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DESIGN AND CONTROL OF CHEMICAL GROUTING: VOL. 3

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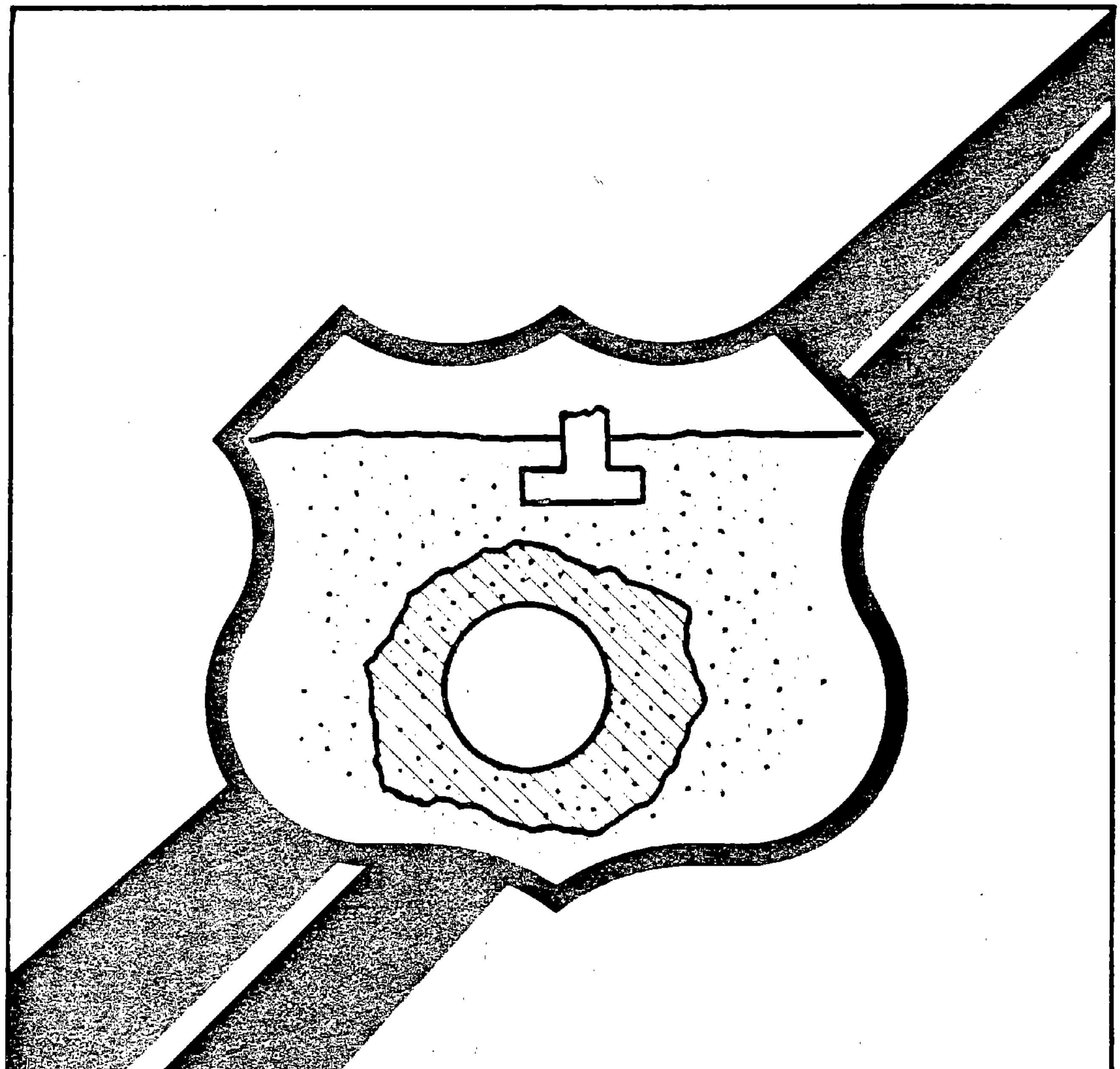

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FOREWORD

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.

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Richard E. Hay
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Office of Engineering
and Highway Operations
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16. Abstract

Recent improvements in the engineering practice of chemical grouting have provided increased confidence in this method of ground modification. Designers can significantly improve the success of chemical grouting by defining their grouting program objectives and focusing on the geotechnical conditions controlling the process. New developments in analyzing the structural behavior of grouted soil masses permit numerical performance predictions prior to construction for more realistic designs and better project economy. Detailed planning of injection locations, staging of grout volumes and injection sequencing is required for an effective work program and facilitates good quality control management. New geophysical methods are explained for evaluating chemically grouted soils, utilizing subsurface radar and cross-hole acoustic velocity profiling. Illustrative case histories and guide specifications are also presented.

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

Vol. No.	FHWA No.	Title
1	FHWA/RD-82/036	Construction Control
2	FHWA/RD-82/037	Materials Description Concepts
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PREFACE

This report presents the background, concepts, and procedures required to perform chemical grouting in a direct, controlled manner. Intended for designers of underground construction projects, the report suggests a rational, methodical approach to the planning and performance of chemical grouting, primarily focusing on structural underpinning and excavation support.

After a brief introduction and discussion of chemical grouting design philosophy, the report discusses important geotechnical parameters, describes chemical grout properties, performance predication methods, and planning steps for the injection process. Quality Control methods are outlined, with emphasis on the need for accurate (preferably automatic) measurements of grout flow rates, pressures, and volumes, and real time evaluation of these data. Chemical grouting Quality Assurance is discussed, contrasting conventional geotechnical testing methods with new radar and acoustic velocity geophysical methods for before and after grouting evaluation. Several brief case histories of chemical grouting applications are presented to illustrate typical design problem areas. A quality control and assurance demonstration case history is also given. Finally, guide specifications are presented, incorporating the proposed design and control methods for chemical grouting.

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APPLICABLE SI TO ENGLISH CONVERSIONS

1 m	=	3.28 ft		
1 mm	=	0.04 in		
1 l	=	$1 \times 10^{-3} m^3$	=	3.785 gal
1 kPa	=	0.146 psi		
1 MPa	=	146 psi		
1 N/m^2	=	1 Pa		
1 MN/m^2	=	1 MPa		

CHAPTER 1—INTRODUCTION AND BACKGROUND

SCOPE

The ideas and recommendations given here are intended to provide the reader with a method for designing, executing, controlling and evaluating the myriad details and parameters that go into a successful chemical grouting project. It is intended that the reader will develop an adequate understanding of the method sufficient to prepare a rudimentary chemical grouting program design for comparison with other alternate construction methods, and to communicate effectively with experienced chemical grouting engineers and contractors.

The suggestions given herein should never be applied directly without considering the fundamental engineering principles involved and their relation to each particular case. The need to avoid a cookbook approach is self-evident if one considers the multitude of soil conditions and grouting purposes that can be encountered, the large number of available grouts, injection methods, underground construction situations and contract types from which one has to choose. The variety of possible chemical grouting programs and designs is almost limitless.

This volume includes a brief background summary of chemical grouting, a design philosophy for performing chemical grouting, a discussion of important geotechnical considerations, a survey of the properties of chemical grouts and grouted soils, a review of performance prediction methods, injection planning considerations, and injection monitoring and control techniques. Finally, several typical chemical grouting case histories are presented to illustrate problem areas and sample specifications are given for chemical grouting for control of settlement and groundwater for soft ground tunneling.

BACKGROUND

Chemical Grouting

Grouting generally refers to several ground modification techniques, including chemical grouting in soils, chemical rock grouting, cement or clay grouting, and compaction or displacement grouting. This report is concerned with chemical grouting in soils. Chemical grouting is the injection of a gelable fluid material into permeable ground. In the liquid phase, chemical grout is a true solution or a colloidal solution. In chemical grouting, the liquid grout is injected into the soil in such a manner that it permeates into the soil interstices without causing gross movements or rearrangement of the soil fabric, and then gels to solid form within the void spaces. During injection, the grout displaces water and air from the soil voids; it may fracture the soil along weak planes, but it will not densify the soil nor displace it significantly.

Chemical soil grouting is used to either strengthen a soil mass (structural), reduce its permeability (water control) or both. The intended purpose of grouting on a job must be determined and clearly stated by the designer, since it is possible to accomplish either objective 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 strong cohesive zone around the tunnel to minimize lost ground and surface settlement may be used in the same situation. The intentions of the designer must be made clear to the specialty contractor and construction manager if the expected results are to be obtained.

Structural Grouting

Structural chemical grouting is used when it is desirable to increase the strength and/or stiffness of a soil mass. Application examples range from the stabilization of running ground prior to tunneling to strengthening of dynamic machine foundations, where the soil stiffness must be changed to eliminate dynamic resonance problems .

Chemical grouts may be used in sand or silty sand containing up to about 20 percent material passing the No. 200 sieve. Less costly particulate grouts, such as portland cement or bentonite clay grout, can be used in very coarse sands and gravels. Fine soils, on the other hand, with high silt or clay contents cannot be grouted 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. Unconfined compressive strengths between 0.2 to 4.0 MPa (30 to 600 psi) can be obtained, depending upon the soil and the grout. Creep or long-term 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 below the creep strength. Dense sands display relatively little increase in stiffness (tangent moduli), while loose sands become as stiff as dense sands upon grouting. Acoustic velocity measurements indicate significant increases in micro-stiffness in grouted sands. This increased stiffness is substantiated by pressuremeter tests. The increase in modulus observed depends not only on the soil that is grouted, but also on the strain level that is used.

Concepts concerning the properties of grouted soils are still being developed, but are nevertheless adequate for the design of civil engineering structures. Variability in soil properties from point to point in a given soil mass causes greater uncertainty in predicting the resultant properties of the grouted soil mass than the lack of data on the characteristics of grouted sand. The designer should obtain soil samples from his site and have them injected with grout and tested. This process is simple, inexpensive, and provides much better data than a review of typical published curves obtained by tests on soils not representative of the site in question.

Structural grouting has been applied to a wide variety of problems. In recent years, structural grouting has been used extensively to protect fragile or important existing structures from movements during soft ground tunneling. It has also been used to stabilize dynamically loaded foundation soils to eliminate settlement caused by densification under vibration, and to stabilize liquefaction-prone soils to protect against earthquake distress. Structural grouting may be applied either before or after construction, and may replace more traditional systems such as mechanical underpinning.

Water Control Grouting

Water control by grouting requires complete grout permeation of the treated zone, such that no "windows" of ungrouted soil remain. This is accomplished by injecting triple line grout curtains or blankets, always involving primary, secondary and tertiary grouting phases. Subjects appropriate for waterproofing are excavations, hazardous waste disposal sites, leachate ponds, and any site where conditions require the cessation of groundwater flow. Typically, if groundwater flow is a problem, the soils are sufficiently permeable 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, high pressure gradients across the grout curtain will develop significant flows of groundwater through the small ungrouted "windows." This may lead to piping and progressive failure of the grout curtain if allowed to grow. In especially critical cases, it may be desirable that the grouting contractor be available to treat such areas if they develop during excavation or later phases of construction. Grouts appropriate for waterproofing need not be as stiff as structural grouts. The use of softer grouts is often desirable if the treated zone will be excavated at some later date.

During the last decade steady improvements have been made in grouting equipment, in injection procedures, and in the wide choice of grouts available on the market. The number of field applications of grouting has increased steadily.

CHAPTER 2—DESIGN PHILOSOPHY

PROBLEM SOLVING PROCESS

Conventional underground construction attempts to deal with ground conditions as encountered in place. Where conditions require extraordinary efforts, or where conventional methods are either too costly or ineffective, special ground modification methods such as chemical grouting should be considered. For example, for a three kilometer long tunnel, it is usually not cost effective to employ chemical grouting as a principal component of the basic tunneling scheme. Rather, the general tunneling method selected should be one that will cope with the majority of conditions encountered over the major length of the tunnel. Chemical grouting should be used to take care of local problems where the primary tunneling method is inadequate. In the situation where two conventional construction methods are being evaluated, such as tunneling versus open-cut excavation, one of these approaches may be greatly aided if the adjacent ground could be rendered stable, impervious, or both, through chemical grouting.

Many small grouting projects have been arranged by simply requesting a local grouting contractor to "come out and see what you can do." Where this invitation is given to a grouting contractor without the involvement (either in-house or third party) of engineering expertise to adequately evaluate the geotechnical conditions, the chances for success are greatly reduced. Although chemical grouting may have been a viable solution in such a case, initial trial and error attempts may have been termed a failure too quickly by the client requesting the demonstration program.

The decision process that leads to the use of chemical grouting in geotechnical construction should be more extensive than a quick trial and error approach. A check list of possible steps to follow in the process of deciding whether to use chemical grouting is shown in Figure 1. The primary steps involved in an engineered chemical grouting program are: (1) the establishment of specific objectives for the grouting program; (2) definition of the geotechnical conditions requiring treatment; (3) development of an appropriate grouting program design and performance prediction with matching specifications; (4) the development and execution of a detailed construction

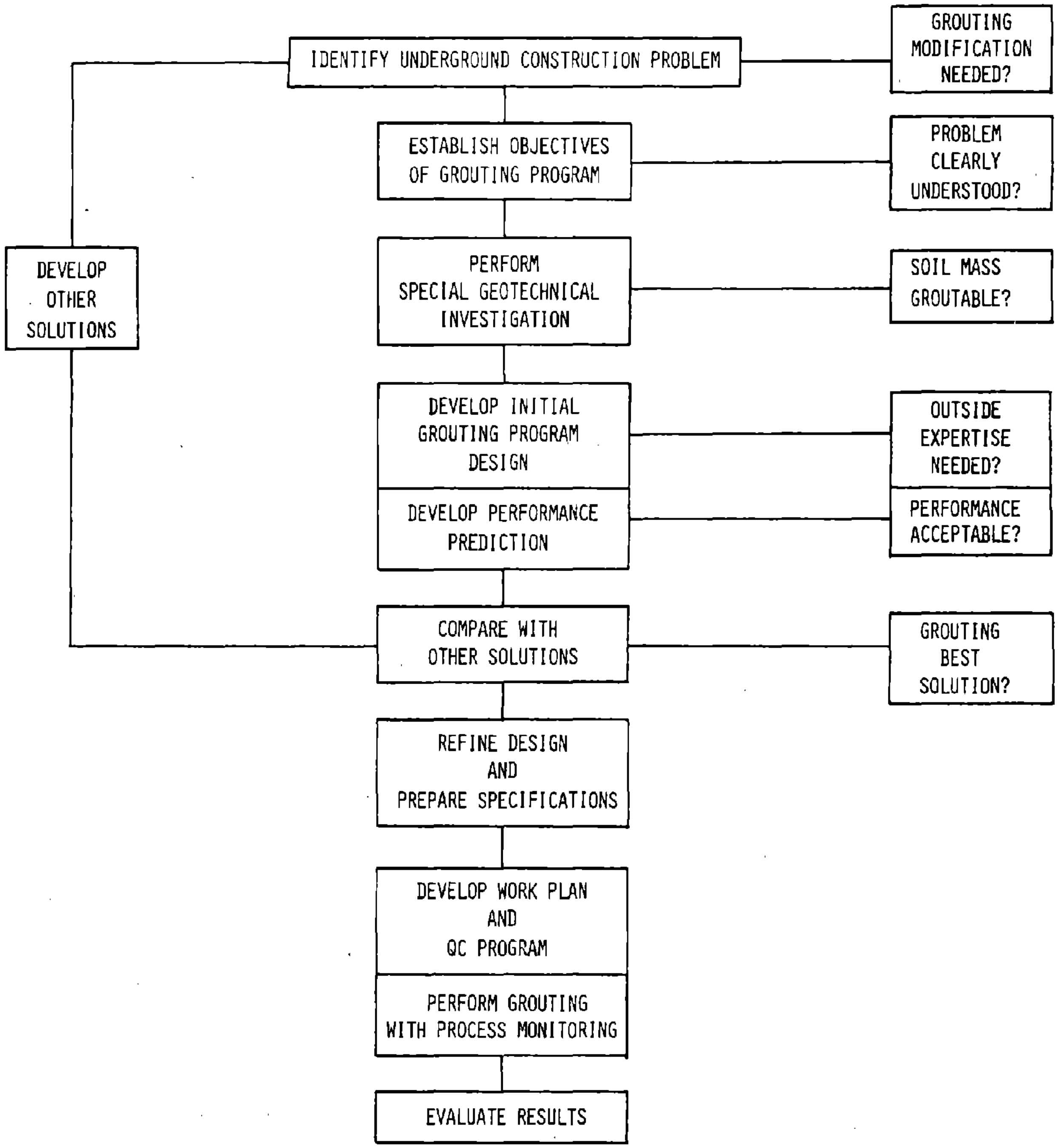


FIGURE 1 – CHEMICAL GROUTING DECISION FLOW CHART

work plan, including a quality control monitoring program, and (5) evaluation of the results of the grouting program. A similar decision-making process would apply to the evaluation of other nongrouting candidate solutions to a particular geotechnical problem. Several fundamental questions must be asked in this decision-making process.

A key question to be asked is whether the soil mass is groutable? Where chemical grouting is attempted without the involvement of the engineering expertise necessary to adequately evaluate the geotechnical conditions, the chances for success are greatly reduced. Selection of grouted zones must be based on feasible grout pipe layout as well as stability considerations. Following the preparation of an initial design and a prediction of the construction performance of the grouted system, it must be asked whether the grouting program will meet the project technical requirements?

Further evaluation of an initial design requires the preparation of an initial grout plan, including injection process planning for grout hole layout, pumping rates and sequences, etc., a quality control program, and a performance evaluation plan. The probable time and money costs can then be estimated, including the impact of the grouting work on other construction schedules. Finally, it is important to ask whether a qualified grouting contractor is available to do the work as anticipated?

CHEMICAL GROUTING AS ENGINEERED CONSTRUCTION

The point of view taken throughout this text is that chemical grouting is to be done as an engineering construction activity. To be so classified implies that enough details concerning the geological conditions and related construction procedures are available so that a clear definition of the problem is possible, and so that an unequivocal statement of the solution objectives can be made. Only then can an adequate preliminary engineering design for chemical grouting be prepared. The generally indeterminate nature of the program requires that the design process assumptions be checked during construction as more and more field data become available. This hands-on design approach, considered to be fundamental to the success of most chemical grouting projects, follows Karl Terzaghi's (1948) admonition for all geotechnical construction:

"...in earthwork engineering, success depends primarily on a clear perception of the uncertainties involved in the fundamental assumptions and on intelligently planned and conscientiously executed observations during construction. If the observations show that the real ... conditions are very different from what they were believed to be, the design must be changed before it is too late. These are the essential functions of soil mechanics in engineering practice."*

GROUTING OBJECTIVES

The early establishment of clear, quantitative objectives to be achieved by a chemical grouting program is a basic prerequisite to good design and a satisfactory, economical performance. All too often, however, the reasons for performing chemical grouting are stated in vague, qualitative terms. This may later lead to lowered satisfaction with the results because of unrealistic and unfilled expectations.

The term "structural chemical grouting" is applied where the purpose of the grouting is to improve the strength and/or rigidity of the groutable soils to prevent ground collapse, reduce otherwise unacceptable ground movement during construction, improve bearing capacity, etc. Many grouting projects have had as the design objective to simply give the normal noncohesive ground (no strength under unconfined conditions) sufficient cohesive shear strength so as to prevent the beginning of collapses or soil "runs" into excavations, tunnels or shafts. Chemical grout underpinning is another application of structural chemical grouting, wherein granular foundation support soils are strengthened so as to permit excavation adjacent to footings. In these cases, the soil strength lost by the reduction in confining stresses is replaced by the cohesion imparted to the soil by the grout.

Previously, the required soil strength of a grouted soil has generally been based on past experience. Recently, the use of finite element analytical procedures has permitted better definition of the required extent and strengths of chemically grouted soils around a soft ground tunnel where the primary objective is to limit surface settlements. Where the design process is refined to this degree, then it is necessary to know the stress-strain-time properties or deformation moduli of the grouted soils, in

*Karl Terzaghi, "Foreword," Geotechnique, Volume I, No. 1, Institution of Civil Engineers, London, England, 1948, pg. 4.

combination with the exact excavation sequence, in order to model the whole process analytically. This puts an additional special requirement on the geotechnical study to define the in-place deformation moduli of the various strata, both before and after grouting.

The term "waterproof grouting" has been used to describe chemical grouting projects aimed at stopping the flow of groundwater, which otherwise would provoke ground movements or the flow of unacceptably large amounts of water into a construction area, or both. Since relatively weak grout gel can be used for this purpose, strength requirements are usually limited to prevention of erosion at the cut face and through ungrouted piping channels. Absolute imperviousness is not an achievable goal with chemical grouting, but ground permeability can easily be reduced a thousand fold or more from the usual permeability range for sand of 10^{-1} to 10^{-3} cm/sec to the range of 10^{-5} to 10^{-8} cm/sec, depending on grout type and other factors.

An important application of waterproof grouting is the establishment of cut-off curtains in alluvial materials below dams and around excavations. Waterproof grouting has frequently been used to prevent the subsurface flow of pollutants or contaminated groundwater away from the source of contamination. Special consideration of this additional requirement must be given in designing the grouting program, particularly in the selection of the grout, to insure that it is chemically impervious or resistant to the polluting substances as well as hydraulically impervious.

The reduction of cut-off effectiveness that results from a few "windows" or leaks in a narrow sheet-pile wall is serious. The presence of "windows" in a grouted cut-off wall one or two meters thick has a much less dramatic effect on flow reduction because of the wall thickness. Nevertheless, elimination of "windows" is important from a stabilizing point of view. The elimination of "windows" in grouted curtain walls is a major goal of the grouting work and requires special efforts.

The life expectancy of the engineering solution represented by the grouted mass needs to be clearly defined in the requirements of the job. Many grouts can be considered to be permanent, i.e., have a service life in excess of 20 years under normal conditions. However, long-term grout permanence is an unrealistic requirement to place on a grouting program unless it is absolutely necessary. There should be a clear outline of the service conditions under which the grouted mass is expected to perform during its "lifetime." For example, silicate grouts may provide excellent waterproofing characteristics and low-strength structural improvement for temporary works, up to several months. They should not be considered to be permanent, with a lifetime of several years, unless the particular silicate-catalyst system has been shown to be permanent under the expected service conditions. For some systems, reversal of the gelling process can occur by a combination of syneresis and loss of the catalyst by leaching. Wet and dry cycles and freeze-thaw cycles can have dramatic effects on the degradation of grouted soils, as can changes in the chemical environment. The ability to sustain load can be sharply reduced for some chemical grouts over long time periods. Thus, each case where permanency is required should be studied carefully. Figure 2 shows schematically several different chemical grouting applications.

PROJECT PERFORMANCE STATEMENTS

Most construction contracts are focused on the legal, commercial and liability aspects of the client-contractor-engineer relationships, and not on the technical intent of the construction project itself. This leads to such language as,

> "...the contractor shall perform chemical grouting to protect the indicated structures from any damages due to adjacent excavation work."

This language tends to hide the designer's intent, which may have been only to provide a temporary structural support system to prevent structural and major cosmetic damage to the building, and not reduce settlements to "zero." In this case, settlements of up to 15 mm may be very acceptable and anticipated by the designer, but not to those reading the specifications.

In such cases, it is recommended that the designer state his goals in engineering terms, such as,

> "...the contractor shall limit vertical settlements of footings of adjacent structures to 15 mm. The chemical grouting scheme shown on the drawings is shown as a typical acceptable plan for accomplishing this purpose. The contractor shall verify that his injection plan will accomplish the stated building settlement reduction purpose."

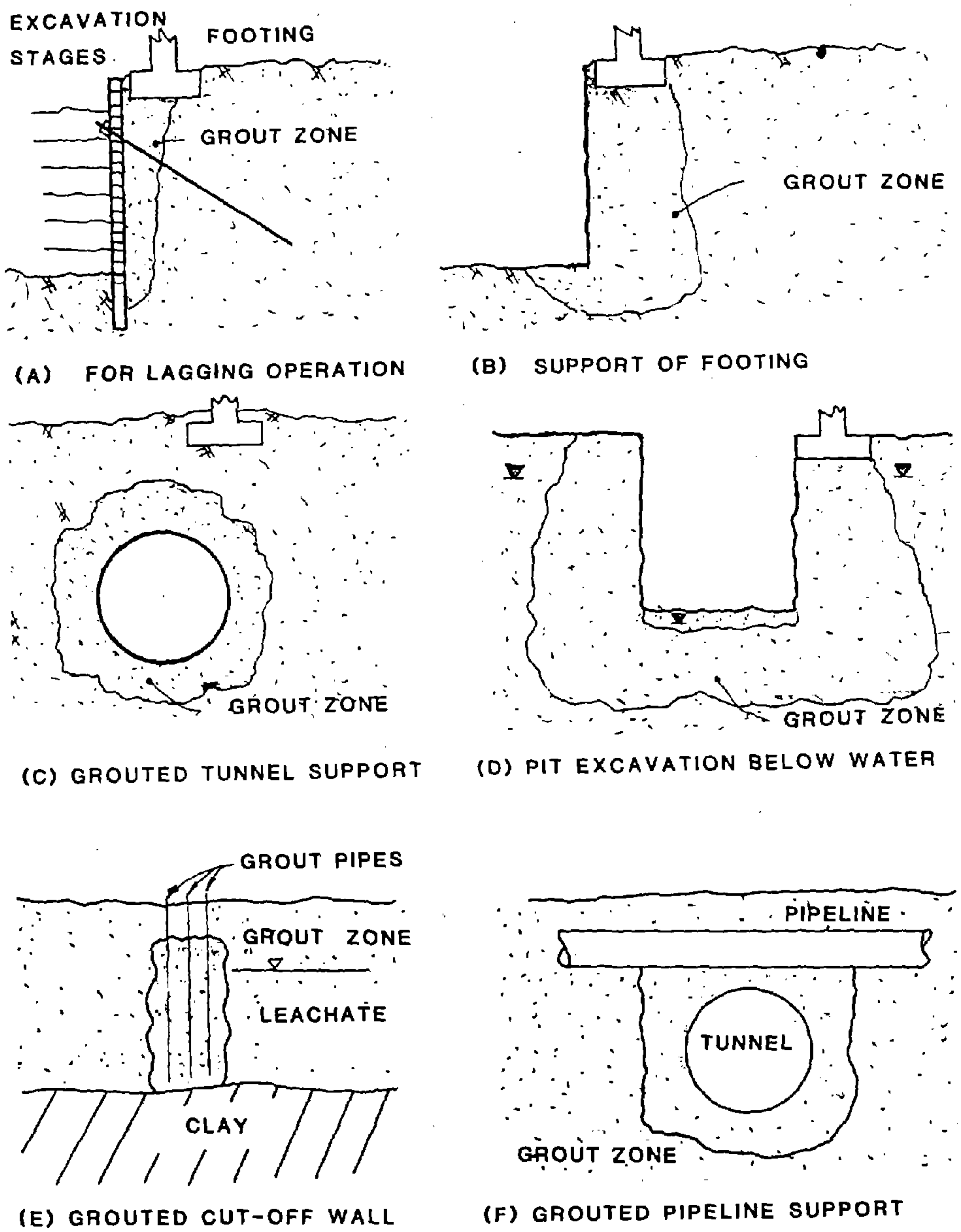


FIGURE 2 - CHEMICAL GROUTING APPLICATIONS

It is helpful in the design phase of the project for the designer to develop a project performance statement. This requires him to list, in order of importance, his major objectives in carrying out the chemical grouting program. For example, in a soft ground tunneling situation where a 6 meter diameter subway tunnel was designed to pass through granular soils and through a point some 3 meters immediately below an important column spread footing, and where the groundwater table was at about springline, the primary objectives of the chemical grouting program could be listed as follows:

1. Prevent large loss of ground during tunneling that would result in near loss of footing support and serious structural damage.

2. Reduce the inflow of groundwater at the face to an amount that can be handled with medium pumps located within the shield, say no more than 200 liters per minute, with no minimal erosion of waterborne soil fines into the face.

3. Prevent structurally damaging settlements to the affected footing by reducing movements to, say, not more than 18 mm.

4. Prevent disruption of services to the affected structure caused by structural damages associated with the tunneling process.

5. Reduce cosmetic damage caused by footing settlement to a minor level (barely visible cracking).

6. Provide for tunnel face ground control that will assure an efficient rate of tunneling advance in the affected area.

After listing the objectives according to the degree of importance, the designer can evaluate the probability of success for any given program and the opportunities for misperformance that would result in failure to meet any particular objective. For example in the above case, even a poorly executed, but properly conceived, chemical grouting program would most assuredly achieve the first objective of preventing a major disaster, but might not achieve the other objectives of water control and elimination of more minor structural damage. On the other hand, a properly designed and performed chemical grouting program in the above situations could meet all of the objectives, with a cost increase of, perhaps, 10% to 25% between achievement of objective No. 1 and achievement of all six objectives. In such a case, the designer would be well advised to design and execute the program for the achievement of all objectives, as an insurance that the primary objective was met with absolute surety.

In developing a project performance statement, it is important to evaluate the consequences of failure of some portion of the system and provide for a back-up position. For example, in the case of structural grouting for underpinning of a spread footing where an adjacent excavation in granular soil was to proceed immediately next to and below a very heavily loaded and important footing, it may be helpful to provide for bracing or lateral earth anchor support of the grouted mass as a back-up system in case a weak zone in the grouted mass tends to deform excessively. In this "belt-and-suspenders" solution, chemical grouting would prevent any raveling or initial loss of ground, and the required extra strength within the foundation support soils would be provided by the mobilization of the frictional strength of the sandy soils by the bracing or anchors.

Another example of a project performance statement would relate to the situation where a tunnel face was to be grouted below the water table with the intention of reducing water inflow. The project objective could be to limit water inflow such that only minor pumping at the face is required, and such that soil inflow at the face is not more than 2% or 3% of the excavated volume. Here, the primary purpose of the chemical grouting is to eliminate the potential for major sudden ground loss during water inflow and thereby preventing large surface settlements. For such an objective it is obvious that minor surface settlements are acceptable. In this case, it would not usually be necessary to specify a high unconfined compressive strength for the grouted soil, because only a low grouted strength is needed to prevent raveling at the face and water movement through the soils.

RISK EVALUATION

For each stated objective, the designer should evaluate the probability of success and the sensitivity of the grouting program to deviations from assumed conditions.

The consequences of failure should be understood beforehand, so that an adequate response to unexpected results can be planned and precautions taken and woven into the work plan to the degree justified.

The following three examples will be used to examine the consequences of failure to obtain the grouting objectives:

1. Chemical grouting underpinning to reduce settlements in a two-story commercial building above and adjacent to a subway tunnel constructed some 7 meters below in granular dense ground which has typically experienced settlements of as much as 3 cm in a previous section of the tunnel.

2. A large diameter tunnel in soft ground passing 3.5 meters immediately below a corner column of a 5-story apartment house.

