# SECTION 6B
# GROUTING TO IMPROVE FOUNDATION SOIL

**JAMES WARNER**


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## *6B.1 INTRODUCTION*

Geotechnical pressure grouting consists of forcing grout under pressure into a subsurface formation. The grout can be a liquid solution, fluid suspension, slurry, or of a stiff mortar-like consistency. It will generally harden at some point after injection so as to become immobile. Prior to the last few decades, most grouting was done to fill discontinuities in rock, with the primary purpose of reducing water movement through the formation. Now, in addition to water control, grouting is also extensively used to strengthen soil, either permanently or temporarily as an aid to construction.

Grouting is not new. In fact, records abound of grouting projects throughout the 1800s and even prior to that. Most of the early work involved injection of aqueous suspensions or slurries, often containing cement, lime, or clay, into joints and seams in the bedrock underlying dams in order to reduce water leakage. As early practice involved filling of seams or voids, the grout had to be very flowable, and the maximum particle size considerably smaller than the thickness of the particular discontinuity to be filled. Because the pore spaces in soils are generally much smaller than typical rock joints, injection of particulate grout had limited success in soil.

Accordingly, low-viscosity chemical "solution" grouts, which could permeate soil formations and chemically harden, were developed. In 1887, a patent was issued to Jeziorski for a sodium silicate based formulation, which could be mixed on-site and injected. Unfortunately, the chemicals would react soon after mixing, requiring very rapid injection, and all too often, would harden in the injection hoses and delivery system. This limitation restricted the application of these early grouts in soils.

To overcome the problem of early hardening in the delivery system, Hugo Joosten, a Dutch mining engineer, developed a two-shot sodium silicate based grout system, which was patented in the United States in 1925. Therein, the sodium silicate base chemical was first injected into the soil, followed by injection of a reactant chemical, commonly calcium chloride, which would cause the silicate to harden. Although Joosten's system was used until the late 1960's, difficulty in achieving complete mixing of the two components in the ground discouraged it's widespread use.

In the 1950s, large chemical suppliers started marketing a variety of solution grout systems, primarily focused at the reduction of water movement through soils but for soil strengthening applications as well. Many of those systems are no longer available due to environmental restrictions or economic considerations. Although commercially available chemical grout systems remain available, sodium silicate based grouts are the most widely used, especially for soil strengthening work. The earlier limitations of rapid setting have been overcome in modern formulations. Many different reactant systems for silicate base grouts are readily available, and it is common for the grouts to be proportioned on the job site by the specialist contractor, using his own particular reactant system and formulation. There are also a number of proprietary sodium silicate base grout systems commercially available. For water control grouting, more sophisticated chemistry is generally involved, and commercially available formulations are almost exclusively used.

Another significant contribution to the current state of the art in grouting of soils was the development of the "mudjack" machine in 1933. The original objective of the machine was the filling of voids under, and raising of, settled concrete pavement. As conceived, a mixture of "loam" or clayey soil was used. As experience with the machine was gained, it was found that the addition of portland cement resulted in a stronger and more durable grout material. Likewise, it was found that by varying the consistency of the grout, a wider range of work could be accomplished. As a natural outgrowth of their work, some of the more inventive operators of the mudjack attempted stabilization of the soil by various means, including pumping of relatively stiff mud mixes into predrilled holes. Although early work was performed on a somewhat "hit or miss" basis, and with little engineering input, a great deal of knowledge was gained and such work was actually a forerunner of the "compaction grouting" process that is now widely practiced.

In the 1940's it was found that mixing a source of calcium with clay soil was beneficial to strengthen and/or reduce the expansiveness of clay. High calcium hydrated lime became the material of choice for such mixing, due to its economy and wide availability. The practical depth for physical mixing was limited however, and a method for deeper mixing was needed. This resulted in use of pressure injection of lime slurries, using grouting techniques. Lime injection continues to be utilized, primarily in geographic areas where swelling clay soils predominate.

### 6B.1.1 Foreign Development

A very significant amount of our present grouting knowledge is the result of foreign development, primarily from the European countries, where extensive use of both solution chemical grouts and particulate suspension grouts has occurred. Unlike American practice, much European work is performed on a design–build basis, and very large projects are commonplace. This has resulted in large multidiscipline firms possessing the capability to perform research and development, design engineering, and the actual construction, as well as design–build and maintain the very specialized equipment often utilized. Such ability to integrate the various disciplines combined with strong financial capability has resulted in many technical advances. There has, however, been reluctance on the part of such firms to share their knowledge, and in fact, it is not unusual for their proposals to be completely void of any mention as to the exact procedures or materials to be used, but rather only the end product to be obtained.

Unique to European work is the use of large central grout plants (Figure 6B.1), often containing a number of different types of mixing and pumping units. Such plants are nearly always provided with automated continuous monitoring and recording equipment, and the operation is usually under the direct supervision of a professional engineer. With exception of techniques that rely on the use of massive equipment such as jet grouting, the European philosophy for grouting has remained largely unchanged in recent times, and although improved and refined, solution and fluid suspension grouts continue to be used almost exclusively.

### 6B.1.2 U.S. Development

Contrary to the European experience, development of soil grouting in the United States was somewhat erratic and primarily the work of small, widely dispersed specialty contractors. The more common grouting of rock for dam foundations was under the control of one of three large federal government agencies: The Army Corps of Engineers, Bureau of Reclamation, and Tennessee Valley Authority. These agencies generally designed and supervised their grouting operations in-house, to the point that the contractors basically furnished only labor, equipment, and materials, performing the work strictly as directed by the agency. This effectively discouraged research or development by the contractors, resulting in little interaction between those contractors doing the more traditional dam foundations and those performing work other than dam-related, which included virtually all grouting of foundation soil. Unfortunately, a shroud of secrecy, often embraced by the latter group,

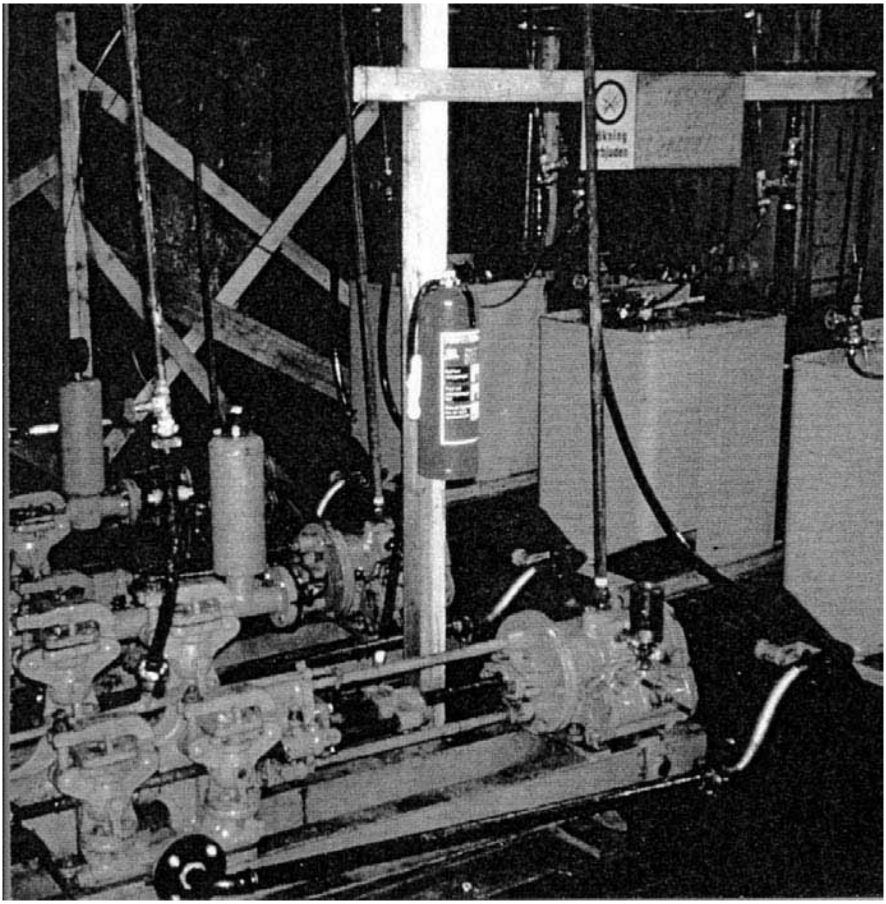

**FIGURE 6B.1** Large European central grout plant.

prevented the sharing of experiences, so that neither widespread knowledge nor use of their developments occurred.

It is only within the last few decades that extensive knowledge of grouting technology has become available. A major contributor to this change has no doubt been the week-long Short Course on Fundamentals of Grouting, now sponsored by the University of Florida and held annually each year since 1980. Additionally, the American Society of Civil Engineers Geo-Institute has sponsored specialty conferences, as well as a number of sessions, at their gathering, on grouting. Sadly however, despite the dissemination and sharing of knowledge, some firms continue to maintain a "shroud of secrecy" in an effort to convey the idea that only they are capable of the "magic touch." Even worse is the sponsorship of publications and seminars that often provide incorrect information, favorable only to the sponsor. Such marketing efforts, often lack technical accuracy and incorrect information is frequently provided. Whereas foundation grouting knowledge, is now fairly well developed, many practitioners, especially design engineers, remain unaware and/or misinformed, of certain well-established technology.

## 6B.2 WHY GROUT IN SOIL?

There are many reasons for grouting in soil, but the most frequent is to strengthen it. This can be accomplished through densification, increasing the cohesion of granular soils, or in the case of clay, altering the chemistry. Another major area of work involves control of subsurface movements of water and reducing the permeability of soil.

### 6B.2.1 Densification to Prevent or Arrest Settlement and Mitigate Liquefaction

In the United States, compaction grouting is the most frequently used procedure for densification of existing soils. It is widely used for arresting settlement of structures of all types (see Figure 6B.2). It also is widely used for predensification of faulty soils, prior to construction, as well as soil densification under existing structures for mitigation of the liquefaction potential during earthquakes. An extension of compaction grouting, which involves essentially the same equipment and grout mixtures, is "groutjacking," which can accurately raise or level settled structures and effect other improvements.

Compaction grouting is uniquely American and is the only major grouting technique to originate in the United States. Its early development was in California in the mid to late 1950s. It is only within the last few decades however, that the procedure has been extensively used. Although much research has occurred and the technology is well documented, some practitioners fail to utilize the proper procedures, resulting in improper and often incompetent performance. It is unfor-

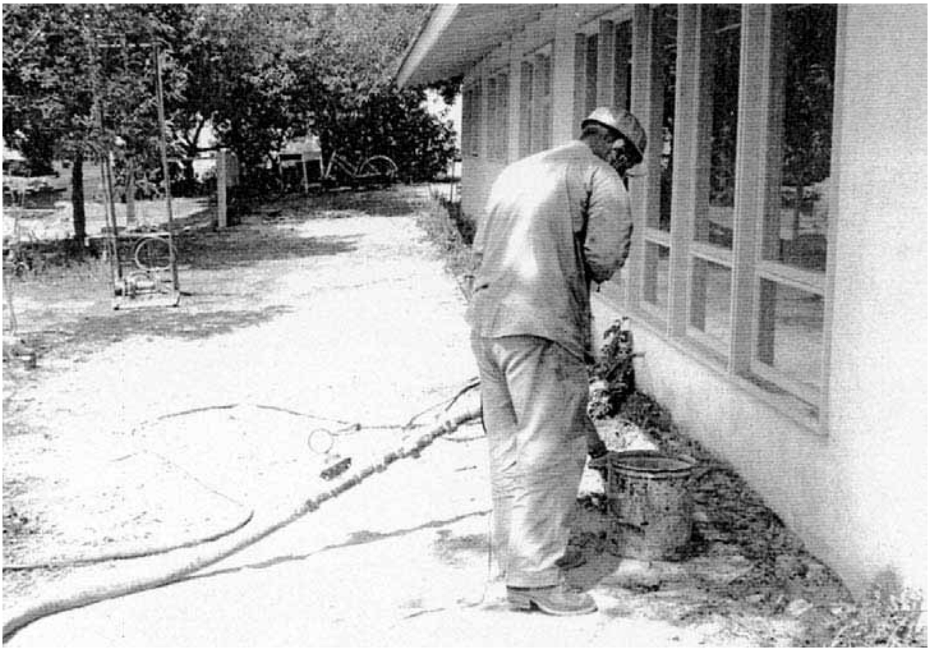

**FIGURE 6B.2** Compaction grouting under a structure to correct settlement damage.

tunate that such poor performance has in many instances resulted in limited effectiveness, and sometimes in actual damage to the structure it was meant to repair. Although the greatest use of compaction grouting remains in the United States, it is now recognized and used worldwide, especially in Asia.

### 6B.2.2 Soil Solidification to Increase Cohesion of Granular Soils

Another form of strengthening is the solidification of sandy soils, wherein the individual grains are bonded together with a chemical solution or a fluid suspension grout (Figure 6B.3). There are a variety of chemical solution grouts, but they all work in essentially in the same manner. A base material, which usually represents the majority of the final mix and is often diluted with water, is combined with one or more reactant materials. The base component is usually fluid, whereas the reactants can be either liquid or powder, depending upon the individual grout formulation. At some period of time after mixing the base and reactants together, setting occurs, and the injected solution turns either to a foam, gel, or solid state. The setting time can be instantaneous to several hours, depending upon the individual system used and the environment at the work site.

Many chemical grouts are mixed and pumped as single solutions, whereas the different components of others are individually pumped and stream mixed at the injection point. There are several manufacturers of proprietary chemical grout formulations, especially those used for control of water movement within the soil or into underground substructures. For strengthening applications, especially where large quantities of grout are required, however, many specialty contractors tend to purchase the individual chemical components separately and mix them on the job site near the point of injection. Although some specialists tend to represent their formulations as something very special and "proprietary" (a word greatly overused, in the writer's opinion), the vast majority of grouts now used for strengthening of soil throughout the world involve sodium silicate as the base materi-

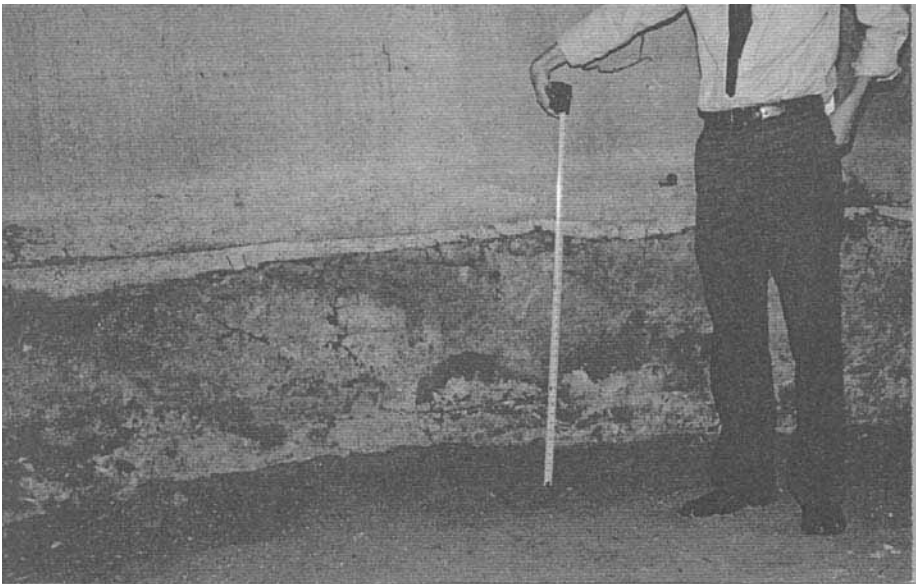

**FIGURE 6B.3** Sandy soil solidified to a solid mass.

al. There are however, a wide variety of different reactant systems, most of which are well known and readily available.

With the increasing availability of ultrafine cements, many of which are able to readily penetrate the pores of even fine sands, suspension grouts of these cements are increasingly being used for soil solidification. They usually result in a stronger and more predictable solidified mass than do the chemical solution grouts, and are often less costly to apply. Also, unlike many chemical grouts, they do not lose strength over time.

European practice for soil strengthening continues to favor chemical solution and/or cementitious suspension grouts. Because solution grouts are usually much more expensive than their common cementitious counterparts, there is a natural tendency to use the latter if feasible. Recognizing that their grain size often exceeds that of the soil pore space, applicators have developed highly refined injection procedures wherein a discreet amount of grout is placed at regular intervals along a grout hole. The grout tends to fracture the soil but remains near its intended location, as only a small amount is injected at each interval. The amount of densification that results from any single hole is limited, as the quantities of grout injected are usually small and considerably less than typically used in compaction grouting. This is necessary, as once grout has created a fracture in the soil, it is virtually impossible to control its deposition distance or direction. Relatively close spacing of the grout holes and limitation of the grout quantity at each hole location is thus imperative in order to maintain control of the injection.

### 6B.2.3 Reduce Permeability, Water Control

Historically, cementitious suspension grouts have been used to stop the flow of water through cracks and joints in rock. The pore size of most soils, however, is significantly smaller than the width of typical rock joints, and except in the case of coarse sands and gravels, it is insufficient for intrusion of a common cementitious, particulate grout. Thus, water control in soils is almost always accomplished with either chemical solution grouts or suspensions based on ultrafine cement. Whereas hard rigid chemical gels are preferable for strengthening applications, some flexibility is usually desirable in water control work, and thus different chemical grout systems are in order. Dilution of the grout with groundwater is always a concern, and where rapid flow of the water occurs, the grout must be resistant to being washed away as well. This often means use of a rapidly setting grout. A variety of such materials are widely available that provide instantaneous setting, as shown in Figure 6B.4. These grouts are usually more chemically complex than those used for strengthening of soil. Some formulations present health risks and require special handling. They are thus most often obtained as proprietary systems from specialty grout producers.

### 6B.2.4 Stabilize and Reduce Expansion of Clay Soils

Unlike granular soils, clay consists of microscopically small, flat plate-like flakes. Clay platelets are generally colloidal in size, which means that they will float rather than sink in water. Water between the individual plates, thus contributes significantly to the volume of a clay soil. Because of the minute size of the individual particles, the movement of water in clays is very slow, and consolidation of clay soils, which results from expulsion of the water component, occurs slowly. Changes in the water content of clays can produce either expansion or shrinkage of the soil.

Mixing a source of cations such as calcium with clay has been shown to stabilize the water content, thus producing a more stable soil material. The most frequently used calcium source is lime, which is available in two forms—quicklime and hydrated lime. When these materials are physically mixed with clay soil, base exchange and pozzolamic reactions occur, which are fairly well understood. When lime slurries are injected at depth (Figure 6B.5), beneficial results are obtained; however the exact response is not predictable. Lime injection has been practiced in the United States for

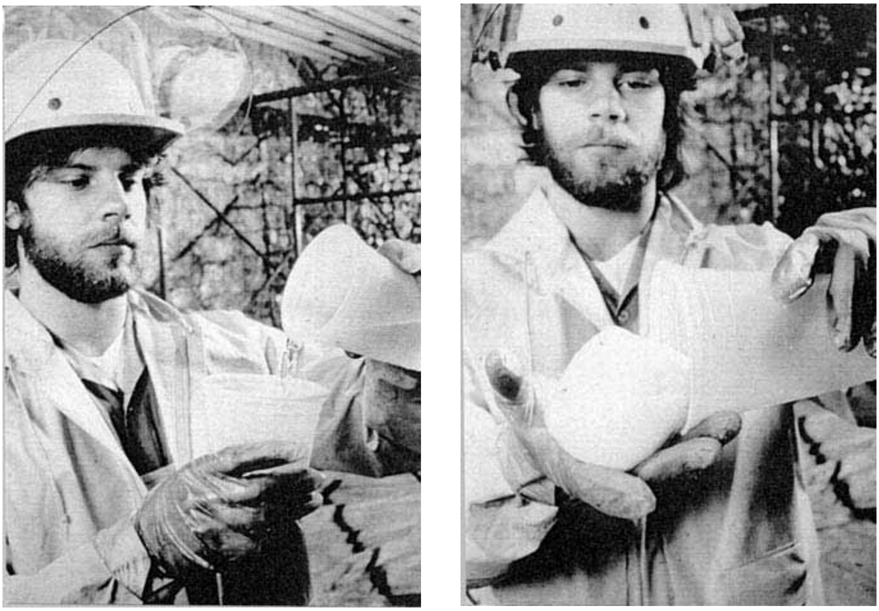

**FIGURE 6B.4** Freshly mixed grout flows freely from one cup to the other (a) but gels midstream a few seconds later (b).

more than 50 years with varying degrees of success. The principal beneficial mechanism is believed to be an ionic exchange that improves the basic chemistry of the clay and facilitates stabilization of the soil moisture content, resulting in a more stable condition.

### 6B.2.5 Compensating for Lost Ground and Filling Large Voids

Grouting methods are often used to fill voids resulting form geotechnical construction in soil. Cementitious suspensions and slurries are widely used to compensate for lost ground adjacent to structures, resulting from loss of support; during soft ground tunneling; or in other underground construction. The work is usually referred to as *contact* or *compensation grouting*. Although the two terms are sometimes used interchangeably, *contact grouting* is preferable for reference to the filling of voids between the structure and the host formation, whereas *compensation* usually refers to larger voids, which may or may not be in contact with the structure.

Where massive voids, such as sinkholes or abandoned mines, are filled, neither specialized grout mixtures nor equipment are required, the work generally being performed with standard concrete pumps and using ready-mixed mortar or grout (Figure 6B.6). Grouting contractors often disguise concrete pumps with modified sheetmetal coverings and sometimes modified hoppers, to give the appearance of special technology. The fact is, however, that standard concrete pumps and ordinary ready-mix concrete and mortar mixes, as well as standard controlled low-density cementitious mixes, can perform quite adequately in such applications, which the writer prefers to refer to simply as *fill grouting*. Concrete technology is highly developed, and established ready-mix concrete suppliers are capable of furnishing mixes with a wide variety of special properties, using established admixture technology.

