permanent recording systems highly advantageous.

Pressure and flowrate should be recorded by stripchart recorder for every injection point. Circular recorders, used extensively in Europe, are difficult to read, while digital recording, whether by automatic data acquisition/printing systems or by clipboard and pencil, require careful interpretation or plotting. The human mind can more readily grasp information presented graphically than as a table of numbers. It is urgently recommended that appropriate instrumentation be required by specification. After injection, the chart paper should be annotated to show location, time and date of injection, and filed as permanent documentation of the project activity. Review/approval of these data should be required either daily or weekly. This will keep both contractor and inspection forces up to date, expose unexpected conditions in a timely fashion and avoid unpleasant surprises.

The volume of grout injected at each point should be plotted daily as shown in Figure 58 - Grout Take Log. This plot provides a graphic display of progress and warns of changing ground conditions. It tends to become the focus of on-site conferences at which project decisions are made.

TECHNICAL QUESTIONS REMAINING

The items called out in the paragraphs above may be implemented by the designer or grouting contractor now, and indeed, in several instances have already seen application. Two technical areas still deserve treatment, as resolution of these questions will significantly impact grouting effectiveness and cost.



Note: 1 Gallon = 3.785×10^{-3} m^3

Figure 58. Log of Grout Volumes Injected in Single Grout Pipe Array

Hydraulic Fracturing

Although strong opinions are held on the advisability of hydraulic fracturing during grout injection, it was not possible to establish a consensus among leaders in the grouting community, or even to identify data supporting one opinion or the other. Arguments for and against permitting hydraulic fracturing are presented in Section 4.0 of this report. Techniques have been developed with which hydraulic fracture can easily be detected, but one cannot state that it either is or is not desirable in all cases. It is expected that study of the question will identify situations in which fracturing should be avoided and others in which it should be actively encouraged. Codification of these areas is needed since both cost and effectiveness are involved.

Hydraulic fracturing should be studied by a program to include analytical study of fractures in material containing low modulus inclusions representative of ungrouted areas in an otherwise well grouted mass, laboratory fracturing tests, and demonstration of a selected production grouting project.

Optimum Injection Patterns

The injection sequence in areal grouting may be varied to achieve particular ends or to suit special conditions. For example, the initial injection of cap and foot grout points to seal the upper and lower boundaries of the grout zone, and the advancing cheveron pattern to express groundwater from the grout zone are injection sequences that serve particular purposes. Because the grouting process itself may be made more effective or more efficient by optimization of injection patterns, study is warranted. The correlation between grout pipe spacing, injection pressure, ground deformation, and project cost should be reviewed for various injection sequences. The use of staggered injections of small volumes at high flowrate appears to be an approach that will significantly increase productivity without increased cost or surface heave.

Injection patterns should be studied, including those serving various purposes, e.g., enhanced productivity, groundwater exclusion, and reduction of horizontal or vertical deformation. These systems should be synthesized, reviewed analytically to discern general trends and effects, and demonstrated on production grouting projects.

Such projects will require appropriate instrumentation, such as level nets and multipoint extensometers, as well as authority to modify injection sequence and parameters to suit the experimental effort.

Corroboration and Documentation

Implementation of research results requires winning acceptance of the civil engineering community with particular emphasis on the criticial design segment of the profession. A research report is not sufficient. Short courses and technical presentations are effective supplements. The most effective approach, however, to disseminate new information is by example. Demonstration projects can be executed at relatively small cost. Production grouting phases of the development efforts proposed above should be used to display the new approaches to designers located nearby. It has been found that job site tours and short presentations in the designer's office are effective and well received. Demonstration projects should be planned with this aspect in mind.

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