3. A subway tunnel passing 1.5 meters below an old, active 1.5 meter diameter sewer main that cannot be bypassed.

In the first example, chemical grouting is specified to minimize settlements and thereby reduce commercial liability caused by cosmetic damage to a structure and associated commercial activity disruption. Failure, however, would not result in loss of life or major loss of operations of the structure. In the second example, major structural damage could likely occur if the chemical grouting were not entirely successful in stabilizing the building foundation, but not so quickly that loss of life would result. In the third example, the consequences of the chemical grouting program failing to protect the sewer could easily be catastrophic if the sewer were to fail and cause an inflow discharge through the ground and into the tunnel, flooding the tunnel.

In some projects, it may be possible to keep a standby grouting crew available for regrouting as the completed work is tested. For example, in waterproof grouting in an excavation where the grouting has been done prior to excavating, it is expected that some "windows" may be encountered in the grouted cutoff wall during excavation. Repair of minor deficiencies in the grouted zone can be made by a standby crew for a small cost before the leakage endangers the work. This is a much more economical procedure than overgrouting the entire job to assure 100% coverage. Such standby repair cannot be done so easily in the case of a large dam, where the detection of poor performance is not simple and the difficulties of redrilling through the dam to regrout a deficient zone are tremendously increased.

CHAPTER 3—IMPORTANT GEOTECHNICAL CONSIDERATIONS

GENERAL

Chemical grouting is typically used to solve special construction problems related to geological anomalies or special environmental conditions. Thus, it often requires additional subsurface information not usually obtained in a conventional soils investigation.

As a minimum, a separate interpretation of all available subsurface information should be made with respect to the potential chemical grouting application. In most cases, it will be necessary to perform additional borings with semi-continuous soil sampling, more detailed soil classification of the samples, and additional special laboratory testing. The purposes of this special subsurface investigation and evaluation is to more carefully define the limits and characteristics of the special geotechnical situation to be solved by the grouting process. Equally important is the clear identification of the geological subsurface conditions which will control and permit the success of the grouting program. This must be done to properly select the best grouting approach, including the type of grouts, grout travel ranges related to grout pipe spacing, optimum pumping rates and sequences, and necessary control techniques.

PERMEABILITY

One of the fundamental questions that must be asked when grouting is first considered is whether the ground involved is groutable. All soils are pervious in an absolute sense. A "groutable" soil is one which will, under practical pressure limitations, accept injection of a given chemical grout at a sufficient flow rate to make the project economically feasible. The permeability of sands may vary as much as 3 or 4 orders of magnitude, from 1 cm/sec for medium grained clean sands to as low as 10^{-5} cm/sec for sand containing 25% or more silts and clays. For very low permeability sands, the injection rate at permissible pressures may be so slow that grouting becomes unfeasible. Thus, chemical grouting is recommended only in predominantly sandy materials with less than 25% silts and clays. Incorrect answers to the original query concerning groutability can defeat an otherwise well conceived plan.

Practical injection rates range from about 2 to 20 liters/min, but they can be as low as 1 liter/min and as high as 40 liters/min. Injection rates higher than 40 liters/min become hard to control, and may suggest that less expensive cement grouting should be used. Injection rates slower than 1 liter/min become impractical, since the volume of grout placed per day at this rate, even with a multiple hole injection system, is very low. In addition, low flow rates require unacceptably long gel times to obtain adequate flow time within the soil for practical grout port spacings.

The injection flow rate increases proportionally to an increase in the injection pressure, up to the point when uncontrolled ground fracturing occurs. When fracturing of ungrouted soil extends more than a short distance beyond the point of injection, the adjacent ground will probably not be impregnated properly with grout. The effect is loss of control of the grouting process, and erratic results. Fracturing of grouted soils is another phenomenon and may be necessary to assure complete grout impregnation of the treatment zone.

The pressure at which uncontrolled, widespread fracturing of ungrouted soils occurs represents an upper limit of permissible grouting pressure. In a recent injection test into sand with 10% fines, fracturing did not occur until the injection rate was increased to about 90 liters/min, far in excess of usual injection rates. Fracturing of a porous medium by liquid injection is a much more complex phenomenon than injection fracturing of a relatively impervious material such as clay or grouted soil. The permeability and relative stiffness (compressibility) of the porous formation appear to be very important in this respect.

Initial soil permeability is the primary guide to establishing the groutability of a soil mass. Soils having permeabilities in the range of 10^{-1} cm/sec to 10^{-3} cm/sec are easily groutable. Soils showing permeabilities in the range of 10^{-3} cm/sec to 10^{-4} cm/sec are moderately groutable. When the permeability is from 10^{-4} to 10^{-5} cm/sec, the soil is usually only marginally groutable and may be ungroutable from a practical point of view. Soils with permeabilities above 10^{-5} cm/sec are considered ungroutable.

A preliminary determination of soil permeability, and thus groutability, can be made by measuring the percentage of fines passing a No. 200 sieve. Soils are initially classified as groutable if they have less than 12% fines, moderately groutable if they have from 12 to 20% fines, and only marginally groutable for 20 to 25% fines. Sands are usually considered ungroutable if they have more than about 25% fines. Figure 3 shows typical grain-size ranges for chemically groutable soils.

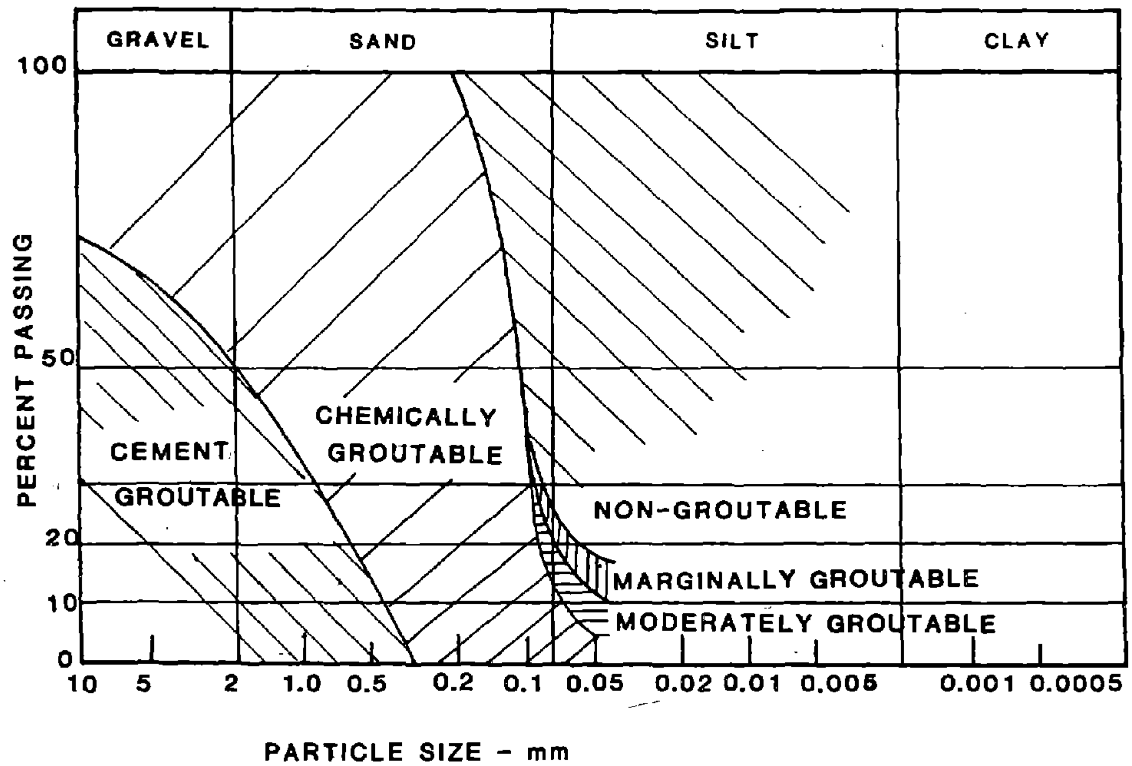


FIGURE 3 - GRAIN-SIZE RANGES FOR CHEMICALLY GROUTABLE SOILS

A more absolute groutability classification can be based on the results of laboratory and field injection tests. The composition of the fines appears to be important. The clay content of fines is more effective than the silt content in reducing the groutability of sandy soils. Where many soil specimens are to be evaluated for the amount of material fines passing No. 200 sieve, it may save considerable time to perform Sand Equivalent Tests (ASTM D 2429) and correlate the results with a few Fines Tests (ASTM D 1140) and Laboratory Permeability Tests.

STRATIGRAPHY

The stratigraphy or the variations in soil materials in the grouting zone is an important controlling factor in the design and effectiveness of the grouting process because ground permeability varies so much between soil types. Thus, it is necessary to have a well-defined picture of the stratigraphy of the area. This will usually require the obtaining of nearly continuous soil samples within the grouting zones. If split-spoon sampling is being performed, at least two 45 cm long drive samples should be obtained for every 75 cm of hole, instead of one sample per 1.5 meters as is the usual practice. Samples should be retained in their entirety for inspection and micro-classification by a geotechnical engineer. Small, fine-grained lenses should be noted, and grain-size tests should be performed on representative samples of separate micro layers. Considerably more descriptive detail should be shown on a boring log for the grouting specialist than is usually shown on a conventional boring log.

The gradation results should be correlated with the stratigraphy. If the total specimen obtained in a split-spoon test is mixed and used to perform a washed sieve analysis, the location of silt layers will be missed. The analysis of grain-size curves should be done in conjunction with a careful understanding of the micro-layering effects present in the soil.

The permeability of the soil in both horizontal and vertical directions should be evaluated in order to predict the relative shape of the grout bulbs. It is common experience to observe elliptically shaped isolated grout bulbs with height to diameter aspect of about 0.80 because the horizontal permeability is greater than the vertical permeability. Soil anisotropy will affect the selection of grout pipe spacings and grout port spacings, as well as the sequence in which primary and secondary holes are grouted.

If unexpected ungroutable lenses occur periodically throughout the design grouting zone, they will control and greatly influence the direction of migration of grout from the grout pipe location. If major ungroutable pockets are encountered frequently throughout the intended grouting zone, their presence, especially if unanticipated, can frustrate the intention of the original grouting program. On the other hand, it is important to determine if the sand occurs in isolated pockets in the zone to be grouted, since the occurrence of sand pockets could limit the ability of the grout to displace the existing groundwater.

Confirmation of the original stratigraphic evaluation can be obtained during the borings conducted for placement of grout pipes. Since wash or blow samples are generally obtained during grout pipe drilling and the drillers may not be experienced in geologic drilling, it is important that they report all observed changes in response to the drilling, including changes in drilling rates and wash water.

GROUNDWATER

Chemical grouting can be performed in pervious soils either above or below the groundwater surface with about equally successful results, provided both the chemical and hydraulic effects of the groundwater are taken into account.

Samples of the local groundwater should be tested for compatibility with the chemical grouts to be used. Groundwater with high pH can be very destructive to sodium silicate based grouts, preventing initial gel formation and/or encouraging grout degradation with time. However, low pH groundwater conditions can accelerate setting of sodium silicate grouts while preventing the setting of acrylamide or acrylate grouts. The presence of organic materials in the ground or groundwater can also have a dramatic effect on the gel times and quality of chemical grouts. Chemical analysis of groundwater is useful in this respect, but should not replace at least one series of grout mixing tests using a groundwater sample in the chemical grout mixture. Of course, additional grout mixing tests should be performed using samples of the actual water source to be used for the job.

As an example of the importance of groundwater evaluation, during the testing program to evaluate the feasibility of a chemical grout cut-off curtain for a large South American dam, all but one of nine chemical analyses of groundwater appeared to be compatible with a sodium silicate grout. The one water sample which showed a very high pH of 11.0 was thought to be contaminated by cement alkaline from the low-grade cement used for cement grouting in the area. Since the proposed chemical grout curtain was to be placed within a previously placed cement-grout cut-off curtain of the same low-grade cement, sodium silicate type grouts had to be ruled out.

In another example, the groundwater below a chemical plant was found to be highly caustic (high pH) due to chemical spillage. For chemical grouting reinforcement of these foundation support soils, it was therefore necessary to select a grout that gelled as pH increased.

During the geotechnical investigation, it is important to establish the directions and rates of groundwater flow, to distinguish between perched water and groundwater, to establish the presence of any artesian pressures, and to estimate the possible effects the injection program will have on the groundwater levels.

POROSITY

In order to calculate the volume of chemical grout needed to treat a given soil volume, it is necessary to have a fairly accurate estimate of the porosity of the groutable soils. Typical groutable soils have porosities of 0.25 to 0.45. For a porosity of 0.35, 350 liters of chemical grout will be required for every cubic meter of soil treated (2.62 gallons/cubic foot). Because a major cost of chemical grouting is the cost of grout chemicals, the porosity has important cost consequences. A correct porosity estimate is also necessary to predict the point at which additional chemical grouting will start to cause heave.

Since more precise data are not usually available, estimates of soil porosity are often obtained from previous correlations with Standard Penetration Test "N" values. Figure 4 shows typical porosity value ranges vs. "N" values. Where relatively undisturbed samples are obtained, unit weight and specific gravity measurements will permit better estimate of soil porosity for use in grout volume calculations.

STRENGTH AND STIFFNESS

Structural chemical grouting involves making load carrying "sandstone" structures (arches, rings, pedestals, etc.) underground. The pregrouting and postgrouting strength and stiffness properties of groutable and adjacent ungroutable soils are needed to design these "sandstone" structures and predict their behavior. A discussion of the properties of grouted soils is given in Chapter 4, Properties of Grouts and Grouted Soils.

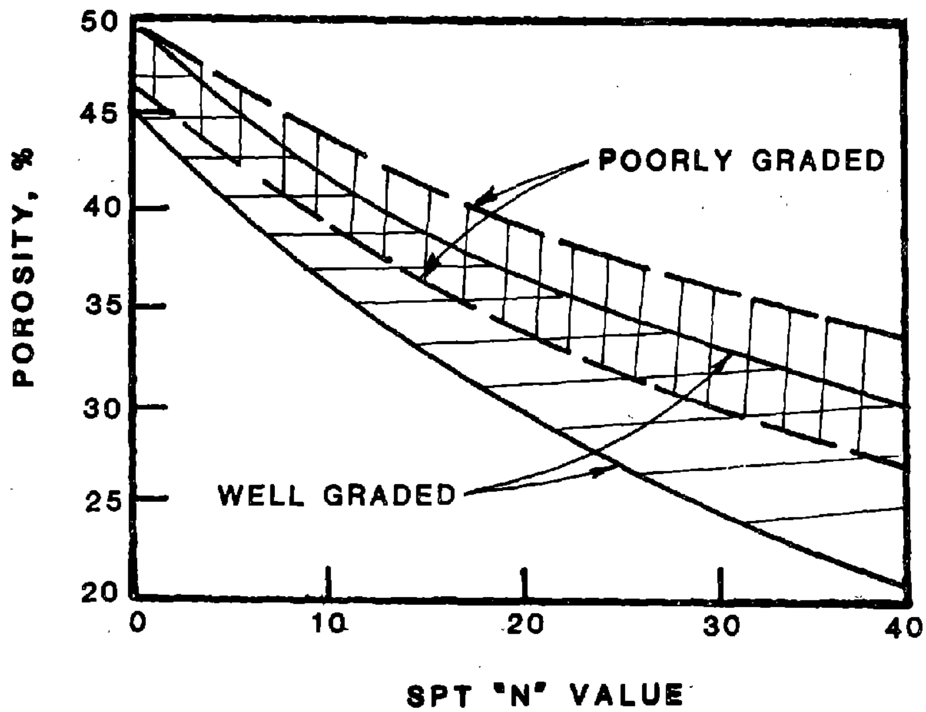


FIGURE 4 - POROSITY RANGES VS. SPT "N" VALUES

ENVIRONMENTAL HISTORY

To define subsurface ground conditions as accurately as possible, it is important to obtain a history of previous construction activities in the area. The presence of old shafts, wells, cisterns, etc., can provide preferred grout migration paths away from the grout zone, rather than into adjacent, less pervious soils intended to be grouted under the job plan. Old topographic maps can be very helpful in piecing together the history.

Quite often, a grouting program is initiated and carried out in order to protect the neighborhood from damages during subsurface construction. Nearby structures may be able to tolerate only a small amount of settlement. The total environment must be studied with respect to the details of the grouting programs, including how the environmental conditions will affect the grouting and how they, in turn, will be affected by the grouting operation. Utility trenches backfilled with gravel or sand bedding materials can provide excellent conduits for migration of the grout away from the intended grouting location.

Grouting technicians and drillers should record every anomaly encountered in the drilling and grouting operations. Such anomalies include a sudden drop in the drill steel rods, sudden increases or decreases in the ease of drilling, sudden fluctuations in grouting pressures (especially after grouting has been proceeding at a particular grout port for some time), and inconsistencies in development of injection pressure with flow rate. These anomalies should be explained and their significance evaluated before conducting any further drilling or grouting.

Consideration should be given to the effect of plugging underground drainage channels and to the additional ground forces which will be created within the grouting zone. For example, an old brick railroad tunnel was protected by chemical grouting from possible subsidence caused by a planned subway tunnel construction below the brick tunnel. Before grouting, the tunnel was serving as a groundwater drainage channel for the area. Grouting to protect the railroad tunnel changed the groundwater regime and raised the groundwater level in the area, resulting in changes in the soil stresses on the tunnel. This had to be considered in the design. It was also important in this case to develop a grout curtain upstream from the tunnel so that subsequent tunneling could be done with as little change in the previous dewatering conditions as possible.

Active drain lines and sewers should be monitored to detect any invasion of grout into these lines. For sodium silicate grouts, this can be effectively done with recording pH meters fitted with audible alarms set to sound when effluent pH reaches a certain level. For silicate grouts, a ten-fold dilution of grout will still result in a substantial pH increase.

Although not generally considered a problem with sodium silicate grouts, some chemical grouts represent toxicity dangers to the groundwater and underground environment. A frequently cited example of this is a case which occurred in Japan several years ago, where an acrylamide grout (free acrylamide monomers have severe neuro-toxic effects) was improperly injected such that it invaded and contaminated a nearby well, resulting in serious health consequences to the users of the well. Lignen or tannin based grouts have been historically gelled with chromium salts and formaldehyde, both of which are toxic. Urea-formaldehyde grouts are considered toxic because of the formaldehyde reactant. Low toxicity chemical grouts are sufficiently available now for most purposes. They should be specified except for unusual circumstances. Sodium silicate grouts are, for the most part, non-toxic. Recently developed acrylate and polyurethane grouts also have very low toxicity.

It is very important that the potential environmental impact of grouts be established on the grout mixture and not on the individual ingredients that are used in the grouts and which are never injected separately.

HYDROFRACTURING

Hydraulic fracturing of the soil mass by a given injection pressure is largely controlled by local effective stresses, strengths and permeabilities. In a real (heterogeneous) soil, hydrofracturing may start at a point of low effective stress and/or low permeability and propagate throughout the zone until it reaches a region of higher effective stress/strength and permeability.

The minor principal stress will determine the pressure at which hydraulic fracturing will occur. Initial hydrofracturing occurs along generally vertical planes. When preconsolidation, either natural or due to previous grouting and fracture prestressing, has raised the lateral earth pressure to be equal to or greater than the overburden pressure ($K_o \geq 1.0$), fracturing may tend to occur along horizontal planes. Thus, heave will eventually occur if regrouting is continued excessively. Distinction should be made between fracturing of previously grouted sands and fracturing of ungrouted, low permeability soils. The former is considered necessary for full grout impregnation of adjacent ungrouted zones. The latter represents loss of control of grouting locations.

SUMMARY

For rational design of chemical grouting, all of the available geotechnical information must be synthesized to define the technical and economical conditions for the accomplishment of the project purposes. The importance of obtaining adequate geotechnical data and the proper interpretation thereof cannot be overemphasized. Selection of the best injection materials and methods is directly linked to knowledge of the job ground conditions.

Important geotechnical parameters related to chemical grouting are shown in Table 1, with the methods employed in establishing their values and typical consequences:

TABLE 1 - IMPORTANT GEOTECHNICAL PARAMETERS

Geotechnical Parameter	Evaluation Method	Consequence
Permeability	Estimate from grain-size analysis; calculate from in-place pump-in tests; and laboratory tests	Determines groutability and injection rates
Micro-stratigraphy	Semi-continuous sampling and visual inspection of samples	Relative shape of grout bulbs; shows preferred grout flow layers
Groundwater	Borehole groundwater readings; piezometer readings; chemical analysis; grout-groundwater gel tests	Influences injection sequences and grout selection
Porosity	Laboratory tests; correlation with density and grain-size data	Determines volume of grout required to impregnate unit volume of soil
Strength and Stiffness	Acoustic velocity profiling; pressuremeter test; SPT; laboratory testing	Deformations under load
Environmental History	Maps, construction excavation observation, inspection pits	Underground anomalies
Injection Fracturing Pressure	Estimate from soil density and permeability; correlate with pressuremeter data; define by injection test with acoustic emission monitoring	Establishes maximum production injection rates for controlled grouting

CHAPTER 4--PROPERTIES OF GROUTS AND GROUTED SOILS

GENERAL

A chemically grouted soil is a "composite material," consisting of a granular soil mass and the chemical void filler that acts as a glue. Grouted soils are always developed in-situ, i.e., the filler (grout) is added to the granular soils by injection as a liquid. Designers need to understand the behavior of the liquid grout during the injection phase and be able to anticipate the behavior of the solid gel (grout) filling the soil voids. They must be able to predict the characteristics of the final soil-grout composite material in the context of time, environment and probable soil variations. In this chapter, liquid chemical grout properties are summarized and the strength, stress-strain and permeability properties of chemically grouted soils are reviewed.

LIQUID CHEMICAL GROUTS

In the early days of chemical grouting, the grout components, the base grout and catalyst, were injected separately in a "two-shot" process. This was necessary since the gel formation reaction was instantaneous with the addition of the catalyst. Therefore, the catalyst was added in the ground. In this early "two-shot" process, concentrated sodium silicate was injected under pressure into the interested zones, followed by the subsequent injection of the catalyst (a calcium chloride and water solution). In the early 1950's, several new chemical grouts were developed which used catalysts or reactants that delayed gel formation long enough to permit premixing and injection of the mixed liquid grout into the desired subsoil zone prior to solidification. Use of these premixed and precatalyzed grouts constitutes the "single-shot" method. Because of the wide availability of reliable "single-shot" grouts, the two-shot method is now virtually abandoned.

In the 1950's there occurred nearly simultaneous development in Europe of single-shot sodium silicate-based grouts and in the United States of single-shot acrylamide-based grouts. Since then a 25-year long worldwide experience with single-shot chemical grouting has developed, stimulating the development of many additional chemical grout formulations and a continually improving application technology. A detailed study of chemical grouts was recently performed by Tallard and Caron (1977).

This two-volume report represents important reference materials for chemical grouting designers.

Chemical Grout Systems

Materials used for chemical grout are typically low viscosity chemical agents which gel after injection into the ground. The properties that affect a grout's injectability, i.e., the ease with which it flows through a porous material under pressure, are not completely understood. Grout viscosity, a measure of shear resistance to shear strain rate, is the main grout property that relates to the injectability of grout in a given soil formation. Surface tension is another important property that has had little industry discussion in relation to its effect upon injectability.

The most common chemical grout is sodium silicate mixed with water and caused to gel by the addition of one of several reactants. In terms of the total volume of chemical grouts employed in geotechnical grouting, it is estimated that sodium silicate-based chemical grouts account for over 90%. Other minor grouts for geotechnical grouting include acrylamide grouts, acrylate grouts, urea-formaldehyde grouts, polyureathane grouts, and resin grouts. A recently introduced acrylate grout (Clarke, 1982) has properties very similar to acrylamide grout without the undesirable neuro-toxic properties that in recent years have resulted in dramatically reduced usage of acrylamide grouts.

Sodium silicate, once called waterglass, is a heavy, syrupy liquid having a pH of 11. Upon reduction of the pH by acidification or saponification, a gel of silicon dioxides and hydroxides 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 slightly silty sand, but not silt or clay. Typical waterproofing chemical grout viscosities range from less than two centipoise for the acrylate and acrylamide grout systems to about three centipoise for dilute (25%) sodium silicate grouts. ASTM Standard D 4016 was recently established for testing grout viscosity.

Reactants

Gel time is controlled by the type and/or amount of catalyst and the amount of accelerator. The change in grout viscosity from the moment of introduction of the reactant (catalyst-accelerator system) until the moment of gel formation is of special interest. Sodium silicate grouts tend to increase in viscosity gradually over time, whereas the acrylate and acrylamide grouts typically remain at about their initial viscosity over most of the time until gel formation, when a dramatic viscosity increase occurs. Figure 5 shows the typical variation in viscosity versus time for silicate, acrylate and acrylamide grouts when left undisturbed.

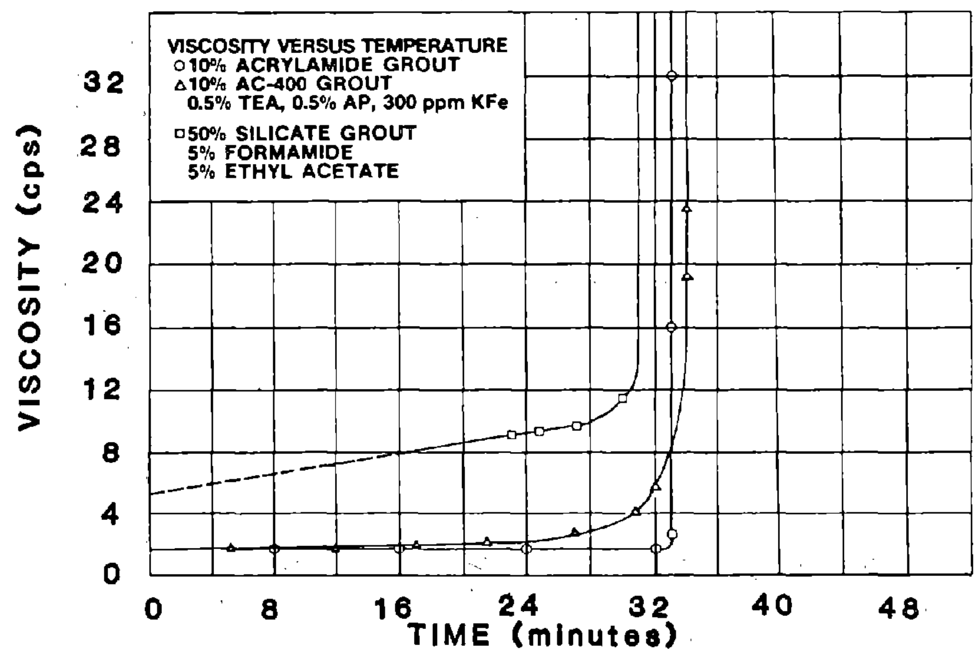


FIGURE 5 - GROUT VISCOSITY VS. TIME

It is well established, however, that sodium silicate grouts, and to a lesser extent the acrylate and acrylamide grouts, have their gel times dramatically increased by continued agitation. Thus, where silicate grout experiences continued agitation because of turbulent flow through the soil, actual gel times in the ground are significantly longer than those taken in cup samples at the ground surface.

Modern silicate grouts can be prepared with reactants that control gel times from a minute to over an hour. SIROC, a once widely used silicate grout, used formamide and dissolved salts as reactants. GELOC-4, TERRASET, and HARDENER 600 mixtures are now widely used non-toxic sodium silicate grouts which use organic diesters as reactants to gel sodium silicate.

Sodium silicate is completely soluable in water. However, most silicate grout reactants do not readily dissolve in water. The addition of surfactants and vigorous mixing to form emulsions are done to overcome this problem. Otherwise such reactant systems may be filtered out in finer sand deposits, resulting in erratic gel times and even ungelled base grout. Filtering out of large silica flocs can also be a problem in fine sands. This problem can be solved by centrifuging this silicate solution to remove large flocs. The flocs eventually plug the porous formation to prevent further grout flow.

PROPERTIES OF GROUTED-SOILS

General

Much progress has been made in recent years in defining the parameters that govern the stress-strain and strength behavior of silicate-stabilized sands (Warner, 1972; Koezen, 1977; Diefenthal, Borden, Baker and Krizek, 1979; Clough, Kuck and Kasali, 1979; Krizek, Beritayf, and Atmatzidis, 1982; Tan and Clough, 1980; and Borden, Krizek and Baker, 1982).

Laboratory Testing

The action of a sodium silicate grout in the voids of a soil can be conceptually described as that of a "glue filler," which bonds soil particles together and initially fills the void volume of the soil. As the grout gels and cures, it undergoes a volume reduction which is directly related to the phenomenon of syneresis (pure silica gel contracts and expels free water). At the same time, the strength and stiffness of the grouted sand vary continuously with time. For example, the unconfined strength and the initial tangent modulus of specimens injected with sodium silicate grout increase with time during the first few days or weeks, depending on the type of grout and the curing environment. It is therefore important from a practical point of view to evaluate the properties of specific soil-grout combinations as a function of curing time.

Injection of sand specimens in the laboratory simulates field conditions only to the extent that coating of the grain-to-grain contacts with grout is prevented. The actual stress path experienced by an in-situ grouted specimen is not accurately simulated during conventional testing. Modeling the in-situ stress history would involve: (a) grouting under confining stresses, (b) curing under confining stresses until the time of sampling, (c) stress relief due to sampling, and (d) testing in the laboratory, possibly under a simulated field state of stress.

Available evidence (Diefenthal, et al, 1979) indicates that the effects of stress history due to sampling are insignificant. Therefore, laboratory injection under at-rest conditions (without the application of confining stress) should yield specimens with mechanical properties that are very similar to those of in-situ grouted specimens, assuming that field density and fabric can be simulated.

The unconfined compression test has been used extensively for evaluating quantitatively the effects of various parameters on the mechanical behavior of grouted sand specimens. Studies using sodium silicate grout indicate that the observed mechanical behavior of specimens injected in split molds in the laboratory depends primarily on the curing time and environment, the silicate content of the grout mix, the rate of strain, the use of appropriate end caps, specimen size, and the grain size and distribution of the sand (see Volume II of this report). Care should be taken to minimize disturbance of the grouted specimens. Do not push or jack the specimens out of their molds or cut or trim their ends. Use split molds whenever possible.

Unconfined compression tests on silicate-grouted sands indicate that the strength, but not stiffness, is dependent on relative density, denser specimens being stronger. Furthermore, strength and stiffness increase with increasing curing time, but this effect is more pronounced during the early stages of curing. The axial strains at failure decrease with increasing curing time and approach an approximately constant value after about one week of curing, regardless of relative density.

Properties of Grouted Sand in Triaxial Compression

Effective Stresses. Grouted specimens have an initial coefficient of permeability which is about two to four orders of magnitude lower than that of the ungrouted sand. However, immediately after injection, silicate grout shrinks with time and interconnected passages develop within the mass of the grouted soil, thereby increasing the permeability. This makes it possible to conduct triaxial compression tests of silicate grouted sands either under drained or undrained conditions. During drained shear, the specimen is allowed to change volume by absorbing or expelling water and excess pore water pressures do not develop. During undrained shear, the volume of the specimen is held constant and pore pressure changes occur. In either case, the specimen should be saturated.