**FIGURE 6B.5** Injection of lime slurry into a clay soil.

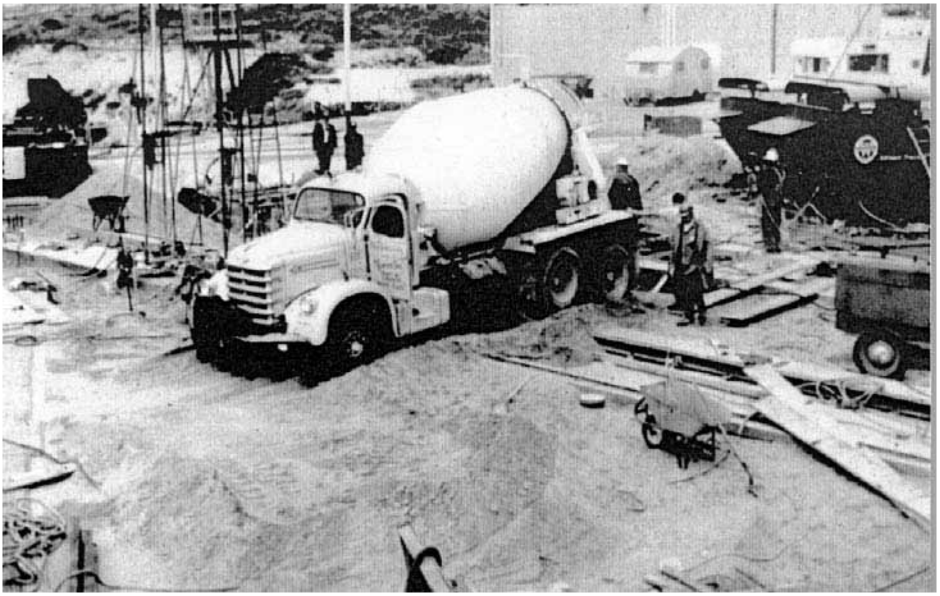

**FIGURE 6B.6** Use of a standard concrete pump for filling a sinkhole with ordinary ready-mixed concrete.

## 6B.3 TYPES AND PROPERTIES OF GROUT

A wide variety of grout materials are available. These range from water-like fluids to very low mobility, stiff mortar-like cementitious mixtures, as shown exiting a grout hose in Figure 6B.7. Selection of the optimal grout material is one of the most important parts of any grouting program. Whereas some job requirements can be adequately accomplished with any of a number of different grout materials, others require very close control of a single one. Some grouting contractors are capable of successful application of many different types of grout, whereas others specialize in only one type of grouting or class of materials. As an example, there are a number of contractors who limit their work to water control grouting, using urethane chemical solution grouts. Others, although strongly favoring one particular procedure or class of material, will offer to perform the work with any that might be specified. Unfortunately, all too often, after a contractor has procured a job, he finds "need" to substitute his preferred grout material or system for that which was originally contemplated, even though it might not be in the best interests of the project (being either more costly or less effective). Available grouts can be divided into the following four classifications.

### 6B.3.1 Fluids and Solutions

These grouts are composed of two or more components that are either premixed and pumped as a single solution or stream mixed, wherein the various components are blended together as they enter the grout hole by way of siamese fittings at the collar of the hole. Some solution grouts are very thin water-like fluids, whereas others can have considerable resistance to flow. Most grout manufacturers and many grouting professionals refer to the *viscosity* of a particular grout, and infer that the in-

**FIGURE 6B.7** Stiff mortar-like grout extruding from grout hose.

jectability is directly related to that property. This is incorrect, however, as viscosity is only one of the properties that contribute to a grout's injectability, which subject will be much more thoroughly discussed shortly. The injectability properties of a proposed solution grout must also be related to the grain size and resulting permeability of the formation to be grouted. As an example, a very highly penetrable grout can "run away" from its intended injection location in a very permeable formation, whereas a less-penetrable grout would remain where intended.

A search of the chemical solution grouting literature will uncover much discussion as to the effect that the soil permeability has upon the viscosity of the grout to be used. This can be quite misleading, however, as such data seldom considers pumping rates in its conclusions. As an example, the writer injected several chemical grouts with widely differing viscosities in a full-scale field test. The test was in a deposit of fine to medium sand containing trace amounts of silt. It was interesting to note that whereas an acrylamide grout with a viscosity of about 1 cps (similar to water) could be injected at a rate of about 7 GPM (27 LPM), a much more viscous sodium silicate based formulation, with a viscosity of about 18 cps, was able to be injected into the same formation, although at a slower pumping rate of about 2 GPM (8 LPM).

Setting times of chemical solution grouts can be instantaneous to several hours, although for most work it is desirable for the material to set soon after injection. This is especially true when working below the water table, or where the water is moving, in order to preclude dilution or washing of the grout beyond its intended place of deposition. Grouts that resist dilution are available and should be used where such a risk exists. Also, grouts are available with extremely short or even zero setting times, and in the case of some urethanes, an instantaneous expansive foam will develop when the grout first comes in contact with water. In instances where moving water is involved, these special grout materials, which can be quite expensive, become very useful and ultimately very cost effective.

It is important to understand the setting behavior of chemical solution grouts, especially where dilution is likely to occur. Some formulations will remain at or near their initial viscosity until shortly before setting (Figure 6B.8, Curve A). Other grouts will start to thicken immediately upon mixing, until they finally set (Fig. 6B.8, Curve B). Either of these setting behaviors can be advantageous, and might even be required, depending upon the particulars of the individual application. Some grouts are hydrophobic, that is, they repel water, while others are hydrophilic and attract water. Still others will immediately foam or gel upon contact with water. These properties can have a significant impact not only upon the injection operation, but the performance of the grout once in the ground. The ability to resist dilution, and/or extrusion or movement of the grout within the formation, or syneresis (a squeezing of the water from the gel), are also important criteria in selecting grouts to be used in saturated soils.

#### *6B.3.1.1 Chemical Solution Grouts for Strengthening Soil*

For the strengthening of soils, the most important parameters for selection of chemical solution grouts are safety, strength, and cost. Many of the chemical grouts historically used have been found to be toxic, and thus were removed from the market. In considering any new grout formulation, it is thus imperative to assess any possible risk that might exist in its use. Hard, rigid gels are desired as they provide the greatest stiffness as well as providing good strength. Cost can also be an important factor, because this type of work is often performed on a massive basis where very large quantities of grout are involved. With these factors in mind, grouts based on sodium silicate are the most frequently used for strengthening applications. Appropriately, the technical literature is filled with publications relative to the use of sodium silicate based grouts. Unfortunately, most of it is quite academic, and some downright misleading.

As an example, several publications present strength data for "sodium silicate grouts," but make no reference whatever to the reactant system. Whereas sodium silicate may represent the largest component of a chemical solution grout, it requires a reactant component to produce the hardened end product. There are many different reactant systems, which may consist of one or several components, as will be shortly discussed, and the properties of the resulting grouts will vary greatly with the different systems. The combination of sodium silicate with *some* reactant systems (such as for-

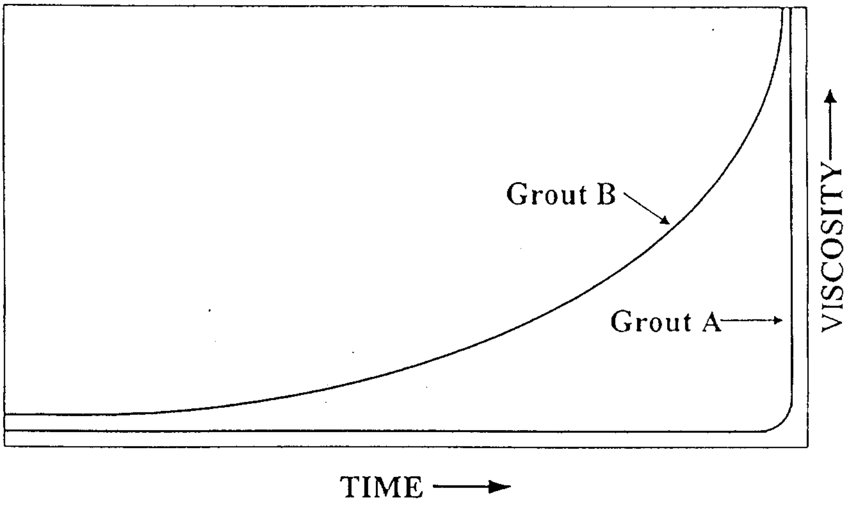


**FIGURE 6B.8** Very different thickening modes for solution grouts.

mamide or inorganic acids) results in a grout that will break down and lose strength with time, while other mix combinations (such as dibasic esters or gyloxol) have proven to be durable over long periods of time. ASTM has produced a standard for the strength of grouted masses, D 4219, "Standard Test Method for Unconfined Compressive Strength Index of Chemical-Grouted Soils." This standard is addressed only to the "short term" strength, which certainly precludes its applicability to permanent installations.

Graf et al. (1982) reported on the long-term aging effect, of chemically grouted soil specimens that were aged from 9 to 11 years. They concluded that "Except for the very weakest samples, the strengths of the stabilized soils tested with no environmental change, showed no change or a modest increase over those measured after one to two years aging."

The strength of the sodium silicate concentration will also affect the final properties of the grout, including long-term durability. Some publications fail to recognize this crucial factor, or in some cases report incorrectly that sufficiently strong solutions either cannot be injected, or cannot obtain a satisfactory hardening time with a given reactant system. Bad results have been experienced in several instances where strongly diluted solutions were used. This not withstanding, the poor results were the result of faulty mix design, and not indicative of the behavior of properly designed sodium silicate formulations.

In 1972, the author reported results of a research program, which was ongoing for several years prior to 1971 (Warner, 1972). That work involved the testing of more than 2500 laboratory prepared specimens of chemically solidified soil, under a variety of both test and environmental conditions. The specimens were made with eight different chemical grout systems, and cured in three different environments. Sodium silicate was the base material in four of the systems. Included were strength test results of about 100 additional specimens, from sodium silicate grouted masses that were procured from a variety of actual field installations. These had been performed between 1965 and 1971, and included several permanent applications.

Although that work was completed a long time ago, many of the results remain pertinent. It was found that many different factors can drastically affect the indicated strength of the grouted specimens. It will vary enormously with different rates of load application. Faster loading rates will produce higher indicated strengths. Thus, comparison of reported strengths obtained from different reports are not valid unless the loading rate is known and identical for all reported results. The moisture level within a specimen also has a substantial influence upon the indicated strength. Dry specimens will always indicate a substantially higher strength than otherwise identical moist ones. But the most significant finding was that the strength that a given mass could withstand under continuous loading was but a small portion of that indicated by even relatively slow loading rates of laboratory compression tests.

The strength obtained under long-term continuous loads was referred to as the *fundamental strength*, and was within a range of 20% to 80% of the *ultimate* strength, indicated by the standard loading rate of the laboratory unconfined compression test. That rate was 20 psi per second as per the provisions of ASTM D 1663. The ultimate and fundamental strengths of several of the grouts evaluated at 5 days are shown in Figure 6B.9, which is reproduced from Warner (1972). Therein, G.V.S., SIROC, and Modified Earthfirm all contain sodium silicate as the base component. The writer has had extensive experience with the Modified Earthfirm, and has found the long-term, fundamental strength of that formulation, which is provided in Table 6B.3, to be on the order of 70% of ultimate.

Another finding of the research was a large variation of the total strain at rupture during loading. The poorest performing material was AM-9 Acrylamide grout, which had a strain of 19.7% at rupture. Although not now easily available, in earlier times this grout was often recommended for strengthening applications. Obviously, unless the solidified masses were fully restrained, less than satisfactory performance could be expected with such great strain deformations. The strain behavior of several different grouts is shown in Figure 6B.10.

Many of the formulations that were reported on were proprietary at the time of publication in 1972. Such rights have now expired, and the mix contents of the three main silicate base formulation are given in Tables 6B.1, 2, and 3. Although this research was performed a long time ago, sadly, nothing comparable has been since produced. Although a limited amount of strength data has been reported, important test criteria, such as loading rate, curing regime, etc., either varied from the previous work, or were not even reported, so that meaningful comparisons are not possible.

In 1983, ASTM released their standard, D4219, "Standard Test Method for Unconfined Compressive Strength Index of Chemical-Grouted Soils," which covers the determination of the *short-*

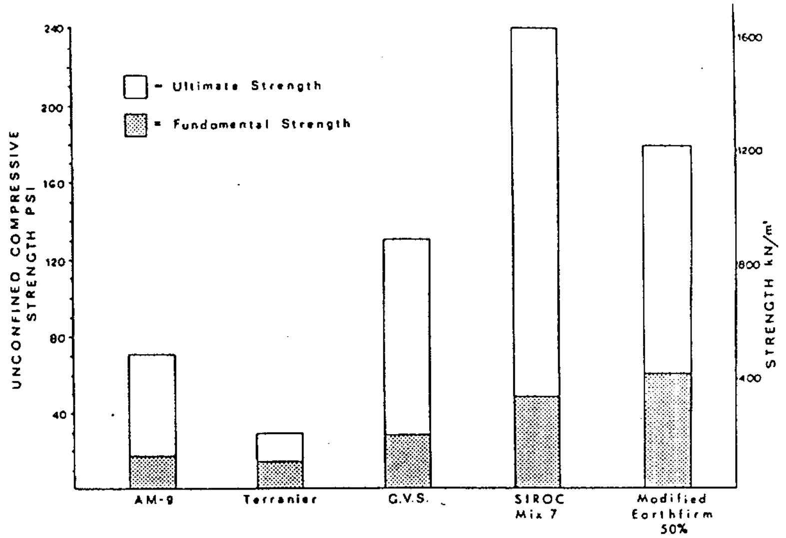


**FIGURE 6B.9** Ultimate and fundamental strengths of five day cured specimens of various chemically solidified soils. (From Warner, 1972.)

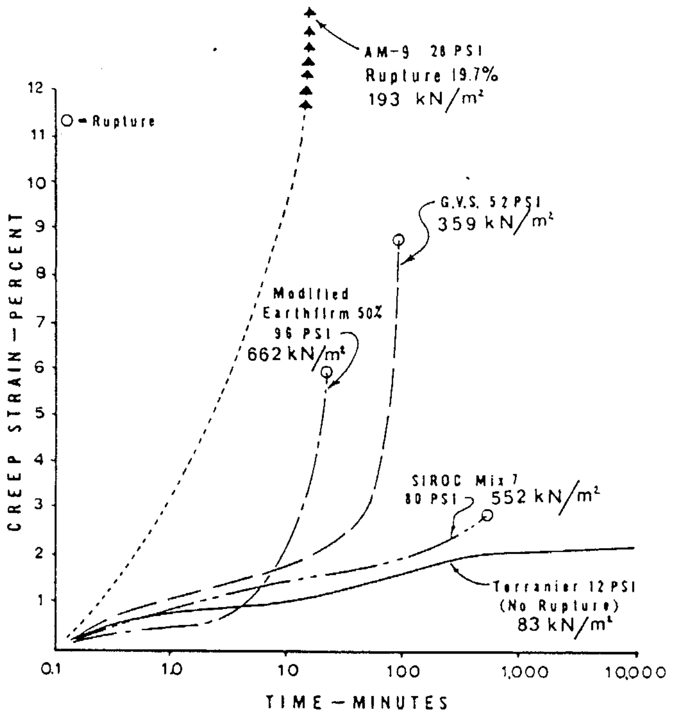


**FIGURE 6B.10** Accumulated strain at rupture of five day old specimens loaded to 40% of their ultimate strength. (From Warner, 1972.)

*term* unconfined compressive strength. That standard provides for a constant but unspecified rate of load or deformation. The only limiting requirements are that failure is not to occur in less than two minutes and the maximum strain rate not exceed 1% per minute. Unfortunately, such wide latitude in the allowable loading protocol precludes meaningful comparison of different specimens, even though evaluated in conformance with the standard, unless identical loading rate were used and noted. An additional provision of the standard is that the reported strength is to be that obtained at rupture of the specimen, or an accumulated strain of 20%. This is a very large strain, and it is unlikely that many structures partially supported by such ground would tolerate the resulting deformations. It must also be noted that the standard pertains only to the *short-term* strength, and thus is not applicable to long-term or permanent strengthening.

There has been much discussion as to the appropriateness of the unconfined compression test

**TABLE 6B.1** Mix Constituents for G.V.S. Chemical Solution Grout.

| Ingredient | Laboratory batch | Field batch |
|---|---|---|
| Sodium silicate | 350 ml | 50 gallons |
| Calcium chloride | 4.5 grams | 6.5 pounds |
| Glyoxal | 35 ml | 5 gallons |
| Water | 315 ml | 45 gallons |

**TABLE 6B.2** Mix Constituents for SIROC Mix 7.

| Ingredient | Laboratory batch | Field batch |
|---|---|---|
| Sodium silicate | 350 ml | 50 gallons |
| Formamide | 63 ml | 9 gallons |
| Sodium aluminate | 4.3 grams | 6 pounds |
| Water | 287 ml | 41 gallons |

**TABLE 6B.3** Mix Constituents for Modified Earthfirm, 50%.

| Ingredient | Laboratory batch | Field batch |
|---|---|---|
| Sodium silicate | 350 ml | 50 gallons |
| Calcium chloride | 6.72 grams | 8 pounds |
| Ethyl acetate | 14 ml | 2 gallons |
| Water | 336 ml | 48 gallons |

for such evaluations, and suggestions that a triaxial test, which provides lateral confinement to the test specimen, would be more suitable. This view has some validity *if* the solidified mass of the particular application is to be continuously confined. A large amount of chemical solution grout solidification, however, is performed for the retention of excavated soil faces, as illustrated in Figure 6B.11. In such cases, lateral confinement does not exist. Even in those situations where the grouted mass is completely confined, extensive laboratory studies by Ata and Vipulanandan (1999) have established little if any effect of the confining pressure on either the shear strength or


**FIGURE 6B.11** Cohesionless dune sand solidified with a chemical solution grout enable support of vertical excavation.

modulus of the grouted soil. Use of the much easier and less costly unconfined test is thus certainly pertinent and justified.

Although, in earlier times, the use of grouts for strengthening applications other than those based on sodium silicate was not unusual, most of the contenders have fallen, due to environmental, economic, or performance limitations. Today, the vast majority of chemical solution grout strengthening applications involve sodium silicate based grouts. While glyoxal, formamide, and ethyl acetate, along with calcium chloride and other salts, continue to be used, the reactants that are now used most frequently, are dibasic and tribasic esters. These can be more easily combined with the sodium silicate, and are much simpler to compound and use than the multicomponent mixtures given in Tables 6B.1, 2, and 3.

Dimethyl esters, which are the most economical and commonly used, are a by-product of the manufacture of nylon. As recovered, they are a combination of dimethyl succinate, dimethyl gluterate, and dimethyl adipate. The proportions of these three components can be quite consistent or it can vary widely, depending upon the source. Whereas for many uses of the solutions, the exact proportioning is of little consequence, such is not the case for use in sodium silicate grouts. Both the gelling time of the grout and strength of the resulting injected mass are affected by the exact proportion of the three components. In order to obtain consistent setting properties of the resulting grout mixture, as well as uniform strength of the grouted mass, the tolerance of the components should be reasonably consistent.

The actual mechanism by which a sodium silicate grout hardens is a reaction between the sodium silicate and an acid. In the presence of water, the esters hydrolyze to acid, which then converts the silicate to a solid. The rate of hydrolysis of the three ester types, and thus the grout gel time, varies widely. Dimethyl succinate hydrolyzes in minutes, gluterate in hours, and adipate in days. The proportions of these three ester types in the particular reactant solution being used can thus have a dramatic influence upon the performance of the mixed grout. Temperature also has an effect upon setting time, but this is generally not a problem in the environments in which grouts are usually used. It can become a significant consideration, however, where ambient temperature extremes exist in either the soil or the surroundings.

The strength, stiffness, and the durability of sodium silicate grouted soils are all related to the concentration of the sodium silicate base component as well as that of the reactant. Sodium silicate solutions are available in a variety of densities and viscosities. The sodium silicate most used in the United States has a density of about 41° Be and a weight of about 11.6 pounds per gallon (5.3 kg/l) at 68° F. It is highly alkaline, with a pH of about 11.3, and as delivered is similar to light syrup, with a viscosity of about 180 cps. It usually comprises from 30% to 80% of the total grout volume. To assure long-term durability for permanent applications, a minimum solution of sodium silicate base of 50% should be used. The proportion of basic ester reactants is usually within a range of 5% to 10% of the total solution, although for strong and durable applications 8% to 10% should be used. The remainder of the grout mix is water.

A typical mixture containing 50% sodium silicate and 10% diester will have a viscosity on the order of 20 cps. Whereas this is much greater than some widely touted low-viscosity grouts, they are nonetheless, quite penetrable. The penetrability can also be enhanced by the judicious inclusion of a surfactant, which effectively reduces the surface tension within the sodium silicate grout. Although grouts employing the basic ester reactants are now widely used, the author is unaware of any documented data relative to their long-term durability, and would suggest that they not be used for permanent solidification until suitable long-term performance data become available.

#### *6B.3.1.2 Chemical Solution Grouts for Water Control*

Whereas hard rigid gels are best for soil strengthening, some flexibility of the gel is often desirable in water control grouts. Also, although long gelling times are often advantageous for strengthening operations, very rapid setting times can be an essential property when grouting in moving water. Those grouts, which are widely used for strengthening, are thus often not best suited for water control work. There are two main generic classes of chemical solution grout that are particularly suited for water control grouting. These are polyurethane and acrylamide/acrylate resins. There are several different formulations within each of these broad categories. The polyurethanes are usually used alone, whereas the acrylamide/acrylate systems are sometime filled with other finely divided materials such as diatomaceous earth, silica fume, and ultrafine cement.

There are many different types of polyurethane grout, which provide a wide range of different properties. They all react when mixed with water, to form either a gel, solid, or foam. The strength of the gel, or density of the foam, is dependent upon the particular formulation and the amount of water with which it has been mixed. Temperature also has an influence upon the final properties, though it is not as significant as the other variables. Some formulations can have a rather complete reaction upon simply coming in contact with water, whereas others require complete mixing therewith.

Urethane grouts can be divided into two main classes, *hydrophobic* and *hydrophilic*. Hydrophobic formulations repel water upon hardening. For this reason, their bond to wet surfaces is not very great. This is not much of a problem within a maze of interconnected soil pore spaces, but can be a limitation where an obvious crack in rock or concrete, or a void at a soil to hard surface interface, is to be filled to stop water flow. Some hydrophobic formulations will immediately foam on contact with water, as shown in Figure 6B.12. This can be very useful in stopping flows of moving water. Because reacted hydrophobic polyurethanes contain no outside water, they are generally free of shrinkage, even when allowed to dry.

Hydrophilic formulations also react with water but continue to attract it after completion of the initial reaction. These formulations will thus continue to expand if possible, forcing the resulting gel into pore spaces and voids, beyond those originally filled. Unfortunately, the strength of the grout is reduced as further water is absorbed. Obviously, they will develop a much greater bond to adjacent surfaces as long as they are not diluted excessively. However, they are subject to shrinkage upon drying out. They are thus best not used in areas that will not be in constant contact with water. Some

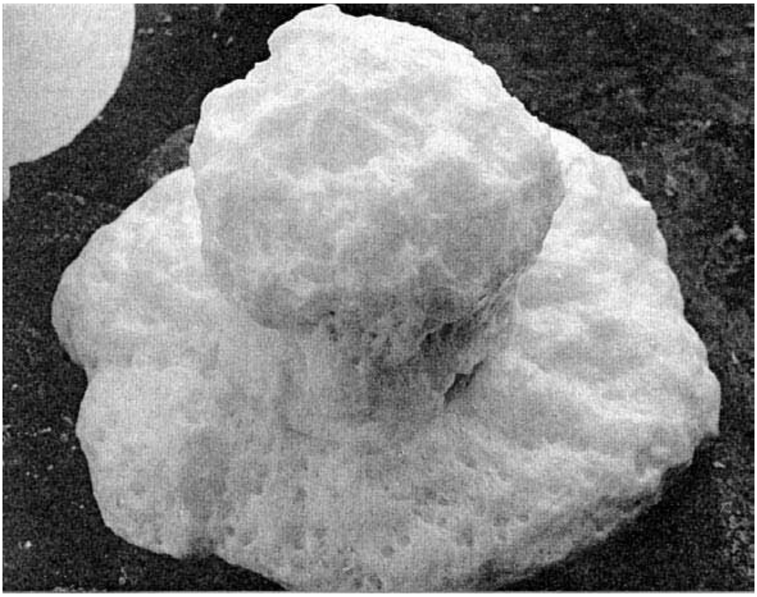

**FIGURE 6B.12** Water dropped onto a small amount of hydrophobic urethane grout in the bottom of container resulted in immediate formation of a closed cell foam.

hydrophilic formulations require a minimum amount of water in order to react fully. This is typically delivered into the grout, during injection, by means of a suitable dual pump proportioning system.

Urethane grouts are of two different categories chemically—those based on toluene diisocyanate (TDI) and diphenylmethane diisocyanate (MDI). The designations TDI and MDI are universally used and well-established terms, and are often used alone for reference. The TDI based formulations, which usually are hydrophilic, will combine with all available water. The result will be a gel or flexible foam, the strength of which will depend upon the proportion of the combined water. They stick tenaciously to wet surfaces and are very hard to remove or clean up should they be deposited upon an exposed surface. As illustrated in Figure 6B.13, spills of hydrophilic urethane grouts are extremely difficult to clean up.

MDI formulations are generally hydrophobic, and while they require water to react, they will only accommodate a specific amount thereof, and will displace any excess. They will thus often not develop a good bond to wet surfaces. They can be formulated so as to provide either flexible or rigid foams, or solid masses.

Although the TDI liquids are generally safe to handle, the resulting vapor pressures in the air exceed established threshold limits. Full protective precautions are thus required with their use. MDI mixtures, on the other hand, have relatively lower vapor pressures, and are thus less hazardous. With either type, however, airborne droplets, such as might result from a leaking delivery line, are hazardous, although the cured gels are not. The safety aspects of working with urethane grouts are thoroughly covered in "Recommendations for the Handling of Aromatic Isocyanates" (1976).

Mixed acrylamide/acrylate grouts are water-like solutions that provide very high penetrability. They are capable of exceptional control of the gelling time. Upon setting, the gel is very soft, enabling it to be pumped out of the delivery system should it gel prematurely. While these factors are benefits from the injection standpoint, they can be limitations for the injected grout. The resulting gel is quite weak, and subject to extrusion within any significant voids or defects in the soil. This

**FIGURE 6B.13** Hydrophilic urethane grout spills are extremely difficult to clean up.

limitation can be offset by inclusion of filler materials; however, the injectability goes down as fillers are added. This group of grouts finds its largest single use in the repair of leaking sewers and pipelines. Although the chemistry of the gelled grout is essentially the same, the origin of these materials is different.

Acrylamide grouts were one of the earliest commercially available chemical solution grout systems and were marketed by the American Cyanamid Company under the trade name AM-9. They were first marketed in 1955 and were extensively used for water control applications until 1978, when they were suddenly removed from the market. This was due to their manufacture being discontinued following a toxicity problem in Japan, which resulted in a ban on their use in that country. Although the cured grouts are not toxic, the individual components are, so the unmixed components or any mixed solution that fails to gel, due, for instance, to dilution after injection, could present a serious risk.

Although not widely promoted, acrylamide grouts are still available in the United States, and continue to see extensive use for sewer maintenance. They are generally sold only to firms that are properly equipped and have personnel who are especially trained in their safe use. The base material is available in either a dry granular form or a fluid slurry. The slurry is considered less hazardous than the dry form. The grout consists of separate solutions of the base material and the reactant system. These components are proportioned and impelled by separate synchronized pumps to the point of injection where they are combined and stream mixed (Figure 6B.14). Both the gel time and strength can be varied by adjusting either the strength of the starting solutions or the final mix ratio.

In order to enable variation of the final mix ratios, a variable proportioning pumping system must be used. The reactant portion of the grout is highly corrosive, so the mixers, pumps, and appurtenances on that side of the pumping system must be of plastic or stainless steel. Most of the work now being done with these grouts involves sophisticated pumping systems, requiring minimal handling of either the unmixed ingredients or the resulting grout.

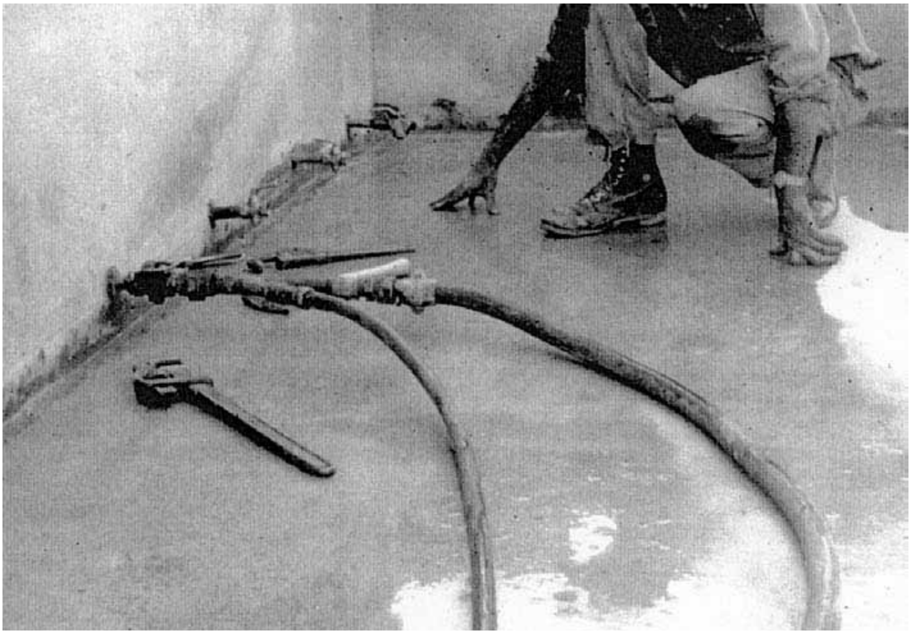

**FIGURE 6B.14** Stream mixing of grout at the point of injection.

Acrylate grout was developed as an exact replacement for the acrylamide grout when AM9 was taken off the market. It uses the same catalyst system and offers many of the advantages of the acrylamide mixtures. The acrylate, which forms the base solution, however, is a prepolymer and is not neurotoxic as is the acrylamide monomer. The mixing and pumping equipment requirements are the same for both grout types.

### 6B.3.2 Fluid Suspensions

Fluid suspension grouts were the first to ever be used, and they continue to be widely employed. The largest area of use for these grouts continues to be the filling of cracks and joints in rock, although their use has been extended greatly and includes controlled injection into soil. In the simplest form a suspension of ordinary portland cement and water is mixed. If all of the cement particles are to be thoroughly dispersed, high-shear mixing is required and the mixed grout must be continuously agitated to maintain the individual grains in suspension.

Cement grout mix design is expressed as the ratio of water to cementitious solids, either by weight or by volume. Volume measurements have been the most often used traditionally, as they were very easy to make when using common bagged cement, which contains one cubic foot of dry powder per 94 pound bag. Where bulk cement is used, proportioning by weight is more convenient. In the United States, grout mixtures as thin as 10:1 by volume have been extensively used historically. Many, including the writer, consider such thin grouts to be little more than dirty water, and extensive testing and research by the author and others (Houlsby, 1982; Weaver, 1991) have shown such thin mixtures to provide little durability. Current thinking suggests that thicker mixtures (maximum W/C 3:1 by volume) are far more appropriate for most applications. In a well-documented discussion of appropriate water to cement ratios, Houlsby (1982) concluded that W/C 2:1 by volume has been used on thousands of projects and is the optimal for general use.

Although unmodified common cement–water suspensions continue to be widely used, they suffer from settlement of the solids, resulting in accumulation of mix water at the top of unagitated grout. This is known as *bleed* and is an important factor to consider in any grout formulation. In this regard, the amount of shear energy imparted during the mixing of the grout has an important bearing upon the resulting bleed. When cement and water come together, the cement grains tend to clump or floc together. To minimize the amount of bleed of a particular grout, high shear mixing will break up the flocks and separate the individual cement grains. This factor was thoroughly discussed by (Kravetz 1959). The essence of his conclusions, as reported by Houlsby (1990) are:

1. Cement grains, when mixed with water, tend to aggregate and form clumps. This slows the wetting process, as does air attached to the grains. The effect of high-speed shearing or laminating, plus the centrifugal effect, is to thoroughly break up the clumps and to separate air bubbles. As a result, each individual grain is rapidly and thoroughly wetted and put into suspension.
2. During cement hydration, needlelike or springlike elements of hydrates form on the superficial layer of each wetted grain of cement. In a high-speed mixer, the laminating effect and high-speed rotation keeps breaking these hydrates away from the grain of cement, thus exposing new areas to the water and consequently causing the formation of new elements. These hydrate elements are of colloidal size, and as the amount of these elements in the mixture increases, the grout becomes colloidal in character.

In reality, virtually no common cementitious suspension grout is truly colloidal, as even well-dispersed cement grains will settle in water unless special admixtures are used, as will be subsequently discussed. High-shear mixing will greatly reduce and can in thick grouts nearly eliminate bleeding. In some places, but generally not the United States, a grout that exhibits little or no bleed is regarded as “stable.” Stable grouts are generally defined as those which exhibit a total of no more than 5% bleed. European practice commonly calls for “stable” grouts, which typically include a few percent of bentonite to act as a suspension agent. Whereas bentonite inclusion will tend to lessen the cement grain settlement and resulting bleed, there are significant disadvantages to its use. Modern admixture technology offers a variety of different compounds that can minimize bleed without the undesirable aspects of admixed bentonite. These will be subsequently discussed in more detail.

### *6B.3.2.1 Soil Strengthening with Cementitious Suspensions*

As previously discussed, the grain size of ordinary portland cement grouts is too great to allow injection into all but the coarsest of sands and open gravels. Extending work of the Corps of Engineers, Technical Memorandum 3-408 (1956), and the work of a number of others, relative to the permeation of soils with cement grouts, King and Bush (1961) concluded that grout would pass freely through a granular mass if the diameter of the particles in the grout was not greater than 6.7% that of the material to be grouted. Their conclusions were the result of an extensive study, which considered not only the largest sized particle in the grout, but also the variation and distribution of grain sizes, as well as consideration of the pore structure to be grouted.

The work was based on the premise that whereas two grains of cement wedged across a pore opening would form a stable bridge, and thus cause blockage, three grains would not be as stable. Although their work was based largely upon spherically shaped particles in the mass to be injected, it did consider the variation in pore structure that could be expected. They performed a number of injection tests in which a neat cement grout was injected into a specimen of natural, round-grained aggregate. Although they included many aggregates in their test program, the finest was a sand, which was completely retained on a No. 8 sieve but passed 100% through a No.4 sieve. It should be noted that currently available common cements are ground somewhat finer than the cements evaluated by King and Bush.

In more recent times, there has been considerable discussion by a number of authors relative to the filling of soil pore spaces with particulate grouts. The discussion has been concerned mainly with mixtures containing ultrafine cement. Theoretical ratios of the grout particle size to the grain size and thus porosity, of a given soil into which a particular cement grout can be injected are emphasized. There has not been a great deal of agreement as to the appropriate ratios, and especially whether they should be based on the mean or maximum particle size of the cement. Although grain size has been emphasized, it has been the author's experience that grain size alone is only one important aspect. The grain shape and surface condition, as well as the overall rheology of the grout, especially the thorough dispersion of the cement, are all significant. Also, of importance are the grout pumping rate and sustained pressure level.

Suspension grouts are often injected under fairly high pressures, and pressure filtration, that is, loss of water from the solids due to the injection pressure, must not be allowed to occur. This is best prevented by inclusion of a high-range water-reducing admixture in the mix, combined with high-shear mixing action, so as to maximize dispersion, and uniform wetting of the cement particles. Whereas in the filling of fissures or voids, the maximum grain size of the grout is not as important as the overall grain size distribution, which will be discussed in more detail shortly, grain size is crucial when injection is into a granular deposit. This is because the particles can form a filter cake where they first enter the deposit, with the largest particles being restrained near the grout hole, followed by those which are progressively smaller, until permeation all but stops.

This is an important consideration, in that ultrafine cement suspension grouts, which are highly penetrable and can form nearly colloidal mixtures, are seeing ever-increasing use for the solidification of sandy soils. Shimoda and Ohmori (1982) described the development of a new ultrafine cement in Japan and reviewed laboratory research as well as two significant application case histories. Suspension grouts made with the then new cement were injected into fine to medium sands.

In more recent times, several manufacturers have introduced ultrafine cements, which are now readily available. Figure 6B.15 shows the grain size distribution for common portland as well as several ultrafine cements. It should noted that there are no standards regulating the actual grain size distribution for common cements. The curves shown in Figure 6B.15, for type I and III cement, are for that produced in one particular batch at a given plant. ASTM C 150, "Standard Specification for Portland Cement," specifies only the Blane fineness, which is a measure of the specific surface area of all of the grains in a given volume of cement. Type III portland cement is required to be finer than type I or II; however, the maximum grain size or grain size distribution, are not specified.

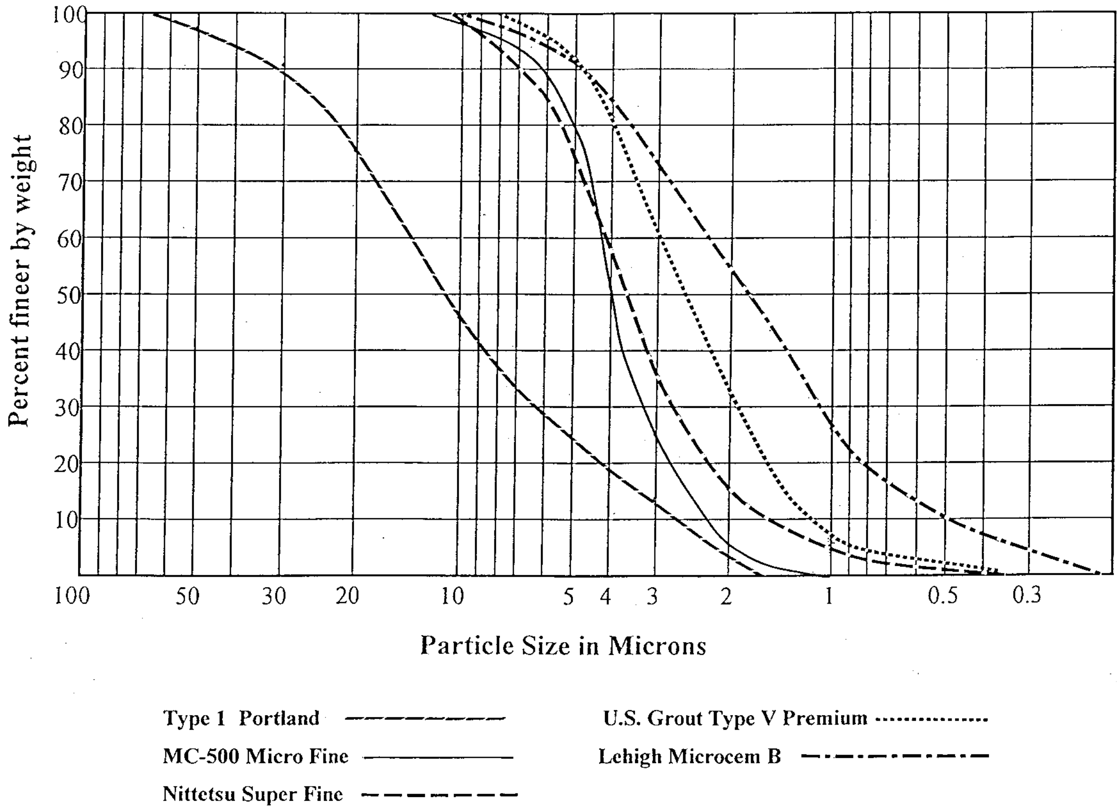


**FIGURE 6B.15** Particle size distribution of several cements used in fluid suspension grouts.

There are two main types of ultrafine cement: those based on slag and those based on portland cement. Although the two are comparable in many respects, there is a fundamental difference in the span of hydration and the resulting setting time. Because of the very high specific surface of ultrafine cements of portland cement origin, hydration activity is high, resulting in rapid setting and strength gain. For this reason, retarders such as citric acid are sometimes included. Portland cement–citric acid combinations are extremely sensitive to temperature changes, so the proportions of the additive must be matched to the temperature of both the working environment and the medium being injected. Problems with flash setting and difficulties with both setting time and rate of strength gain have been reported.

Cements based on slag, however, tend to set and gain strength much slower than their portland cement cousins. Thus, retardation is seldom a problem, but accelerating admixtures may be required in some instances. Well-established accelerators are available and their behavior is quite predictable, so setting times are not much of a problem with the slag ultrafine cements.

These factors were well illustrated during a demonstration, which was part of the 21st Annual Short Course on Fundamentals of Grouting, held in Denver, Colorado, September 12 to 17, 1999. Therein, vertical, transparent tubes, eight inches in diameter and five feet high, were filled with sand (Figure 6B.16). The sand was of quartz origin and was graded such that 100% passed a number

**FIGURE 6B.16** Sand-filled columns used to evaluate cement grout penetrability.

30 mesh sieve and 40% passed a number 40 sieve. It was carefully poured into the tubes such that all tubes were filled equally and had equal density.

Five different grout mixtures were injected into the sand, from the bottom of the columns. The injection was made at a pressure of 10 psi, which was maintained for twenty minutes on each column. The degree of penetration of the various grouts was noted. All of the grout mixes had an identical water to cement ratio of one. Mixture content and penetration rates for the various grouts are shown in Table 6B.4. Interestingly, the greatest degree of penetrability was not achieved with the finest cement. This is consistent with real-world experience on many projects where ultrafine cement grout has been injected into sandy soil. Obviously, there remains much to be learned about these cements, and certainly, particle shape and surface characteristics are major contributors to the penetrability of grouts in which they are used.

The thickest grout that can be readily mixed with the mixers commonly used in grouting is on the order of 0.5:1 by volume. Lower W/C ratio grouts result in a mixture with pastelike consistency, generally referred to as a slurry. Slurry consistency grouts are sometimes used for filling small voids (0.3 to 3 inch) as well as for filling of controlled fractures in soil. They have also seen some

**TABLE 6B.4** Penetration of various ultrafine cement grouts into experimental sand columns.

| Grout mix/cement type | Penetration from base (inches) |
|---|---|
| Type I Portland | 4 |
| Type I Portland + 1% naphthalene superplasticizer | 8 |
| Portland based ultrafine | 9 |
| Slag based ultrafine | 60 (flow from top in 7 minutes) |
| Portland–pumice based ultrafine | 57 |

use in filling of large voids, such as abandoned mines (although this is inappropriate in the writer's opinion). Slurries are generally too thick to be mixed in commonly available high-shear mixers. They are thus most often prepared in horizontal-shaft paddle mixers, such as those commonly used to mix plaster and mortar, and thus are often particularly sensitive to bleed.