Saturation is usually ascertained by checking the value of Skempton's pore pressure parameter, B, which, for soils with skeletal compressibility that is negligible relative to the compressibility of water, approaches unity as the degree of saturation becomes very high. However, a series of tests on laboratory grouted specimens indicates that the maximum B values ranged between 0.8 and 0.9 and never approach unity. This is because the compressibility of the solid skeleton cannot be neglected relative to that of water. Accordingly, a grouted sand can be considered fully saturated when the value of the B parameter does not change for successive increments of back-pressure and cell pressure in a triaxial chamber.

Volume Change Characteristics. In contrast to the well known different volume change characteristics of ungrouted loose and dense sand subjected to shear stresses under drained conditions, the volume change characteristics of silicate-grouted specimens (regardless of relative density or confining stress) are similar to those of a dense sand. The volume of grouted specimens decreases and reaches a minimum at an axial strain of about 1% to 2% and then increases continually until the point of failure.

Pore Pressure Response. Similarly, regardless of relative density or confining stress, grouted specimens subjected to shear stresses under undrained (no-volume-change) conditions exhibit a pore pressure response which is similar to that of ungrouted dense sand. The pore pressures increase with increasing axial strain and reach a maximum at an axial strain of about 1% to 2%; subsequently, the pore pressures decrease continuously until failure.

The reduction in pore pressure during the undrained shear of grouted specimens leads to pore water cavitation, regardless of relative density or confining stress. As failure is approached with increasing axial strain, negative pore pressures develop in the grouted specimens, dissolved air is expelled from the pore water, air bubbles are formed, the volume of the grouted specimen changes, and undrained (no-volume-change) conditions cease to exist. The volume changes and pore pressure responses characteristic of chemically grouted sands in triaxial shear are shown in Figure 6.

Shear Strength Parameters. The Mohr-Coulomb failure criterion has been found to describe reasonably well the strength behavior of grouted specimens subjected to drained or undrained triaxial compression. The Mohr-Coulomb failure envelopes obtained on the basis of effective stresses at failure yield the same effective cohesion and effective angle of internal friction, regardless of drainage conditions during testing. In general, the magnitude of the cohesion intercept obtained for loose grouted specimens is somewhat smaller than that of dense grouted specimens. This could be attributed to the smaller number of grouted (bonded) grain contacts associated with loose specimens. Grouting has little measurable effect on the angle of internal friction of the sands. Loose and dense grouted specimens exhibit approximately the same angle of internal friction as loose and dense ungrouted specimens, respectively. These effects can be seen in Figure 7.

Development of Shearing Resistance. A limited experimental investigation using drained and undrained triaxial compression tests revealed a definite trend in the development of the Mohr-Coulomb shearing resistance parameters (C and ϕ) of grouted sands as a function of axial strain. These trends are shown schematically in Figure 8. The effective cohesion increases rapidly with axial strain until it reaches a maximum value at axial strain levels between 0.5% and 1%, at which point it decreases and, at an axial strain at about 2%, attains a value which remains practically constant until failure. The effective angle of internal friction also increases with increasing axial strain, but at a rate which is somewhat lower than that for ungrouted sand, and

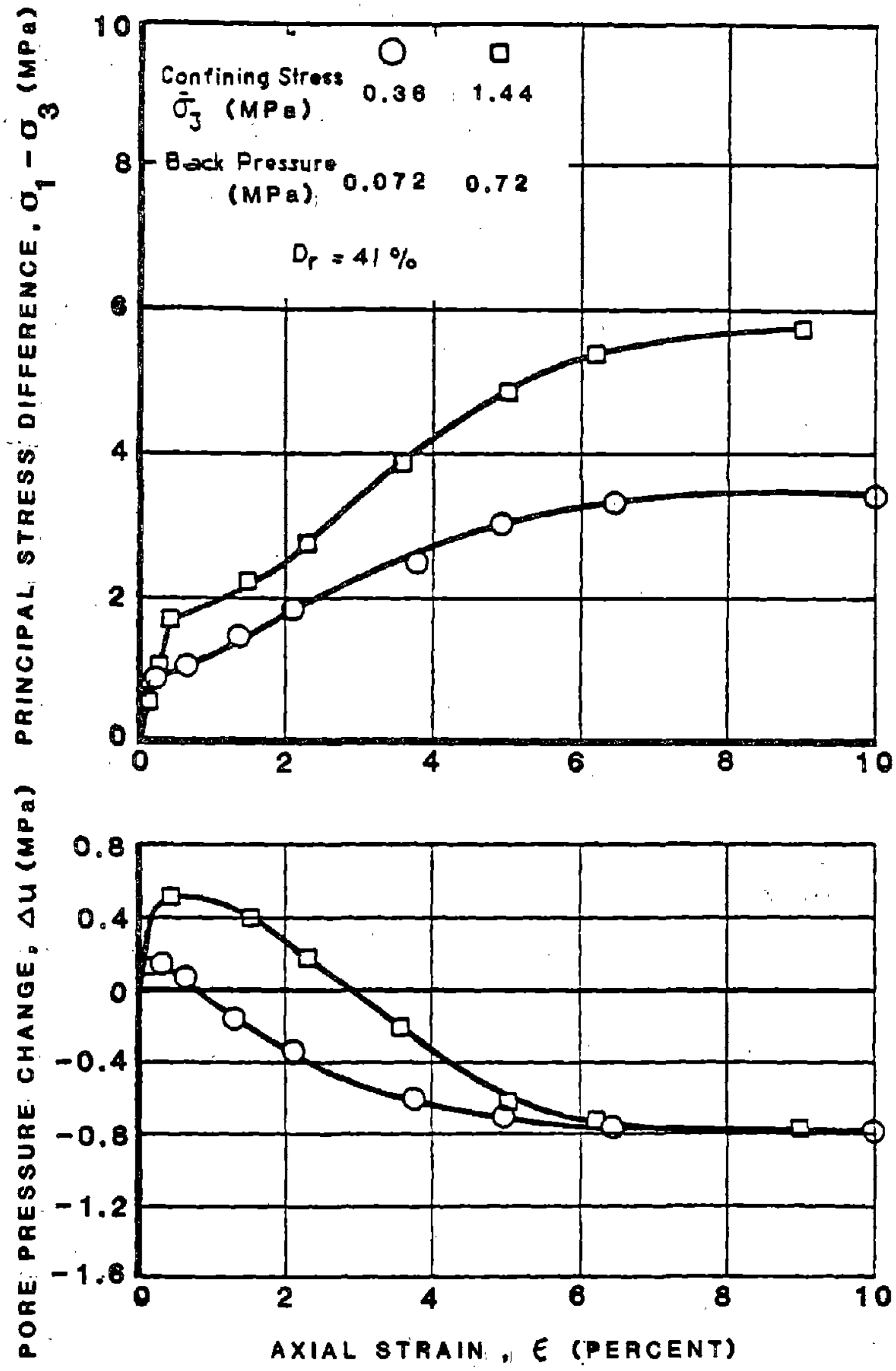


FIGURE 6 - SHEAR STRESS AND PORE PRESSURE RESPONSE FOR GROUTED SANDS

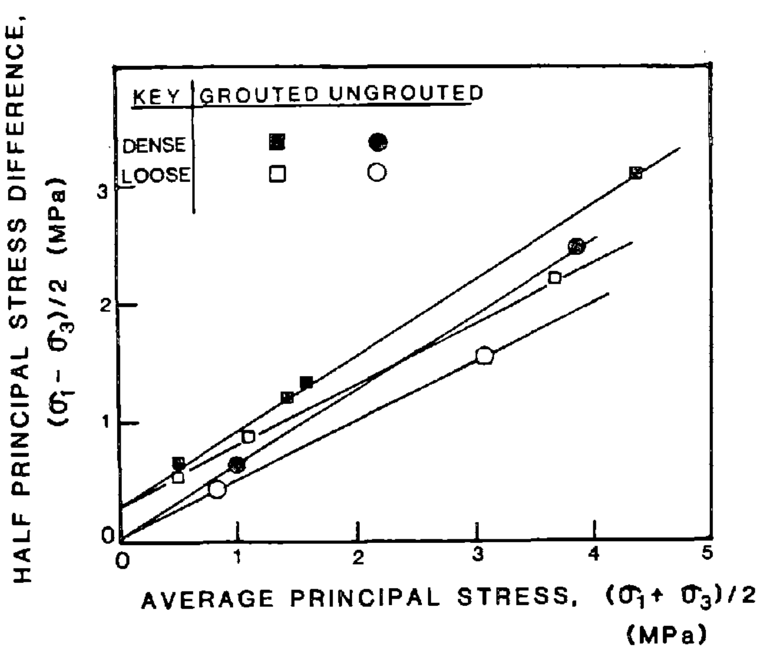
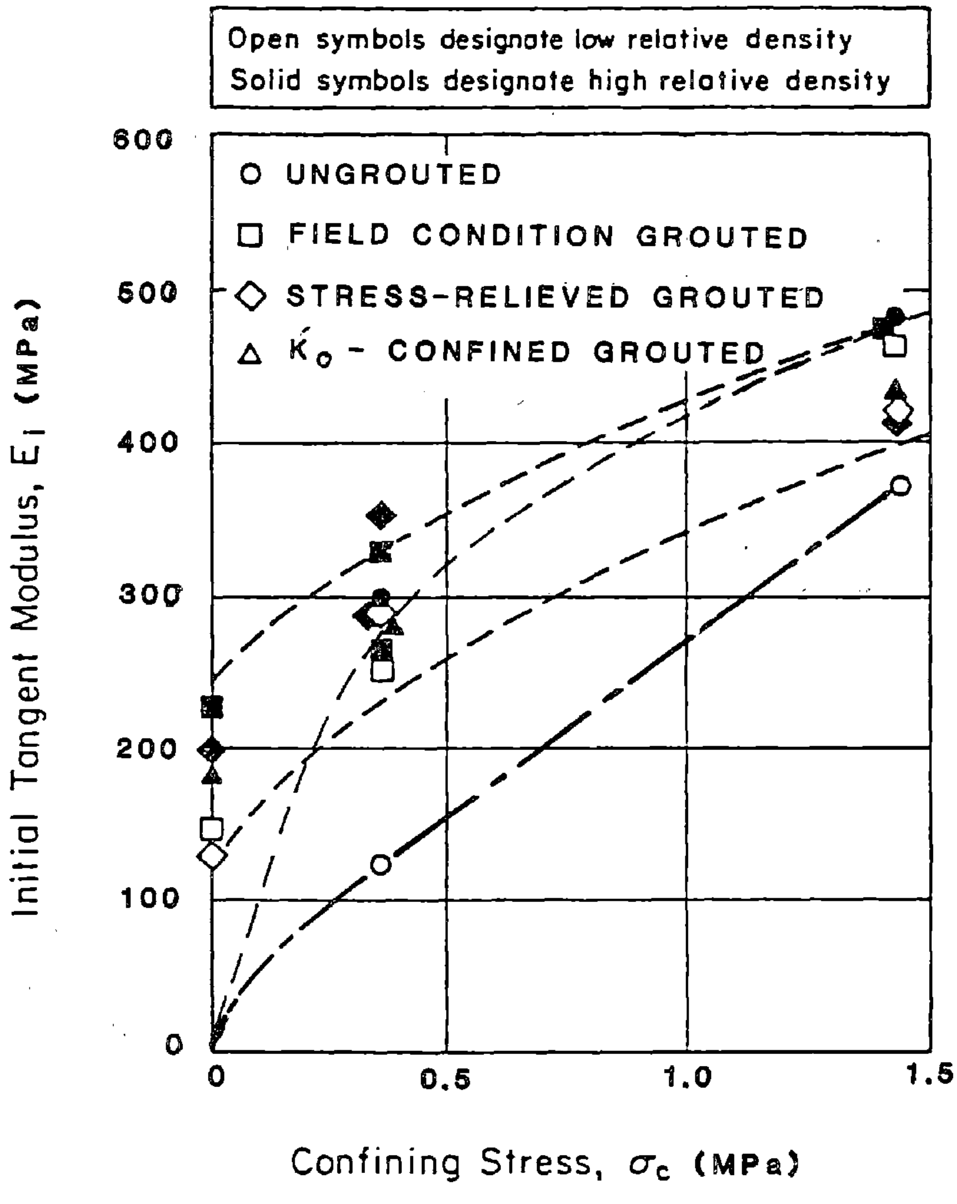


FIGURE 7 - MOHR-COULOMB ENVELOPES FOR GROUTED AND UNGROUTED SANDS

reaches a maximum value at the point of failure. The combined shearing resistance due to cohesion and friction is maximum at the point of failure. These trends for the development of shearing resistance resemble those that have been documented for cohesive soils and appear to be independent of relative density, confining stress, and drainage conditions.

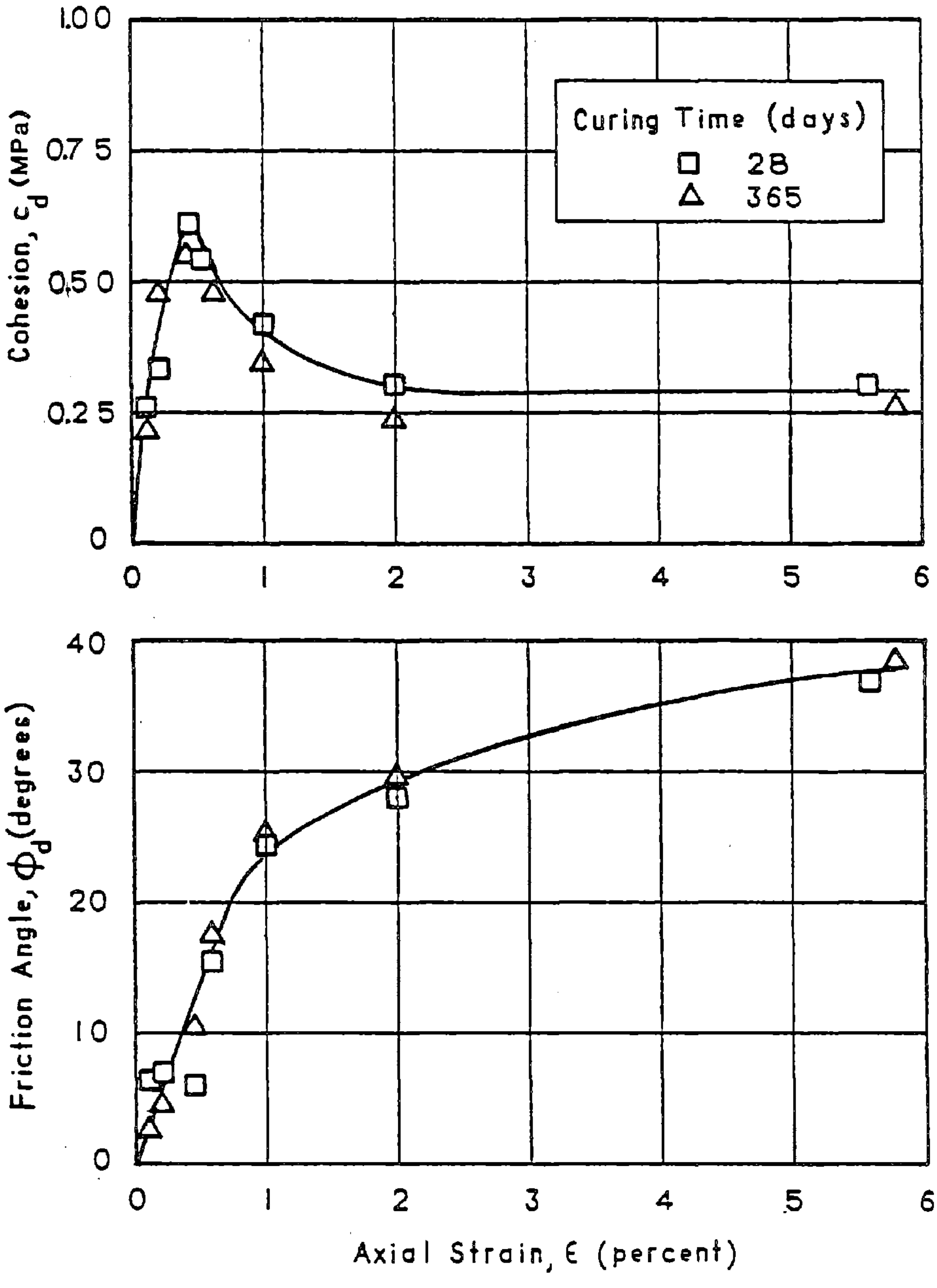
Although drained and undrained triaxial compression tests yield essentially the same values for the effective cohesion and angle of internal friction, the maximum shear strength is not the same. Grouted specimens tested in undrained shear exhibit a maximum shear strength which is larger than that of similar specimens in drained shear. This is due to the development of negative pore pressure during undrained shear causing the effective confining stress to increase substantially as the axial strain becomes large and failure is approached.

<u>Tangent Modulus</u>. The stiffness of grouted specimens can be described quantitatively by using an appropriate modulus. Figure 9 shows the relationship between the confining stress and the initial tangent modulus. Grouted specimens (regardless of the relative density) tested under higher confining stresses exhibit a stiffness approximately equal to that of the ungrouted dense sand . One explanation for the observed



Note: $1 MN/m^2 = 1000$ kPa; $1 kN/m^2 = 1$ kPa

FIGURE 8 - TANGENT MODULUS VS. CONFINING STRESS



Note: 1 kg/cm^2 = 98.07 kPa

FIGURE 9 - DEVELOPMENT OF C AND Φ FOR GROUTED SANDS

behavior is that, when a specimen is subjected to confined compression, the gelled grout in the soil voids has sufficient shear strength and flow resistance to inhibit reorientation and collapse of the soil fabric during the initial stage of a shear test, thus tending to produce similar stiffnesses in specimens with both high and low relative density.

Time Effects. Since the development of the mechanical properties of neat silicate grouts is a time-dependent process, the mechanical properties of grouted soils should logically be a function of time. The strength and stiffness of the gelled grout increase with curing time for up to about one week, thereby increasing the strength and stiffness of grouted soil masses. The results of drained and undrained triaxial compression tests on loose and dense grouted specimens which had been cured in a high humidity environment for one week to up to one year indicate an insignificant effect of curing time on the volume change behavior, pore pressure response, stress-strain characteristics, and shear strength of the grouted specimens after about 30 days.

The limited information available indicates that silicate grouted specimens subjected to unconfined compression creep loading could fail after being subjected for only a few days to a load equal to 30% to 50% of their rapid-loading unconfined compressive strength. This characteristic of grouted sands is of extreme importance in field situations where an unsupported grouted mass may be required to sustain loads for several months. However, in evaluating the time-dependent behavior of grouted sands, care should be exercised to differentiate between the effects of curing and the effects of sustained loads. This can be accomplished by allowing sufficiently long curing times (up to 30 days for sodium silicate) to allow the mechanical properties of the grouted soil to fully develop.

Figure 10 shows the Stress Level Ratio plotted versus the logarithm of time. In the figure it can be seen that the long-term creep strength of unconfined grouted sand may be only 30% of the rapid-loading unconfined strength.

However, when confined even ungrouted sands show considerable strength due to the frictional sand response. Since the frictional component of grouted sand is essentially independent of time or strain rate, it follows that failure of grouted sands will never occur at stress levels below the frictional strength level. Borden, et al

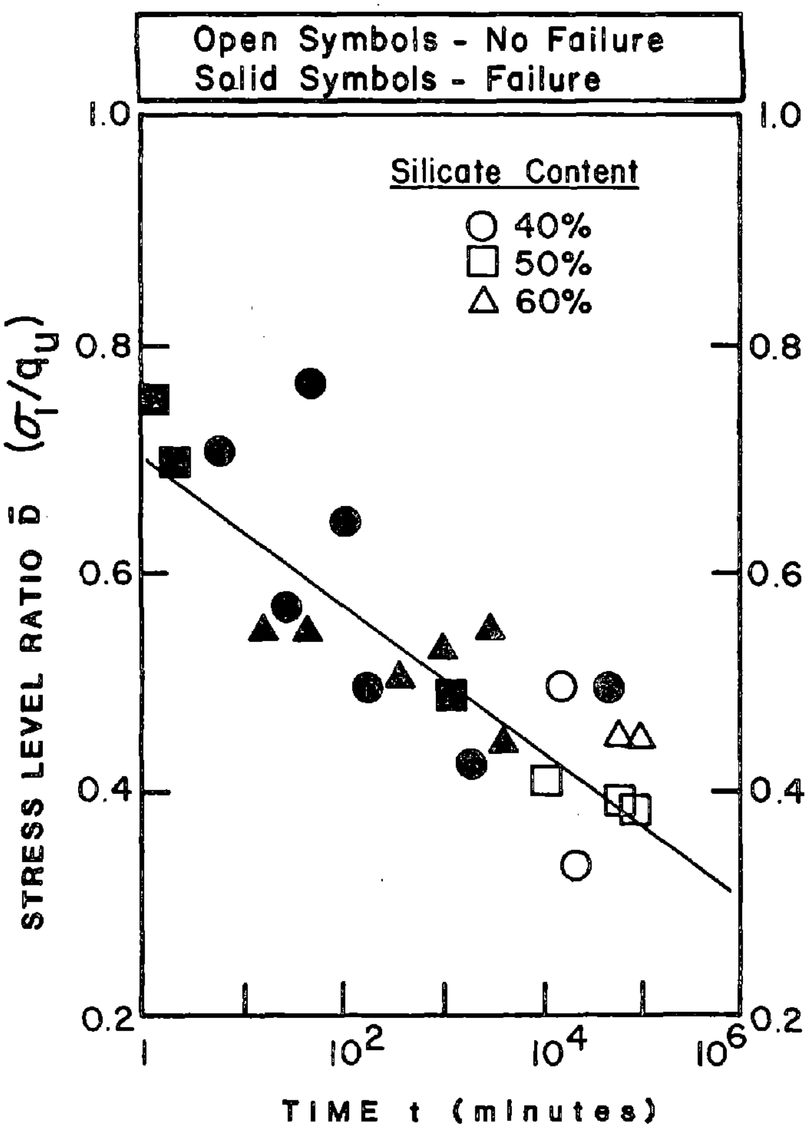


FIGURE 10 - STRESS LEVEL RATIO VS. LOGARITHM OF TIME FOR UNCONFINED CREEP

(1982) have suggested that the time dependent portions of the confined strength of silicated-grouted sands may be viewed in terms of the ratio of the applied stress in excess of the frictional strength (excess stress) to the unconfined strength. This ratio, termed the Modified Stress Level Ratio, is shown in Figure 11.

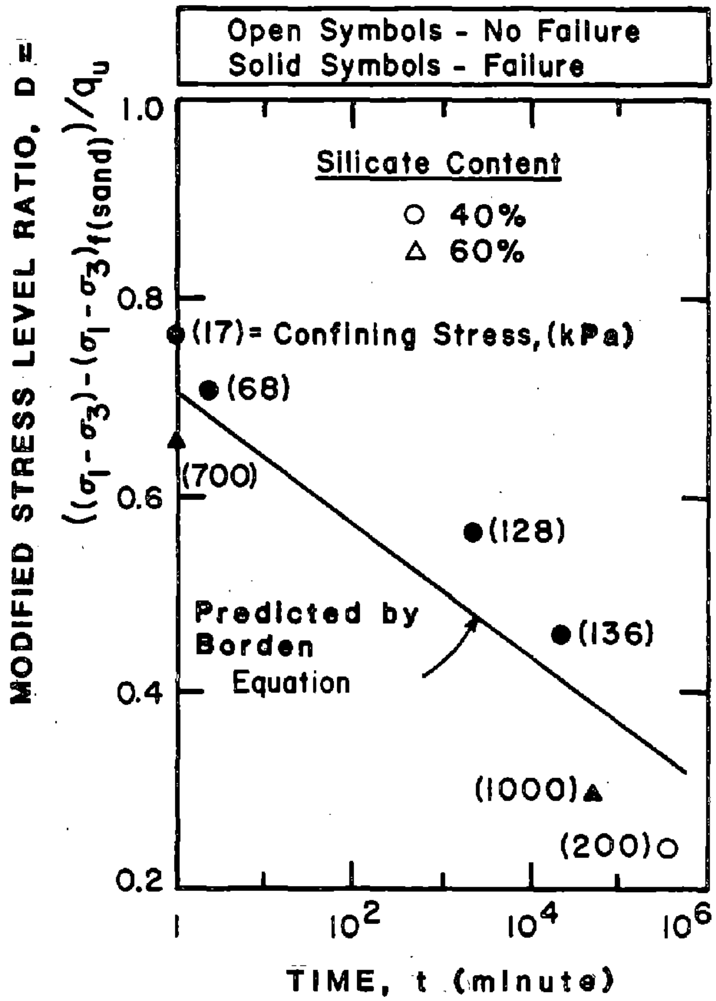


FIGURE 11 - MODIFIED STRESS LEVEL RATIO VS. LOGARITHM OF TIME FOR CONFINED CREEP

SUMMARY

A grouted mass in the field is usually subjected to a three-dimensional state of stress and is frequently located below the groundwater table. The behavior of a saturated grouted mass under loading depends on the rate of loading and prevalent drainage conditions. Drained and undrained triaxial compression tests are considered to represent two extremes in the behavior of silicate grouted masses. In general, the grouted mass has a tendency to dilate for rates of loading higher than the rates of drainage; pore pressures decrease because of dilation of the sand and associated cavitation of the pore water; and temporarily high effective confining stresses and shear strength develop. Eventually, as pore pressures dissipate, the behavior of the grouted mass is realistically represented by its behavior measured in a drained triaxial compression test.

Unconfined compression tests conducted under carefully controlled specimen preparation, handling, and testing conditions can be used when performing parametric studies. However, the results of such simple and expedient tests cannot yet be used with confidence to predict the behavior of grouted soils under more complicated stress fields. For this reason, triaxial compression tests should be used to evaluate more realistically the mechanical behavior of grouted specimens. Every effort should be made to test saturated specimens to facilitate the interpretation of the results in terms of effective stresses. Drained triaxial tests simulate the long-term behavior of a grouted soil mass better than undrained tests. Undrained tests with pore pressure measurements offer the advantage of expediency and yield stress-strain and strength characteristics that are similar to those obtained from drained tests. The development of cavitation must be properly taken into account. Finally, appropriate consideration must be given in design calculations to the significant reduction in the creep strength of chemically grouted soils relative to the strength measured in an unconfined compression test conducted in a normal quick test.

Chemically grouted soils are composite materials with mechanical properties that depend on the properties of the individual components (neat grout and sand) and the interaction between them. For practical engineering purposes, it can be said that (a) the mechanical properties of grouted soils develop as a function of grout curing time; (b) the observed mechanical properties of grouted soils depend on the type of test conducted; (c) Mohr-Coulomb failure criteria is a useful method of representing the strength of silicate-grouted soils; (d) every effort should be made to simulate field conditions during a laboratory investigation of grouted soils; and (e) time effects should be taken into account when describing the strength and stiffness of grouted soils.

CHAPTER 5—PERFORMANCE PREDICTION METHODS

INTRODUCTION

The rational design of chemical grouting requires some prediction of the project performance, including evaluation of soil stresses and strains. In the past decade, several large chemical grouting projects were carried out for American subway construction (Clough, Baker, and Mensah-Dwumah, 1979; Ziegler and Wirth, 1982). These projects have largely been designed on the basis of geometric intuition, with no analytical evaluation of soil stresses and strains, resulting in apparently large but actually unknown degrees of conservatism. This underscores the fact that until the recent work by Tan and Clough (1978, 1980) performance prediction procedures for structural chemical grouting around tunnels were not generally available.

A prediction of performance of chemically grouted structural soil masses requires the following:

1. A useful model of the construction environment and sequence;

2. A stress-strain-strength model of the grouted and ungrouted soils;

3. An understanding of the structural behavior of the chemically stabilized mass in resisting loads and deformations; and

4. Validation of the approach by comparison of analytical results with case histories.

The first three of these four aspects of performance predictions are discussed in this chapter. Tan and Clough's procedure for settlement prediction of chemically grouted tunnels is specifically discussed in the section on Structural Behavior of Grouted Masses. Several case histories were compared to analytical predictions by Clough, Baker and Mensah-Dwumah (1979).

CONSTRUCTION PROCEDURES

The structural behavior of the chemically grouted zone is dependent to a large extent on the particular construction excavation procedures used. Consider, for example, the frequent case of chemical grouting underpinning of a conventional footing to permit adjacent excavation to several footing widths below the bottom of the footing. If the excavation is to be made in one lift with no lateral bracing to the footing or cut face, the role of the chemically grouted mass would be to provide total vertical support to the footing and lateral support to the retained soils. This case is illustrated in Figure 2-b. Due to the reduced creep strength of chemically grouted soils, it is important to know how long the area will be left exposed before final lateral support is provided.

On the other hand, if the footing is located within an area that will be laterally supported by conventional soldier piles and lagging, the role of the grouted mass may be reduced to providing limited vertical footing support near the cut face and to prevent any loss of ground (and associated loosening of formation support soils) during the actual lagging process. In this case, the amount of laterally unsupported ground would be limited to one lagging lift between soldier piles. Long-term strength would not be a factor. This situation is shown in Figure 2-a. It is apparent that for this case the role of the chemically grouted mass is much less critical than in the former example.

When chemical grouting is used to reduce settlements for tunnels in sandy soils, as shown in Figure 2-c, the specific tunneling procedures used are important. Face control, use or nonuse of a shield, size of overcutting bars, and tail shield thickness, among other things, all affect the maximum potential for ground movement. Tail void grouting or liner jacking procedures also are important in determining the standup time required of the grouted zones prior to the performance of backpack grouting. If a portion of the tunnel is below the groundwater table and no dewatering has been provided, the chemically grouted mass will be expected to provide water control as well as structural ground support.

STRESS-STRAIN-STRENGTH MODELING OF SILICATE-STABILIZED SANDS

Much progress has been made in recent years in defining the parameters that govern the stress-strain and strength behavior of silicate-stabilized sands (Warner, 1972; Koezen, 1977; Clough, Kuck and Kasali, 1979; Diefenthal, Borden, Baker and Krizek, 1979; Tan and Clough, 1978 and 1980; Borden, Krizek and Baker, 1982; and Krizek, Benitayf, and Atmatzidis, 1982). This subject has been treated briefly in Chapter 4, Properties of Grouts and Grouted Soils.

The principal points resulting from these studies that relate to stress-strain and strength modeling are:

1. The variation of shear strength with normal stress generally follows the Mohr-Coulomb cohesion and friction angle criteria. The friction angle ϕ of the grouted sand is practically the same as the ungrouted sand. The presence of the silicate grout is primarily reflected in the addition of a cohesion component of strength to the soil.

2. Above about one atmosphere of confining pressure, the medium strain stiffness increases with confining pressure in about the same manner and following similar values to that of an ungrouted dense sand.

3. The stress-strain response is non-linear, with ductility increasing as silicate concentrations are increased.

4. The proportions of strength and stiffness related to the silicate grout component are time dependent, decreasing in value as loading rates decrease. Thus, creep occurs under sustained loading and creep rupture occurs under high load levels exceeding the frictional strength.

5. For a given silicate grout-sand combination, all of the above are functions of grout curing time and conditions.

Performance predictions employing a time-independent grouted soil response in the analysis should use "operational" soil-grout parameters , i.e., ones which reflect possible changes over a given performance period (Tan and Clough, 1980, p. 107). The operational shear strength for a grouted underpinning system is always less than that determined by a rapid laboratory strength test. Tan and Clough (1980) have modified non-linear stress-strain soil models to describe silicate grouted sands and employed these models for finite element analysis of the chemically stabilized soft-ground tunnel problem. The basic parameters for silicate-grouted sands used in these finite element models are shown in Table 2.

TABLE 2 - BASIC GROUTED SAND PARAMETERS FOR SILICATE-GROUTED SANDS USED IN FINITE ELEMENT ANALYSES BY TAN AND CLOUGH (1980)

Sand Density	Relative Designation	Operational Unconfined Comprehensive Strength, q_u kPa (psi)	Ratio of Stiffness of Grouted to Ungrouted Soil K_r*	Friction Angle ϕ, Degrees
Loose	Weak	60 (8.7)	1.50	36
	Medium	150 (21.8)	2.25	36
	Strong	300 (43.5)	3.50	36
	Very Strong	480 (69.6)	5.00	36
Medium	Weak	125 (18.1)	1.50	38
	Medium	315 (45.7)	2.25	38
	Strong	630 (91.4)	3.50	38
	Very Strong	1010 (146)	5.00	38
Dense	Weak	265 (38.4)	1.50	40
	Medium	660 (95.7)	2.25	40
	Strong	1320 (191)	3.50	40
	Very Strong	2110 (306)	5.00	40

*Stiffnesses are defined in terms of the initial tangent modulus at a confining pressure of one atmosphere.

For time-independent analysis, they employed a modified hyperbolic model, (Duncan and Chang, 1970), in which they made adjustments for soil-grout stiffness values at low confining pressures and accounted for small tensile strengths. For the much more complicated time-dependent analyses, Tan and Clough have used the empirical Singh-Mitchell model (Singh and Mitchell, 1968).

STRUCTURAL BEHAVIOR OF GROUTED SOIL MASSES

During excavation, in a typical structural grouting application, various ground stresses, foundation loads, and hydraulic (groundwater) forces are applied to the chemically grouted mass, which in turn is expected to limit ground and/or foundation deformations and possibly control water flow.

For the case of a circular tunnel opening in chemically grouted sand as shown in Figure 2-c, Tan and Clough (1980) have shown that the grouted zone acts primarily as a compression ring, absorbing most of the stress changes caused by the excavation. Using their model in an appropriately configured finite-element network analysis, the effects of grout thickness, grout strength, tunnel depth, groundwater, ungrouted zones, etc. were studied. From the results of numerous different case analyses, simplified design charts were developed that permit rapid initial selection of grout strength and thickness to control soft ground tunnel subsidence using silicate-grouted masses. The analyses assume liner support of the tunnel, a limiting maximum inward soil movement around the tunnel, and full heading stability. Based on an observed correlation between the average level of mobilized strength (Average Principal Stress Difference or APSD) in the grouted zone to the surface settlement, Tan and Clough developed the following approach:

STEP 1 -- Select an Operational Unconfined Compressive Strength of the silicate-grouted sands as weak, medium, strong or very strong, based on the values shown in Table 2.

STEP 2 -- Determine the APSD for the chemically stabilized zone as a function of tunnel depth, d, tunnel diameter, D, stabilized zone thickness, t, and relative stiffness (strength value) of grouted to ungrouted soil. Depth-normalized APSD values (NAPSD) for a given tunnel diameter of 7 meters are shown in Figure 12 for various grout thickness to tunnel diameter ratios. APSD values for actual tunnel crown depths are obtained by multiplying the NAPSD values given in Figure 12 by the actual tunnel crown depth in meters.

STEP 3 -- Define the ratio of the APSD value with the assumed Operational Unconfined Compressive Strength of the grouted soil as the Mobilized Strength Index. Finite element correlations of Mobilized Strength Index values with maximum surface settlement, S_{max7}, are shown in Figure 13, for dense, medium and loose sands. These curves are for a tunnel diameter of 7 meters. For a different tunnel D, adjust the S_{max7} values for 7 meters by:

$$S_{max} = 0.03D^{1.8}\left(S_{max\ 7}\right)$$

Where S_{max} is the surface settlement in millimeters and D is the tunnel diameter in meters.

Using these correlations as outlined above, the specific preliminary design procedure for a single, chemically-grouted tunnel consists of the following steps:

1. Classify the sand masses to be grouted as loose, medium, or dense.

2. Select a trial value of Operational Unconfined Compressive Strength and categorize according to Table 2.

3. Select a trial value of grout-zone thickness, t.

4. Based on the trial grout-zone properties, find the corresponding depth-normalized average principal stress difference (NAPSD value) from Figure 12. (APSD = NAPSD x Tunnel crown depth in meters)

5. Find the APSD value by multiplying the NASPD by the actual tunnel crown depth in meters.

6. Calculate the Mobilized Strength Index (MSI), which is the ratio of the APSD to the trial Operational Unconfined Compressive Strength.

7. Determine the maximum ground surface settlement, $S_{max\ 7}$ for a 7 meter diameter tunnel from Figure 13.

8. Adjust the maximum ground surface settlement for the actual tunnel diameter by the Equation $S_{max} = 0.03D^{1.8} S_{max7}$

9. Evaluate the acceptability of the predicted maximum ground surface settlement, S_{max}, for the assumed grout properties and thickness. If too large, try increasing the grout zone thickness or increasing grout strength, or both. If much smaller than tolerable, try reducing the grout zone thickness or grout strength.

10. Determine the required short-term grout strength for use in specifications.

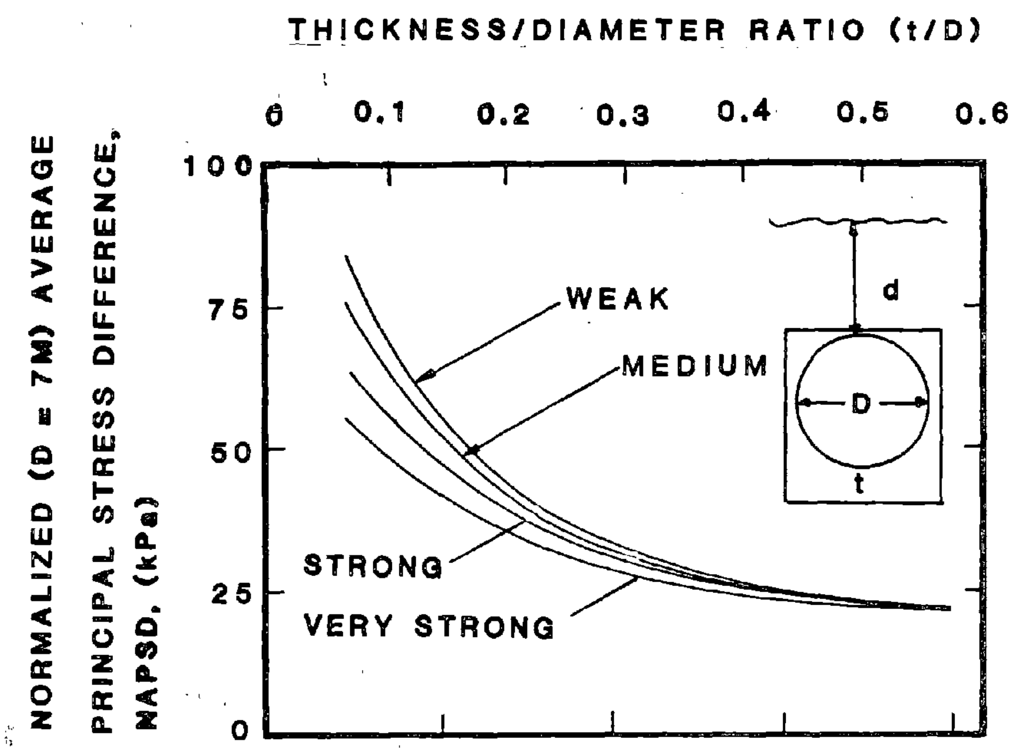


FIGURE 12 - NORMALIZED AVERAGE PRINCIPAL STRESS DIFFERENCE (NAPSD) VALUES VS. GROUT THICKNESS/TUNNEL DIAMETER RATIO

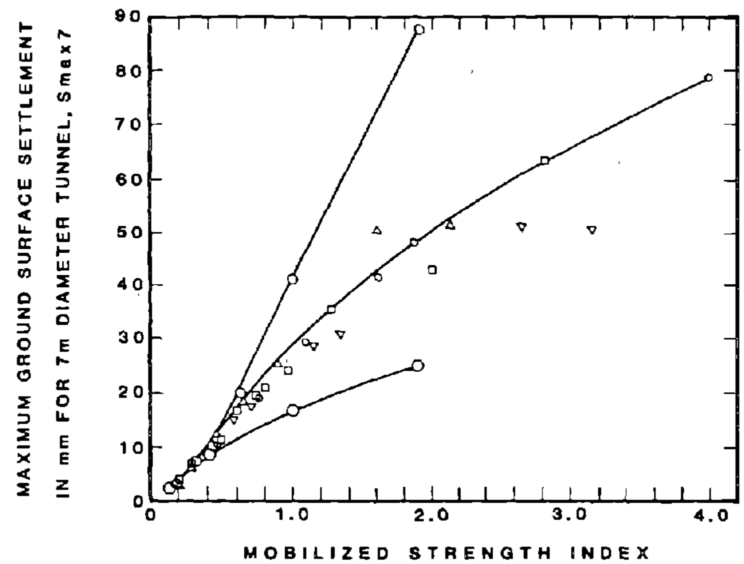


FIGURE 13 - MAXIMUM SURFACE SETTLEMENTS VS. MOBILIZED STRENGTH INDEX VALUES

In many practical cases, the presence of ungroutable layers within the idealized grout zone may have important effects on the behavior of the grouted system. Obviously, the properties of the ungroutable layers are not changed or improved by grouting. In Figure 14, the relative effects of variable thickness clay layers (ungrouted zone) at the crown, springline, and invert are shown. This figure and actual tunnel grouting experience indicate the following:

1. The location of a clay layer or ungrouted zone at the invert, regardless of thickness, has little effect on settlement.

2. Location of a clay layer (ungrouted zone) at the crown or the springline can substantially increase settlements, depending on the clay layer thickness.

3. With a clay layer at the crown or springline approaching the tunnel radius in thickness, increasing or stiffening the grouted zone will have little effect in reducing settlements.

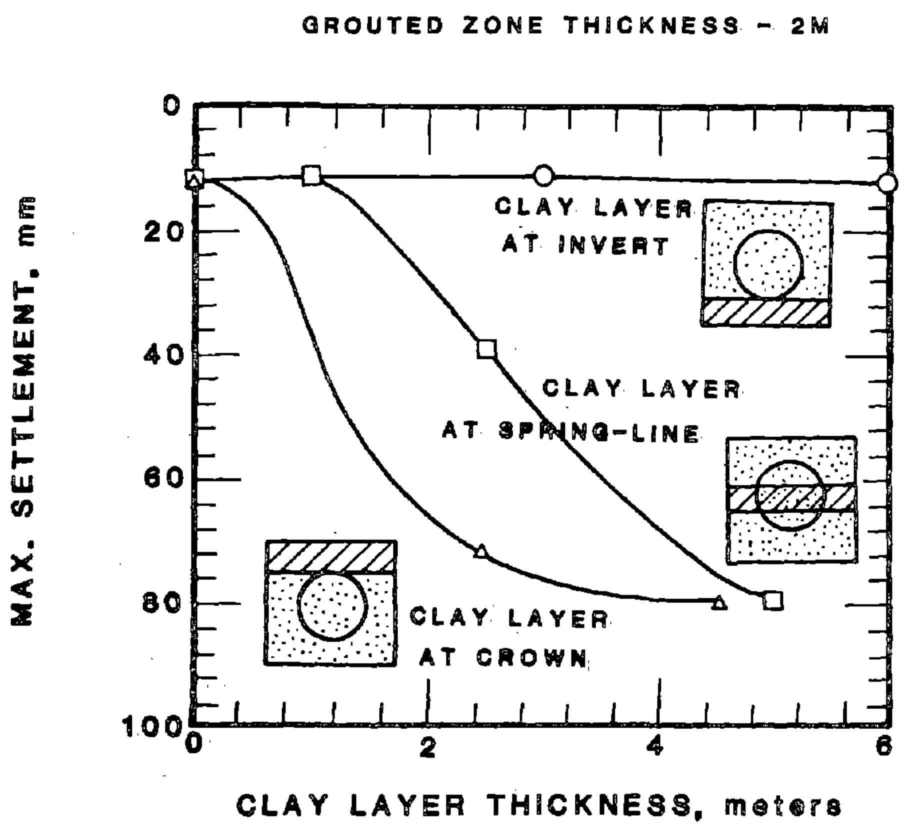


FIGURE 14 - EFFECT OF UNGROUTABLE ZONES ON SURFACE SETTLEMENTS

The above chart-based preliminary design procedure for the selection of size, shape, and strength of chemically stabilized soil zones is the first analytically-based design methodology for chemically grouted tunnels. The interested reader is advised to study the original article (Tan and Clough, 1980) in detail before using the method.

CHAPTER 6—INJECTION PROCESS PLANNING

GENERAL

It is assumed at this point that the general design steps for a chemical grouting program have already established the required properties of the grouted soil mass and selected a grout. The next step is planning the injection process.

Planning the injection program requires a fundamental understanding of how the liquid grout flows in the porous soil from the injection point. With this understanding, the planning aspects for the actual injection portion of a chemical grouting program involve the following four major steps: (1) Definition of the shape and size of the grouted zone; (2) Estimation of the total liquid grout volume required for injection of the grouted zone; (3) Definition of the grout pipe layout, including location, spacing, and pipe installation scheme; and (4) Establishment of an injection staging and sequencing procedure, including the indication of the order in which the various pipes and grout ports will be injected and the partitioning of the estimated grout volumes among the individual grout pipe ports.

In practice, the above four injection planning steps are followed cyclicly through several iterations, as the selection of injection stages affects the grout volumes apportioned to individual injection ports, which in turn affect previously selected grout pipe spacings, which in turn affect the actual grout zone shape, etc.

INJECTION CONCEPTS

Fluid chemical grout injected into pervious granular soil under pressure will permeate the soil following the paths of least hydraulic resistance. This means that grouted three-dimensional masses form radially around each injection point. Their shapes are irregular spheroids or ellipsoids (called grout bulbs) for the first stage of grouting. Where a very pervious channel is intercepted, a much larger proportion of grout will be found. Moving water will also displace the grouted mass in the direction of flow. The most important factors affecting the location of the grouted zones are the <u>locations and arrangements of injection points</u>, which in turn are a function of the grout pipe location and type. Grouting sequence, i.e., the order in which the various grout points are injected, can also be very important in determining where the grout goes.

The final grouted soil mass is produced by the geometrical arrangement of small, contiguous grouted masses. Second and third stages of grouting are often performed to fill in the small ungrouted zones between the original grout bulbs. Gel time, grout viscosity, pumping pressure, water flow, and soil anisotropy all affect the eventual grout distribution.

The injection of chemical grouts into granular soils for either structural grouting or waterproofing purposes is generally expected to follow the basic principles of Darcian flow through porous media where the flow rate, v, for a given soil and permeant, is considered to be directly proportional to the pressure gradient, i, such that

$$v = ki$$

The proportionality constant, k, is termed the coefficient of engineering permeability. Darcy's Law has generally been found valid for laminar flow of water through sands. It has been suggested (Huang, Borden, and Krizek, 1979) that for laminar flow conditions the engineering permeability of the medium can be related to the effective grain size, D_e, the soil porosity, n, the viscosity, u, and unit weight, w, of the permeant, by the equation

$$k = CnD_e^2\ w/u$$

where C is a constant, probably related to the shape of the sand particles. For fine or silty sands, chemical grout flow is expected to be generally laminar, except perhaps very near the grout pipe at high injection pressures. Where these two equations are approximately valid, the following general phenomena will be observed for chemical grouting in permeable soils:

1. Grout will flow generally radially away from the source of injection pressure. This means that no preferential direction of flow is possible, and that grout "spheres" tend to be developed from point injection sources such as sleeve-ports. Grout "cylinders" develop from line sources such as result when a grout pipe is withdrawn from the ground. Tangentially spaced grout spheres leave about one-half the affected zone ungrouted. Thus, the integration of these grouted spheres and columns into a continuously grouted mass of the desired shape requires (a) an imaginative distribution of grout volumes at the various injection points, and (b) an intelligent sequencing of the various grout stages.

2. The rate of grout injection at any given time will be directly proportional to the injection pressure. Increasing the injection pressure is thus an obvious means of increasing productivity and reducing the labor costs of grouting. Practical limits to the maximum permissible grouting pressure result, however, when uncontrolled ground fracturing occurs due to excessive injection pressures and rates.

3. The rate of grout injection will vary directly with the apparent permeability of the soil.

4. The apparent permeability will vary inversely with the viscosity of the grout.

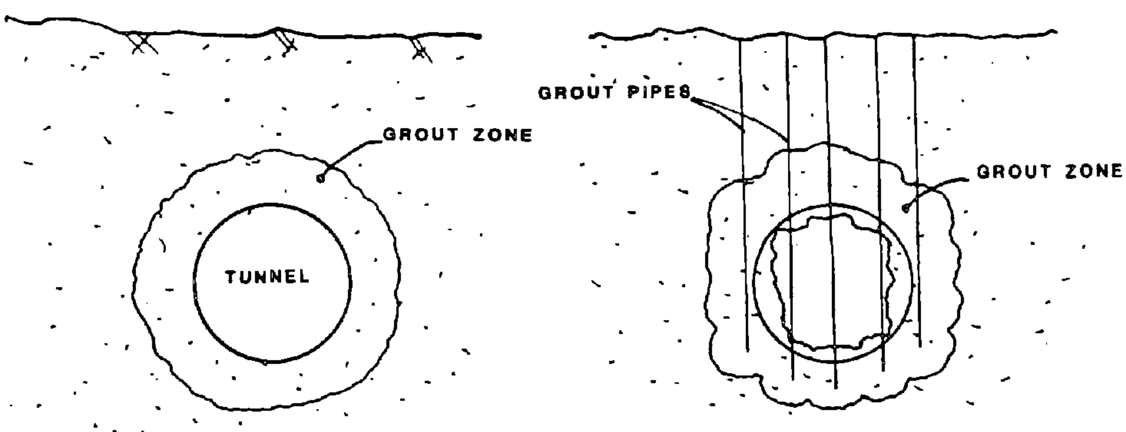
5. The apparent permeability of sand will vary directly with the square of the effective grain size (D_{10}).

GROUT ZONE GEOMETRY

The first step in planning the actual injection phase of a chemical grouting program is to establish the idealized shape and size of the chemically grouted zone. This is established in relation to the associated construction and excavation details of the project. For each trial grout zone geometry, an estimate of the grouted soil-structure interaction response is required.

Where chemical grouting is used to reduce settlements caused by the construction of large-diameter (subway) tunnels, for example, an initial estimate of the desired thickness of the grouted soil around the tunnel is needed. For underpinning of shallow footings adjacent to an open-cut excavation, the idealized cross-sectional shape and depth of the underpinning pedestal is needed. When planning for a groundwater cutoff floor, the thickness and extent of the floor must be estimated.

In Figure 15-a, a typical idealized cross section of the required grouted zone for a subway tunnel is shown, as frequently appears in construction specifications. In Figure 15-b, the actual grouted cross section is shown, as dictated by the location of a typical grout pipe array placed from the ground surface. The difference between the two is due to the fact that the grout ports are located linearly along the grout pipes, and it is not possible to truncate a grout sphere that invades the ground beyond the idealized cross section. The actual grouted cross-sectional shape exceeds the idealized shape and requires more grout than the neat dimensions of the idealized shape would indicate.



(A) IDEALIZED GROUT ZONE (B) ACTUAL GROUT ZONE

FIGURE 15 - IDEALIZED AND ACTUAL GROUT ZONES

ESTIMATION OF LIQUID GROUT VOLUMES

The volume of liquid grout required to chemically solidify a given volume of granular soil is generally calculated based on the assumption that the grout must fill almost all of the soil voids. It is assumed that the grout flows radially away from the injection point, and that the grout flow is continuous, leaving no ungrouted zones in its path, and that some grout will penetrate beyond the idealized faces of the grout zone because of the generally spherical shape of the grout bulbs. This is represented in the equation

$$\text{Liquid Grout Volume} = V_z(nF)(1 + L)$$

where V_z is the total volume of the treatment zone, n is the soil porosity (0.25 to 0.50), F is the void filling factor (0.85 to 1.0), and L is the grout loss factor for grout placed outside the treatment zone (0.05 to 0.15). Thus the liquid grout volume can range from 22% to 57% of the total soil volume to be treated, or a factor of over two and a half.

The porosity of granular soils may vary from 25% for dense, well-graded silty sands to almost 50% for loose, uniform sands. The actual porosity within these extremes depends on the grain-size distribution and the relative density of the deposit.

In practice, it has been found that typical sodium silicate-based grouts fill from a minimum of about 85% of the void volume up to nearly 100% of the void volume, depending on various factors. Generally, the lower percentage of void filling occurs for well-graded sands with fines, where only a single stage or limited second stage of grouting is performed under low pressures. Some correlation seems to exist between injection pressure gradients and the percentage of void filling that occurs in the grouting process. Although the liquid grout volume estimate could be in error by as much as 15% because of an incorrect assumption of the percentage of void filling, by far the greatest error will be caused by an incorrect estimate of the actual soil porosity.

It is also necessary to estimate the volume of grout that will be lost beyond boundary of the expected grouted cross section. This grout loss factor grout may vary from about 5% to 15%, depending upon the shape of the grouting zone, the frequency of injection points per unit volume, and on the presence of highly porous layers within the groutable soils.

GROUT PIPE LAYOUT

Spacing

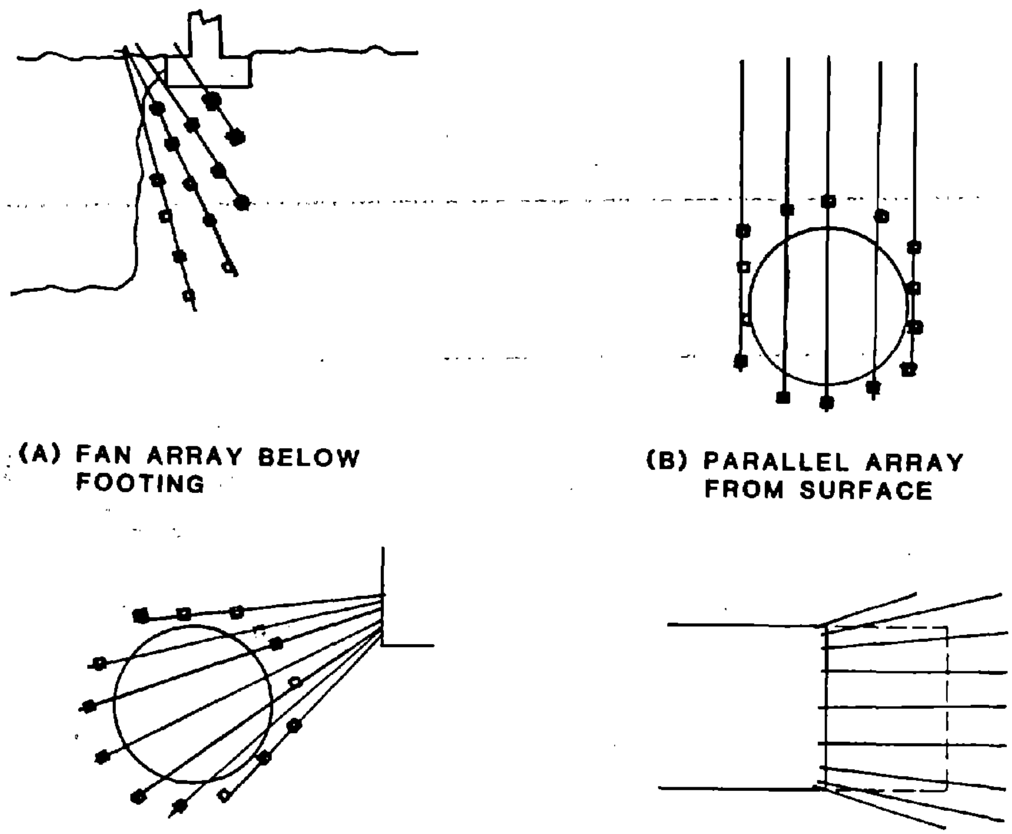
Grout pipe spacing is arranged to provide for primary and secondary injection locations with secondary locations generally falling halfway between the primary locations. Grout pipes spaced too closely to each other will result in excessive drilling costs. Grout pipes spaced too far apart will require extremely long pumping times to place the necessary volumes of grout at each location, with loss of control of the grouting operation as the location of the grout front becomes unknown. In practice, spacing between grout pipes varies from a mimimum of about 0.5 m to a maximum of 2.5 m, with most grouting projects having pipe spacings on the order of 0.8 to 1.5 m.

The simplest grout pipe array occurs when pipes can all be placed parallel to each other. In many cases, however, available working space and construction constraints require grout pipes to be placed in a fan array. This results in considerable additional drilling and pipe placement requirements since near the fan apex, pipes are too close together for all to be used effectively. Figure 16 shows different group pipe layouts required by specific site conditions.

Grout Pipe Installation

Next to the proper location of the grout injection point, the most important factor affecting the successful distribution of grout in the intended grout zone is the proper mechanical grout pipe installation. The grouting engineer anticipates that grout under pressure will flow evenly in all radial directions away from the grout injection point, filling the voids in the porous granular media. Actually, the liquid grout under pressure will flow along the path of least resistance, and any open channel left by the grout pipe installation procedure will permit the grout to flow away from the intended injection zone. Thus, success requires that grout pipe installation procedures permit no unexpected flow channels to remain within the borehole.

Many simple grouting projects have been successfully completed by using the expedient grout pipe installation method of temporarily sealing an open-ended pipe with a drive point, driving the pipe to the desired grouting depth, removing the point, and injecting grout while withdrawing the grout pipe. As the grout pipe is withdrawn, it is assumed that all of the grout is entering the ground at the elevation of the grout pipe tip. After the pipe has been withdrawn some distance during the injection process leaving an open channel in the ground, this likely is not the case. Control of the injection location has been lost especially if a more porous strata is encountered and the grout can travel to it along the space created by withdrawal of the pipe.



(A) FAN ARRAY BELOW FOOTING

(B) PARALLEL ARRAY FROM SURFACE

(C) FAN ARRAY FROM PIT

(D) FAN ARRAY FROM SUCCESSIVE HEADINGS

FIGURE 16 - GROUT PIPE LAYOUT PLANS

These problems can be partly resolved by the use of needle pipes, long tapered drive pipes with grout holes located near the pipe tip and which are driven successively downward as grouting progresses. The needle grout pipe is advanced downward through continual driving rather than withdrawn, and can provide effective control over the actual injection location.

Graf (1978) reported that good grout location control has been obtained by grouting through an undersized soil bit while it is being rotated and driven down. The grout serves as the actual flushing fluid, and the cuttings block the hole and stop the upward flow of grout, providing an effective injection point within a few inches of the drill bit. The drill is actuated and moved downward periodically, with no escape of grout to the ground surface. If any loss of pressurization is encountered, a slug of bentonite is temporarily used as a drilling fluid to block the upward flow along the pipe.

A common drawback of all of these pipe-driving procedures is that during second and third stage grouting, the ground has become considerably stronger and the grout pipes may actually encounter refusal in the previously grouted zones through which they must penetrate.

All of the above problems are effectively resolved by the use of sleeve-port grout pipes, sometimes known by the French term, "tube-a-manchette." This system consists of a 25 to 50 millimeter (1 to 2 inch) diameter pipe that has small, periodically spaced grout holes drilled through the pipe wall at the preferred grouting locations. The grout holes are in turn covered by snugly fitting rubber sleeves, which act as one-way check valves when grout is injected one port at a time from the inside of the pipe out into the adjacent soil. The entire rubber sleeved grout pipe system is installed in an oversized borehole. The annular space between the sleeve-port pipe and the borehole wall is then sealed with a brittle mortar to prevent movement of grout along the grout pipe. This sheath of mortar is fractured when the sleeve is expanded during grouting from the inside of the pipe. An internal double packer pushed ahead of a separate small diameter grout pipe is then used to locate the sleeve-ports one by one, and to inject the desired quantity of grout at each sleeve-port location. The sleeve-ports can be injected in any sequence or may be reinjected as desired. The sleeve-port system is shown schematically in Figure 17.

The resulting excellent control that is possible over grout injection locations using the sleeve-port system generally justifies the additional installation expense and effort required. Actual project economies may even result, because the sleeve-ports can be regrouted without subsequent pipe installation cost and a primary-secondary grouting effect can be obtained in a single grout pipe by alternating sleeve-ports during the two stages of grouting. Initial high injection pressures are necessary for a few brief moments to crack the mortar sheath around the sleeve-port and initiate the flow of grout into the ground. The exact grouting pressures acting in the ground cannot be measured at the grout pipe header, because of the grout pressure dissipated in keeping open the sleeve-port. This "cracking pressure" can be estimated by plotting the flow rate versus pressure curve for the hole during initial stages of grouting and tracing the flat portion of the curve back to zero flow. The pressure intercept at zero flow then approximates the pressure dissipated within the sleeve-port. In Figure 18, a view of a grouting face with the sleeve-port systems installed and multiple pipes being grouted simultaneously is shown.