### 6B.3.3 Mortarlike Low-Mobility Grout

Very stiff mortarlike grouts were first developed in the United States in the mid 1950s. The original use of these very low mobility grouts was for the then developing technology of compaction grouting. With the advent of the modern concrete pump in the 1960s, it became practical to pump and place under pressure typical concrete and mortar mixes. In recent times, admixture technology has provided the ability to alter conventional concrete and mortar mixtures, creating a wide range of special properties. Of these, greatly increased plasticity, cohesion, and resistance to bleed are most germane to grouting.

Although many consider all mortarlike grouts to be equal, subject only to the slump, as determined by ASTM C 143, such thinking is erroneous. As an example, the low-mobility grouts used for compaction grouting must exhibit high internal friction in order to provide placement control and prevent hydraulic fracturing of the soil into which they are being injected. Conversely, conventional concrete and mortar mixtures ideally possess relatively low internal friction, in order to provide maximum workability. Stiff mortarlike grouts of the same slump are not necessarily equal in mobility when under pressure, either in the delivery system or in the ground. Whereas slump by itself may be a sufficiently accurate measure for conventional concrete or mortar, and whereas such mixes are quite satisfactory for fill grouting of large voids, the mix requirements for compaction grouting are much more demanding, as will be discussed subsequently.

## 6B.4 GROUT RHEOLOGY

Simply stated, rheology is the study of the deformation and flow of materials. Perhaps more germane to grouting, the American Concrete Institute, in their publication ACI 116 *Cement and Concrete Terminology* (1990), defines rheology as "The science dealing with flow of materials, including studies of deformation of hardened concrete, the handling and placing of freshly mixed concrete, and the behavior of slurries, pastes, and the like." Satisfactory rheological properties of a grout in both the as-mixed as well as in the hardened state are fundamental to successful completion of any grouting project. The mixed grout must provide appropriate properties to enable proper injection and travel within the particular formation to be improved. Obviously, the durability and long-term performance of the hardened grout is fundamental to achieving the intended performance. The flow properties of grout materials vary widely, and a basic understanding of both fluid and plastic flow is requisite to either the study or understanding of grout flow.

Many grouts are subject to thixotropy, which is the performance of a material as an immobile paste or gel when at rest, but as a fluid when energy is exerted on it. A common example of thixotropic behavior is the pouring of catsup from a bottle. The catsup has a high resistance to flow out of the bottle at first attempt, but upon rapid shaking, flows readily. Many grouts are thixotropic in that they require a positive pressure of some magnitude to initiate movement within the delivery system, but flow freely at lower sustained pressure, once movement is initiated. This factor is important in considering allowable injection pressure for a given application, and an adequate initial pressure level to start flow must be provided. The flow behavior of fluids is a complex science. Here, only those properties that are fundamental to a basic understanding of grouting will be explained in as simple a manner as possible. Those who might desire a more technical explanation are referred to Tattersall and Banfill (1983) and Mehta and Monteiro (1993).

Fluid and paste flow are described as either Newtonian or Binghamian. In Newtonian flow, the shear stress, which is the force required to move the fluid, is essentially constant, regardless of the rate of movement, which is technically referred to as the shear strain. Water is an example of a Newtonian fluid. Bingham fluids, on the other hand, possess some thixotropy, and require a measurable force (shear stress) to start movement. The magnitude of that force is usually referred to as the Bingham Yield Value, and sometimes, in reference to grout, the *cohesion*. With such grouts, the shear stress typically increases as the rate of shear (shear strain) increases. Typical stress–strain curves for both Newtonian and Bingham flow are shown in Figure 6B.17. The slope of the Bingham curve indicates viscosity, which property will be dealt with in more detail later. Whereas some chemical solution grouts perform as Newtonian fluids, most fluid suspension and other grouts exhibit Binghamian behavior.

Behavior of non-Newtonian grouts, is not always as simple as indicated by the straight line relationship of the applied shear stress and the shear rate or strain, as shown on Figure 6B.17. Although the relationship of shear stress to shear rate can remain constant, as shown, it can also vary either up or down, depending upon the tendency of the material to thicken or thin with an increase in the shear rate. Figure 6B.18 demonstrates an instance of shear thickening where resistance to flow (pressure) increases with an increase in shear or flow rate. Some grouts will behave quite differently, wherein their resistance to flow will decrease as the shear or flow rate increases (shear thinning), as shown in Figure 6B.19.

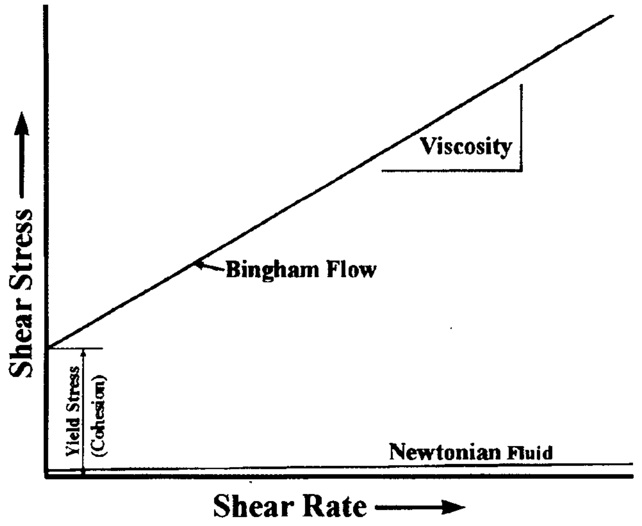


**FIGURE 6B.17** Newtonian and Binghamian flow indicating Bingham yield stress (cohesion).

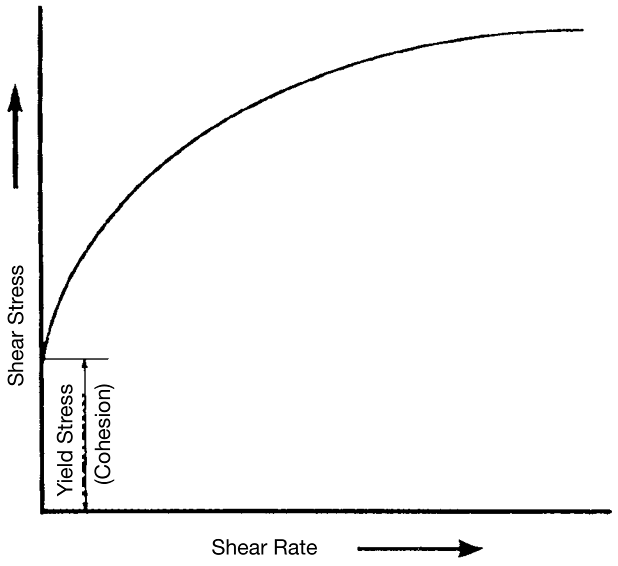


**FIGURE 6B.18** Shear thickening.

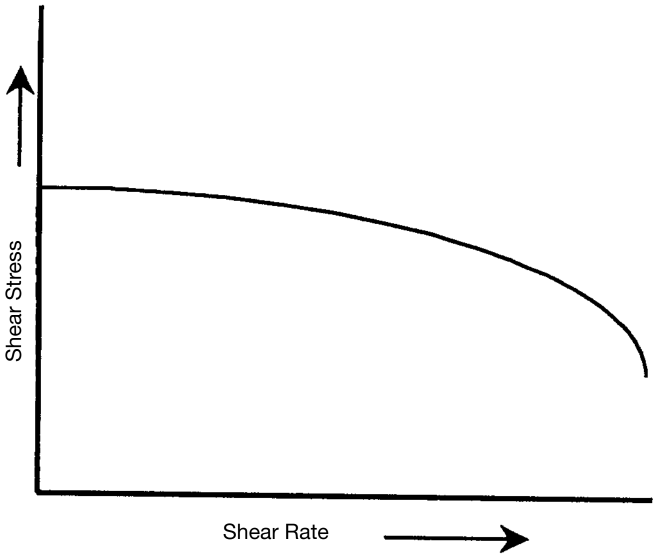


**FIGURE 6B.19** Shear thinning.

Although incorrect, all too frequently only one or a few of the rheological properties pertaining to a particular grout mixture are used to describe the performance of that grout. One of the most frequent errors, is using the *viscosity* alone to define the injectability, when in fact knowing the viscosity is of little value unless other important properties such as surface tension and the resulting wetability are known. Grout rheology is very complex, and one must consider many important properties. The writer suggests that the use of all-inclusive, simple terms to describe the properties of a grout are of more practical value, and recommends the descriptive terms *mobility*, for relatively thick grouts, and *penetrability*, for more fluid grouts, that are intended to fully permeate soil or rock. Each of these terms include many rheological influences and effects that must be evaluated in total in order to rationally evaluate the propriety of that grout for a particular application.

### 6B.4.1 Mobility

Mobility denotes the ability of a grout material to travel through the delivery system and into the desired voids or intended deposition area within the geological formation being grouted. Of, equal importance is the ability to limit travel, so as to *not* flow beyond the zone of desired deposition. Thus, for proper performance, the grout should be of sufficient mobility to penetrate and fill the *desired* geotechnical defects, but it must be sufficiently limited so as to *not* flow beyond the desired injection zone. When injection is made into soils, the mobility will also affect the manner in which the grout behaves during injection.

When grout is placed in soil under pressure, it acts in one of three ways:

1. As a penetrant filling pore space, if the pumping rate is equal to or less than the rate at which the pore structure will accept the grout
2. As a fluid causing hydraulic fracturing of the formation if the pumping rate exceeds the permeation rate, or if the grout consistency does not allow it to permeate
3. As an expanding mass pushed by fluid-like pressure at the source of injection, so as to compress or compact the soil (Warner et al., 1992)

A common misconception is that a thick grout is always of low mobility, whereas a thin grout is always highly mobile. Depending upon the individual mix constituents, thin grouts can be of relatively low mobility, whereas even very thick low slump grouts can be highly mobile and behave in the soil as fluids. An example of the former would be fluid suspension grouts that contain blocking agents such as sawdust, mill feed, etc. (Technical Memorandum 646, 1957). Prior to development of the ability to pump low-mobility plastic consistency grouts, such low viscosity fluid grout mixtures, designed to restrict grout travel, were quite common.

Even very thick or essentially no-slump grout mixtures can be of sufficiently high mobility as to act like a fluid when injected into soil. Such behavior would typically be expected of grouts containing clay components or admixtures such as some concrete pumping aids. When a grout injected into soil behaves as a fluid, causing hydraulic fracturing, control of the grout placement is lost, and negative occurrences are likely. This subject will be much more extensively discussed subsequently in relation to compaction grouting.

In 1980, the American Society of Civil Engineers published the “Preliminary Glossary of Terms Relating to Grouting,” wherein they defined compaction grout as:

> COMPACTION GROUT—Grout injected with less than 1 in (25 mm) slump. Normally a soil–cement with sufficient silt sizes to provide plasticity together with sufficient sand sizes to develop internal friction. The grout generally does not enter soil pores but remains in a homogeneous mass that gives controlled displacement to compact loose soils, gives controlled displacement for lifting of structures, or both.”

Although, a standard test for “slump” was not defined by ASCE, many grouters have assumed that the well-established slump cone used in ASTM C 143, (see Figure 6B.20), which is the most common

**FIGURE 6B.20** ASTM C 143 slump test in progress.

slump test, was intended. Many have ignored the inclusion of "sand sized" and "internal friction" in the definition, and defined any grout with a low slump using the C 143 slump cone to be appropriate.

Notwithstanding the definition by ASCE, or the interpretation by grouters, the *slump* of a grout alone, is *not* a valid measurement of its rheological properties or appropriateness for compaction grouting, or, for that matter, any grout application. The test was originally developed for appraising the workability of concrete that includes only clean sands and large aggregate. Even for use with that material, for which the test was developed, ASTM states in their document, ASTM Special Technical Publication 169B (1978),

> Slump Test—The slump test . . . is the most commonly used method of measuring consistency or wetness of concrete. It is not suitable for very wet or very dry concrete. It does not measure all factors contributing to workability, nor is it always representative of the placeability of the concrete.

A mixture with a slump of only one inch is certainly "very dry," and is a mixture that, according to ASTM, the slump test is "not suitable" for. Typical low-mobility grouts, such as those used for compaction grouting, contain silt-size particles, which tends to make them sticky. It is nearly impossible to properly fill the slump cone with such material, let alone obtain repeatable results. Unless one is dealing with concrete that is not "very wet or very dry", the slump test should not be used, and it most certainly is *not* appropriate for judging the rheology of most thick mortar-like grout mixtures.

## 6B.4.2 Penetrability

Penetrability defines the ability of a grout to permeate a porous mass, such as a sand or soil, or fill fractures or small voids. As previously discussed, viscosity alone is sometimes equated to the penetrability of a given grout. In actual practice, however, penetrability of a grout into a given soil is dependant upon a combination of viscosity *and* wetability. Within the viscosity ranges commonly found in solution grouts, wetability, which is defined by the contact angle of a drop of the fluid with the receiving surface, is a far more important and significant property than viscosity. Wetability is a function of the surface tension between the formation or granular surface and the solution at their interface.

This surface tension depends on the chemical composition of the grout being used and the chemistry and physical conditions of the individual soils being treated. The affinity of the grout, which results from its wetability behavior in contact with the formation into or thorough which it is being driven, is thus of critical importance. This affinity, which includes the grout's surface tension properties, is fundamental to its penetrability. A simple example of the concept is the relative immobility of a drop of water placed on a newly waxed surface, compared with the rapid dispersion of a drop of the same water placed on a similar but heavily oxidized surface.

An excellent example of this occurred during an experimental test program to develop the optimal grout for injection into several thousand feet of ⅜ inch OD tubing embedded within the embankment of a earth dam. The tubing had been installed in the embankment during the original construction, as part of an extensive instrumentation system. The test program was to include pumping of grout through several thousand feet of tubing, to determine injection parameters that would be required for the actual work. Although the original tubing was made of saran plastic, such material was no longer available, so a PVC plastic tubing of identical dimension was used.

As part of the evaluation, an eight foot length of tubing was secured to a slanted board, as shown in Figure 6B.21. Various trial grouts were then run through the tubing, with the flow times being noted. Because of a concern that the different tubing materials might react differently to the grout, an eight foot long piece of the original saran tubing was removed from a gallery in the dam. It was secured to the slant board adjacent to the PVC tube, and fed with grout from a common funnel, as illustrated in Figure 6B.22. Interestingly, the grout flowed more easily in the PVC tubing, as can be

**FIGURE 6B.21** Tubing attached to slant board for grout flow evaluation.

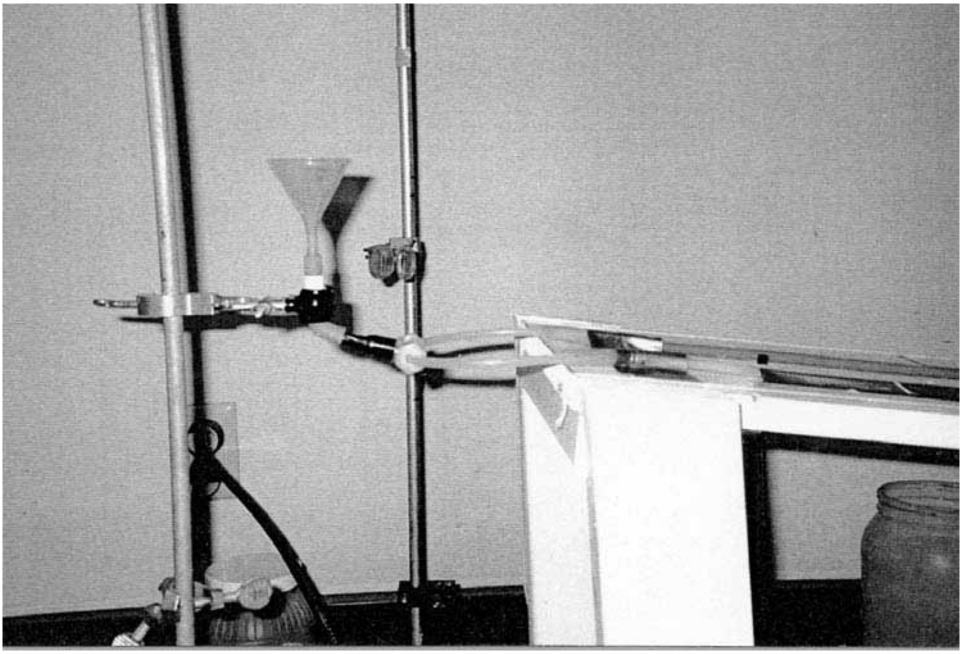

**FIGURE 6B.22** Both tubes are simultaneously filled through a common funnel.

seen in Figure 6B.23, wherein the graduated cylinder on the right is filled much higher than that of the left, below the saran tubing.

The most promising grout mixes were then pumped through several thousand feet of the PVC tubing, Figure 6B.24. As a result of the slant board tests, a correction factor was developed for the saran tubing, allowing an accurate calculation of the grout pressure that would be required to inject the saran tubing existing in the dam. As confirmed by these tests, the penetrability of a grout is not solely dependent upon its viscosity. Low-viscosity grouts can have poor penetrability, whereas highly viscous grouts can be very penetrable, in a given delivery system or formation.

In the case of grouts that contain solids, such as fluid suspensions and slurries, the grain size of the solids also affects the grout's ability to penetrate. Theoretical research and discussion abound, relative to the ratio of maximum grain size that can penetrate a given crack and/or joint width in the grouting of rock. Although the subject is somewhat contentious, in rock grouting there is general agreement that the minimum dimension of the defect must be at least three to five times that of the maximum grain size in the grout. Although the maximum grain size unquestionably affects the penetrability of an obvious defect, the author suggests that it is only one consideration, and perhaps not the most important.

As an example, should a grout contain only a small percentage of the maximum size particles, and those particles were well distributed so they were not adjacent to or in near proximity of each other, the maximum size would not be of great significance, as long as their dimension were less than that of the void being filled. Conversely, if the grout contained a larger proportion of the large-size particles and/or they were in near proximity of each other, the risk of blockage would be much greater, and thus the maximum size of the particles more significant. Regardless of the grain size, of particular importance in all cases is the amount of shear energy applied during the grout mixing. It

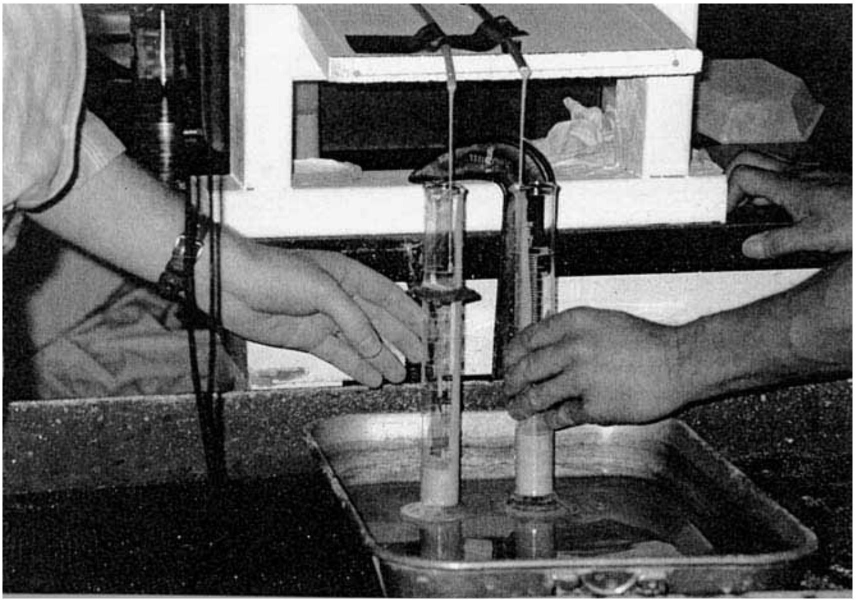

**FIGURE 6B.23** Cylinder on right is filling faster than that on the left.

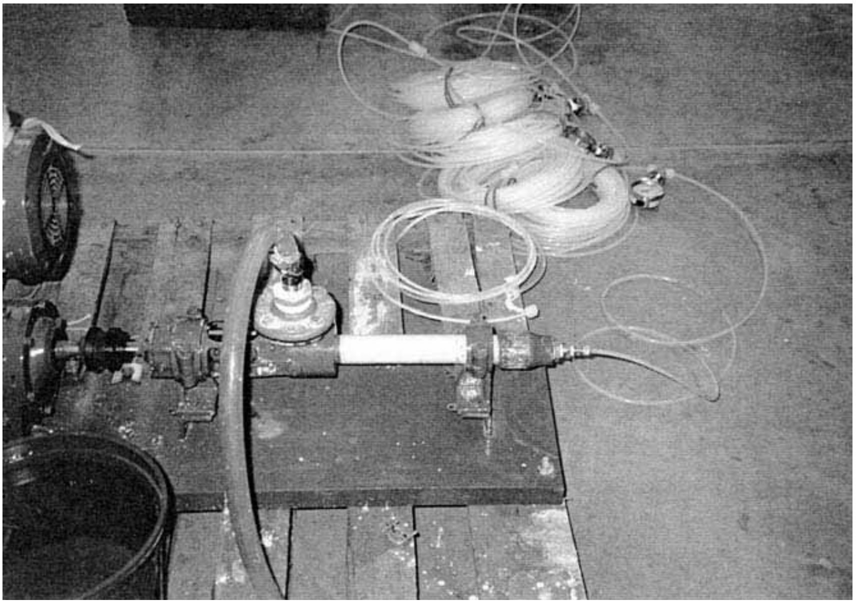

**FIGURE 6B.24** Test grouts were pumped through several thousand feet of coiled tubing to simulate conditions actually existing in the dam.

has an important influence on the dispersion of the individual grains, and thus the penetrability of the resulting grout.

The shape of the particles and condition of their surface are also of immense importance. Rough, angular particles are far more likely to form blockages within a pore space or defect than are smooth round ones. Experience has repeatedly shown that cementitious grouts containing large amounts of pozzolanic materials, such as fly ash or silica fume, are more penetrable than those containing only cement. This is no doubt the result of the near spherical shape of most pozzolans. Round shaped grains tend to roll past and over each other, rather than bind together as would rough angular shapes.

Although the terms mobility and penetrability are perhaps the most all-inclusive for describing the rheological properties of a grout, there are other factors that must also be considered to assure the proper selection and applicability of a grout mix.

### 6B.4.3 Cohesion

Lombardi (1985) reported on a simple field test to determine the "cohesion" of a grout. What Lombardi is actually evaluating is the Bingham Yield Value for the grout, albeit with a simple test. It involves dipping a 10 cm square, slightly roughened metal plate approximately of ⅛th inch thick (Figure 6B.25) into the grout and measuring the weight of the grout that remains on the plate after removal.

### 6B.4.4 Bleed

When at rest, the individual particles of a fluid suspension grout will tend to settle out of the solution. This is referred to as bleed. To prevent bleeding of the grout prior to injection, it is usual-

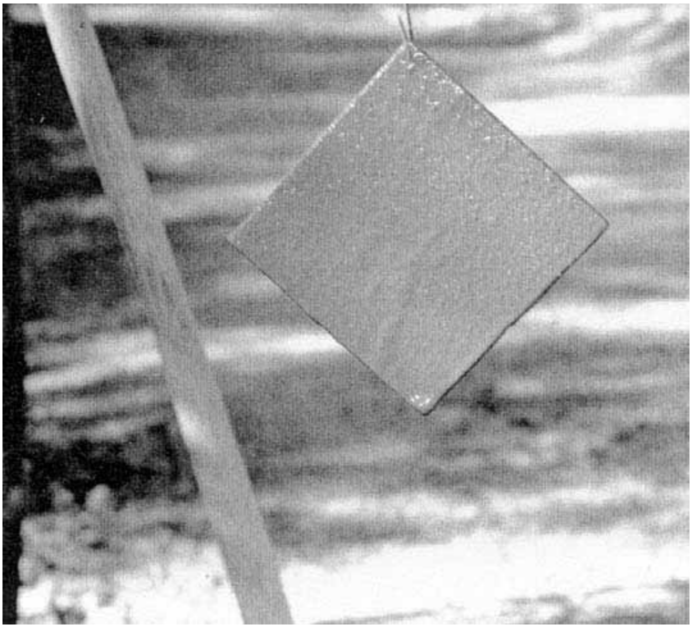

**FIGURE 6B.25** Lombardi cohesion test.

ly continuously agitated after mixing. Whereas excessive bleed will not occur under proper agitation, it can occur when the grout is essentially at rest, either during or after injection. This is of particular significance where large quantities of grout have been injected into a single location or void, or where the injection rate is very slow. As bleed occurs, the solids will settle to the bottom of the void or defect being filled, and the top portion will be filled initially with water, and eventually with air, leaving void space. The bleed potential of a grout can be greatly reduced by very through, high-shear mixing, as well as through thoughtful mix design, including the judicious use of admixtures.

### 6B.4.5 Setting Time

In many cases, control of the time required for a grout to set or harden can be crucial to proper performance. It is imperative that grout injected into a given hole has set or become immobile prior to the drilling of adjacent holes. Rapid setting times are usually desirable when injection is into moving water, so the grout mass will set before it is washed away. Even where the water is not moving, a rapid setting will minimize the opportunity for dilution of the grout. In many cases of soil strengthening or groutjacking, rapid strength development is required in order to limit downtime of a facility. Conversely, where injection is to be made through a very long delivery system, or into large linear void spaces, extension of the setting time might be required, so as to prevent hardening of the grout within the delivery system or the initially filled portions of the void. Admixtures are commercially available that can either accelerate or delay the setting time of all cementitious grouts. In the case of chemical solution grouts, the setting time can often be controlled through judicious proportioning of the mix ingredients, or the use of set controlling components.

### 6B.4.6 Solubility

When mixing a solution grout, rapid dilution of all the components is beneficial and shortens both the amount of time and energy required for mixing. If the grout is to be placed into a saturated formation or void, however, the ability of the mixed grout to easily dissolve is undesirable. Under such placement conditions, and especially where the water is moving, a grout of low solubility in water is desirable. It is also important to assure that the particular grout to be used will not be adversely effected by the chemistry of the formation or ground water into which it is to be injected. An example would be attack of the grout curtain under a tailings dam by retained acidic water.

Properties of grout in the hardened state are also often very important. Is the purpose of the grouting short term, such as solidification of sandy soils to permit tunneling or excavation, or must the improvement be permanent? If strengthening of the soil is desired, the grout or the grouted mass must be of sufficient strength and/or stiffness. In many applications, the dimensional stability or shrinkage of the grout must be considered. Nearly all chemical solution grouts shrink when allowed to dry, as do most cementitious compositions.

Durability of the hardened grout, under the conditions that exist at the injection location, should also be evaluated. In this regard, chemistry of both the soil and the ground water, to which the grout might be exposed, should be appraised, as well as any unusual chemical exposures that might occur during the lifetime of the installation. Exposure to unusual temperature variations must also be considered, as the setting time of virtually all grouts is temperature dependent, as is the durability of many. Both the strength and durability of chemically grouted masses are dependent upon their moisture state. The ability to resist syneresis (the exudation of liquid from the grout) or extrusion of the grout from its intended location is also of importance, especially when the purpose of the grouting is the blockage of high heads of retained water.

Most reacted water-control grouts will shrink upon drying. Some will swell upon being rewetted; however, there is often a substantial time lag for complete expansion, and often times it will not be to the full volume that existed prior to drying. Although this is of little concern where the grout will always be in contact with water, it is an important consideration in arid regions or other locations where drying might occur.

## 6B.5 TESTS FOR EVALUATION OF GROUTS

Simple field test methods to assure the quality and propriety of the grout are unquestionably one of the greatest needs in geotechnical grouting. Unfortunately, few simple tests exist for either very thin or very thick grouts. A number of tests do exist, however, for mid-range grout consistencies, which are generally fluid suspensions.

### 6B.5.1 Flow Cones

Several different configurations of funnel devices have been proposed or used to evaluate the flow properties of grout, two of which have become somewhat commonly used. The "flow cone," Figure 6B.26, originally developed in the early 1940s, has been used for many years by the U.S. Army Corps of Engineers under the designation CRD-C611, for both research and field quality assurance. In 1981, it was adopted by ASTM as part of their C939 "Standard Test Method for Flow of Grout for Preplaced Concrete Aggregate." The Marsh funnel, Figure 6B.27, long used to evaluate the flow properties of drilling fluids, is now also used for pourable grouts.

Flow evaluations are made by filling the funnels while holding a finger over the outlet. The finger is then released, allowing the funnel to empty. The time is noted to the nearest second, providing the number of seconds required for the cone to completely empty. This is known as the efflux time. The C939 cone is typically made of cast aluminum, whereas the Marsh funnel is usually made of

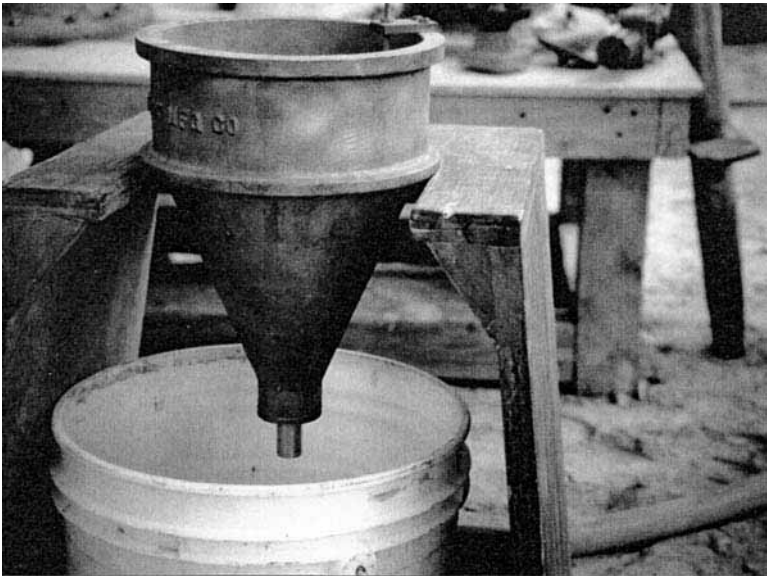

**FIGURE 6B.26** ASTM C939 Flow Cone.

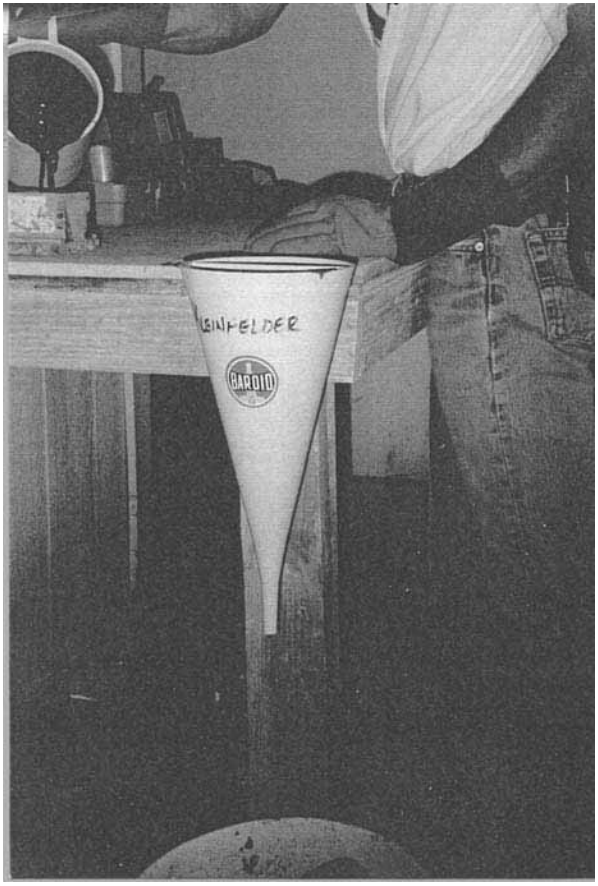


**FIGURE 6B.27** Marsh funnel.

plastic. Neither material can have wetability properties at all similar to the wide variety of geomaterials into which a grout might be injected. They do, however, provide for easy confirmation of the uniformity of different batches of grout and are widely used in quality assurance testing.

### 6B.5.2 Specific Gravity

Another testing device that was originally developed for the evaluation of drilling fluids is the Baroid Mud Balance, Figure 6B.28. It is a simple and very rugged device for determining the density of drilling mud or grout. In application, the cup and beam are removed from the fulcrum, and the cup is dipped into the grout until it is completely filled. Any excess grout that has come in contact with the beam or outside of the cup is removed. The beam and cup are then placed on the fulcrum, and the weight is slid along the beam until the bubble of an attached spirit level is centered. The density is then read directly from a scale on the beam. The mud balance is a particularly useful device for quality assurance testing of cement–water suspensions, in that the water to cement ratio can be easily and accurately determined.

**FIGURE 6B.28** Baroid mud balance used for evaluation of grout density.

## 6B.5.3 Evaluation of Bleed

Controlling the bleed of suspension grouts is also important, especially when slow pumping rates are used or large voids are being filled. The bleed potential of a suspension grout can change quite markedly with changes of water to cement ratio, shear mixing energy, or changes in properties of the cement or other grout constituents. Bleed evaluation is easily accomplished by filling a transparent tube or jar with the grout and observing the amount of clear water that collects on the top after about two hours. Where accurate determinations are desired, 1000 ml graduated cylinders, as shown in Figure 6B.29, can be used. These can be used in a manner similar to that provided in ASTM C 940, "Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory." Whereas this procedure is useful for laboratory evaluations, the required settlement time often renders it impractical for the control of injection parameters in the field.

## 6B.5.4 Pressure Filtration

Pressure filtration is essentially bleed forced by the pumping pressure imposed upon the grout. It can be evaluated for common cement grouts with a standard Gelman pressure filter. The 47 mm diameter test chamber shown in Figure 6B.30, fitted with a disposable glass fiber filter, works well on

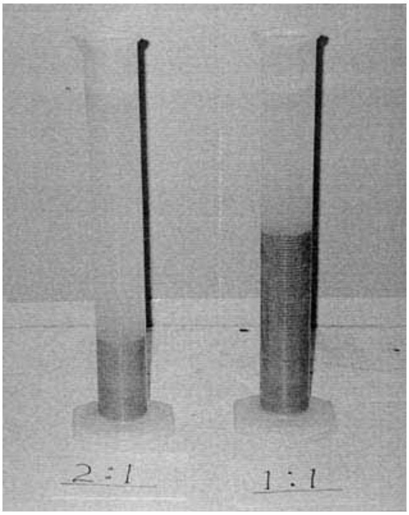


**FIGURE 6B.29** Evaluation of the bleed of two cementitious water suspension grouts with different water to cement ratios.

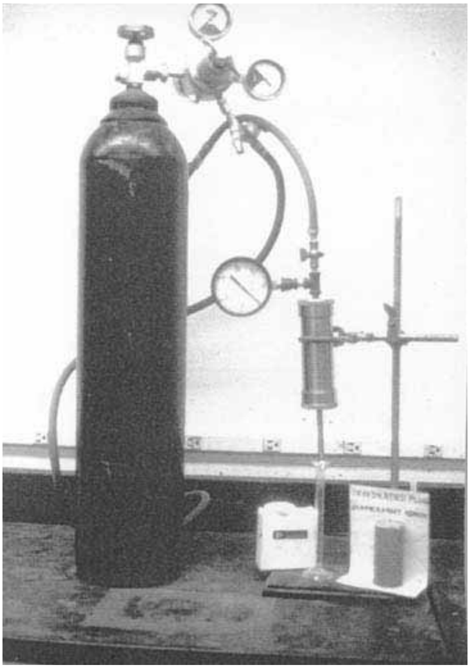

**FIGURE 6B.30** Pressure filtration evaluation with Gelman filter apparatus.

grouts based on common cements. Various applied pressures on the grout can easily be imposed by pressure regulation of bottled nitrogen gas, which is typically used.

### 6B.5.5 Slump

As previously discussed, standard slump tests are not appropriate for either very wet or very dry mixtures, and certainly not for the rather sticky mixtures that are common in grouting. Except for the rather rare instances of *fill grouting*, in which standard concrete or mortar is being injected, the slump test should be neither specified nor used. Where it is mentioned, however, the particular standard should be provided. There are several different established slump tests that involve molds of different sizes and configurations than the most common ASTM C 143 test, shown in Figure 6B.20.

### 6B.5.6 Strength Tests

In most cases of soil grouting, grout strength is of little concern as long as it is at least that of the soil. In fact, a mistake frequently made is specification of cementitious grout strength, based on typical concrete or mortar standards. Specifying unnecessarily high strength for the grout is in most cases of no benefit, can present difficulties for the contractor, and usually results in increased costs. In those cases where strength is of importance, either standard unconfined or triaxial compression tests can be made. Specimens for grout are usually standard 2 by 4 inch cylinders or two inch cubes.

In the case of permeation grouted masses, it is not the strength of the grout that is of importance, but rather that of the resulting solidified soil mass. In such cases, any laboratory tests, to be meaningful, must involve saturation of the proposed grout into a confined mass of the particular soil. ASTM Standard D 4220, "Test Method for Laboratory Preparation of Chemically Grouted Soil Specimens for Obtaining Design Strength Parameters," is applicable for the preparation of such specimens. Because virtually all laboratory tests rely on remolded specimens, the indicated strengths may not compare closely to that of the in-place grouted soils, however the indicated strength should give a good idea of the strength levels that can be reasonably expected.

In order to verify field strength performance, specimens should be carved from the actual grouted mass for evaluation. This will require excavating to the grouted zone for access. Core drills can be used to obtain specimens of many masses of cement grouted soil and those soils that have been solidified with higher strength chemical solution grout, provided the soil zone where the core is taken has been completely permeated and essentially all of the pore space filled. Grouted soils are usually of much lower strength than grouted rock, and some erosion and disturbance of the core surface often results from the drilling operations. It is sometimes useful to procure a core of greater diameter than is required for the test and then carve it down to the required size. Be aware, however, that very often it is not possible to obtain a proper core specimen of grouted soil. This is especially so where weaker grouts are used or the soil pore space is not completely filled. The strength of many grouts and virtually all chemical solution grouted masses are significantly affected by their moisture condition. It is thus crucial to preserve the moisture condition of the specimen as obtained, and often precludes the use of core drilling for specimens that are above the water table.

Because chemically grouted masses are subject to significant creep, and the fundamental strength is often significantly less than that indicated by the rather rapid loading of the ASTM D 4219 test, which was previously discussed, variable strain, constant stress compression tests, such as shown in Figure 6B.31, are preferable. The simple device illustrated operates on compressed air, and the imposed load level is easily controlled by varying the air pressure with a regulator, such as shown on the right, bottom, of the figure. Such simple equipment can easily be taken to the job site and tests run immediately upon retrieval of the specimens.

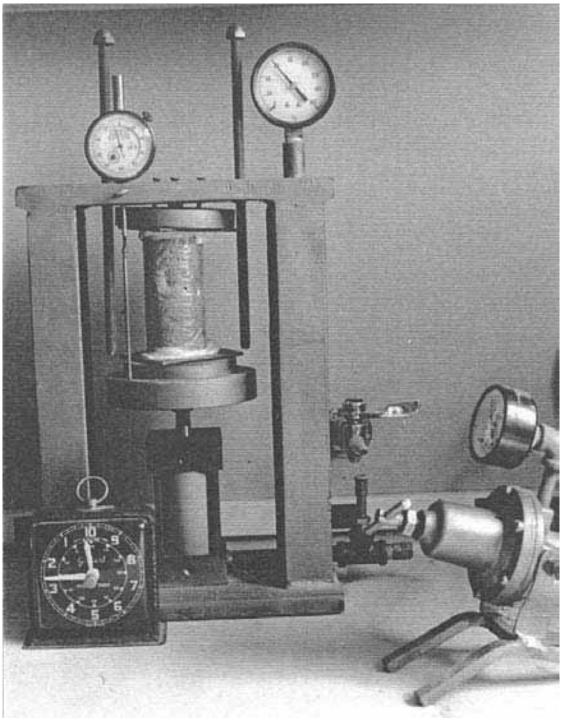

**FIGURE 6B.31** Constant stress testing apparatus with test in progress.

## 6B.6 TYPES OF GROUTING

Classification of the many different types of soil grouting is a difficult task at best. This is due to the wide variety of different grout materials and end uses, as well as the many different injection methods and procedures. Additionally, much confusion results from the promotion of a variety of "proprietary" terminology by grouters, in an attempt to make their activities appear to be something special. Perhaps the best classification for different methods of soil grouting was presented by Xanthakos, Abramson, and Bruce (1994). Therein, four different categories of soil grouting are identified:

1. Hydrofracture (or claquage)
2. Compaction
3. Permeation
4. Jet (or replacement)

### 6B.6.1 Fracture/Claquage Grouting

Fracture grouting involves the intentional hydrofracture of the soil by a fluid suspension or slurry grout, with the intent of producing a network of interconnected grout-filled lenses to act as reinforcement. The procedure was developed in France (origin of the term claquage), in an effort to overcome the limited ability of most soils to accept the particles common in suspension grouts due to insufficient pore size. In theory, the procedure should be quite effective; however, in practice, control of the direction and extent of the fracture system is nearly impossible. Due to this limitation, the procedure has not been extensively used outside Europe, although some notable exceptions exist. To create the hydrofractures, high pressures are generally required. In the experience of the writer, pressures of up to 1200 psi (1.7 MPa) have been used, although pressures as high as 2760 psi (4 MPa) have been reported.

Because uncontrolled hydrofracture can be very damaging to both the soil and adjacent structures, it is crucial to maintain control of the grout deposition to the greatest degree possible. This requires a strict limitation on the quantity of grout injected at any one location (not more than about two cubic feet), necessitating relatively close spacing of the grout holes and injection intervals. In fracture grouting, *sleeve port pipes* (tubes-à-manchette) are used; they are simply tubes with drilled ports, spaced at regular intervals, typically within a range of one or two feet, as shown in figure 6B.32. The tubes are typically made of 1¼ to 1½ inch plastic pipe. The regularly spaced ports are covered on the outside with a thin rubber sleeve so as to prevent any soil or encasement grout from entering the tube. They are placed into oversized drilled holes. A weak grout, which is typically of cement–bentonite composition, is used to fill the annular space between the tube and the oversized hole.

For grout injection, a double packer is used to isolate any one of the ports for injection, which can be used in any desired order. The injection pressure breaks the rubber sleeve and the containing grout seal, such that the grout exits at the desired vertical interval. The writer has investigated many

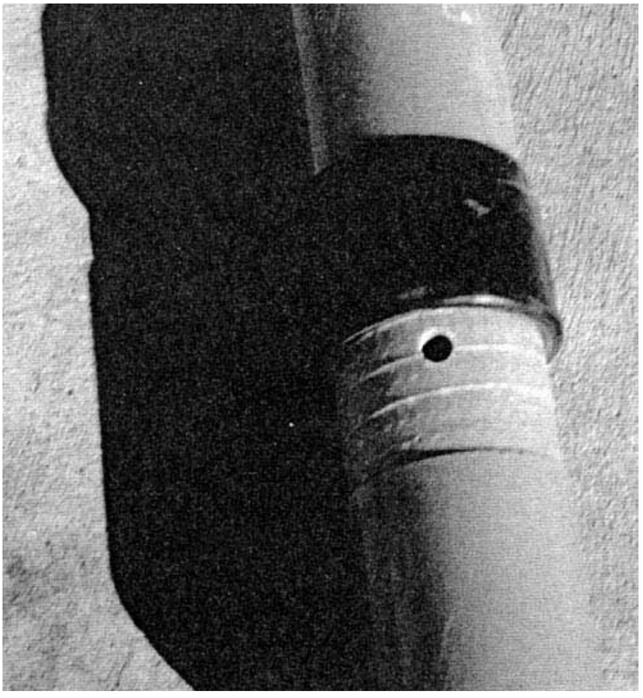

**FIGURE 6B.32** Portion of a sleeve port pipe. Rubber sleeve has been folded back to expose one of the drill hole ports.

instances of damage resulting from uncontrolled hydrofracture grouting, and is of the strong opinion that where performed, strict limitations of grout quantities, combined with meticulous observation and control of the grout pressure and other injection parameters, are essential. Fracture grouting can be performed in virtually any soil and to any depth.