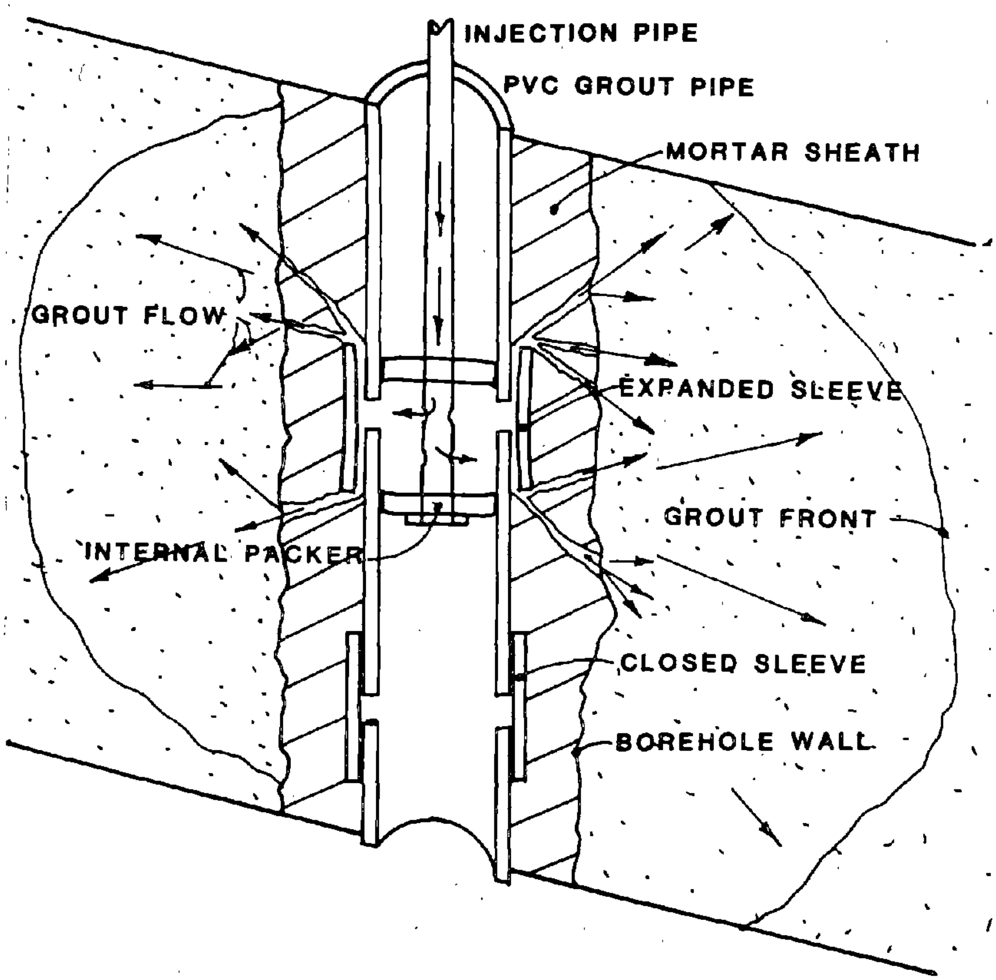
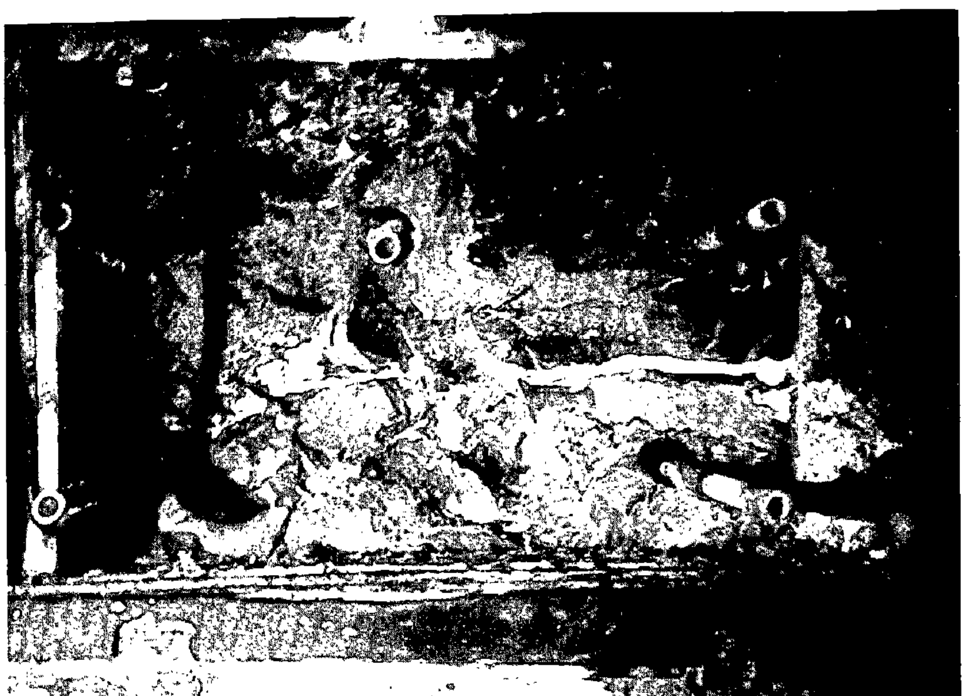


FIGURE 17 - SLEEVE-PORT GROUT PIPE

(A) MULTIPLE PIPES INJECTED FROM PIT



(B) SLEEVE-PORT PIPES EXPOSED IN TUNNEL HEADING

FIGURE 18 - VIEWS OF SLEEVE-PORT GROUT PIPES

INJECTION STAGING AND SEQUENCING

"Injection staging" refers to separate episodes of grouting done at different times in the same zone. "Injection sequencing" refers to the sequence or direction in which grouting occurs. Primary stage chemical grouting refers to the initial volume of grout placed in essentially ungrouted zones where the grout spheres or ellipsoids do not contact each other or contact adjacent previously grouted zones only over a small proportion of their boundary surface. Secondary stage chemical grouting refers to the next grouting episode following primary grouting in the same zone. Tertiary, quaternary, etc., stages refer to the third, fourth, etc., subsequent grouting episodes in the same zone.

If the initial injection spheres were to exactly touch, they would occupy 52.4% of the theoretical cubes in which they are contained. Primary injection is usually carried out so that adjacent grout sphere radii overlap some 10% to 15%. Injection spheres that overlap adjacent cube spaces by 10% occupy about 70% of their cube space. It is a typical practice to inject about 65% to 75% of the planned total liquid grout volume in the primary injection port and to inject the remaining 25% to 35% of the total grout volume in secondary and tertiary ports. After primary grouting, large ungrouted spaces still exist in the intended grout zone. Secondary grouting is usually intended to inject all but the final 5% to 10% of the anticipated grout volumes. After secondary grouting, a very large proportion of the intended grout mass is fully impregnated with hardened grout.

This grout volume distribution is depicted in Figure 19. Tertiary grouting then necessarily requires careful selection of grout pipe ports so as to be near any ungrouted zones. During secondary and tertiary grouting, it is usually necessary to provoke localized fracturing of previously grouted ground, to permit migration of the liquid grout from the injection point to nearby ungrouted regions. It should be noted that for structural grouting where no waterproofing effect is needed, complete tertiary grouting may neither be necessary nor cost effective. Tertiary grouting is often used as a form of check to verify that the major proportion of a zone has been effectively treated.

In grouting above the water table, injection sequencing does not have the same importance it does when grouting below the water table where trapped water can prevent the migration of the grout to the ungrouted zones. Injection below the groundwater table must be done in such a way that escape locations are always provided for groundwater to be pushed or "herded" ahead of the advancing grout front. Injection grouting must proceed methodically either from one side completely across to the other side of a grouted mass, or from the center outward, from the top down, or any such sequence that will finally expel any groundwater from the interior of the grouted mass. If injection sequencing is improperly done such that water is trapped in the center of the grout zone, then increasing grout pressures will incorrectly indicate complete grouting, without the full use of the anticipated grout volumes.

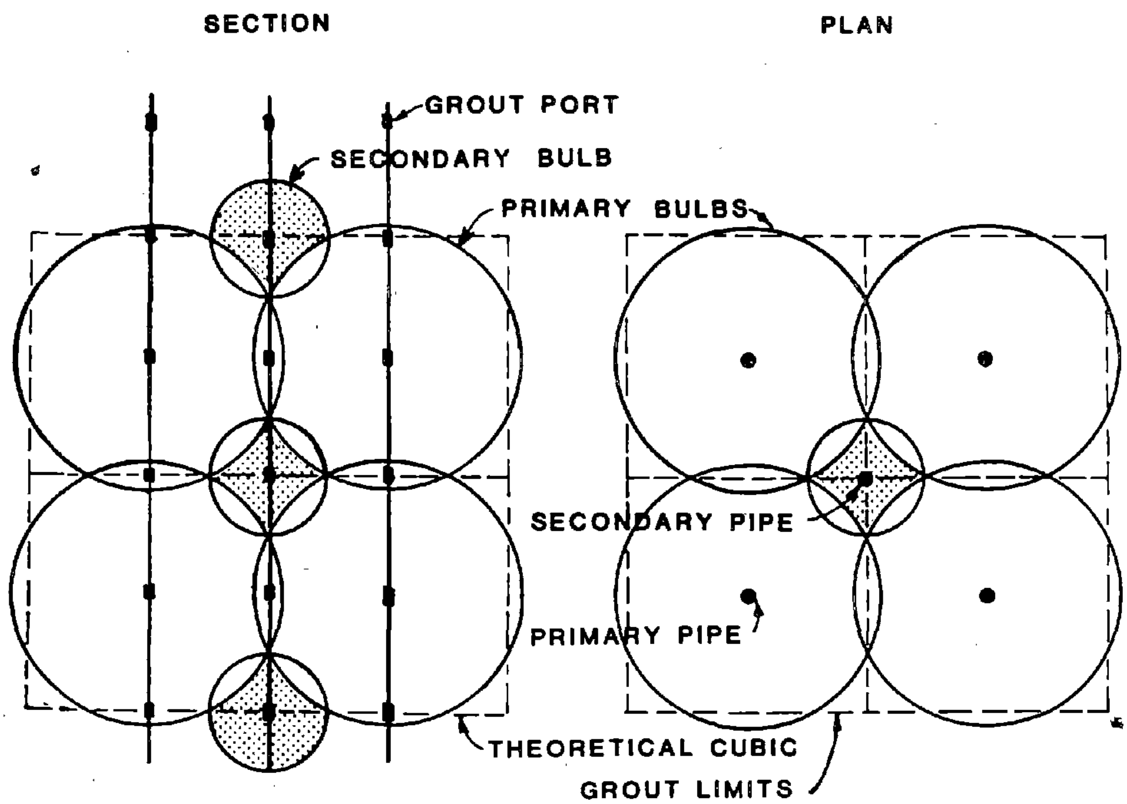


FIGURE 19 - GROUT DISTRIBUTION AT PRIMARY AND SECONDARY GROUT PORTS

SUMMARY OF INJECTION PROCESS PLANNING

Planning steps before actual grout injection include:

1. Establishment of grouted zones in idealized shapes.

2. Establishment of preferred sleeve-port grout pipe arrangement to cover idealized shape.

3. Adjustment of idealized shape to reflect actual grout pipe locations.

4. Adjustment of sleeve-port spacing for soil anisotropy and layering effects.

5. Calculation of soil volume to be treated and estimation of total required grout take using estimated soil porosities and grout loss factors.

6. Distribution of grout volumes between separate sleeve-ports and injection stages.

7. Establishment of grouting sequences to avoid trapping groundwater.

8. Adjustment of design grout volumes in the field during initial grouting according to observations, including indications of injection pressure increase, grout refusal, heave, and fracturing.

CHAPTER 7—MONITORING AND EVALUATION

INTRODUCTION

The most rationally designed chemical grouting plan is eventually subjected to the reality of field injection practice. The real time verification of the field injection activities is termed Quality Control. After injection, typical questions arise concerning the boundaries of the grouted zones, the completeness of treatment, and the properties of the injected soils. Post-injection evaluation of the grouted soils is termed Quality Assurance. This chapter details the quality control and quality assurance functions of chemical grouting.

Recent developments in electronic grouting procedural controls provide for much improved quality control for chemical grouting of soils, and recent applications of geophysical profiling techniques to chemically grouted soils permit more comprehensive quality assurance programs. These improvements permit designers to take a more confident approach to the achievement of design objectives and should result in both reduced costs and superior technical results. Through improved quality control and quality assurance, both the designer and the owner/client can be assured that construction is satisfactory and that design objectives have been achieved.

QUALITY CONTROL

The designer can reasonably require that a competent specialty chemical grouting contractor provide the following:

1. Grout pipes are accurately placed and properly installed.

2. Grout components are properly formulated and thoroughly mixed to give the required neat grout mix and gel times.

3. Grout volumes are accurately injected as planned to the specified grout ports, in a logical sequence and with acceptable flow rates and pressures.

4. Injection process data are recorded and used as feedback to determine that grout is where and in the condition it is expected to be.

5. Through final quality assurance testing performed after completion of the injection work, grouted soil acceptance criteria are satisfied.

The above specific items should be verified in the quality control plan.

The accurate measurement of grout flow rates, injection pressures and total grout volume with time per injection location are fundamental quality control requirements of any chemical grouting project. In conjunction with the known geometrical array of grout port locations, these data can be used to infer the location and behavior of the grout underground. Grouting flow rates, pressures and volumes should be used in real time by the grouting technician to decide whether to reduce or increase flow and pressure at any given moment and to decide when to end injection altogether at a given port. In addition, if properly documented and displayed, the flow rate, injection pressure, and grout volume histories can be used to review the contractor's activities and responses to dynamic field conditions at a future date. Thus, any adjustments to the design program can be based upon a clear picture of what has been accomplished to date. It is clear that any quality control test can be used as a quality assurance test, given appropriate documentation. For example, a strip-chart recording of injection pressure and flow rate versus time, annotated to show injection point and date, and properly filed, is both a quality control and a quality assurance tool.

Specifications should clearly require that a detailed grout monitoring program be submitted by the grouting contractor for approval by the Engineer. For important projects, automatic electronic recording of injection data should be required. The detailed equipment and methods for measuring grout flow rates, pressures and volumes and recording these items are usually left up to the contractor; however, they should be measured, recorded and analyzed continually during grouting and must not be optional.

Grout Pipe Verification

The accurate location and correct placement of sleeve-port grout pipes are so critical to the success of the injection process that special efforts should be made to verify this work. As a minimum, selected grout pipes should be plumbed to confirm that they are being installed to the depths shown on the as-built drawings. Selected water injection tests should also be performed to verify that the sleeve-ports are located at depths anticipated and that cracking pressures and injection pressures are as anticipated. Finally, grouting and inspection personnel should be very observant during the actual injection work to notice any surface leakage that may occur around grout pipes, indicating improper installation and sealing of the annular space around the pipe. This improper sealing problem can sometimes be solved by letting the leaking grout gel, thus reestablishing the seal.

Grouting Systems

Grouting systems are distinguished by the particular combinations of grout mixing methods (batch vs continuous) and grout injection methods (open pipe vs grout ports) employed. Typically, each grouting contractor is limited by his equipment and experience to one or two of the four system combinations available. Because control and evaluation depend upon the particular grouting method used, these factors should be considered when establishing the Quality Control and Quality Assurance plans.

Chemical grouting by continuous mixing uses metering or proportioning pumps and totalizing meters for grout components. 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 45 to 90 minutes, whereas gel times used with the continuous mixing systems are usually 10 to 30 minutes. Silicate grout gel times are lengthened by agitation in the ground when high injection flow rates are used. Combining the continuous mixing system with short gel time avoids the formation of large pools of ungelled grout in the ground or loss of grout migrating downward away from the design zone.

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 shown in Figure 20. The sandy material 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.

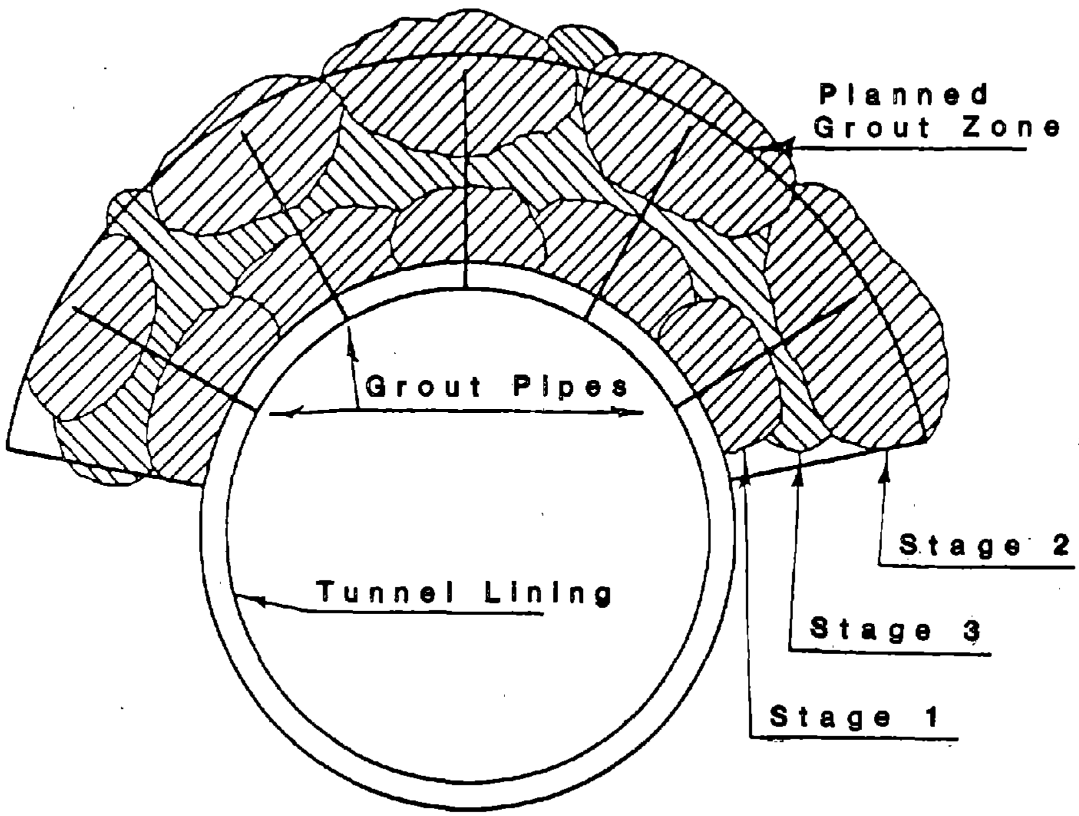


FIGURE 20 - CROWN GROUTING WITH SHORT GEL TIME

Further argument for short gel time grout is provided by Karol (1968), 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 Measurements

Volume. Accurate volume measurements of individual grout components are required to confirm proper grout mix proportions and to calculate total grout volumes for pay items. For both batch and continuous mixing type grout plants, positive displacement meters should be used for this purpose. Conventional water meters or waffle-plate meters cannot provide required accuracy for the variable flow rates and viscosities involved. Meters should be provided with filters and be protected from overpressuring.

Pressure. Electronic transducers convert liquid pressures to electical analog or digital signals for use with electronic recording systems. Transducers accurate to 3 kPa (0.4 psi) and rated to 3,000 kPa (440 psi) are available so that overpressuring is not a problem.

Where continuous electronic recording is not done, pressure measurements should be made by bourdon tube pressure gages. Indicated pressures should be recorded periodically on a data sheet. Bourdon gages will not register below about the lowest 5% of their range. Thus, high pressure gages must not be used to measure low grouting pressures. Positive gage protection against plugging by gelled grout and against overpressuring during grout port cracking must always be provided.

Flow Rate. Grout flow rates at individual grout ports must be measured in an easily interpreted form. This preferably involves an electronic direct reading device such as an acoustic or magnetic flowmeter that permits strip-chart recording.

Also used for this purpose are flow-column meters, mechanical turbines, or positive displacement meters. Back calculation of flow rate based on the rate of liquid drop in chemical tanks is not considered accurate enough for good control. Flow rate measuring devices must be constructed so as not to have dead spots where grout can gel and be trapped, to later break loose and jam flow paths.

Continuous Monitoring. Injection pressures and flow rates should be continuously monitored. The grouting technican should not have major duties other than monitoring the injection process itself. When manual systems are used, pressure and flow rate records are made for each grout port whenever injection pressure changes more than 25 kPa (3.6 psi) or flow rate changes more than one liter per minute occur, or at least every 10 minutes. For simultaneous multiple hole grouting, special automatic recording equipment should be used (Mueller, 1982).

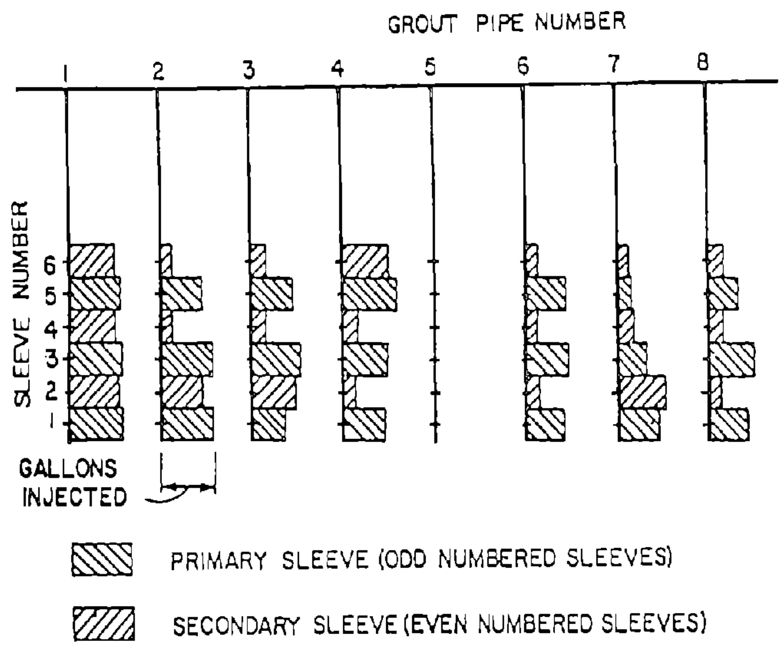
Data Recording and Evaluation

The accurate measurement of grout flow rates, injection pressures and total grout volume with time per injection location are fundamental quality control requirements of any chemical grouting project. In conjunction with the known grout port locations, these data are used to infer the location and behavior of the grout underground. Grouting flow rates, pressures and volumes are used in real time by the grouting technican to decide whether to reduce or increase flow and pressure at any given moment and to decide when to end injection altogether at a given port.

Specifications usually require that a detailed grout monitoring program be submitted by the grouting contractor for approval by the Engineer. The detailed methods for measuring grout flow rates, pressures and volumes and recording these items are usually left up to the specialty grouting contractor.

Graphical Grout Take Log. An important adjunct to field record keeping is the use of a graphical grout take log, showing grout volume injected at each injection point. Upon a cross-sectional display of injection pipes and ports, a graphical representation of actual injected grout volume is drawn at each injection point. Linear, logarithmic, or circular representations of grout volumes have been used. Color coding can be used on the graphical grout take log to highlight areas of serious over and under grout takes, making it an excellent tool for contractor and inspector alike for visualizing project progress. By referring to such a graphical log as the job proceeds, variations in injection conditions can be observed and any unusual conditions spotlighted. A typical log is shown in Figure 21. Grout volumes are drawn at each grout port proportional to the volume injected. It is clear that grouting of pipe 5 has not been completed and that the upper primary grout port at pipe 7 was undergrouted.

Secondary-Tertiary Grout Port Test. An important tool used to test the adequacy of grouting is the secondary-tertiary grout port test. Grouting is traditionally conducted in stages in the same zone, with primary, secondary, and sometimes higher level injection points.



Note : 1 gallon = 3.785 litres

FIGURE 21 - GRAPHICAL GROUT TAKE LOG

If the secondary stage injection points produce a rise in pressure and reduction in flow as the projected volumes are reached, this is taken as an indication of nearing complete grout saturation and that the primary grouting is satisfactory. Figure 22 shows typical injection records. For the primary stage grouting curve, the flow rate/pressure curve is about constant, showing no grout saturation "closure." For the curve displaying a strongly decaying flow rate pressure ratio with time, grout saturation or "closure" was inferred and injection was terminated when the designed secondary grout volume had been injected, with good confidence in the grouting effectiveness in this particular example. If the flow rate pressure ratio does not drop off during the second stage, a third stage of grouting may be necessary to verify the completeness of the second stage. This response is not typical of a single row grout pipe array, or grout ports on the perimeter of the grout zone.

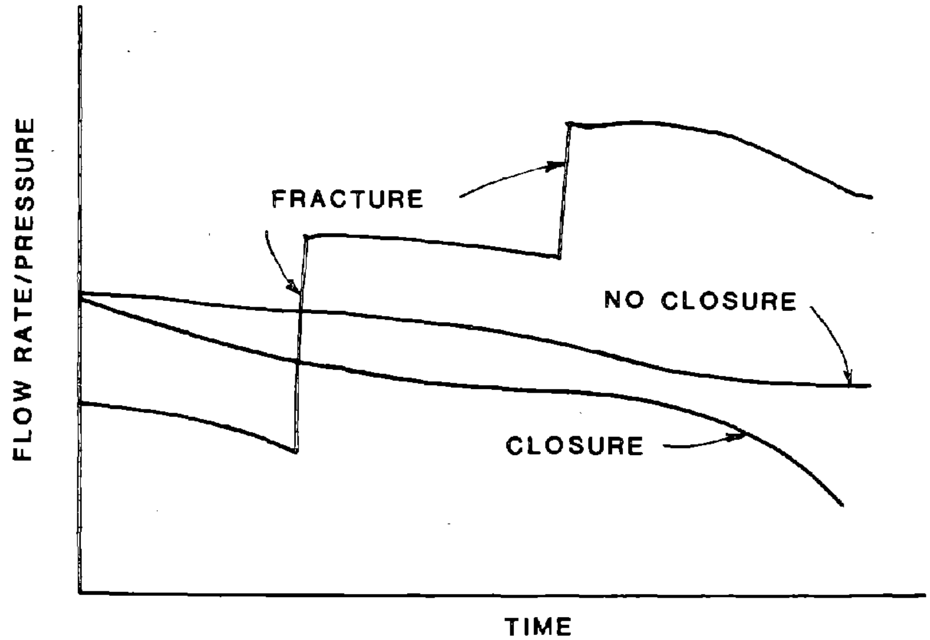


FIGURE 22 - TYPICAL FLOW RATE-PRESSURE RATIO CURVES

Hydrofracturing of Grouted Soils

Hydraulic fracture is indicated by large increases in flow rate with only small pressure increases. That is, the flow rate/pressure ratio curve is concave upward or discontinuous with time. Though relatively easy to diagnose during injection into impermeable materials or previously grouted materials, hydraulic fracture is more difficult to identify in permeable soils that have been grouted. Figure 22 shows a typical flow rate/pressure curve where fracturing occurred on two occasions.

In the past, the traditional opinion held that hydraulic fracturing of ungrouted or grouted soils during injection should not be permitted. In contrast, current opinion holds that hydrofracturing of grouted soils is not necessarily detrimental, but is necessary to obtain complete grout impregnation. Studies at Locks and Dam 26 (Woodward-Clyde, 1979) , provided the first available objective study of the subject. It showed that hydrofracturing of previously grouted soils due to reinjection of the same grout port will occur at pressures as low as one-half the overburden pressure, or may not occur until pressures reach three or four times the overburden. Thus, limited hydrofracturing on grouted soils is a necessary feature of effective grouting practice.

Hydraulic fractures will run through previously grouted sand until an ungrouted volume is intersected, and then permeate normally into the untreated sand. Thus, access to adjacent small, ungrouted zones may only be possible by limited hydrofracturing of previously grouted zones. This feature may be used to promote thorough grouting. Used carelessly, it can result in grout traveling outside the grout zone and being wasted. Hydraulic fractures usually initiate in the plane of the borehole, and may exert considerable force by virtue of the hydraulic pressure distributed over the fracture area. Since hydrofracturing causes some horizontal precompression of the ground, a nearby basement wall or lateral retaining structure for an adjacent excavation may be endangered by hydrofracturing. There is little risk of initial fractures causing heave of the ground surface unless the grout holes are horizontal or fracturing occurs between layers of different soil types. Overgrouting will eventually cause ground heave. It is not known what effect fracturing has on the strength of grouted masses. On being excavated, a grouted soil mass will sometimes break along the fracture, other times across the fracture.

Acoustic Emission Monitoring of Injection Pressure

Acoustic emission (AE) monitoring, a relatively new geotechnical monitoring tool, may be used to detect structural distress in geotechnical materials. In grouting, it may be used to detect hydraulic fracturing and therefore allow for control of this phenomenon. (See Volume I, "Construction Control" of this report.) Indications of fracturing are bursts of microseismic noises "heard" by the system, denoted by increased acoustic emission count rates.

High grouting pressures and the high flow rates can reduce grouting costs to the owner's benefit. The critical pressure at which fracturing is initiated can vary by a factor of three or more at points less than a meter apart. The common rule of limiting injection pressure to one psi per foot of depth (20 kPa/m) will not completely eliminate the risk of fracturing, and the use of higher and more efficient injection pressures might be completely safe over much of a grouting site. What is required is not a rule of thumb, but instrumentation that will detect fracturing immediately as injection pressure is raised. Acoustic emission monitoring can be used to detect the grout pressure causing hydraulic fracturing and therefore allow for control or prevention of this phenomenon.

An effective AE sensor is a hydrophone placed in a water-filled grout pipe. Placing the hydrophone underground reduces surface noises which otherwise would cause spurious alarms. The AE system should have adjustable filters that can eliminate frequencies below 1,000 hz., which includes most construction noise.

The time of integration of the AE system is usually between 10 seconds and 1 minute. The AE system should have an annotatable strip chart output recorder, visible to the grouting technician, so that he will immediately see any large pulses in the AE output. An audio output (earphones) may be used so that any noise sources detected may be more readily identified in the field and noted on the AE strip chart.

The procedure for field use of acoustic emission (AE) monitoring to detect hydraulic grout fracturing is as follows:

1. At the start of grouting, set the filters on the AE system so that construction or other cultural noise on-site will not be recorded.

2. In a noncritical area of the grout zone, conduct a hydro-fracture test. Set the threshhold and gain controls so that only hydrofracturing is recorded. Typically, the sound of hydrofracturing is several thousand times more intense than background noise levels if the AE sensor is placed at the grouting depth in a nearby grout pipe.

3. During grouting, set the AE system so that the grouting technician can see the recorded output. He can then increase the injection pressure at each injection point until fracture begins, and then back down to a comfortable safety margin.

4. If fracturing occurs during grouting, the grouting technician can reduce the pressure on the several injection points one at a time to identify the one causing structural distress.

The AE system controls should not be changed frequently. They should be set so that little if any cultural noise is heard, but fracturing is. It is a great temptation for the operator to increase the gain until extraneous construction or background noise is detected. This produces confusing data and should be avoided. Acoustic emission systems should be operated by experienced personnel. The array of controls and indicators on the face of an AE monitor is confusing and intimidating to the untrained construction worker. However, individuals having some experience in instrumentation or electronics can be quickly trained to use AE equipment.

GEOTECHNICAL QUALITY ASSURANCE TOOLS

Quality assurance methods include those systems of measurement and documentation that are useful in proving that the specified project objectives were or were not accomplished, since one cannot "see" the grouted ground. This section discusses those systems which are strictly quality assurance tools, and are not used for quality control purposes. In addition to conventional site exploration tests, selected geophysical tests are now being used to evaluate grouted soils. These include borehole radar and cross-hole acoustic velocity, which can be used to determine the grout location and condition underground.

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, and the excavation of test pits. These systems are useful under some conditions, but under other conditions, they are ineffective or even misleading.

The Standard Penetration Test (SPT) is a favorite site exploration tool in spite of its crudeness. The primary advantage of the SPT appears to be the wide familiarity geotechnical 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 strength.

The borehole pressuremeter can be used effectively in grouted soil where drilling does not disturb the adjacent soil and where a smooth clean borehole can be obtained. This is presently not possible in grouted gravelly sand. 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 conventional pressuremeter (Fernandez, 1980). Interesting data might be obtained by pressuremeter creep tests, but this has not been attempted.

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 often unsuccessful, due to small gravel particles or broken pieces of grouted sand working their way into the core barrel and abrading the sides of the sample, usually breaking it in flexure. Even if a sample is recovered intact, it is of questionable value because of the rough handling it undergoes during the coring process. It is difficult to hand 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.

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 presence of high pH silicate grouts. While test pits are both destructive and expensive, they are most effective conventional grout evaluation methods.