There are other grouting methods, which are usually both faster and more economical than closely controlled fracture grouting, for the strengthening of most soils. A clear exception where fracture grouting is applicable however, involves the pressure injection of hydrated lime slurry to stabilize the moisture in, and reduce the expansive potential of, clay soil. Because the chemical reaction requires contact of the lime with the soil, closely spaced fractures are needed. This type of injection is usually done on a fairly large-scale basis by contractors who specialize in it. The equipment tends to be massive, possessing several probes with perforated pointed tips, which are pushed into the soil simultaneously, with either continuous or discrete injections of the lime slurry (Figures 6B.33 and 6B.34). As can be observed, the process can be quite messy, and avoiding lime deposition on either the equipment or surrounding surfaces is nearly impossible. Although the slurry has a distinctive white color, and is quite alkaline, with a pH of about 12, it presents no serious health or environmental risks, and is fairly rapidly diluted by rainfall or washing, and then absorbed into the surface soil. The high alkalinity, however, can cause burns on human flesh if allowed to remain in contact for an extended time. Spills should therefore be promptly washed off the skin.

The depth of treatment depends upon the expansive potential of the subject soil. It is usually on the order of 5 to 20 feet, but applications as great as 40 feet, have been reported. The slurry consists

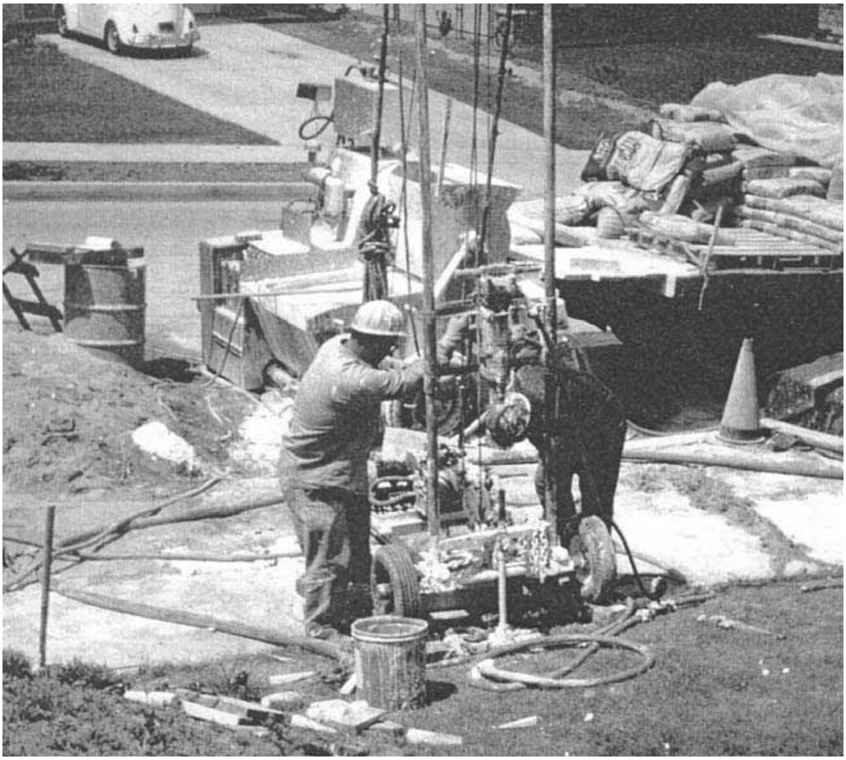

**FIGURE 6B.33** Small-scale lime injection

**FIGURE 6B.34** Large-scale lime injection.

simply of water mixed with about 25 to 30% by weight of dry hydrated lime. A surfactant wetting agent is often included to improve penetrability of the slurry. It is typically injected at pressures within a range of 50 to 200 psi. Injection is continued until either refusal at the maximum pressure or surface leakage occurs, or the predetermined amount of slurry, which is typically about 10 gallons per vertical foot of hole, has been placed. Because the injection slurry consists primarily of water, which can be absorbed into the clay, some accelerated expansion can take place, immediately after injection, unless the initial soils moisture content was near its maximum level. This expansion tends to stabilize shortly after the injection, however, usually within a period of two or three weeks.

Lime injection has been subject to considerable promotion by those who perform the work as well as the National Lime Association, a trade organization for manufacturers of commercial quicklime and hydrated lime. The association has made available a publication by Boynton and Blacklock (1986) that includes a comprehensive bibliography of 51 references.

In 1967, the writer was involved in the remediation of damage to the slab on grade floor of three large dormitories at the Hollygrove Orphanage, in Hollywood, California. An investigation disclosed that the floor slab had experienced upward movement of as much as 1.5 inches. An additional heave of 2.75 inches was predicted. The cause of the upheaval was determined to be a layer of highly expansive clay, underlying the floor at a depth of two to seven feet. Because funding was restricted and moving the occupants out of the structure was not possible, lime injection was adopted as the most reasonable remedial method, although it was considered somewhat experimental at the time.

As previously mentioned, lime injection is inherently messy, so great care was taken to perform the work with as little damage to the structure as possible. Injection holes were located on a three foot grid, normal to the module of the existing 12 inch square floor covering tiles. The work was accomplished with only minor disturbance and the structures were occupied throughout the work. Limitation of floor covering replacement to only those tiles at the hole locations, and waxing of the entire floor upon completion of the work, resulted in a large cost savings to the owner. The replacement tiles were of bright colors to provide an accent, which actually improved the esthetics of the original construction.

The slurry consisted of high-calcium hydrated lime mixed with water, at the rate of about 4.2 pounds of lime per gallon of water, to make a nearly 50% mixture by weight. The injection rate was held to a relatively low value of about 0.6 cubic feet per minute, in order to minimize leakage. Injection pressure varied between 50 and 250 psi. Reading from a report prepared after completion of the work:

> The slurry was injected from the top down through special injection needles made especially for this work. The needle was driven downward from the surface until the clay layer was "felt," after which a metered quantity of 0.25 cubic feet (7 L) of slurry or 7.5 pounds (3.4 kg) of lime was injected. The needle was then continued downward in 6 inch increments, the metered amount of slurry injected at each, until the bottom of the clay layer was "felt" or minus 7 feet was reached. In some cases it was not possible to inject the full measure of slurry due to uplift of the structure walls or floor.

Several elevation surveys had been taken prior to the work. Others were taken immediately following completion and at 30, 90, 180, and 360 days thereafter. Immediately following the injection, an additional upward movement of about 0.25 inch (6 mm) was recorded. No further movement was noted thereafter, and, in fact, after one year a portion of the original 0.25 inch (6 mm) of heave was recovered.

### 6B.6.2 Compaction Grouting

Compaction grouting involves injection of very stiff, mortar-like, low-mobility grout at high pressure into discrete zones of soil. Because of the low mobility of the grout, much higher pressures are used than in traditional grout injection. It is not unusual to experience pressures of several hundred psi when injection is made at depths of only five or ten feet. Properly placed, the grout remains in a homogeneous, expanding mass, which displaces and thus compacts the adjacent soil, increasing its density. Fundamental to the success of the procedure is deposition of the grout in such a manner that it remains in a globular mass at the injection location, with a distinct grout–soil interface. In deed, one of the primary advantages of the technique is that absolute control of the grout deposition location is possible.

The work is virtually always done in stages; that is, only a few feet of the grout hole are injected at any one time. The staging can be from the top down (downstage) or from the bottom up (upstage). The upstage method is the fastest and most economical, and thus, the most frequently used, especially for deep injection. For shallow injection (less than about 15 feet), working downstage has the distinct advantage, in that each injected stage provides additional restraint and containment for those that follow. Thus, higher pressures, which enable a greater quantity of grout to be injected, and thus greater densification, can be used in the deeper stages, after the overlying soils have been strengthened. Whereas upstage injection is nearly always accomplished in one continuous operation, when working downstage, each stage is allowed to harden before the next one is drilled and grouted.

Grouting upstage involves:

1. Drilling a hole to the bottom of the zone to the grouted.
2. Placing casing to within a few feet of the bottom of the hole. The casing should be a snug fit and may require pushing or driving into place. Sometimes it is driven entirely, the predrilling being eliminated.
3. Injection of the grout is continued until essential "refusal" is reached. Refusal is usually considered as: a) a slight movement of the overlying ground surface or improvements, b) injection of a

predetermined amount of grout, or c) reaching a given maximum pressure at a given pumping rate.

4. Raise the casing a regular increment, usually one or two feet (0.3 or 0.6 m)
5. Resume grout injection until refusal.
6. Repeat steps 4 and 5 until the top of the zone to be grouted has been reached.

Grouting downstage involves:

1. Drilling an oversize (usually about three inch diameter) hole, to the top of the zone to be densified or a minimum of about four feet (1.2 m) deep.
2. Insert a casing (usually two inch internal diameter), into the hole and fill the annular space outside the casing with rapid setting grout.
3. Drill through the casing and advance the hole several feet for the first stage. Typical stage lengths are on the order of three to six feet (1 to 2 m).
4. Inject grout until refusal is reached, as described above.
5. Repeat steps 3 and 4, after the previously placed grout has hardened, until the bottom of the zone to be injected is reached.

Grout holes are usually spaced on a grid of six to twelve feet, although closer spacing is occasionally used. Alternate primary holes should be injected before the intermediate secondary holes. As a general rule, the injection should start at the outside of the area to be improved, working toward the interior portions. When grouting near a down-slope or retaining wall, the holes nearest these features should be injected first. Holes should generally be vertical, as inclined holes provide a greater horizontal effective area (Figure 6B.35). This results in refusal, due to surface heave, at lower grout pressure, and thus less injected grout and resulting compaction. Also, a vertical column of grout and compacted soil provides better support than one that is inclined. Where inclined holes are used, they should generally not be more than about 20 degrees off vertical.

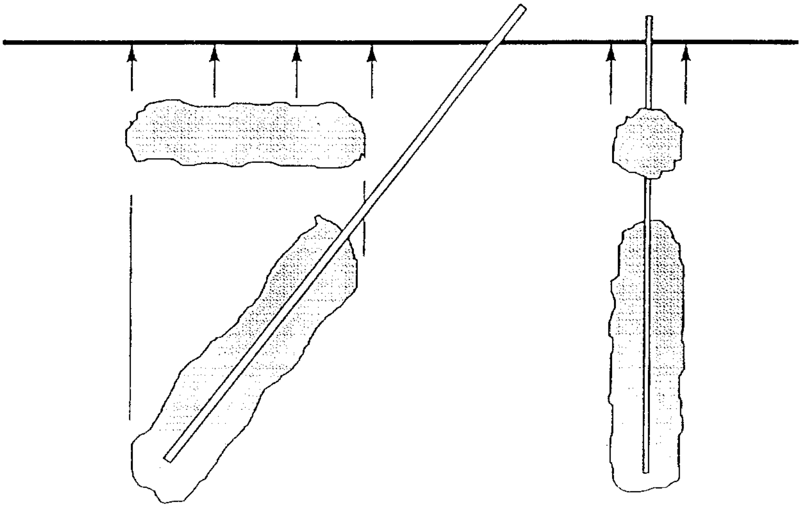

**FIGURE 6B.35** Effect of grout hole inclination on surface uplift. Total uplift force is the product of pressure times *horizontal area.*

Compaction grout casing is typically two to four inch (5 to 10 cm) ID steel tubing. Some contractors use proprietary casing, while others use either standard flush wall drill casing or iron pipe. The outside surface of the casing should always be of a uniform diameter, which usually means a flush wall threaded coupling. When working upstage, the casing, and especially the couplings, must possess sufficient strength to resist the considerable extraction forces that are often required. Extraction, is most often provided by pairs of hydraulic jacks (Figure 6B.36). From the standpoint of grout effectiveness, larger casing can be used. However, in casing larger than about three inches, the pressure at the bottom of the hole resulting from the weight of the grout will usually exceed the restraint provided by line friction and may result in excessive pressure being exerted at the tip. A few contractors attempt to perform the work with casing of an internal diameter less than two inches. This is not advisable, however, at it is sometimes difficult to establish grout flow in such a small hole and the risk of hole blockage is significant. Also, if working upstage, casing smaller than 2 inch ID, often lacks sufficient strength to withstand the extraction force, and joints are more prone to breakage.

Because of the very low mobility grout that is used in compaction grouting, all valves and fittings must provide full flow openings. Appropriate gage savers must be supplied for all pressure gages and should have a minimum dial size of three inches, so they can be easily read. Standard pipe fittings should be avoided, and wide sweep bends used as required. A high-pressure, two inch internal diameter hose is the most often used, however one and one half inch hose is sometimes adequate. Where especially long runs are required, the use of rigid pipe will decrease the resulting line pressure. Appropriate quick connect couplings, such as those used for concrete pumping, should be used. Figure 6B.36, shows a typical connection to a hole during grouting.

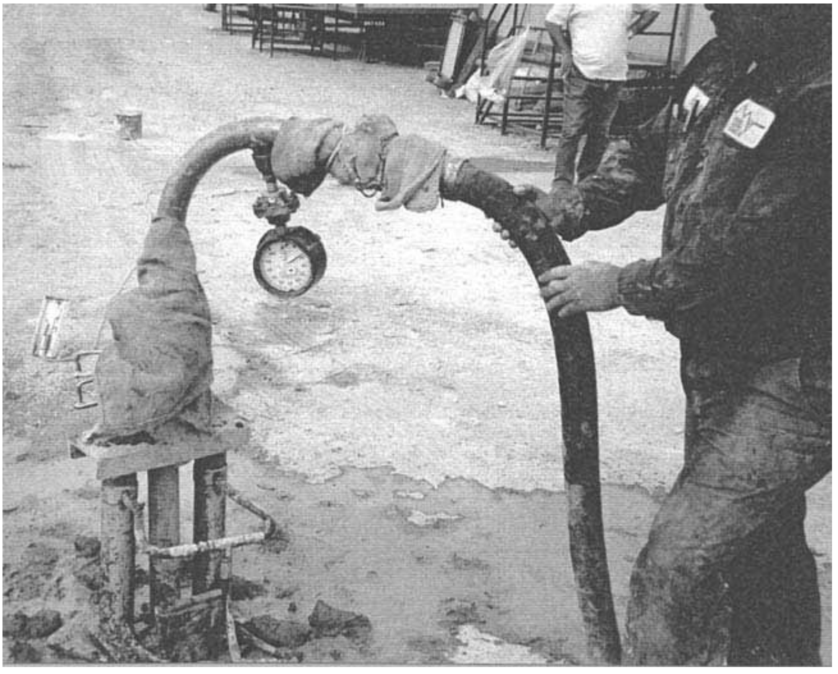

**FIGURE 6B.36** Typical connection to a grout hole during upstage injection. Note pair of hydraulic jacks used for casing extraction.

Regardless of the method used or the slump or stiffness of the grout, should it be overly mobile or act as a fluid under pressure, hydraulic fracturing of the adjacent soil can occur, which will result in loss of control of the zone of compaction. Hydraulic fracturing not only interferes with orderly compaction, but can also result in unacceptable displacement or other damage. Should it occur in near proximity to a down-slope, retaining wall, or other substructure, or within a water retaining embankment, severe displacement or complete failure could result. Compaction grouting has been successfully used in most types of soil and to depths of more than 400 feet. Injection into saturated clays is dependent upon their ability to drain, however, and very slow injection rates are usually required. This increases the amount of time required to perform the work as well as the cost and often precludes its use. The procedure also has limited effectiveness in clean, coarse sands and gravels.

Compaction grouting originated in California, where the procedure has been practiced for more than forty years. Its application is now extensive throughout North America, and it is often used in many other countries, especially in Asia. The first publication on the procedure (Graf, 1969) presented a theoretical description of the process, hypothesizing that the injected masses would be generally spherical and would densify the injected soil radially, in all directions. The first report describing the actual mechanism, and providing data derived from the excavation and removal of full-scale test injections, was made by Brown and Warner in 1973. It reported columnar injected masses, with essentially horizontal, radial densification.

Subsequent to this early work, a large number of masses have been injected, exposed, and evaluated, in research demonstrations as well as in connection with actual or proposed projects. Evaluations have included determination of the increase in resulting soil density, utilizing standard penetration, cone penetrometer, and laboratory tests of split spoon specimens, both before and after grout injection. Large-scale load tests and long-term elevation monitoring of structures overlying treated soils have also been performed on many applications. Additional studies have been directed at the effect of lateral forces exerted in the soil mass and upon adjacent structures. This substantial research and investigation has resulted in compaction grouting becoming one of the best understood grouting technologies. The process is especially suited to the densification of soils, which is its only use.

The single most important parameter to assure effective soil densification is obtaining a globular grout mass, which will typically be either columnar or pear shaped. Cracking or hydraulic fracturing of the soil mass, with resulting thin lenses of grout, should generally be avoided. One measure of regularity of the obtained grout mass is the Travel Index (TI) of the grout. The TI is the maximum radial travel of the grout from the point of its injection, divided by the minimum radial distance to a grout–soil interface, and is thus representative of the grout's propensity to remain in a controlled mass and at the intended location. Grouts with a low travel index (less than about three) remain in relatively symmetrical masses, with clear interfaces of the surrounding soil. Loss of placement control and hydrofracture of the soil, however, is virtually assured with grout travel indexes exceeding about five.

Importance of the shape of the injected grout mass was first reported by Brown and Warner (1973). They described a research program performed in the 1950s that involved the injection and subsequent excavation of more than 100 test grout masses. The effort involved a variety of different mix designs, consistencies, sand materials, and injection rates. A photograph of two of the excavated test grout columns depicting desirable shape was provided and appears here as Figure 6B.37. It is interesting to note the much smaller diameter of the lower portion of the mass shown on the right. Whereas the upper deposits of the test site consisted of mixed soils, the grout holes extended into an underlying clean sand layer, which was not subject to the same degree of compaction as the upper mixed soils. This resulted in a much smaller mass diameter in the sand. The paper concluded with examples of many projects, successfully completed with grouts, that were found to be optimal. The grouts were reported to consist of: ". . . fine sand combined with about 12% cement and water to form a very stiff mortar like mixture." Emphasized were criteria such as "The greatest amount of grout injected, and thus greatest densification achieved, resulted from the use of very stiff mixtures," and "A slower pumping rate resulted in significantly higher grout takes."

The same authors expanded on their experience in a further report, "Planning and Performing Compaction Grouting," (Warner and Brown, 1974). Therein, the significance of the grout composi-

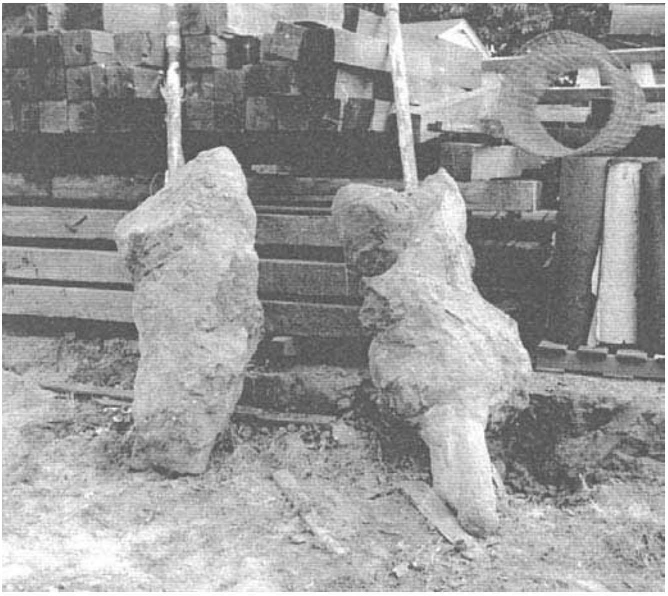

**FIGURE 6B.37** Grout "columns" from two test injections (Brown and Warner, 1973).

tion, and in particular, a strict limitation of any clay content, was further emphasized. An entire section was devoted to describing the "sand material," which included a gradation envelope, "Preferred Limits of Gradation for Sand Used for Compaction Grout," given here as Figure 6B.38.

Noteworthy is the zero allowance for clay size constituents. The significance of grout consistency was again emphasized: "It is preferable to use the least amount of water that will provide a very stiff plastic consistency grout. A rule of thumb is the stiffer the grout, the more effective its injection will be."

Included was a photograph of such grout extruding from a grout hose, included here as Figure 6B.39. Further emphasized were the risks of excessive pumping rates, which could result in "rupture" of the soil. The first two of their concluding statements for proper performance stressed the importance of the aggregate gradation and resulting grout rheology, as well as the injection rate: "Proper gradation of the sand material which accounts for 80%–90% of the total grout volume is imperative" and "Absolute control of grout rate is imperative . . . within a range of 0.3 cu ft (0.009 $m^3$/min to 2 cu ft (0.06 $m^3$/min. . . ."

Quite surprising to this writer, the salient conclusions of that early research have proven to be applicable to the present day. Furthermore, that applicability has been substantiated by extensive recent research and literally thousands of successfully completed projects.

As part of the grouting demonstration included in the 12th annual short course, Fundamentals of Grouting, now sponsored by the University of Florida, and held in Denver, Colorado in 1990, ten grout test injections were made and exposed. Two different grout mixtures, designated "A" and "B," were utilized. They were identical except that 5% bentonite by weight of the sand material was included in the "B" mixes. Notwithstanding the recognized inappropriateness of the slump test for such grouts, such tests were made in as careful a manner as possible. The two grouts were injected at ASTM C143 slumps of one, two, three, and four inches (25, 51, 76, and 102 mm), and at a constant injection rate of 1.5 $ft^3$ (0.04 $m^3$) per minute.

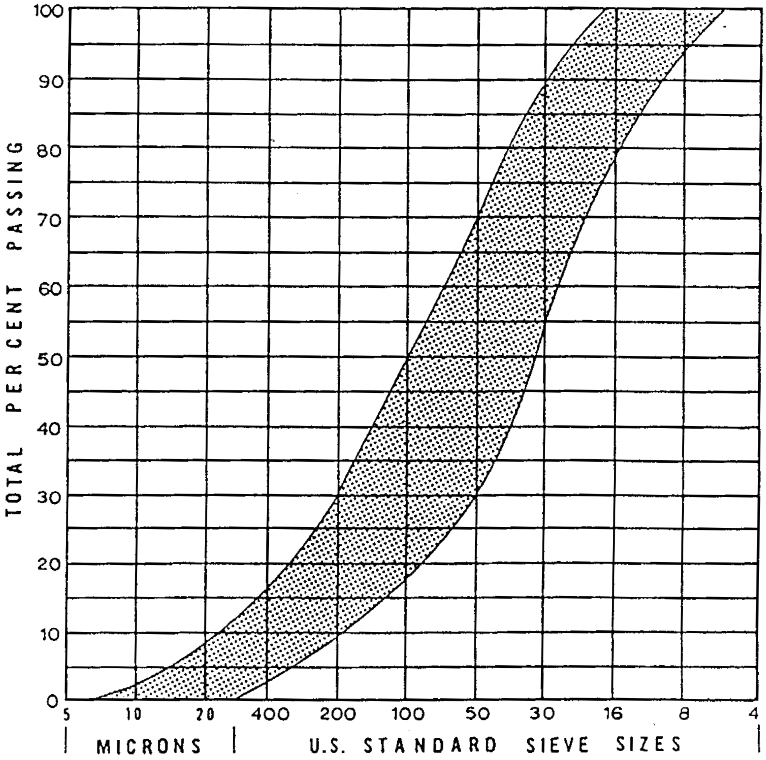


**FIGURE 6B.38** Preferred limits for sand material used for compaction grouting (Warner and Brown, 1974).

**FIGURE 6B.39** Very stiff plastic consistency grout (Warner and Brown, 1974).

The test site consisted of highly stratified stiff to dense clayey silts and sands, which exhibited negligible settlement potential. Untreated, they would provide adequate support for most normal foundations, and thus provided an extreme for evaluation of compaction grouting. Several of the injections resulted in hydraulic fracturing of the soil; however, the incidence and extent of the fracturing, was directly related to the grout rheology. All of the "B" mixes resulted in hydraulic fracturing and revealed very high TI's. Of particular interest was the markedly better performance of the "A" grout at a four inch slump than the "B" grout at a slump of only one inch.

An extensive and very well documented research effort was conducted in San Diego, California in 1991 (Warner et al., 1992). Eighteen grout masses were injected, with three different grout mix designs, using different grout consistencies and injection rates. The soils at the test site were predominantly fine to medium grained sands, with approximately 20% of the material passing a #200 (0.007 mm) sieve. The minus #200 (0.007 mm) portion consisted of approximately 70% silt and 30% low-plasticity clay. The soils were of waste fines from a quarry, and were deposited as an uncontrolled fill. Their in-place densities varied from 79% to 96% of maximum dry density as determined by ASTM D 1557.

The grout mixes, which were designated A, B, and C, were all identical except for the aggregate fraction, which was of different gradations. The A and B aggregate as obtained from the pit were identical. They contained about 7% clay, and the minus #40 sieve (0.04 mm) fraction was plastic, with a liquid limit of 35 and plasticity index of 10. The B grout mix contained the material as received from the pit, whereas the A mix had about 30% minus ¾ in (19 mm) gravel added With the exception of a component of about 4% nonplastic clay, the C aggregate was close to the gradation envelope recommended by Warner and Brown (1974) (Figure 6B.38). Each of the grout mixtures was injected using three different injection rates: one, two, and four cubic feet per minute (.0283, .0566, .085 and $m^3$/min).

Of interest was the development of three differently shaped masses of the injected grout. These were closely related to the aggregate material gradation, especially the clay content. They were radially symmetrical columnar, as shown in Figure 6B.40; four vertical "wings," extending out at about 180 degrees from a columnar mass at the hole alignment (Figure 6B.41); and two wings, with or without formation of an initial grout column, resulting in hydraulic fracturing, as illustrated in Figure 6B.42. Details pertinent to the resulting grout masses are provided in Figure 6B.43, and the travel indices are m shown in Figure 6B.44. The detrimental effect of clay in the grout was again clearly illustrated by the thin wings of grout, observed in Figure 6B.42. further evidence of hydraulic fracturing is clearly delineated by the high travel indices, enumerated in Figure 6B.44.

In 1994, the 1991 research effort of was extended with an additional eleven grout injections. These were made on the same site, immediately adjacent to the 1991 work. In order to establish the influence, if any, of the beginning soil density, the entire test site was excavated and backfilled, with minimal compaction being exerted. As in 1991, the grout mixes were identical except for the gradation of the aggregate. Three different aggregates, designated D, E, and F were used, which contained 0%, 1%, and 4.5% bentonite clay, respectively. In spite of the obvious benefit of including gravel in the grout aggregate, as demonstrated by the earlier work, because many contractors do not have the ability to pump the larger aggregate, it was not included. Exposure revealed grout masses very similar to those of the previous work. Again, a close correlation between the clay content of the aggregate and the shape of the resulting grout masses was illustrated. Figures 6B.45 and 6B.46 show typical examples of the D and F grout masses, respectively. Note the very thin, long wing length of the mass in Figure 6B.46, which resulted in hydraulic fracturing and a very high travel index of 20.

In South Africa, a mill building at a diamond mine had experienced serious differential settlement. The structure was founded on a marginally compacted mine waste fill, composed of sandy silt containing about 6% gravel. Compaction grouting was determined to be the best remedial method, but since the technique had not been previously used in the area, initial injections were excavated to assure proper performance. The grout used was a very stiff mixture of aggregate falling within the envelope of Warner and Brown (1974) mixed with about 10% cement. It was injected at a rate of about (1.5 $ft^3$) 0.04 $m^3$ per minute. As can be observed in Figure 6B.47, the resulting grout mass was in a nearly perfect column.

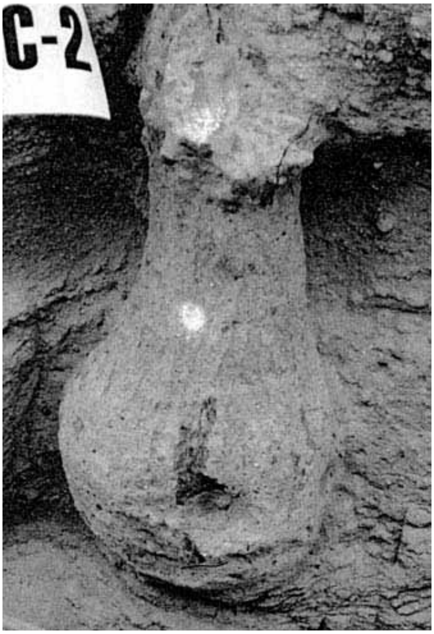


**FIGURE 6B.40** Symmetrical grout mass.

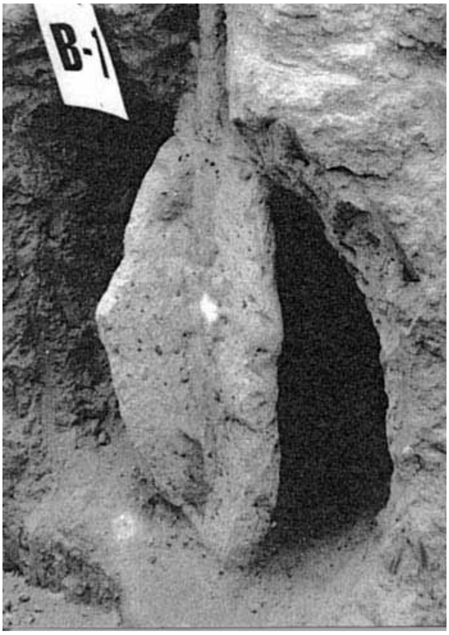


**FIGURE 6B.41** Four wing grout mass.


**FIGURE 6B.42** Two wing grout mass resulting in hydraulic fracturing of the soil.

| Hole No. | Maximum Radial Travel in | Maximum Radial Travel mm | Maximum "Wings" Width in | Maximum "Wings" Width mm | No. "Wing" Sets | Minimum Column Radius in | Minimum Column Radius mm | * |
|---|---|---|---|---|---|---|---|---|
| A-1 | 14 | 356 | 5-7 | 127-178 | 2 | 7.5 | 191 | P/S |
| A-2 | 14 | 356 | 6-7 | 152-178 | 2 | 6 | 152 | P |
| A-3 | 22 | 559 | 5-7 | 127-178 | 1 | 5 | 127 | P/S |
| A-4 | 16 | 406 | 5-7 | 127-178 | 2 | 7 | 178 | P |
| A-5 | 34 | 864 | 12 | 305 | 1 | Negative | | P/S |
| A-6 | 36 | 914 | 8 | 203 | 1 | Negative | | P/S |
| B-1 | 12 | 305 | 4 | 102 | 2 | 6 | 152 | S |
| B-2 | 20 | 508 | 3-4 | 76-102 | 2 | 4 | 102 | P/S |
| B-3 | 15 | 381 | 2-5 | 51-127 | 1 | 3.5 | 89 | P |
| B-4 | 24 | 609 | 6 | 152 | 2 | 3 | 76 | P/S |
| B-5 | 21 | 533 | 4 | 102 | 1 | 2 | 51 | P/S |
| B-6 | 38 | 965 | 6 | 152 | 1 | Negative | | P/S |
| C-1 | 6 | 152 | N/A | | 0 | 6 | 152 | S |
| C-2 | 12 | 305 | N/A | | 0 | 12 | 305 | P |
| C-3 | 22 | 559 | N/A | | 0 | 10 | 254 | S |
| C-4 | 21 | 553 | N/A | | 0 | 11 | 279 | P |
| C-5 | 16 | 406 | 5 | 127 | 2 | 5 | 127 | P/S |
| C-6 | 18 | 457 | 3 | 76 | 2 | 4 | 102 | P/S |

* Sequence - P = Primary; S = Secondary; P/S = One adjacent hole completed.

**FIGURE 6B.43** Grout mass properties (Warner et al., 1992).

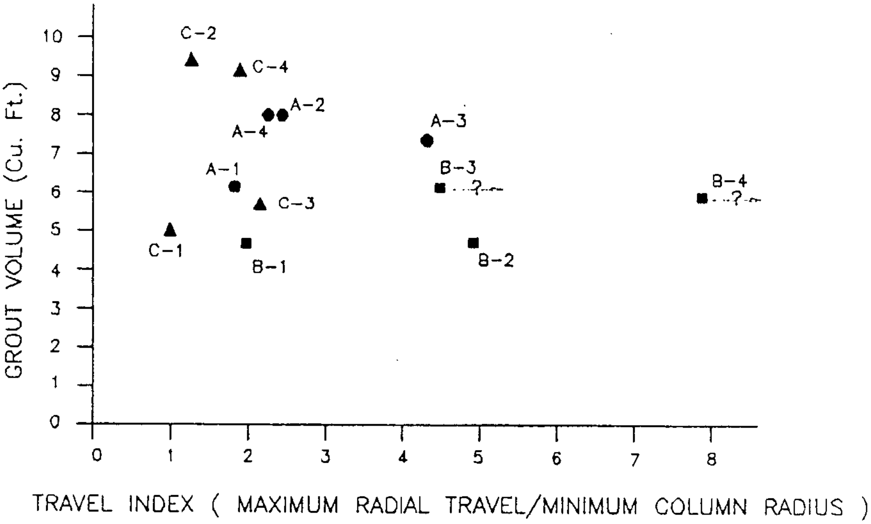


**FIGURE 6B.44** Grout travel index (Warner et al., 1992).

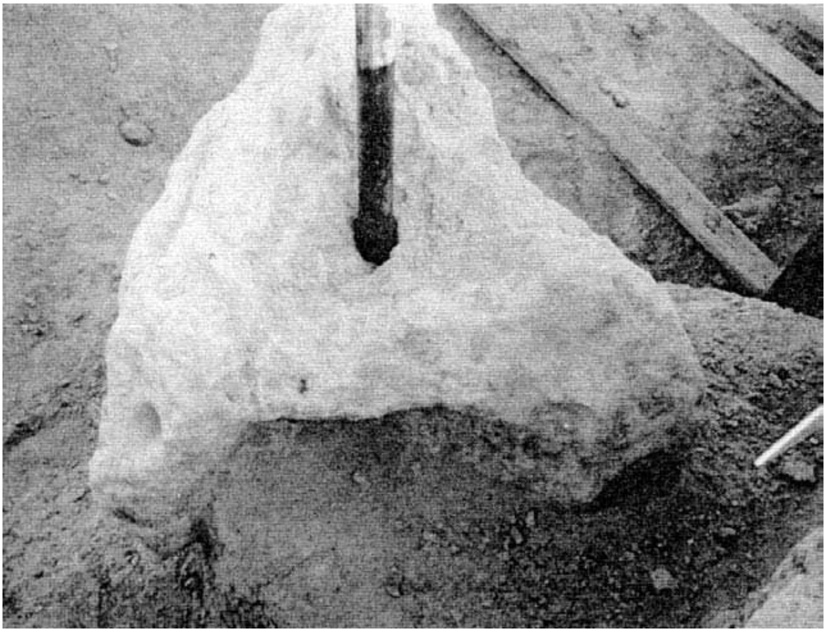

**FIGURE 6B.45** "D" grout mass.

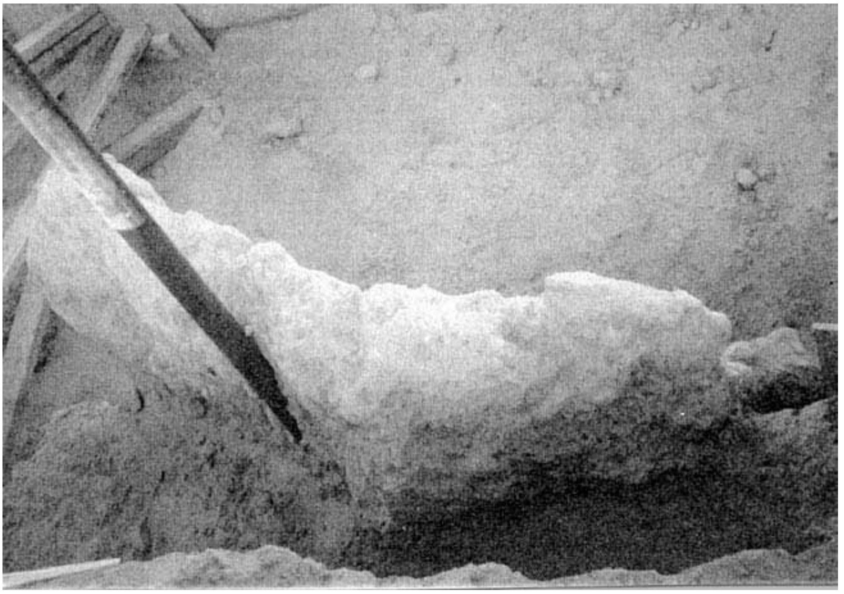

**FIGURE 6B.46** "F" grout mass.

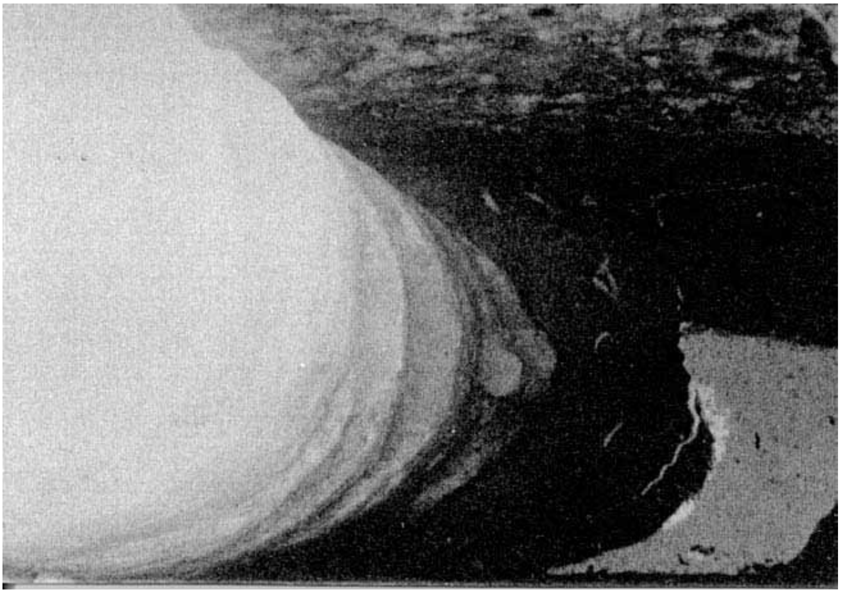

**FIGURE 6B.47** Near-columnar grout mass.

Recent experimental work (1996) in Korea to develop a method to contain underwater bay mud has involved many experimental injections, both on dry land and underwater. The grout typically used is a very stiff mixture similar to that reported by Warner and Brown (1974), and is injected at rates less than 0.06 $m^3$ (2 $ft^3$) per minute. Resulting columnar masses, of nearly one meter (3.3 ft) in diameter have been routinely obtained, as illustrated in Figures 6B.48 and 6B.49. Figure 6B.48 shows an underwater grout mass that has been excavated and is being raised by a derrick barge. The grout mass shown in Figure 6B.49, was injected into loose sand. Upon exposure, the sand fell off, revealing a near perfect column.

The writer has witnessed a large number of excavations made on a variety of compaction grouting projects and visually inspected the particulars of the resulting grout masses. In the early use of the procedure, this was frequently done to increase knowledge of the technology, as well as for quality assurance. In many situations, examination of either full-scale test injections or early production work has been done as a requirement for qualification of the procedure for a particular project. In several instances, where grout injection work has failed to perform as expected, excavations allowing visual inspection of the grout masses have also been made, in order to better understand the cause of the poor performance.

In applications that are in near proximity to a down slope or retaining wall, a risk of displacing the slope or wall always exists. Damage of this type, which results from an inappropriate injection sequence, use of excessively mobile grout, or too high a pumping rate, has unfortunately been experienced on many projects within the last few decades. The initiation of such displacements will *al-*

**FIGURE 6B.48** Grout mass from underwater injection.

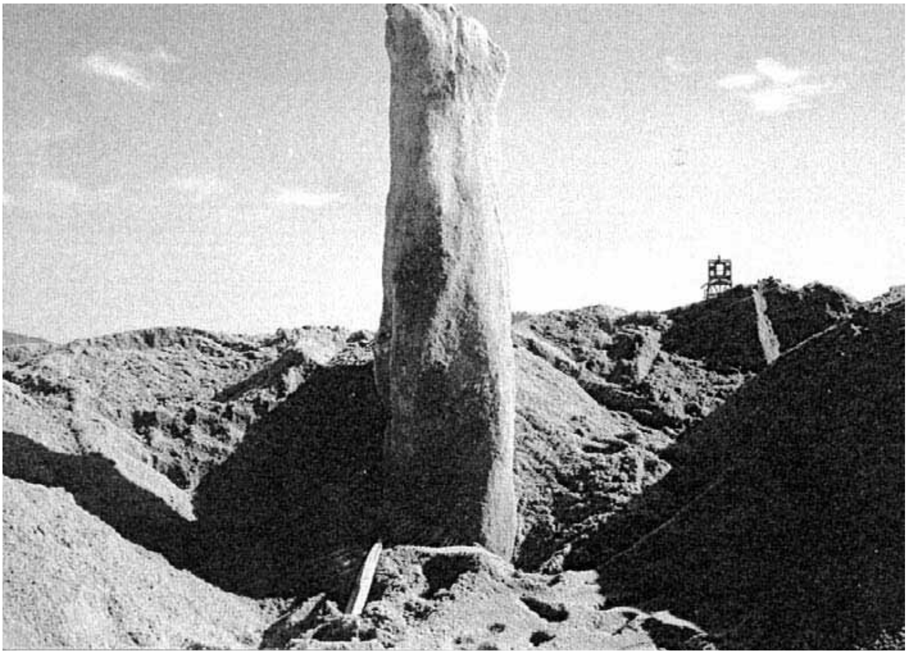

**FIGURE 6B.49** Near-perfect column.

*ways* be indicated by a sudden loss of injection pressure. Competent grouting crews will thus promptly check for such movement at every pressure loss, especially when working in sensitive locations. If movement is detected, corrective procedures such as halting injection or reducing the pump output (which lowers the pressure) should be immediately taken, precluding the occurrence of destructive movements.