To summarize, some conventional site exploration tools can be used to obtain a qualitative idea of the grout location and condition, but even with them it is difficult to obtain quantitative data. In most cases, even intuitive judgments based on extensive experience with the particular tool in question can be misleading because of the sensitive nature of grouted sand.

GEOPHYSICAL QUALITY ASSURANCE TOOLS

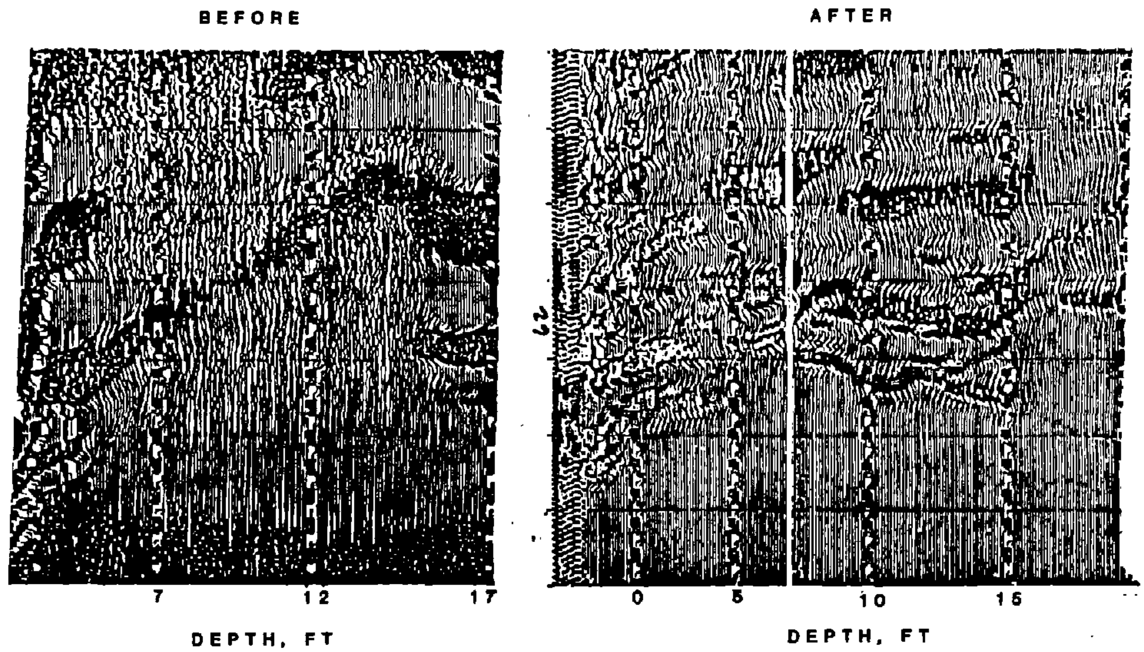
General

The geophysical tests that have proven most useful in evaluating grouted soils include cross-hole acoustic profiling and ground probing radar. These geophysical methods are well suited to defining increases in soil stiffness and grout presence and are being used increasingly to monitor soil grouting.

Borehole Radar

Borehole radar profiling involves the transmission of microwaves through the ground from one borehole to another. Silicate grouted sand becomes opaque to the microwave transmission, such that loss of signal is interpreted as a sign of good grout penetration. Recent development of small-diameter (35 mm) down-hole transmitters and antennas now permit economical use of plastic sleeve-port grout pipes as monitoring holes. This eliminates the need for expensive special instrumentation holes and greatly reduces the cost of radar profiling of grouted masses. Data processing equipment is needed to allow on-site interpretation of the before- and after-grouting radar logs by non-specialist technical personnel.

Borehole radar may be used in either transillumination mode or transmit/receive mode. In transmit/receive mode, a single borehole instrument is used which transmits a pulse, and then listens for the reflected signal. Because both portland cement and silicate are "lossy" materials, they are typically poor reflectors, and are difficult to see using transmit/receive mode. In transillumination profiling, a transmitter is lowered down one borehole, and a receiver down the adjacent borehole, both to the same level. The instruments are then raised simultaneously, so that the signal path between them is level, and the received signal is recorded as a "radar profile." By taking radar profiles before and after grouting, the effects of grouting can readily be seen in the comparison of the profiles. Transillumination radar is best used to determine the grout location, and to obtain an indication of the amount of grout present. Figure 23 shows a typical before and after grouting radar profile image pair.



Note: 1 foot = 0.305 m

FIGURE 23 - BEFORE AND AFTER GROUTING CROSS-HOLE RADAR IMAGES

<u>Equipment</u>. Earth probing radar is available in a variety of forms. For effectiveness in grout monitoring, the radar and grouting systems should have the following features:

1. The grouting system should use plastic grout pipes through which the radar can see, and which are available for radar surveys before and after grout injection.

2. The radar system must have borehole antennae which fit in the grout pipes. Surface radar is ineffective.

3. Transillumination radar, which has a transmitter in one borehole and receiver in another, must be used.

Criteria 1 and 2 above insure that radar surveys are possible in existing grout pipes without the expense of extra boreholes intended specifically for the radar surveys. Transmitting and recording equipment manufactured by Xadar Corporation and Geophysical Survey System, Inc., have been successfully used. Small-diameter antennae suitable for use in 40 mm (1.5 inch) diameter PVC grout pipes are not yet commercially available and must be custom manufactured. Figure 24 shows ground probing radar being used in sleeve-port grout pipes at the Pennsylvania Department of Transportation Sewer Undercrossing in Philadelphia.

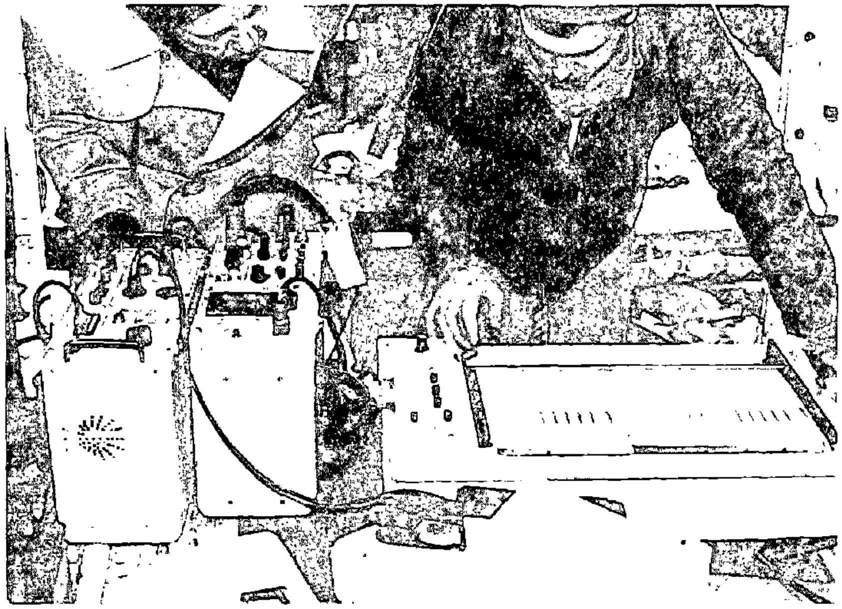
Operation. Borehole radar can see through ungrouted soil, but not through well-grouted soil. Interpretation of single run borehole radar survey results is difficult. When the system is used before and after injection, the changes caused by grouting can be readily discerned, even by inexperienced personnel. Use of borehole radar equipment should be supervised by technical personnel with geophysical experience. Properly used, it is capable of determining whether the area between two grout pipes has been grouted. The recommended sequence of steps is as follows:

1. Prior to grouting, conduct borehole radar surveys of selected PVC grout pipe pairs, noting operational parameters on the radar system. The survey should extend the full depth of the boreholes, starting with the antennas in the air above the borehole. One pair of "calibration holes" should be established outside the grouting area.

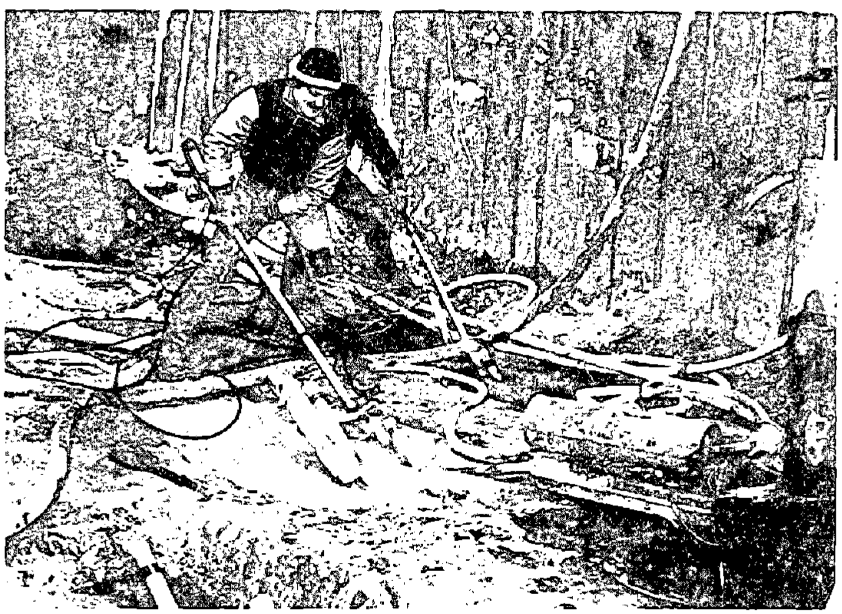
2. After grouting, repeat the surveys in the same boreholes, using the same settings on the radar controls. Confirm that the radar equipment is adjusted and operating properly by surveying the "calibration holes" and verifying that the "radar profiles" are similar.

3. Make side-by-side comparisons of the before and after radar surveys. Areas which were grouted will show much reduced signal strength, while the areas not grouted should show similar geologic features on both surveys.

Where the surveys extend from the bottom of the grouted zone to the ground surface several feet above the grout, the before and after radar profiles should be similar near the surface. This is evidence that the before and after surveys were in the same pair of boreholes and that the radar was working well. The top of the grouted zone should be clearly delineated by decreased signal strength. With such data, even those who are not familiar with borehole radar can understand and appreciate the data.



(a) ON-SITE RECORDING EQUIPMENT



(b) TRANSMITTER AND ANTENNA BEING PLACED IN PVC GROUT PIPES

FIGURE 24 - VIEWS OF RADAR PROFILING EQUIPMENT USED IN GROUT PIPES

Cross-Hole Acoustic Velocity

Geotechnical acoustical velocity measurements involve the evaluation of the rate of travel of mechanical pulses through the ground. The acoustic pulses are distinguished as either compression (P)-waves or shear waves. Work reported herein refers exclusively to compression (P-wave) measurements. At the time of this writing (1982), actual job applications have shifted to the use of shear waves, thought to be more independent of groundwater levels.

Cross-hole acoustic transmissions are used to measure acoustic velocity and a spectra of received signals. Profiles are obtained between two boreholes in much the same fashion as in transillumination radar profiling, except that the signal is a mechanical rather than electro-magnetic pulse. The acoustic system is set so as to determine whether a significant increase in acoustic velocity occurred upon grouting, and whether the transmitted spectrum indicates an improved acoustic medium after injection of the voids with grout. Attenuation of acoustic energy in soil is highly dependent upon the stiffness of the ground. Structurally, grouted sands are known to increase in micro-strain stiffness, and thus show two to ten-fold increases in acoustic velocity.

Cross-hole acoustic surveys are used to determine qualitatively the strength of the grouted zone. Like radar, it is used before and after grouting, and the ratio of flight times is compared to indicate <u>relative</u> changes in acoustic velocity. Thus, distances between test holes are not measured. The acoustic sounder and receiver are used in grout pipes, so that special survey holes are not required. Acoustic velocities through ungrouted soils typically are several hundred meters per second (600-1,400 ft/sec). After grouting, velocities as high as two kilometers per second (6,600 ft/sec) may be observed. This factor of two to ten increase is diagnostic of change from soil to weak rock, and indicates well grouted material. P-wave acoustic velocity measurements taken at the Demonstration Site are shown in Case History No. 6 of Chapter 8.

Requirements for cross-hole acoustic tests in grout are as follows:

1. Grout pipes are available for cross-hole surveys prior to and following grouting.

2. The acoustic tools are sized to pass through the boreholes.

3. Cross-hole acoustic velocity profiles are run the full length of the grout pipes.

4. The after survey should be long enough after grouting to permit full gel formation of the grout, say 48 hours.

SUMMARY

Detailed planning of injection locations, grout volumes staging, and injection sequencing can provide a specific work program against which subsequent quality control programs can be planned.

Quality control of chemical grouting requires real time verification of grout proportions, and injection rates, pressures and quantities. Continuous recording of these measurements is necessary for important projects, and requires automatic electronic monitoring equipment.

New geophysical methods of evaluating chemically grouted soils, using subsurface radar and cross-hole acoustic velocity profiling, can provide for quality assurance that the chemical grouting job was successful.

CHAPTER 8—CASE HISTORIES

PREVIEW

Many case histories of successful chemical grouting projects are available in the technical literature (Clough, Baker and Mensah-Dwumah, 1978; Ziegler and Wirth, 1982) to amply demonstrate that chemical grouting is an effective Ground Modification technique, and that highly qualified speciality contractors are available to do the work. However, little candid discussion of design and planning problem areas are found in the geotechnical literature. Numerous personal interviews were conducted with geotechnical designers, inspectors and chemical grouting contractors, to learn of the problems they most frequently encountered. The results of these surveys are discussed in Volume I, Construction Control. The first four of the six chemical grouting case histories presented in this chapter have been selected to illustrate the four specific difficulties that were most frequently mentioned during these interviews, and that arise when rational design and planning procedures are not carried out.

The four frequent problem areas are: (1) failure to anticipate the need for chemical grouting prior to excavation construction; (2) failure to identify ungroutable soils in the critical grout zone; (3) failure to carry out or perform predictions, leading to overly conservative design; (4) failure to integrate the results of a subsurface instrumentation program with a grouting program. The fifth case history describes chemical grouting done to guarantee a stable ground condition where a loss of ground would have been potentially disastrous. The final case history describes a demonstration program where the results of this FHWA research study were applied in the field on an actual, full-scale chemical grouting job for the Baltimore subway.

The exact locations and identities of the first five of the six projects described herein have not been disclosed, to permit more candid commentary. It should be understood that, with the exception of the demonstration test program, Case History #6, the examples cited generally represented typical design state-of-the-practice, at the time of the job. Since that time, United States chemical grouting designers and specialty contractors have dramatically improved their state-of-practice, coming considerably closer to the world-wide state of the art, which has itself been advanced by recent United States developments.

CASE HISTORY #1—EXCAVATION PRIOR TO GROUTING

Site Conditions

During construction of an underground subway station for a major rapid transit system, five near-surface vent structures had to be excavated beyond the station wall neat line, in close proximity to several brick buildings. The five vent structures extended outside the previously constructed temporary bracing shaft system. Construction of these structures had to be achieved without seriously damaging light one-story structures located only 0.6 to 1.2 meters away. A profile of the intended grout zone is shown in Figure 25.

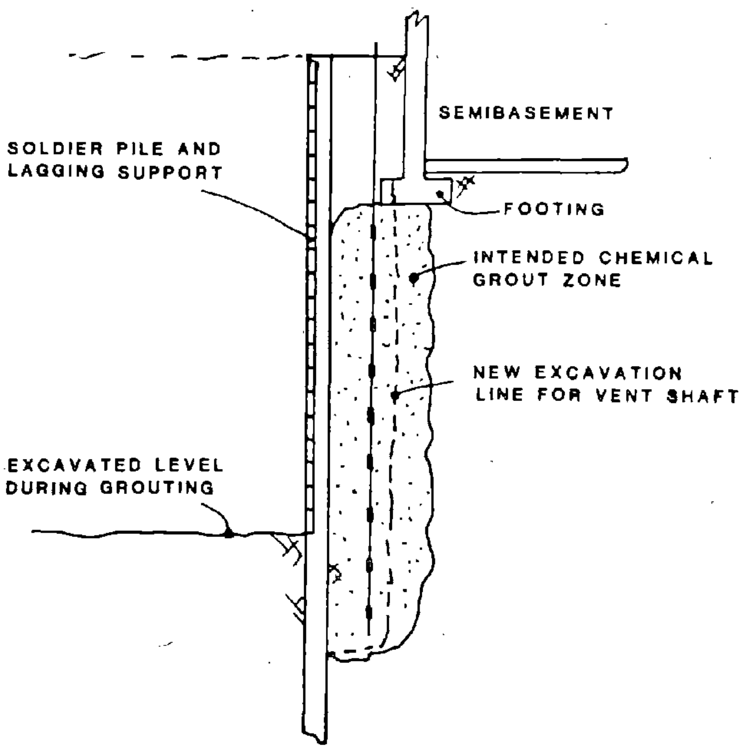


FIGURE 25 - SUBSURFACE PROFILE - CASE HISTORY 1

The Program goal was stabilization of underlying granular soils so that vent shaft excavations could be carried out without extensive and costly underpinning of the adjacent structures. The chemical grouting program was combined with a compaction grouting program in order to densify and fill any voids that may have occurred as a result of installation of the soldier piles and lagging, prior to the chemical grouting.

Grouting Plan

The zone directly below the front of the structures and throughout the vent shaft area was chemically grouted to depths varying from 4.0 to 15.0 meters below sidewalk level. Plastic sleeve-ported grout pipes were installed at a slight batter along the front of the structures, typically on 1.2 meter centers. Approximately 300 meters of drilling were performed and about 150,000 liters of Geloc-4 grout (42% sodium silicate) were injected.

Quality Control Program

Routine chemical grouting procedural controls were employed. This included manual recording of pumping rates, pumping pressures, and grout take at each sleeve. Samples of neat grout were obtained periodically to check grout quality, gel times and neat grout strength.

Results

Success of this program was limited, due to loss of chemical grout flowing through loosened soils to nearby lagging spaces. This was caused by injecting in sleeve ports too close to adjacent soldier piles and lagging. Guniting of the lagging was eventually tried in an attempt to seal the lagging, but was not effective. Full chemical stabilization of soils near the existing lagging was therefore not achieved. Nevertheless, no serious building damage occurred during the vent shaft excavations, and surface subsidence was minimal.

Problem Illustration

Chemical grouting programs usually fall into two categories: those that have been designed specifically for grouting and incorporate grouting into the initial specifications, and those that are remedial in nature and are employed only after unforeseen problems occur on a project--often not until the final success of the project is jeopardized. This project clearly falls into the latter category. Grouting was employed here as an afterthought, and a grouting contractor was not called in until extensive excavations had already taken place in the area. The excavations, with the presence of soldier piles and laggings, made control of the chemical grouting process extremely difficult. Thus, significant grout loss through the soldier piles and lagging was inevitable. This could have been a relatively routine grouting job, with anticipated excellent results, had the need for grouting been foreseen prior to excavations in the area. However, the ultimate results were mixed, and the program proved to be more difficult, more costly, and less effective than necessary.

CASE HISTORY #2—SOILS UNGROUTABLE IN CRITICAL ZONE

Site Conditions

Twin rapid transit tunnels were scheduled to be mined through a mixed-face, soil-rock transition zone under a major railroad tunnel. The new 5.5 meter diameter subway tunnels were to pass some 2.4 meters below the old railroad tunnel, which had a diameter of 12.2 meters.

Since the railroad tunnel carried all of that railroad's passenger traffic, their engineers specified that all noncohesive soils below the railroad tunnel and above the transit tunnels be chemically grouted to prevent tunnel subsidence. The theoretical grouting zone began at the subway tunnel crown approximately 2.4 meters below the railroad tunnel and grouting was extended up the sides of the railroad tunnel to 1.5 meters below its crown. Mixed soils were present in the specified grout zone, and subsequent soil borings and laboratory testing by the grouting subcontractor indicated that only between 45% and 55% of the targeted soils were groutable. See Figure 26.

Grouting Plan

Specifications required that a zone 22 meters wide by 46 meters long, symmetrical to the intersection of the tunnels, be chemically grouted to prevent subsidence of the railroad tunnel. The longest dimension of the grouting was parallel

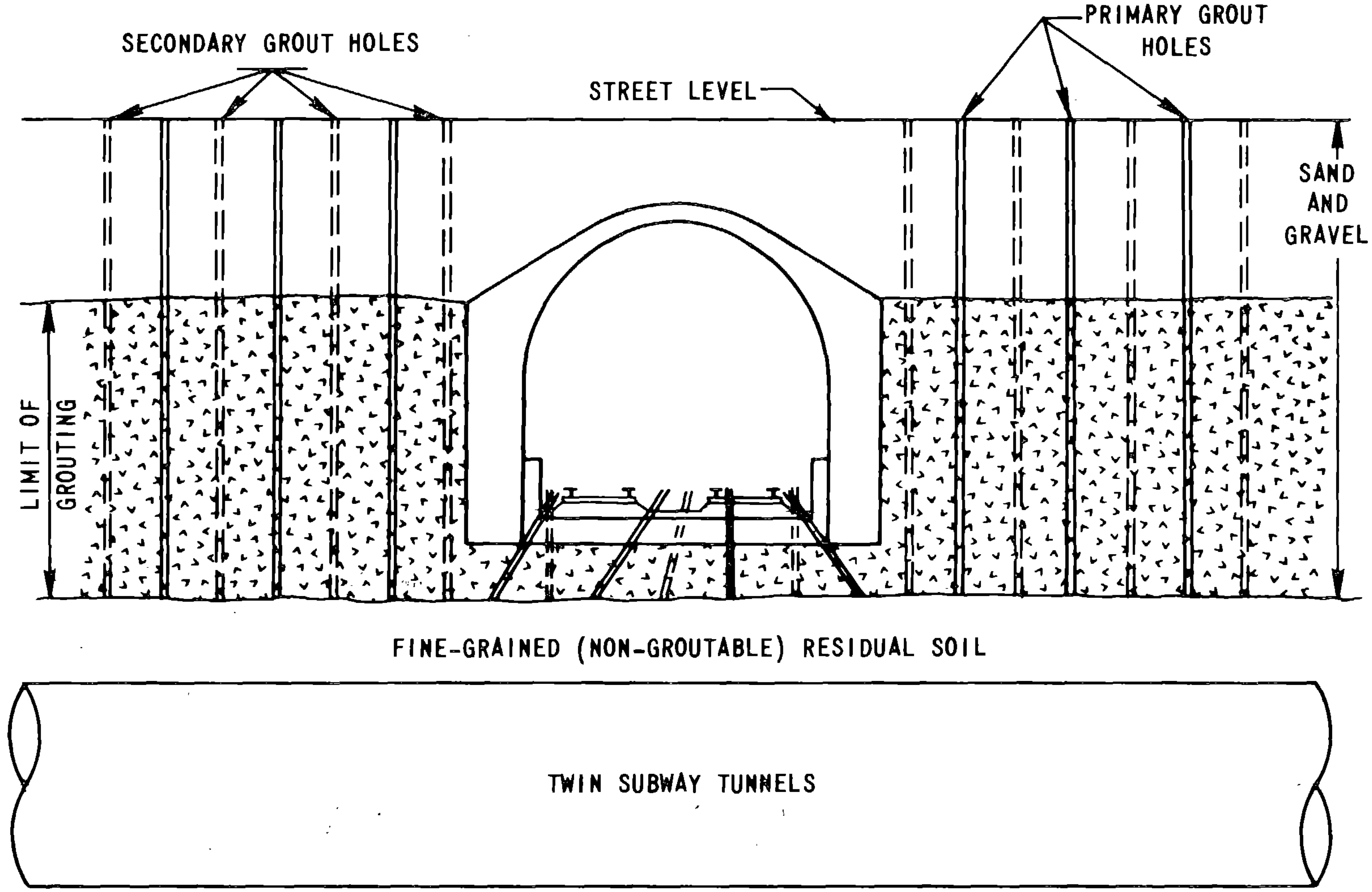


FIGURE 26 - SUBSURFACE PROFILE - CASE HISTORY 2

to the new tunnels, extending some 16.8 meters beyond the edge of the railroad tunnel. The targeted zones involved a volume of 6,185 cubic meters of groutable and non-groutable soils. Grout holes were drilled from the surface and from inside the train tunnel. This resulted in a final hole spacing of 2.2 m x 2.2 m on centers. Primary injection points were grouted to theoretical design volume. Once all the primaries were completed, the secondary pipes were pumped to their design volume or to refusal whichever occurred first. This technique confirmed complete saturation of the specified zone, but with takes of only 70% of theoretical grout volumes. Pumping pressures were limited by specification to a maximum of 6 kg/cm^2 (588 kPa).

Total drilling quantities were 2,500 lineal meters from the surface and 380 meters of tunnel drilling. A total of 1,180,000 liters of 40% sodium silicate base grout were required to complete the stabilization program.

For injection, a packer device within the sleeve-port pipe was placed at each specific port, to insure the precise location of the injected volume of grout. The grout pumping rate and injection pressure at each hole were measured by a flowmeter and gauge.

Continuous mixing of chemicals, as opposed to batch mixing, was used. Typically, gel times were set at about 30 minutes. Gel times of test samples ranged from 15 to 60 minutes. Grout refusal was considered to have occurred when the flow rate at a given sleeve dropped below 2.0 liter/min at pumping pressures of 6 kg/cm^2 (588 kPa).

A particular challenge to this job was conduct of the ground stabilization program without disruption to traffic, both on the street and in the railroad tunnel. To keep traffic disruptions to a minimum, a staging area was established in a yard near the site. Chemical storage tanks were used to supply the chemical mixing plant with materials, which in turn were pumped to the injection site via underground pipes.

Quality Control Program

Conventional monitoring techniques, which included elevations on bench marks and heave gages for monitoring of the train tunnel, were used routinely. Manual recording of pumping rates, pressures, and grout takes at each grout sleeve was also routine. This allowed special notice of any sudden increases or decreases in grout pressures, indicating the presence of large voids, or grout intrusion into utility lines. The quantities of grout placed were measured by positive displacement meters at the grout plant. Comparisons of the nominal grout takes for each grout hole, as indicated by the flowmeters, and meter readings at the grout plant, were made on a daily basis. A graphical grout take log was kept and updated daily as an adjunct to field record keeping. This graphical progress display, showing a plot of grout volumes injected at each injection point, served as an excellent tool for visualizing job progress and the variation in grout takes from point to point. Standard Penetration Test borings, water injection tests, and cored samples were performed to evalulate the results.

Results

A small amount of surface heave occurred during grouting. This was observed as an extension of the soil volume near the active grout port during pumping, followed by rebound in the hours after the end of a day's injection. Motions during pumping were sometimes as large as 5 mm, of which about 75% would rebound after pumping stopped. Permanent deflections after all primary and secondary injections were completed also ranged from near zero to about 5 mm. At no point was any structural damage observed as a result of these small deflections.

At the time that the twin transit tunnels were mined through the grouted zone beneath the railroad tunnel, subsidence was observed in a range from 12 to 20 mm. Again, no structural damage to the railroad tunnel occurred, but concern was expressed over the possible extent of the subsidence. This was a probable result of the ungroutable soils present in the critical zone above the tunnel crown, which prohibited the construction of a continuous stabilized zone surrounding the tunnel opening.

Problem Illustration

Recent research has indicated that there is substantial difference in the mechanics of support provided by a grout stabilized zone when it is continuous around a tunnel opening, and when it is interrupted by the presence of ungroutable layers in the critical zone (Tan and Clough, 1980). In the first case, the grouted zone appears to act as a structural element that absorbs stress changes and minimizes distress in the ungrouted soil. In the second case, loose or soft ungroutable soils above the springline can disrupt the formation of an arch action in the stabilized zone. If the ungroutable layers are coherent and strong, they will have a positive influence upon the stabilized soils. If they are weak or incoherent, subsidence problems may occur. In an attempt to compensate for the ungroutable layers in the crown area, simply enlarging the stabilized zone has been a frequent design response to this problem, and was attempted in this case. According to Tan and Clough, however, this is seldom successful since the structural action of the stabilized zone cannot be mobilized. These conclusions seem to be borne out by this case. Here the soils in the transit tunnel crown were composed of fairly competent, fairly granular but ungroutable, weathered material, causing interruptions in the stabilized zone. When the transit tunnels passed through the area, settlements in the range of 12 to 20 mm occurred in the railroad tunnel above. These deflections happened quickly and did not result in structural damage. Had the ungroutable layers been less competent than these were, greater subsidence could have been expected.

It should be noted that this project was designed and executed several years prior to finalization of the research by Tan and Clough, cited above, and represents the state of the art in soil stabilization design and practice for that time.

CASE HISTORY # 3—OVERLY CONSERVATIVE DESIGN

Site Conditions

During construction of a subway system, twin, soft-ground subway tunnels, 5.8 meters in diameter, were driven at an average depth of 18 meters so as to pass perpendicularly below a 90-year old, masonry arch railroad tunnel. The old tunnel carried a principal freight track for a major east coast railroad. With less than 2.1 meters of soil between the crowns of the transit tunnels and the base of the old railroad tunnel, it was vital that maximum control of settlements be exercised. Since the masonry structure of the railroad tunnel made conventional underpinning techniques problematic, an extensive chemical grouting program was undertaken. The situation plan is shown in Figure 27.

The chemical grouting program goals were stabilization of soils to reduce structural settlement of the masonry arch tunnel and protection against possible run-ins to the transit tunnels under construction in the soft-ground below. The arch tunnel support soils were chemically grouted along 64 meters of the arch tunnel axis and some 25 meters along the axis of the transit tunnels.

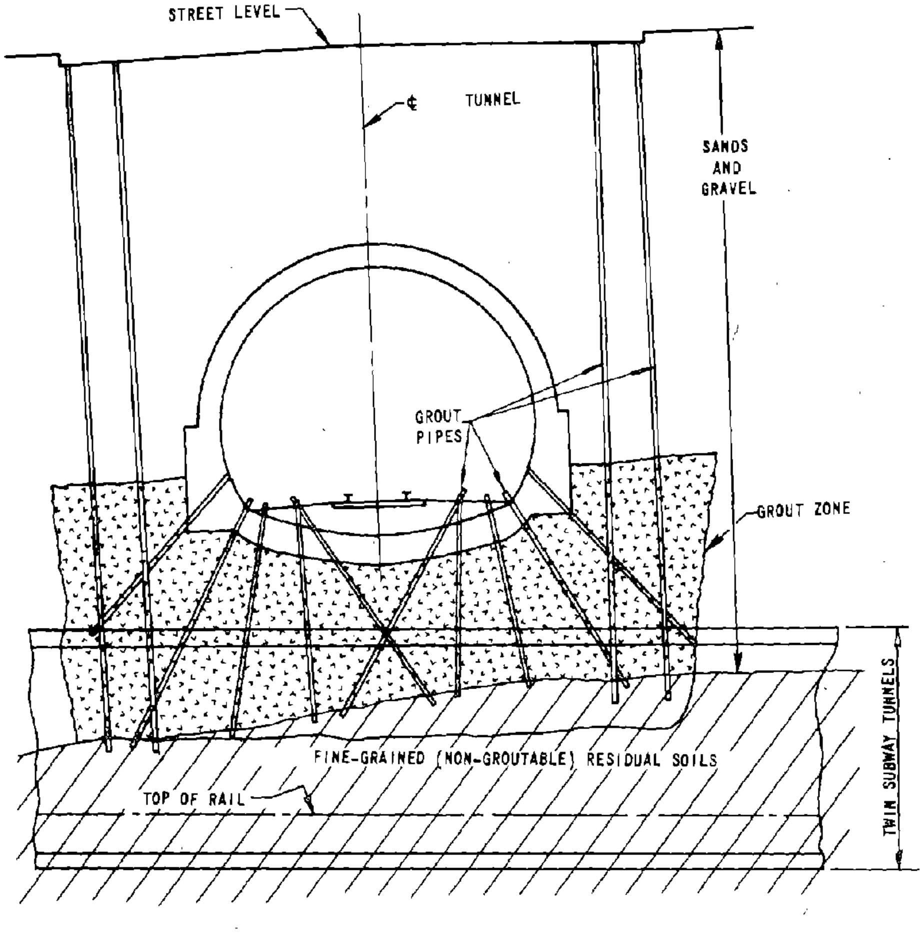


FIGURE 27 - GROUTING ZONE PLAN - CASE HISTORY 3

Grouting Plan

Approximately 600 grout pipes were placed, both from the street surface above the tunnel intersection area, and from inside the active railroad tunnel. At the surface, two rows of vertical grout pipes 1.5 meters apart and on 1.5 meter centers were placed along each side of the arch tunnel. From the surface, some 600,000 liters of sodium silicate medium strength chemical grout were injected. From within the tunnel 43 grout pipe fans (10 grout pipes each) were placed on 1.5 meter centers. Some 750,000 liters of sodium silicate medium strength grout were injected from inside the masonry arch tunnel. A total of 3,860 cubic meters of soil was thus stabilized, using about 1,350,000 liters of Geloc-3 chemical grout (40% sodium silicate catalyzed by 7% reactant mixture of ethyl acetate and formamide). A total of 4,725 meters of drilling was performed, 60% by drill rig from the surface and 40% by hand drills from within the tunnel.

The chemical grout plant was set up on a nearby vacant lot and consisted of large chemical storage tanks and an automatic proportioning pump unit. Metered amounts of chemicals were pumped under pressure to the injection sites, through hoses on the surface street level or through pipes drilled into the masonry tunnel. Total volumes and individual grout proportions were measured by positive displacement meters at the pumping unit, which operated independently of the viscosity or rate of flow of the individual materials. Chemical components were stream mixed in a special mixing hose before being discharged into a multiple outlet grout manifold. The rate of injection into each grout pipe was determined by a valve, gauge and flowmeter assembly placed at the manifold.

Pumping pressures up to 6 kg/cm^2 (588 kPa) was used initially. However, at one point during secondary grouting from within the railroad tunnel, localized heave occurred at the center of the tunnel floor, causing minimal disturbance. After that point, pumping pressures were limited to 3 kg/cm^2 (294 kPa). All drilling and grouting was done under stringent traffic restrictions from congested surface traffic and from frequent train traffic within the tunnel.

Quality Control Program

Conventional monitoring techniques, which included settlement points and routine surveys for control of settlements and surface heave within the tunnel itself, were employed. In addition, records of pumping pressures, pumping rates, and grout takes at each grout sleeve were kept. This allowed short-term warning of any sudden increases or decreases in grout pressures which might indicate the presence of a large void, or grout intrusion into a utility line. Grout samples were obtained periodically to check grout quality, gel times and neat strength.

Results

When grouting was complete and the subway tunnels were finally pushed through, the settlements in the tunnel were negligible, less than 0.2 mm.

Problem Illustration

In the absence of a rational analytical design procedure, and lacking Quality Control/Quality Assurance programs which would allow evaluation of grouting adequacy, the designer for this project felt required to rely on highly conservative design parameters as insurance against possible failures. In addition, the precedent of a previous large chemical grouting program for mainline railroad track tended to set, in the eyes of the railroad officials reviewing the plan, the "required" grout zone dimensions for this work unnecessarily large.

The grout zone supporting the masonry arch tunnel extended well beyond the boundaries necessary for adequate structural support. This resulted in increased grouting costs to the client of perhaps up to 50%. Such overdesign, with the resultant increases in project costs, tends to limit the feasibility of grouting solutions, and encourages the development of other design alternatives. Utilization of Quality Control and Quality Assurance programs, as outlined in Chapter 7, that can be integrated into the design and construction phases of a grouting project, can considerably reduce the uncertainties formerly associated with the use of this technique.

CASE HISTORY # 4—EXCESSIVE INSTRUMENTATION AND MONITORING

Site Conditions

Installation of a pumping station to upgrade a sanitary sewage treatment system necessitated construction of an 2.45 meter diameter force main through a residential district of a major city. Soils in the area ranged from granular fill to organic clays, with groundwater extending over the crown of the proposed sewer and only 2.0 to 2.5 meters of soil cover above the crown. Because of the historic nature and importance of the area, specifications mandated that settlement rates along several blocks of three story homes be maintained at "zero." In addition, infiltration of groundwater into the excavation area was to be minimized. To achieve these goals, an area 400 meters long was selected for chemical grouting.

Grouting Plan

The total tunneling program for this four block area of the city was designed for grouting from its inception. The original plan called for use of liner plates within the tunnel excavation and the grouting program was designed accordingly. Plastic sleeve-ported grout pipes were placed three across the tunnel on 1.4 meter centers for the length of the grouted section. The sleeve pipes were to remain in place for the duration of the program, in case remedial work became necessary. A nominal one meter thick annular ring of 35% sodium silicate grout was injected along the length of the tunnel to a depth of 5.5 meters. Quantities of grout at each hole were predetermined during the design process, based on soil samples. Grout take and pressures during pumping were uniformly consistent with design volume. Gel times ranged from 10 minutes to 60 minutes and the unconfined compressive strength of the grouted soils was set at 7 $kg/cm^2 \pm 2\ kg/cm^2$ (686 $\pm$ 196 kPa). A total of 4,500 cubic meters of soil was stabilized in this program. Although the prime contractor, with approval from the client, changed from the liner plate system of the original design to a permeable ring and lagging system used in conjunction with a closed face tunneling machine, no soil losses occurred. Nor did any daylighting occur in the grouted zone, despite the shallow depths of the tunnel excavation.

Quality Control Program

To monitor the effectiveness of the grouting program, an intensive instrumentation program was conducted. A 1.6 kilometer long section of the sewer force main was instrumented, although only 400 meters of the excavation was actually grouted. This was done to contrast any settlements in the grout zone with settlements that occurred in the ungrouted areas. The instrumentation program included deep and shallow settlement points, inclinometers, tilt plates, tape extensometers, piezometers, and borings to determine the strength of the grouted soils. In addition, the grouting contractor maintained the usual records of pressure and flow rate at each grout port. Graphical display models, illustrating the grout takes at each hole, were also maintained.

Results

The most dramatic proof of the effectiveness of the grouting program was obtained when a definite tunnel trough occurred in the ungrouted sections. Cave-ins and broken utility lines were also frequent occurrences outside the grouted sections of tunnel. This project well illustrates that a properly designed and executed grouting job can meet program goals without the delays and problems routinely encountered on projects where grouting is only brought on-line after subsidence problems have developed.

Problem Illustration

Despite the comprehensive nature of the instrumentation program undertaken on this project, the actual data collected had little impact on the course of the grouting work. At the conclusion of this tunnel project, the data collected had not been completely reviewed. In addition, the instrumentation program was not integrated into the design program and had no feedback loop, either to the project designer, the grouting contractor, or to the inspector on site.

The instrumentation program cost 15% of the total construction costs and were not accessible for review in time to take corrective action. As an academic exercise and as a learning tool for future similar projects, the data collected on this job will have no doubt proven its value, once it has been reduced and the results publicized. A program tailored to design objectives with more direct impact on job performance would have been more appropriate.

CASE HISTORY # 5—GROUTING FOR INSURANCE PURPOSES

Site Conditions

Construction of a large rapid transit underground station encountered a number of soil stability problems typical to a congested urban site. In particular, support of five high-rise commercial buildings and their foundation soils during deep excavation directly adjacent to the building lines presented problems. Initially, conventional pile jacking techniques were attempted, but had to be abandoned due to numerous boulders found beneath the buildings. A slurry diaphragm wall, originally planned as support for the lateral earth pressure created by the building loads, was redesigned to provide underpinning as well as excavation support. However, due to the proximity of the slurry wall to large spread footings (some of which had to be cut to allow for the slurry wall excavation) and the frequent fluctuation in slurry level at the top of the trench, concern developed over possible soil losses under the footings. To guard against this, chemical grouting was selected for use immediately below the remaining footing area to avoid sand raveling during construction. A secondary consideration may have been increased bearing strength for the reduced area of the foundations. A typical cross-section is shown in Figure 28.

Grouting Program

A continuous grout curtain, approximately 3 meters deep, was placed directly below all spread footings which faced the slurry wall. Higher loads under one building necessitated a 5-meter deep grout curtain. Because of site constraints, a variety of drilling problems were encountered and drilling techniques had to be tailored to each of the five buildings. Thus, some grout pipes were placed from inside the basements of the buildings while others were drilled from outside the buildings, through the structure into the underlying soils. All grout pipes were placed on 120 cm centers. Both hand and air track drilling were employed. Lack of storage space, at a premium on the site, required the use of a self-contained pump-tanker, which could move easily

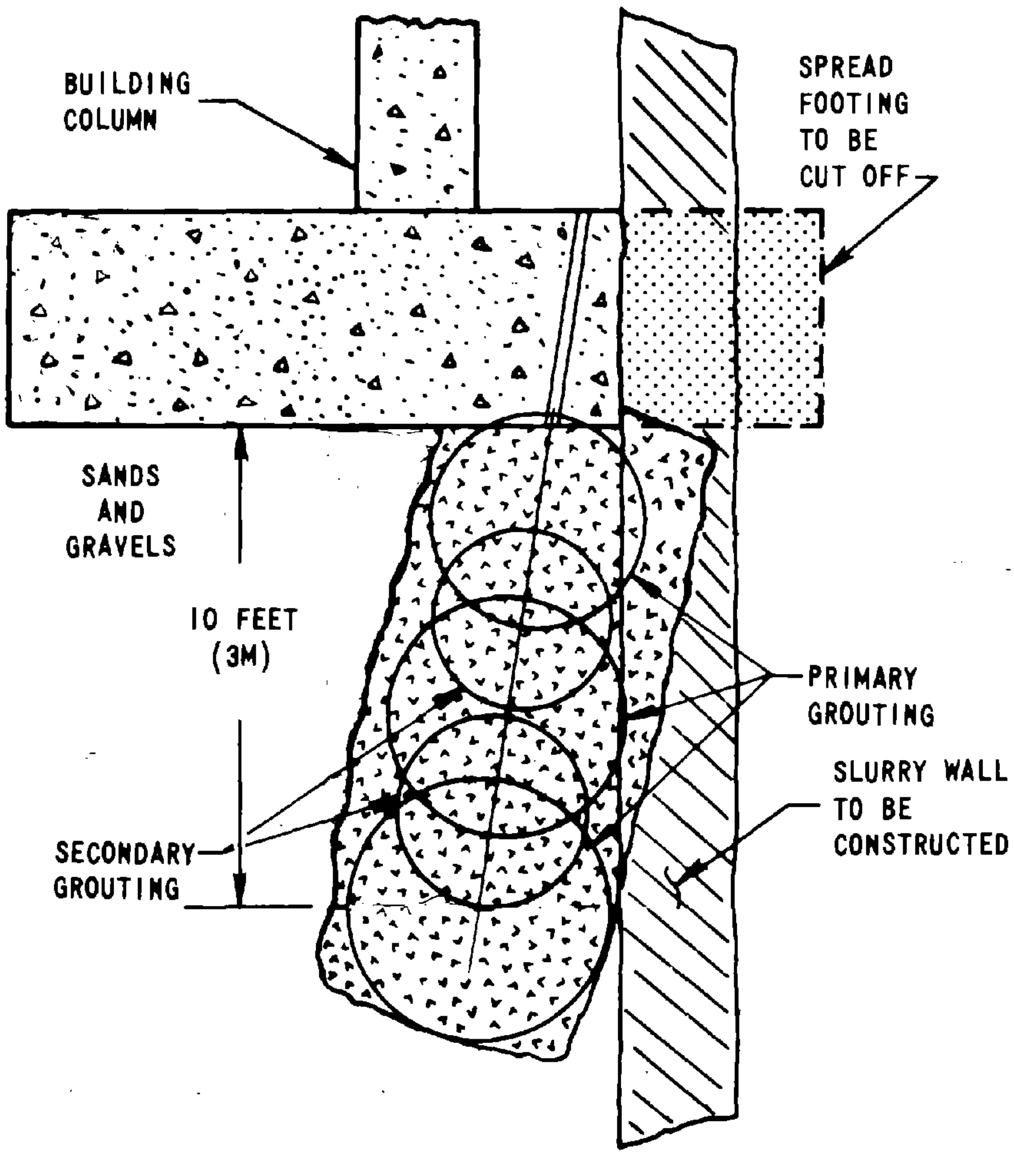


FIGURE 28 - GROUTING ZONE FOR SLURRY WALL CONSTRUCTION - CASE HISTORY 5

from building to building, as the grout pumping progressed. A total of 285,000 liters of GELOC-4 (40% sodium silicate) grout were pumped to treat an estimated soil volume of 1,250 cubic meters.

Quality Control Program

Routine procedural controls were employed, including manual recording of pumping rates, pumping pressures, and grout take at each sleeve. Samples of neat grout were obtained periodically to check grout quality, gel times, and neat grout strength. A graphical grout take log was also maintained and updated daily as an adjunct to field record keeping. Conventional monitoring techniques, including elevations on bench marks and heave gages for monitoring building settlements were used routinely.

Results

Excavations in the area took place several months after grouting. Consistency of the grout curtain was excellent, no raveling occurred and stand-up time was adequate to complete excavation without difficulties. No building settlements were noted at the completion of excavations in the area.

Problem Illustration

The effectiveness of a chemical grouting program is often made dramatically obvious, as in cases where settlements occur during excavation in ungrouted zones while settlements are maintained within acceptable limits in directly adjacent grouted areas. Just as often, however, it is virtually impossible to prove or evaluate the effects of a grouting job. Although in this instance, the consistency of the grouted curtain was excellent, and no settlements or soil raveling were noted during or after excavations, the possibility still exists that settlements would not have occurred even without the grouting. Since there is no real way to evaluate the efficacy of this particular grouting program, the role of grouting remains somewhat ambiguous. Nevertheless, when the risk of settlements or subsidence exist and the consequences would be severe, as in the case of these five important high-rise structures, the cost of grouting as insurance against dangerous risks can be justified.

CASE HISTORY #6--DEMONSTRATION PROGRAM

Site Conditions

The monitoring techniques developed in the course of the laboratory and field research phase of this FHWA research program were ultimately taken into the field and applied in a full-scale grouting project. Specific objectives of this demonstration program included:

1. Demonstration and evaluation of the new techniques within the context of a production grouting project,

2. Exposure of any weaknesses in the new techniques that might be brought out by actual practice, and

3. Sharpening of skills and methods by additional use.

The site selected for the demonstration was the Union Trust Building adjacent to planned cut and cover construction of the Charles Center, an underground station on the new Baltimore subway system. A number of soil stability problems typical to a congested urban site were encountered here. In particular, the support of a 100 year old granite commercial building and its foundation soils during deep excavation directly adjacent to the building line presented problems. Conventional jacked pile underpinning techniques were not possible due to very dense ground and boulders. The standard excavation support system of slurry diaphragm walls was not feasible in a short section of wall because of immovable utilities in the area. As an alternative, the general contractor selected a support system of soldier beams and lagging, combined with a chemical grouting program to provide stand-up time during the lagging operations. This stiffer lateral support system was expected to eliminate the need for underpinning. Lack of room, however, made it necessary to cut off portions of the existing spread footings prior to excavations, so a secondary consideration was to increase bearing strength for the reduced area of the foundations. The plan for this project is shown in Figure 29.

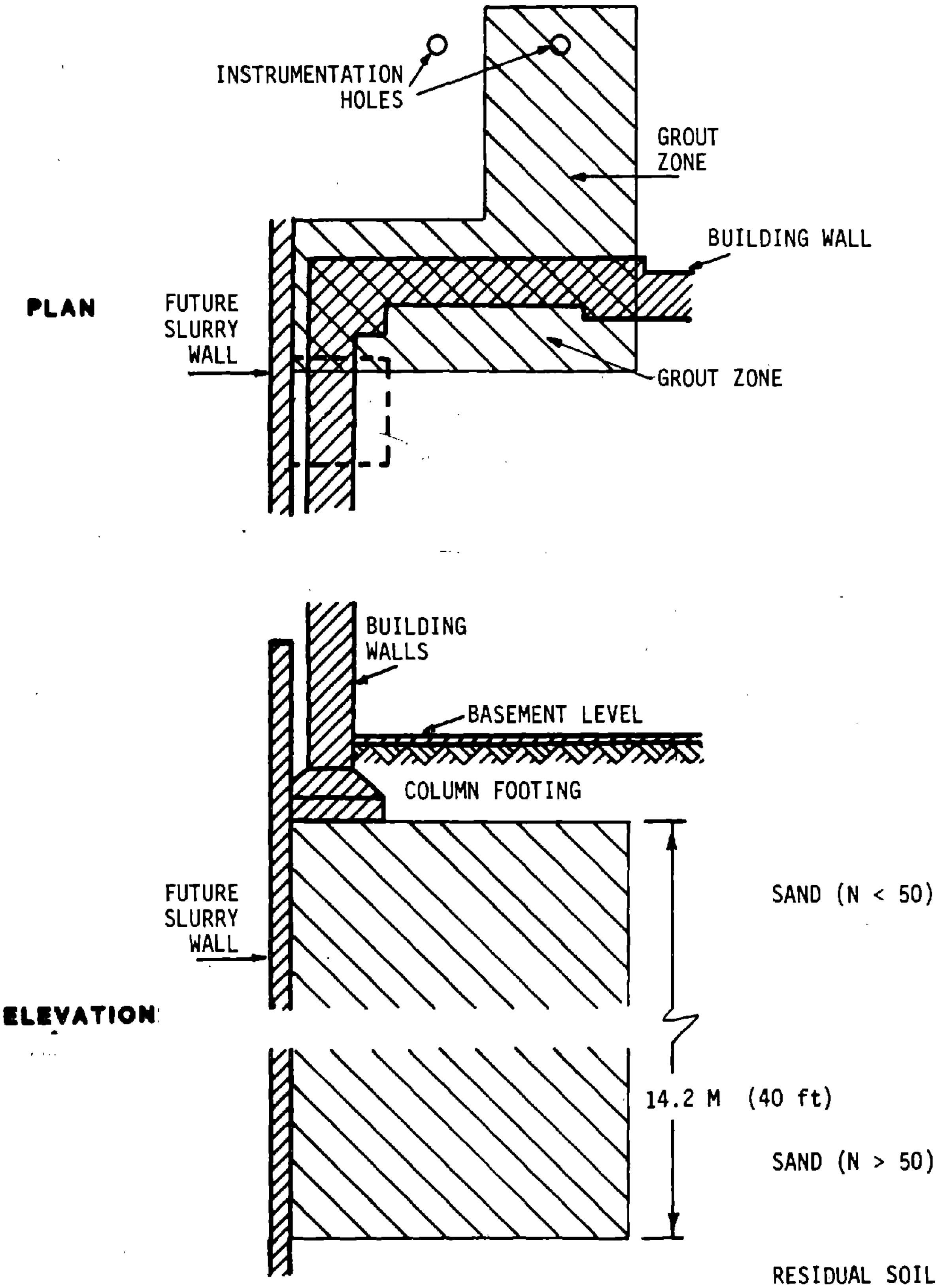


FIGURE 29 - PLAN OF DEMONSTRATION GROUTING SITE - UNION TRUST BUILDING

Grouting Plan

Grouting was to take place from the footing level down to the residual material, approximately 17 m below street level. The area beneath the footings was reached by battered (slanted) grout pipes, so that it would not be necessary to drill through the basement floor. The density of utilities in the area made drilling difficult, but the grout sleeve pipes were successfully placed from outside the building. Had it been necessary, this work could have been done from inside the basement, but at the cost of considerable inconvenience to the occupant. Two instrumentation holes for radar and cross-hole acoustic sensors were drilled in the grout zone outside the building. Into these holes were placed 3-inch diameter PVC pipe. (Radar antennae capable of passing through a normal 38 mm PVC sleeve-port grout pipe are now available, so that extra instrumentation holes are not required.)

Grout injection was carried out using the sleeve-port pipe method. Prior to injection at each grout point, the weak mortar jacket around the grout pipe was ruptured by a brief injection of water at high pressure. The actual injection of silicate grout was held at low pressures to reduce the possibility of hydraulic fracture.

Quality Control Program

The monitoring program included ground probing radar, acoustic velocity, acoustic emission (AE) monitoring, and procedural controls. The geophysical methods were used to determine whether the area between the instrumentation holes was well grouted, and to evaluate the stiffness of the grouted mass by the change in acoustic velocity. During grout injection, AE monitoring was used to check for hydraulic fracture. Procedural controls included manual recording of pressure and flow rate data.

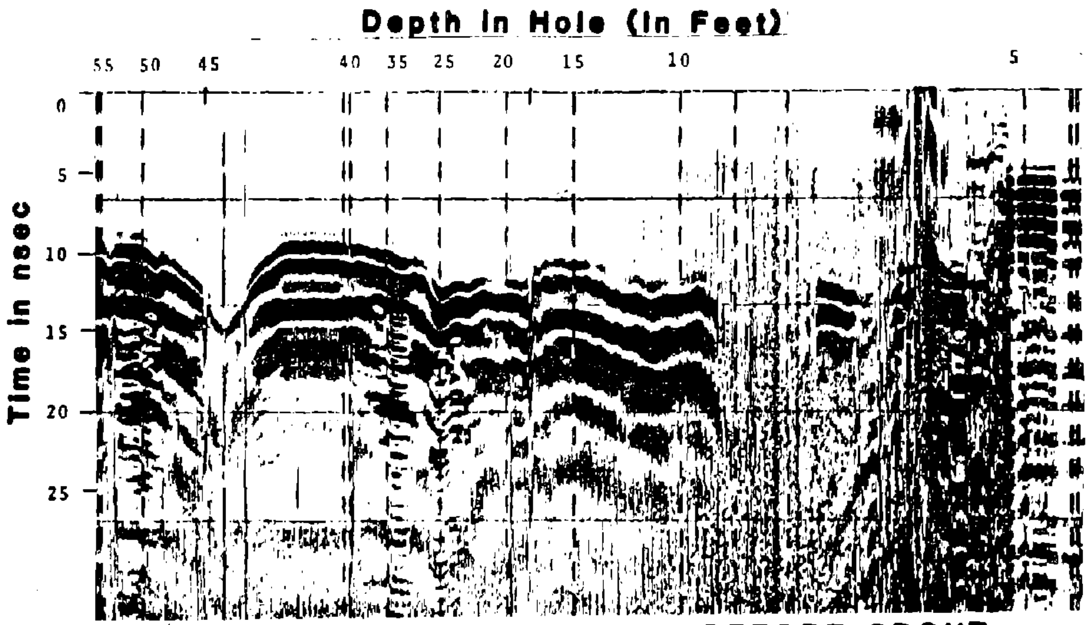
The demonstration included limited transillumination radar and cross-hole acoustic velocity (P-wave) tests between the two special instrumentation holes. One of these holes was inside the grout zone; the other was outside, so that the entire cross section between the two holes was not fully grouted. AE monitoring was conducted by placing the system hydrophone at the bottom of an inactive grout pipe. The pipe was then filled with water to provide good mechanical coupling, and the top was plugged to reduce air-borne noise. The primary function of AE monitoring is to detect structural distress in the soil mass being grouted. Previous tests had shown emission rates of several thousand to tens of thousands of counts per minute with this instrument when hydraulic fracturing occurs. Since the injection of silicate grout was held to relatively low pressures to reduce the possibility of hydraulic fracture, it was expected that the AE system would detect the initial sleeve rupturing operation, but would not be able to hear normal grouting. Injection pressure and flow rate were manually recorded from the grout injection manifold and plotted as time histories at a later date. Recording by strip-chart recorder would have reduced the cost of data collection and made the data immediately available.

Geophysical Data Radar

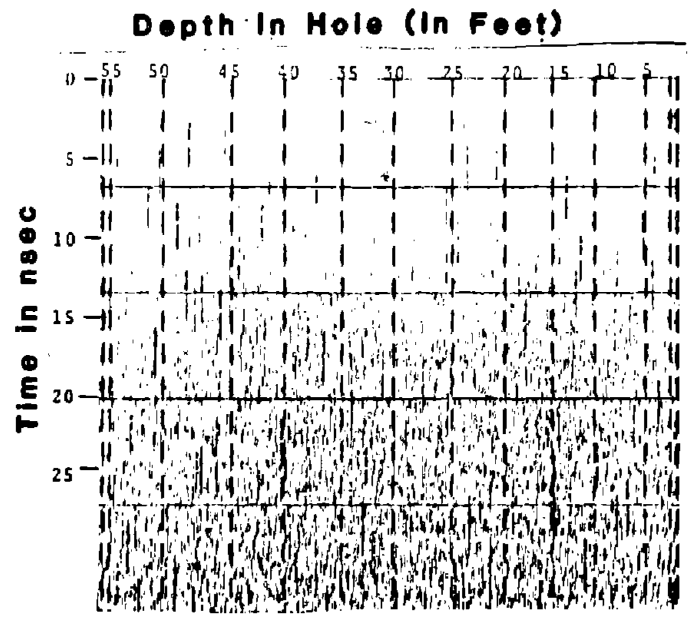
Images from the transillumination radar surveys conducted before and after grouting are shown in Figures 30. While the before-grout survey appears like a routine survey, the after-grout survey indicates a lost signal. Tests were conducted that proved that the antennas were working, or a system malfunction would have been suspected. Since the antennas were working, it is clear that the grout completely absorbed the radar signal. The before-grout survey is interesting. Near the surface, the signal is greatly confused by the utilities and variations in soil and backfilling material. At several points, the signal is blocked completely in this range. At depths greater than 3 meters, the first signal arrival is clearly seen. There is a general trend toward higher velocity materials at increasing depths, with one low velocity anomaly at a depth of about 13 m, which may indicate a clay or a water-filled seam. The critical factor, however, is not the interpretation of the before-grouting survey, but the great change brought about by grouting.

Cross-hole Acoustic Velocity

The cross-hole acoustic data, shown in Figure 31, were very noisy but indicated that a general increase in acoustic velocity resulted from grouting. Fly-by tests, with the receiver placed at a depth of 12.2 m in one hole, and the transmitter moved to various depths between 7 and 17 m in the other hole, are shown in Figures 31. These figures show actual time of flight as a function of transmitter depth. These data are reduced to show average velocity along the sloping signal path as shown in



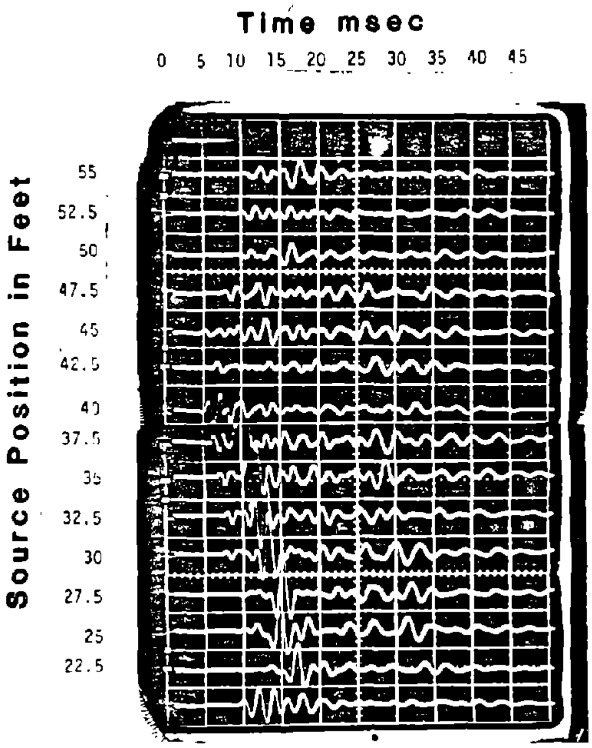
BETWEEN HOLES A AND B BEFORE GROUT



BETWEEN HOLES A AND B AFTER GROUT

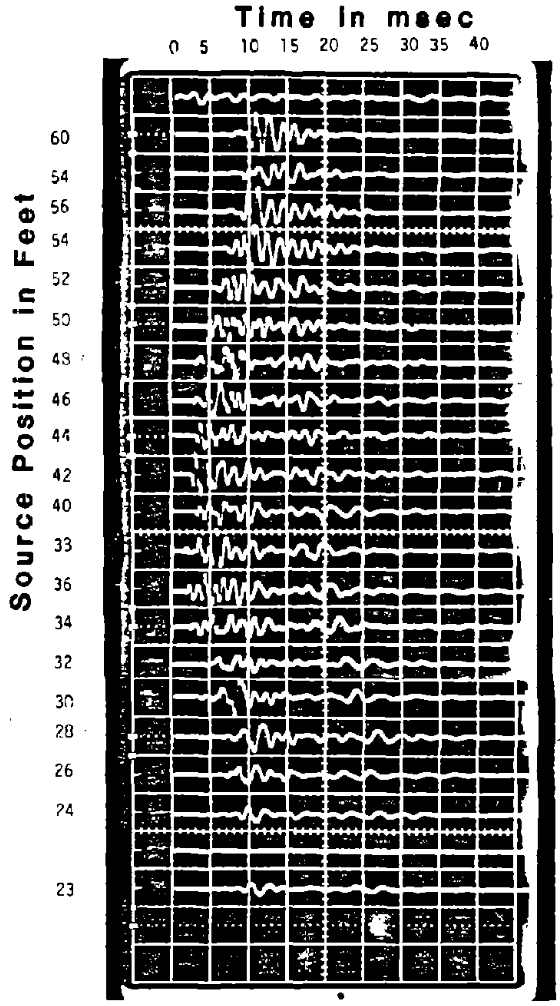
Note: 1 foot = 0.305 m

FIGURE 30 - RADAR IMAGES AT DEMONSTRATION SITE



Receiver Located at 40 Feet

BETWEEN HOLES A AND B
BEFORE GROUT



Receiver Located at 40 Feet

BETWEEN HOLES A AND B
AFTER GROUT

Note: 1 foot = 0.305 m

FIGURE 31 - CROSS-HOLE ACOUSTIC VELOCITY PROFILES AT DEMONSTRATION SITE

Figure 32. The before data show acoustic velocities in the general range of 0.5 to 0.7 km/sec. The after-grouting velocities are generally higher, ranging from 0.75 to 1.5 km/sec. These data are also less precise, due to the faster times of flight, which increases the relative error in determination of velocity, as well as the fact that refraction could vary widely from point to point. Had both the test holes been fully inside the grout zone, the after-grouting velocities would have been more consistant and somewhat higher.

The test holes for the geophysical tests were deliberately placed to make these data difficult to obtain. This was done by placing the test holes straddling the edge of the grout zone. In spite of this, the radar indicated the presence of grout (total absorption of the signal) and the acoustic cross-hole shooting indicated significantly higher velocities (stiffer ground) after grouting.

Acoustic Emission Monitoring

The Acoustic Emission (AE) count rate trace for the 12 February shift is shown in Figure 33. Note that the particular instrument used here integrates the count rate over a five-minute interval, so that sharp peaks are traced with several minutes of decay curve. One of the objectives was to determine how best to use the instrument on a noisy urban construction site. Since equipment and other urban noises are generally low in frequency, especially if the sound passes through a moderate distance of soil, the system was set to record emissions lying in that portion of the spectrum between 2,000 hz and 50 kHz. To show the effect of equipment noise, the lower frequency threshold was reduced to 1,000 Hz for several minutes at 10:06 (peak A in Figure 33) and then returned to 2,000 Hz. At 13:50, the low frequency threshold was momentarily reduced to 200 Hz (peak G). The amount of cultural noise that is heard at low frequencies is apparent. On a quiet rural site, the frequency window can be lowered down to 1 Hz without objectionable interference.

Not all equipment noise can be eliminated by filtering. At 12:10, a crawler-mounted pavement breaker began demolishing an existing brick and concrete sewer under the temporary street decking only a few meters from the grout pipes. This produced count rate on the order of 900 counts per minute. By 13:20, this machine had worked its way to about 15 m away and the noise level was much reduced. Peak F, at 13:45, was identified as an air hammer.

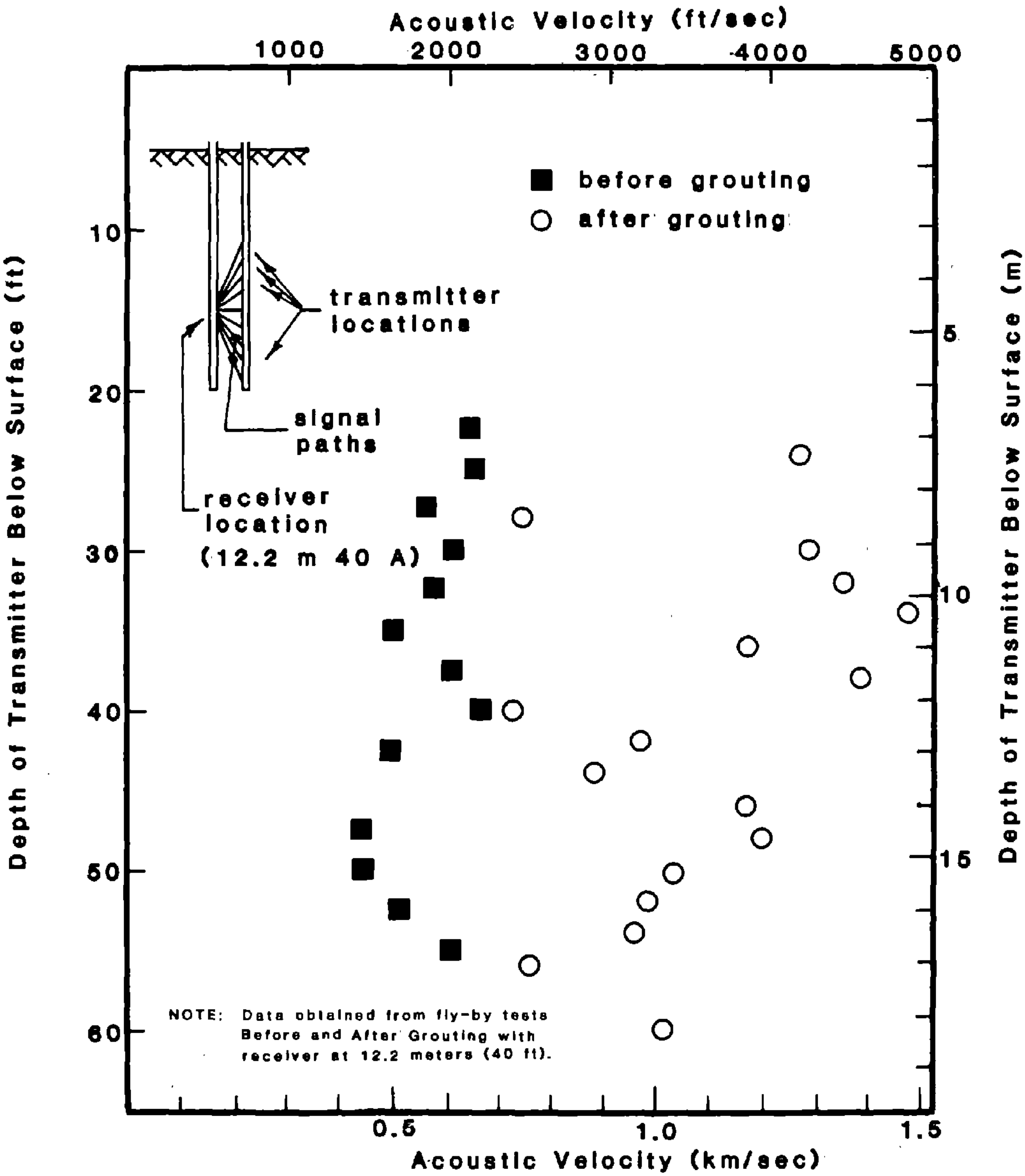


FIGURE 32 - AVERAGE CROSS-HOLE ACOUSTIC VELOCITY DATA FOR DEMONSTRATION SITE

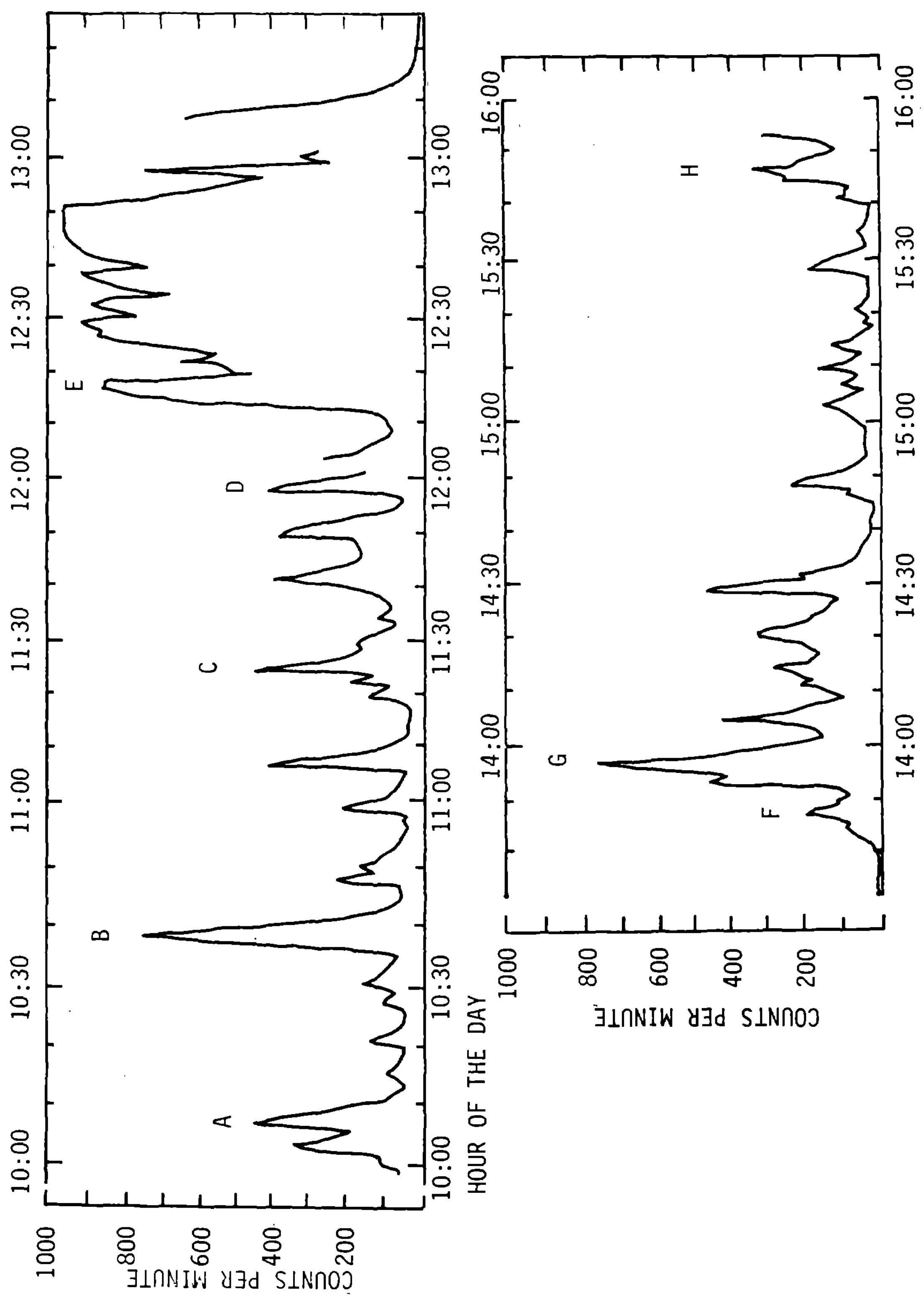


FIGURE 33 - ACOUSTIC EMISSION COUNT RATE TRACE FOR DEMONSTRATION SITE

When injection is started at a new injection point, the first step is to inject high pressure water for a few minutes to break the mortar jacket around the sleeve pipe. Depending upon the location of the AE sensor, this can produce count rates up to 1,000 counts per minute, (CPM). Peaks B, C, D and H, and possibly some of the others, were caused by breaking mortar jackets.

Because the injection specifications prohibited the use of high injection pressures on the grouting contract, hydraulic fracturing tests could not be conducted. In regular commercial use, the AE system should be used at lower sensitivity than was the case at this site. Reducing the sensitivity by a factor of ten would limit interference to tolerable levels and still detect fracturing. The tendency among AE operators is to operate at high gain so that any events in the general area can be detected. This produces noisy records like the one in Figure 33, which show various equipment and other noise sources extraneous to the central purpose of detecting hydraulic fracture.

While these are satisfying to the skilled AE operator, who can point to and identify many events, they are confusing to the engineer who would like to be able to detect hydraulic fracture and nothing else. For this reason, the AE system should be set at a reduced gain so that only emissions from hydraulic fracturing events are likely to be detected. This is possible if the AE operator has a certain amount of experience, and it is much easier if fracturing tests can be conducted at each site, so the level of acoustic emissions will be known with confidence.

Procedural Controls

Pressure and flow rate for four grout lines during one shift are plotted in Figure 34. When the grout lines are fed in parallel from a single pump, the flow rate summed across the grout lines must equal the pump flow rate at all times. If the pump should change speed, all grout lines will reflect this change. If the flow at one grout line is reduced without changing pump speed, the flows at the other grout lines must increase. This is demonstrated by two events in the pressure and flow rate data shown in the figure. Event A, from 11:07 to 11:56, was caused by a gradual reduction in flow rate at hole 3-A as the valve feeding that line was slowly closed. At 11:52 the valve was opened widely, and then trimmed back at 11:56. Thus, the pressure at that hole is seen to creep downward, spike up at 11:52 and then drop back to an intermediate

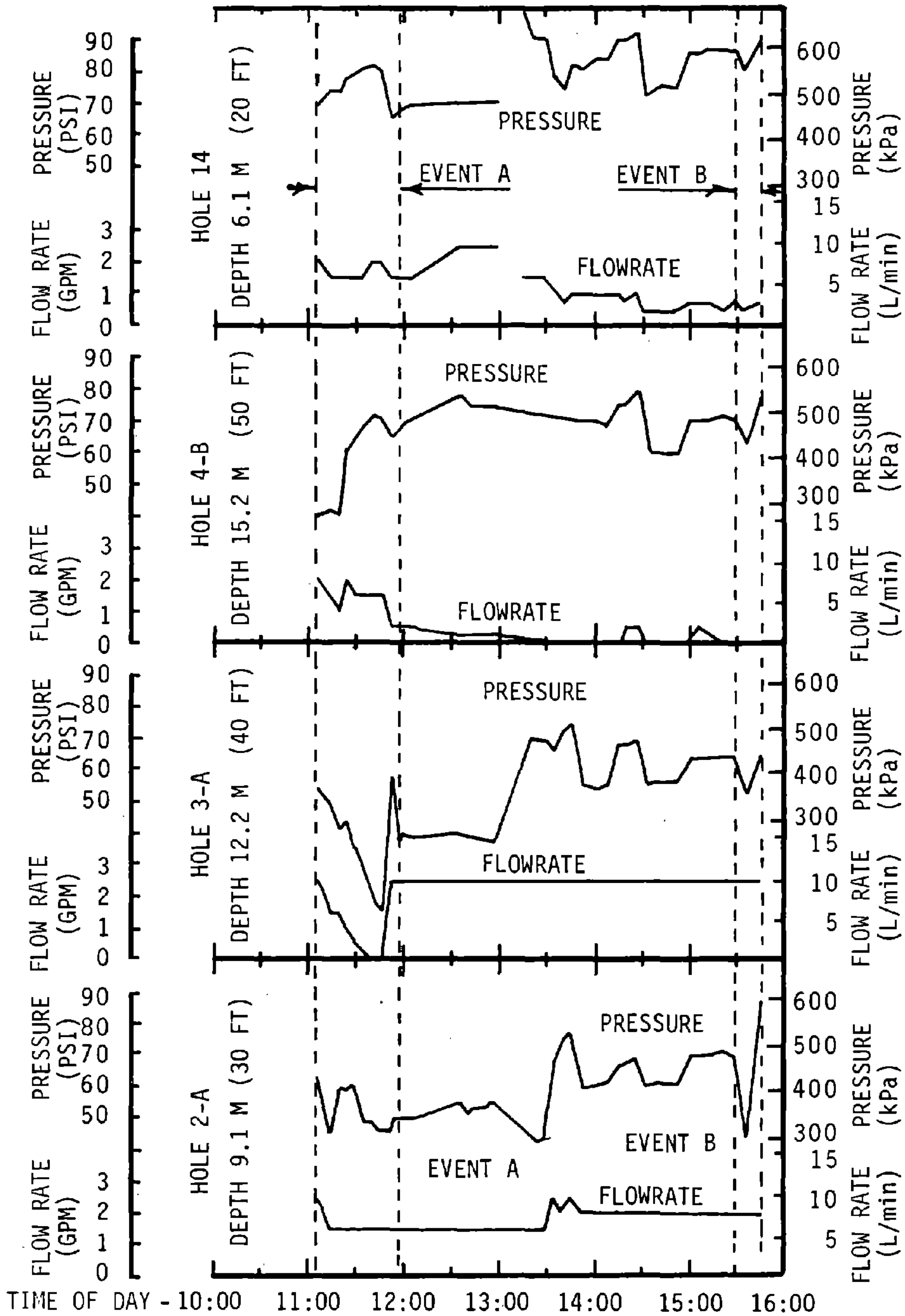


FIGURE 34 - TYPICAL FLOW RATE AND PRESSURE VALUES VS. TIME FOR DEMONSTRATION SITE

value at 11:56. The interaction of the other grout lines is clearly seen at holes 14 and 4-B, which display just the opposite pressure curve.

The interaction at hole 2-A is confused by manipulation of that valve at 11:30. Event B was caused by a momentary reduction in pumping speed between 15:30 and 15:45. It can be seen that all the grout lines respond together, indicating that this event is caused by the pump rather than one of the grout lines. These data show that a detailed recording of injection pressures provides a history of the grouting operations in considerable detail which may be reviewed at any future time.

Hole 4-B displays a characteristic signature of grout refusal--gradually increasing pressure and decreasing flow rate. Had a plot of these data been available in the field, this hole would have been declared to have been grouted by 12:30, and another injection point started in the afternoon. Unfortunately, while the trend is clear on a time-history plot, it is much less obvious when reviewed on manually recorded clipboard data.

A possible example of grout refusal is given at hole 14. This is less clear, however, because the flowmeters used in the manual read-out have very poor resolution, pointing to the need for using flow rate transducers, which have much greater precision than the manually read gages. These have a resolution of about 2 L/m (½ GPM) at best, and were operating near the low end of their range. A flow rate transducer, on the other hand, typically has precision five times better than this, and would show the trends in time much more clearly. The most important factor, however, is that the curves plotted in Figure 34 were not available in the field during injection. Strip-chart recorders should be required if maximum control over the injection process is desired.

Results

Each of the three monitoring techniques met the goals of the monitoring program. The geophysical tests were conducted between two boreholes which were intentionally located to put these systems at a disadvantage by forcing them to look through the edge of the grouted mass. In spite of this, complete absorption of the radar signal after grouting showed that the grout was present, and increased acoustic velocities after grouting indicated that the soil was made stiffer by the grout.

In general, radar may be used to determine the grout location and acoustic velocity may be used to determine the increased modulus of the grouted mass. Because the acoustic signal may be strongly refracted, it is less precise than the radar in determining grout location.

The geophysical tools used in the demonstration required the use of 75 mm diameter PVC cased boreholes. Production work should make use of borehole instruments that can be used in conventional 38 mm diameter grout pipes. Since the completion of the demonstration, custom designed, small diameter radar antennas suitable for this purpose have been fabricated. The acoustic cross-hole velocity should be determined using a high frequency transmitter/receiver pair. Acoustic velocities in grouted soil are high and the path lengths are short. Good high frequency response is required to resolve the short times of flight. Because grouted soil is a good acoustic medium, high signal levels are not critical.

Acoustic emission monitoring confirmed that no hydraulic fracturing events occurred during the intervals monitored. The AE system was operated at high sensitivity, and numerous extraneous events and equipment noises were recorded. In a commercial application of this method, the sensitivity of the instrument should be reduced to eliminate extraneous noises not germane to the task of monitoring for structural distress in the soil.

Detailed records of injection pressure and flow rate identified grout refusal events, and detected the activities of the grouting forces as they maintained control of the operation. Because these data were collected manually and plotted only later, the effects of not having a strip-chart recorder on site were also displayed. The automatically plotted data would have detected trends indicating grout refusal some time before they were seen in the field.

Problem Illustration

The demonstration successfully displayed the ability of the various monitoring systems to operate on an urban grouting project, and exposed weaknesses in the new techniques as they were implemented in actual practice. In addition, skills and methods were sharpened as small errors were made and rectified in the course of the work. As a result of this experience, it became apparent that monitoring instruments that can be used in standard PVC grout pipes will greatly facilitate the monitoring work. Strip-chart recording of actual injection data was shown to be an essential step in providing timely quality control of the injection process.

CHAPTER 9—SPECIFICATIONS

GENERAL

Specifications for chemical grouting differ from conventional construction specifications in that the desired final results require the unusual field expertise of a specialty grouting contractor skilled in chemical grouting. This special expertise is needed for the following steps: (1) development of a grout pipe layout scheme and installation of sleeve-port grout pipes in a precise pattern; (2) development of a rational injection sequence plan with proper allocation of grout volumes to the various grout ports; (3) proper operation of the grout mixing and injection system in harmony with the actual ground response; (4) continuous recording (preferably automatic) and graphical display of the injection data; and (5) quality assurance acceptance testing. The integration of the technical and mechanical skills required by the above is so complex as to preclude the design engineer from directing the exact details of the work.

The following Guide Specifications for chemical grouting are written to require the grouting contractor to bring the necessary expertise to the job and to perform and organize his work according to the above five-step outline so as to accomplish the established purpose. Done in this way, the work can be easily monitored by the construction management staff on the job, and performance problems will be quickly highlighted and more easily corrected. Accordingly, these specifications define the intent and extent of the work, establish specialty contractor qualifications, set criteria for grout selection, describe acceptable pumping equipment types and operating procedures, specify grout pipes, define injection procedures and quality control, and establish the basis for acceptance and payment. These Guide Specifications reflect the principles presented in previous chapters, generally applied to structural chemical grouting. They should never be used directly, but should be adapted to the specific conditions and needs of a particular project.

GUIDE SPECIFICATIONS FOR CHEMICAL GROUTING

1. SCOPE

The work covered by this section consists of furnishing all supervision, labor, materials and equipment necessary to perform the required chemical grouting as hereinafter specified and as outlined on the contract drawings.

2. INTENT

The purposes of the chemical grouting program are either (1) to increase the strength and stiffness of the groutable soils in order to reduce surface settlements due to subsequent excavation operations to less than the values shown on the contract drawings, or (2) to impermeabilize the affected soils below the existing groundwater table so as to permit excavation without dewatering or, (3) both.

2.1 Structural Chemical Grouting

In the zones specified for structural chemical grouting, the chemical grouting shall be performed in such a way as to produce a continuous mass of structural chemically grouted soil as shown on the drawings. Some zones will require combined structural and waterproof chemical grouting effects.

2.2 Waterproof Chemical Grouting

In the zones specified for waterproof chemical grouting, the chemical grouting shall be performed in such a way as to produce a continuous wall of impervious chemically grouted soil below the water table, to act as a plug or dike, so as to prevent the flow of water beyond.

3. QUALIFICATIONS

The work shall be performed by a Grouting Subcontractor specialized in chemical grouting. The Contractor shall establish, to the satisfaction of the Engineer, that the planning for chemical grouting and the actual placement of grout pipes and the mixing and injection of chemical grout is performed by an experienced Grouting Subcontractor that has completed at least five (5) chemical grouting projects of similar scope and purpose, and that is experienced with the use of the specified continuous mixing procedure, automatic recording equipment, and types of chemical grout. The Contractor shall also establish to the Engineer's satisfaction that the on-the-job supervision of all chemical grouting is under the direction of a Grouting Engineer with at least three (3) years actual on-the-job supervision in similar applications, assisted by an experienced chemical grouting foreman on each grouting shift.

4. CHEMICAL GROUT MATERIAL

4.1 Structural

Structural chemical grout will be composed of liquid sodium silicate, approved reactant, water and accelerator, if required. The design chemical grout mix shall be such that, when injected into medium dense Ottawa 20-30 sand, the unconfined compressive strength of the grouted soil shall average at least 7 kg/cm^2 (686 kPa) and the unconfined initial tangent modulus shall average not less than 7 kg/cm^2 (686 kPa).

4.1.1 Sodium Silicate

The base material for the structural chemical grout shall be liquid sodium silicate, which shall have a specific gravity of 1.4 to 1.5 (41.5^{o} to 48.3^{o} Baume) and a silicate to soda ratio in the range of 3.20 to 3.35. The minimum sodium silicate concentration shall be 50% of the mix by volume. The sodium silicate should be delivered in sealed containers or certified tank truck and shall be accompanied by the supplier's certificate of origin. Sodium silicate in ungelled liquid form, while not considered toxic, is strongly alkaline and shall be handled by authorized personnel only.

4.1.2 Reactant

The reactant shall be of organic base type and shall, when properly mixed with the other grout components, provide a permanent, irreversible gel with controllable gel times. The resulting gels shall exhibit less than 15% syneresis in 30 days when mixed with appropriate amounts of sodium silicate, water and accelerator, and shall not exhibit objectionable odors such as ammonia. The reactant shall be delivered in sealed containers accompanied by the supplier's certificate of origin.

4.1.3 Accelerator

The accelerator, if required, shall be technical grade, water soluble calcium chloride or other approved metal salt and shall contain a minimum amount of insolubles.

4.1.4 Water

Water used with grout shall be free of impurities that will affect the grout.

4.2 Waterproof Chemical Grout

Waterproof chemical grout may consist of approved structural chemical grout or any alternate non-toxic chemical grout with good gel time control and which, when combined with 20-30 Ottawa sand, will provide an unconfined compressive strength of 2 kg/cm^2, (196 kPa) will not show more than 15% syneresis during the service life of the grouted zone and will reduce the permeability of the grouted sand to at least 10^{-6} cm/sec. Acrylamide-base grouts will not be permitted.

5. EQUIPMENT

5.1 General

All chemical grouting equipment shall be of a type, capacity and mechanical capability suitable for doing the work. The equipment shall be maintained in first class operating condition at all times. Any grout hole that is lost or damaged due to mechanical failure of equipment, inadequacy of grout supply, or improper injection procedure shall be properly filled and replaced by another hole, drilled by the Contractor at his expense.

5.2 Pumps

The chemical grout plant shall be of the continuous mixing type and shall be capable of supplying, proportioning, mixing and pumping the grout with a set time between 5 minutes and 50 minutes. Batch-type systems will not be permitted. Each main pump shall be equipped with recording, positive displacement meters. The meters shall be constructed of materials that are non-corrodable for the intended products and shall operate independent of the viscosity of the metered fluid. The pumping unit shall be capable of varying the rate of pumping while maintaining the component ratios constant.

5.3 Piping and Accessories

The pumping unit shall be equipped with piping and/or hoses of adequate capacity to carry the base grout and reactant solutions separately to the point of mixing. The hoses shall come together in a 'Y' fitting containing check valves to prevent backflow. The 'Y' fitting shall be followed by a suitable baffling chamber. A sampling valve shall be placed beyond the point of mixing and the baffling chamber, and shall be easily accessible for sampling mixed grout. A water flushing connection or valve shall be placed behind the 'Y' to facilitate flushing the grout from the mixing hose and baffle between grouting sessions. Distribution of proportioned grout, under pressure, to the grouting locations shall be monitored by separate, automatic recording, flow rate indicators and gages.

5.4 Chemical Tanks

Chemicals shall be stored in metal tanks, suitably protected from accidental discharge by valving and other necessary means. Tank capacity shall be sufficient to supply at least one day's worth of grouting materials so as not to interrupt the work in the event of chemical delivery delays.

5.5 Testing

The Contractor shall provide at the site all necessary chemical quality control testing apparatus, including but not limited to: hydrometers, balance scales, graduates, viscometers, and all other devices that are required to conduct chemical material acceptance tests, chemical proportioning tests, and grout quality tests for proper quality control of the work. The Grouting Subcontractor shall submit certified laboratory testing results documenting the required performance of the proposed chemical grouting at least 30 days prior to the commencement of injection operations.

6. INSTALLATION OF GROUT PIPES

Grout pipes may be installed horizontally, inclined, or vertically to obtain the specified minimum grout coverage, with a maximum average spacing between adjacent grout pipes of 1.5 m. The grout pipes shall be of the sleeve-port type, with grout ports at minimum 50 cm centers covered by expandable rubber sleeves. After being placed in a borehole, the sleeve-port grout pipes shall be encased in a continuous brittle mortar sheath. An internal double packer shall be used to inject grout at a specific sleeve-port.

7. CHEMICAL GROUTING PROCEDURES

7.1 Work Plan

At least 30 days prior to the start of the drilling work, the Grouting Subcontractor shall submit a detailed Chemical Grouting Work Plan, specifying the chemical grout to be used, grout-hole and grout-port locations, grout-pipe installation procedures, grouting equipment, injection procedures and sequences, recording equipment, data reporting methods, and schedules. The Plan shall show the basis for establishing grout target volumes at each primary and secondary grout port.

7.2 Grout Mixing Method

The method of injection for chemical grouting shall be the continuous mixing method, with the proper amounts of sodium silicate base material, water, reactant, and accelerator automatically proportioned and continuously supplied at proper flow rates and pressures. The batch system of mixing grout shall not be permitted. The base material and the water-accelerator-catalyst solution shall pass through parallel separate hoses to a suitable baffling chamber near the top of the hole. A sampling cock, to allow frequent gel time checks, shall be placed after the baffling chamber. Suitable check valves shall be placed in the grout lines at the proper locations to prevent backflow.

7.3 Injection Procedures

Using double packers, chemical grouts shall be injected into the design zones through grout ports in the sleeve pipes. The grouting pressure for any one pipe shall not be more than 0.3 kg/cm^2 (29 kPa) per meter of depth, unless acoustic monitoring is to be performed in adjacent boreholes to detect hydraulic fracturing, in which case pressures may be increased as desired up to 0.75 kg/cm^2 (74 kPa) per meter of depth. Detection of excessive hydraulic fracturing as determined by using acoustic monitoring equipment placed in adjacent grout pipes will require reduction of injection pressure. Surface elevation monitoring will be carried out continuously during grouting. Injection procedures will be adjusted as needed to prevent excessive surface heave. Temporary very high injection pressures will be permitted to crack open sleeve-ports, but these pressures will not be permitted for longer than 1 minute duration. In any event, the rate of injection into any port shall not exceed 40 liters per minute.

7.4 Gel Times

All grouts shall have a gel time between 5 minutes and 50 minutes, with most grout having gel times in the range of 10 to 40 minutes. Samples shall be obtained for gel time checks at least one for every half hour of pumping or for every 2,000 liters of grout, whichever is more frequent. Gel samples shall be properly labeled and stored until the completion of the project.

7.5 Record Keeping

Accurate and timely records of all chemical grouting shall be kept by the Grouting Subcontractor and submitted to the Engineer. These records shall include, but not be limited to, grout mix, gel time, injection date and time, injection pressure and rate, injection volumes, and exact injection location. In addition, these data shall be displayed in an acceptable chart-type format that facilitates rapid visual evaluation of the results of the work. This display shall be updated daily.

7.6 Quality Assurance

Prior to the commencement of tunneling thru a grouted zone, the Contractor shall demonstrate, using either soil sampling methods or geophysical methods such as radar, acoustic velocity measurements, or other means satisfactory to the Engineer, that the grouting zones have been thoroughly impregnated and stabilized with chemical grout. Tunneling thru grouted areas shall not commence until the chemical grouting work has been completed and accepted by the Engineer.

8. PAYMENT FOR CHEMICAL GROUTING

Payment shall be made for the work based on the following unit prices:

8.1 Mobilization/Demobilization

The cost of assembling all plant, personnel and equipment at the site preparatory to initiating the global chemical grouting program, and the cost of removing it there from when the chemical grouting program has been completed, will be included in the contract lump-sum price for "Mobilization and Demobilization, Chemical Grouting."

Eighty (80) percent of the contract lump-sum price for "Mobilization and Demobilization, Chemical Grouting," will be paid following completion of moving onto the site, including complete assembly in working order, of all equipment necessary to perform the required chemical grouting operations. The remaining twenty (20) percent of the contract lump-sum price will be paid when all equipment has been removed from the site and the areas cleaned up.

8.2 Placement of Grout Pipes

Grout pipe placement shall be measured for payment on the basis of the number of meters of sleeve-port grout pipe properly placed, measured from the ground surface or face of excavation to the bottom of the pipe.

8.3 Injection of Chemical Grout

Injection of chemical grout shall be measured for payment by the liter of liquid chemical grout properly mixed and injected.

8.4 Injection of Waterproofing Chemical Grout

Waterproof Chemical Grouting will be measured for payment by the liter of liquid waterproofing chemical grout properly mixed and injected.

8.5 Combined Structural and Waterproof Chemical Grouting

Combined Structural and Waterproof Chemical Grouting, in which both structural chemical grouting and waterproof chemical grouting effects are required, shall be measured for payment by the liter of liquid grout properly mixed and injected.

8.6 Quality Control and Testing

Quality control and testing of the grouted zones will be paid under the chemical grout items per liter and will not be paid separately.

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