Unfortunately, not all compaction grouting projects have been satisfactory. The writer has investigated many instances where far less than acceptable performance has occurred. In virtually every instance, the poor performance was directly linked to one of four faults:

1. Failure to treat the faulty material for its full depth. Compaction grouting adds considerable weight to the treated zone. It is imperative that grout not be deposited over a soil formation that is unable to support the combined weight of the original soil and new grout. In this regard, experience has continually revealed the culprit soil zone in apparent fill failures to be in the bottom of the fill or in the original soil immediately thereunder.
2. An inappropriate injection sequencing. Soil settlement often results in lateral spreading, which causes open cracking on the ground surface or structures thereon. Initial injection should always be started at the furthest limits of the soil, which has influence on the surface spreading. As an example, settlement occurring in near proximity to a retaining wall or downslope generally has lateral movement in the direction of that feature. Initial grout injection should thus be in rows of holes nearest to the wall or slope. It is usually possible to push the soil laterally so as to close such cracks.
3. Excessively mobile grout. As previously discussed in detail, grout that is excessively mobile, or acts as a fluid in the ground under pressure, will cause hydraulic fracturing. Such a fracture will always be parallel to and near an area of weakened restraint, which is most often a retaining wall or downslope.

4. Too rapid a pumping rate. Soil deformation (compaction) is time dependent. Forcing it to occur too rapidly with grout under pressure will cause disruption and possible hydraulic fracturing. This will also be in the direction of least restraint.

Use of grout aggregates containing clay, or the deliberate addition of clay, to improve pumpability has been a continuing problem within the grouting industry. This is often done to impart lubricity or water retention to the grout, in order to allow an otherwise inadequate pump to function. Unfortunately, use of excessive injection rates are always a temptation for both contractors and their working crews, as less pumping time means earlier job completion and higher profits. Sadly, poor performance, is not often discussed by those involved, and the results of investigations as to the cause are often sealed as a result of legal proceedings.

Deficiencies routinely encountered include the displacement of down-slopes and retaining walls. The slope illustrated in Figure 6B.50 was displaced to such an extent that the exterior wall of the adjacent building (Figure 6B.51) was dislodged some eight inches toward the slope. Clay in the grout, an excessive pumping rate, and probable poor injection sequencing resulted in lateral displacement of the end of the building. The occupant, who was present at the time of the damage, stated to the writer: "The pumping was going real good in the middle of the room when all of a sudden the floor split and everything opened up."

In a similar case, the building shown in Figure 6B.52 was seriously distorted, requiring extensive structural repair. This happened as a result of a retaining wall (Figure 6B.53) some eight feet away being blown out by an incompetent grouting crew. Again, clay in the grout and an excessive pumping rate were to blame.

In another case, a residential structure continued to settle even though a very large amount of grout had been injected under it, about a year before. While inspecting the structure, the writer happened to observe an excavation being made on an adjacent property. There exposed was a vertical fracture filled with grout extending from a previous grout hole (Figure 6B.54). It ran from the point of injection, a distance exceeding 12 ft (3.6 m), onto the adjoining property. Legal considerations

**FIGURE 6B.50** Displaced slope caused by excessive pumping rate and clay in the grout.

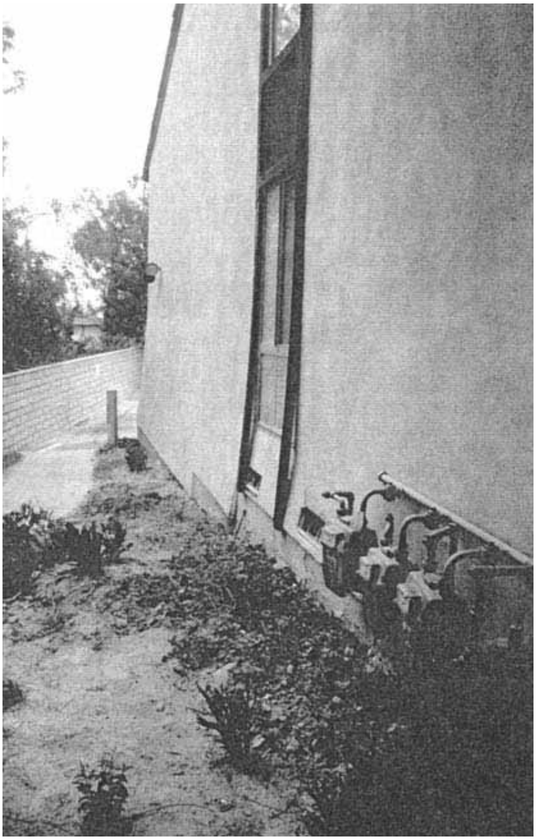

**FIGURE 6B.51** Exterior wall of structure dislodged when slope was blown out.

precluded following it further, but it was obvious that most of the grout that had been injected "under" the structure actually traveled a long distance therefrom. No records of the grout injection had been kept, but it was known that the work was done with ready-mixed grout using a standard concrete pump. Because the fines contained in a proper compaction grout aggregate tend to make a somewhat sticky mix that builds up on the fins of a truck mixer, it is virtually impossible to obtain compaction grout of the proper rheology therewith. It is thus a reasonable assumption that the above example was the result of using an inappropriate grout in combination with an excessive injection rate.

The relatively high grout pressures used in compaction grouting would suggest the development of high lateral forces in the ground. Such is often given as an excuse for damage, as discussed above, and the refusal of some contractors to work in such situations. This notwithstanding, experience with properly performed compaction grouting, in literally hundreds of applications in near proximity to retaining walls or unsupported down-slopes, would indicate otherwise. In fact, a common requirement of the procedure is densification of faulty backfill material.

Satisfactory compaction grout mixes can be made with aggregate material conforming to that indicated on Figure 6B.38. Research and experience have proven, however, that increased control re-

**FIGURE 6B.52** Structural distortion caused by blow out of a retaining wall.

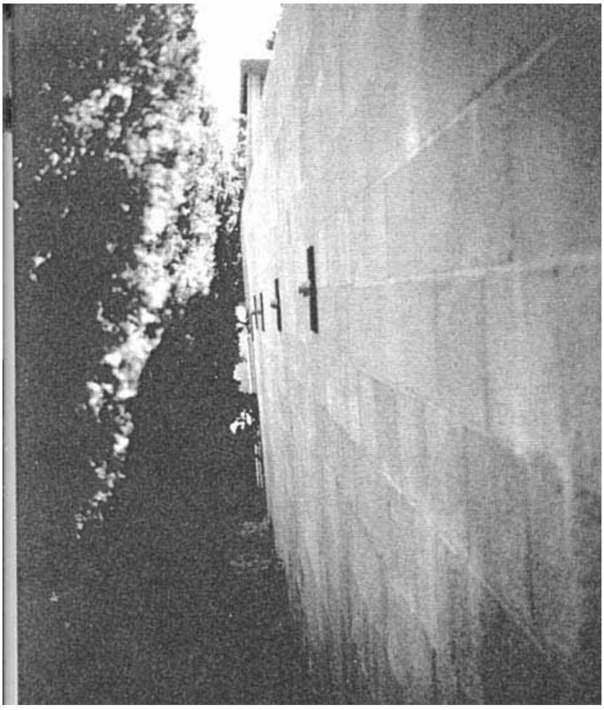

**FIGURE 6B.53** Retaining wall displaced by inappropriate grout injection.

**FIGURE 6B.54** Thick hydraulic fracture some 12 feet from the intended grout injection location.

sults from the provision of gravel in the aggregate. It has also been found that the inclusion of significant gravel will to some degree mitigate the propensity for high travel indices and resulting hydraulic fractures caused by clay in the grout. Because it is often difficult to obtain sands with the required silt content that is totally free of clay, the inclusion of gravel, which can be either blended into the sand or separately batched into the mixer, becomes even more advantageous.

The benefits of this were vividly observed during grouting of a test hole, where a large diameter injection casing was used. During injection, a distortion of the pressure behavior indicated some sort of a grout leak. When the casing had been raised about thirty feet, the cause of the distortion was readily evident. The casing had split, allowing grout to escape in an uncontrolled manner as shown in Figure 6B.55. Washing the grout from the casing with a water spray revealed gravel tightly packing those areas of the split that were less than about an inch wide, as shown in Figure 6B.56. The preferred range of gradation for grout material, which includes the gravel fraction, is provided in Figure 6B.57. Use of the preferred material is strongly recommended where suitable pumping equipment is available, and its use should be mandatory on sensitive projects.

For most compaction grouting, about 10% common portland cement is mixed with the aggregate. The water is limited to that which will provide a very stiff mortar-like consistency, as shown in Figure 6B.58. Such a mixture will provide an unconfined compressive strength of 400 psi (2760 kPa) or more, which is more than adequate for most work. Where required, higher-strength grouts can be formulated by increasing the cement content and using an aggregate with reduced fines. Such is seldom justified, however, and it is not recommended for most work. It must be recognized

**FIGURE 6B.55** Grout escaping from split casing.

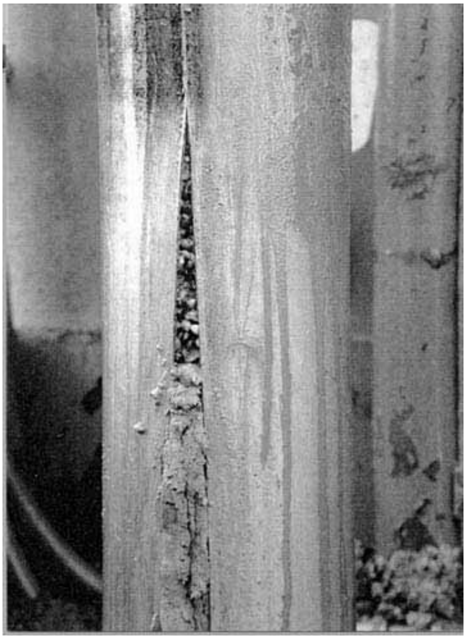

**FIGURE 6B.56** Gravel bridges top of split.

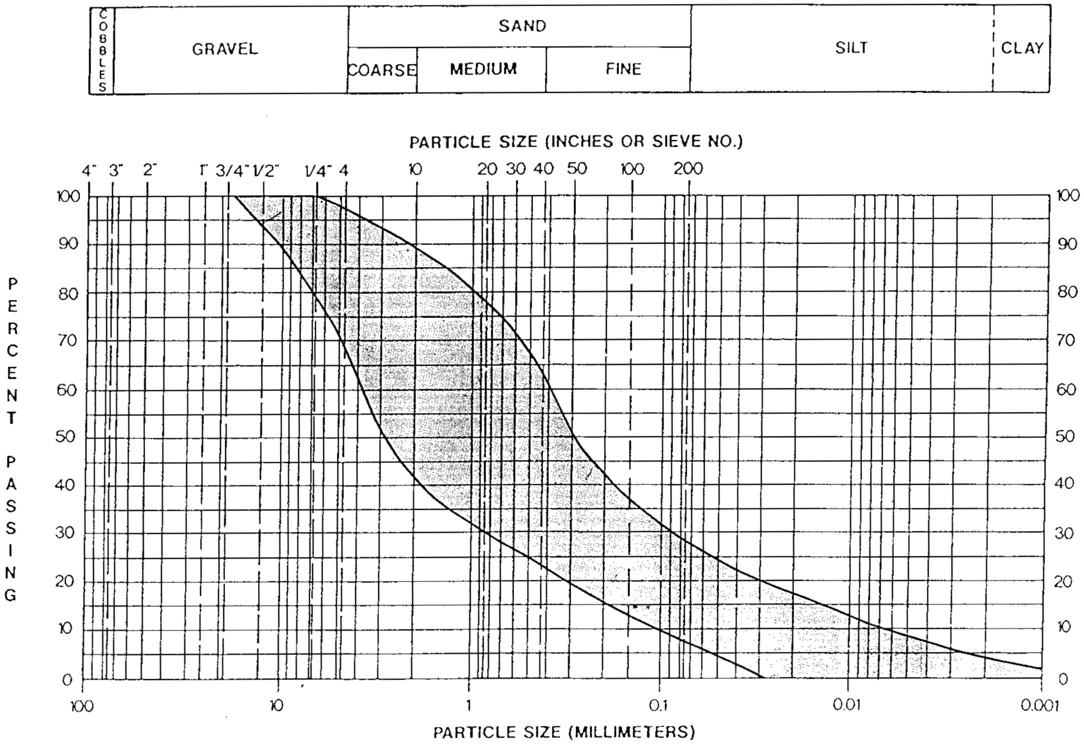


**FIGURE 6B.57** Preferred gradation for compaction grout aggregate.

that a sufficient amount of fines in the aggregate is necessary to provide the required pumpability. Very high grout strengths are seldom justified, and their unnecessary specification is not advisable.

In those instances where only a small amount, or even no cementation is desired, another fine material such as hydrated lime, or a pozzolan, can be used in place of the cement. It is also possible to compound a completely suitable grout with no cementing material at all by increasing the fines content of the aggregate. Increasing the fines will, however, require more mix water, so trial mixtures with the available materials, should be assessed before specifying such a mix.

Because the grout injected by properly performed compaction grouting stays in a homogeneous mass near the point of injection, the process allows for controlled soil improvement. It has the advantage of execution without creating a great deal of mess or interference with the normal operations of a facility. Large equipment is not required near the injection location, which allows the work to be performed in areas of confined or poor accessibility. This results in a very wide range of advantageous applications, and no doubt accounts for the procedure's widespread use in North America, where it originated.

Whereas the greatest use of compaction grouting is in connection with the correction of settled buildings, significant applications have been performed on a wide variety of other structures. The process also is used for site improvement, prior to construction of new structures, such as the mitigation of the potential for soil liquefaction. By far, however, the most common applications are in connection with the repair of structural settlement. Because groutjacking involves essentially the same equipment and grout material, settled structures are usually raised to their proper elevations as part of the program to improve the underlying soils (see also section 7.B.2).

**FIGURE 6B.58** Very stiff, mortar-like consistency grout.

Faulty foundation soils and resulting settlement of literally thousands of light residential structures have been corrected with compaction grouting. Included have been many large and even very heavy buildings, including structures founded on both pile and large continuous raft foundations. Many successful applications have been made where settlement caused structural damage and resulted in vacation and in some cases condemnation of the structures. The four year old building of Manitou Springs Junior High School in Colorado had to be vacated in 1980, due to severe structural distress resulting from several inches of differential settlement. Compaction grouting not only improved the underlying faulty soil, but also jacked the building back to its proper elevation.

A portion of a five story high wing of a concrete building in Rapid City South Dakota had settled several inches, resulting in serious structural distress. In 1986, faulty soils to a depth of fifty feet were remediated under the shallow foundations, and the settled areas raised to their original grade while the structure remained fully occupied. In 1991, the four story high Inage Welfare Center building in Chiba City, Japan remained fully functional and occupied during the correction of nearly three inches of differential settlement.

The first use of compaction grouting in connection of with a pile supported structure occurred in 1966, in Hollywood, California. Settlement of the seventeen buildings of an apartment complex still under construction (Figure 6B.59) was occurring. The structures were built on a deep canyon fill, which had been deposited over a period of several decades. The fill consisted primarily of a variety of uncompacted soils, but also included many large boulders, and a minor amount of organic waste. The foundation consisted of concrete grade beams supported by end bearing, bell bottom, cast in place concrete piles. Investigation revealed that several of the piles, which varied in depth to more than eighty feet, did not extend to a competent bearing layer as planned, and some actually terminated in massive boulders embedded in the fill. It was considered imperative that no grout be deposited above the pile tip elevation, as this would act negatively and increase the load on the already overloaded end bearing piles.

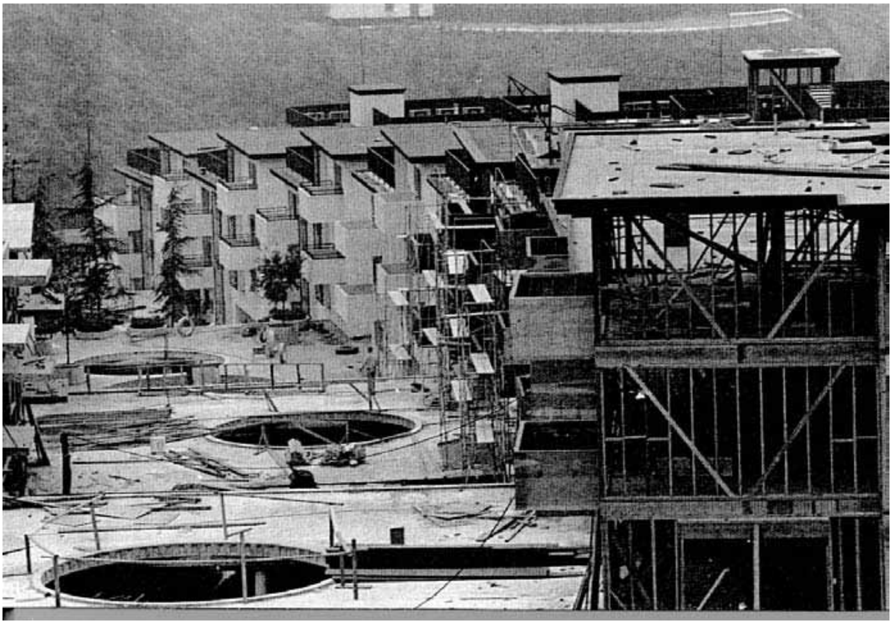

**FIGURE 6B.59** Differential settlements of several inches occurred in the 17 buildings of this apartment complex while still under construction.

Accordingly, prior to adoption of a full-scale grouting program, several holes were grouted adjacent to four selected piles. The grout holes were spaced six feet apart, and placed either in a triangular pattern of three holes or a square pattern of four. Grout was injected from the top down, in vertical stages of four feet, starting at the level of the pile tip. Between 12 and 22 cubic feet of grout was injected for each foot of depth. Following grout injection, large test borings were excavated adjacent to the four piles, and extending beyond the pile tips, allowing visual inspection. The photograph shown as Figure 6B.60, was taken from one of the borings, which was about 60 feet deep. As can be observed, grout was under the pile tip, as required, and only extended above that elevation by less than one foot.

Because this was a landmark project in the development of the compaction grouting process, extensive other testing and observation was performed. Load tests (Figure 6B.61) to a level 1.5 times the design capacity were performed on several of the grouted piles, with virtually no further settlement. Regular inspections by the structural engineer and second-order optical surveys were made for a period of 10 years following completion and occupancy of the buildings. No structural damage was found during that period, and the maximum vertical settlement observed was less than 0.1 inch.

In projects such as this, where the grout deposition zone must not extend above a given level, it is important to “seal off” the boundary by at least two stages of grout, injected from the top down. This is especially important in such instances as that cited, as injected grout will naturally tend to flow into the weaker soils. Where a considerable depth under the piles needs improvement, bottom-up procedures can be used once the upper boundaries have been established.

In another noteworthy case, friction piles were used for support of the West Orange County, California, Municipal Courts Building (Figure 6B.62). The original geotechnical investigation provided for the piles to extend from the surface to depths of 25 to 40 feet. Subsequent to the original investigation and prior to construction, the building was relocated approximately 100 feet to the west. It is

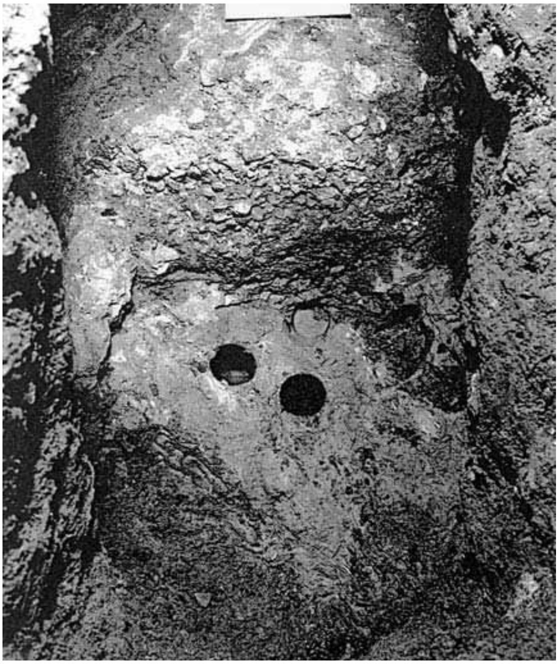

**FIGURE 6B.60** Grout under pile tip.

**FIGURE 6B.61** Load test in progress.


**FIGURE 6B.62** West Orange County Municipal Courts building underlain by faulty soil.

of contemporary design, basically a reinforced concrete frame with both concrete and masonry walls. The floor and roof slabs are of reinforced concrete. Approximately two-thirds of the structure is one-story with a slab on grade floor, the remainder consisting of two structural floors above a basement. The foundation system consists of friction piles, cast in place, in driven corrugated steel shells. At the time of construction, the groundwater level was approximately at the basement floor elevation and, in fact, an extensive underfloor drainage system was installed to prevent the development of uplift pressures.

Distress was noted shortly after completion of construction. The building nonetheless remained in full service until a major reinforced concrete roof girder fractured, eight years later. An investigation disclosed that five inches of differential settlement had occurred, and that the structure was suffering from significant structural overstress. By moving the building from the originally planned location, the west half had inadvertently been placed over a wedge of very low density peat that existed only a few feet deeper than the pile tips, as illustrated in Figure 6B.63. As part of the investigation, individual piles were test loaded in increments up to twice their design capacity. Even with supporting the imposed loads for five days, no significant deflection of the piles was observed. It was concluded that the settlement involved not only the structure and its pile foundation, but also the entire block of soil above the peat layer.

Compaction grouting was thus performed in the peat layer underlying the pile tips. Two inch I.D. casing was placed to the top of the peat layer, and grout injected in stages from the top down, until good bearing soil was encountered. Interestingly, the building remained open and in full service during the work. The work was divided into five phases, each involving one courtroom and ancillary facilities. Access was through windows in the office areas adjacent to the courtrooms. Drilling of the grout holes, which were as deep as 65 feet ( m) was by hand-held, rotary wash, equipment (Figure 6B.64), which could be quickly moved and operated throughout the otherwise restricted area. The grout pump remained outside the structure, and up to two hundred feet of hose was used to reach the furthest holes (Figure 6B.65).

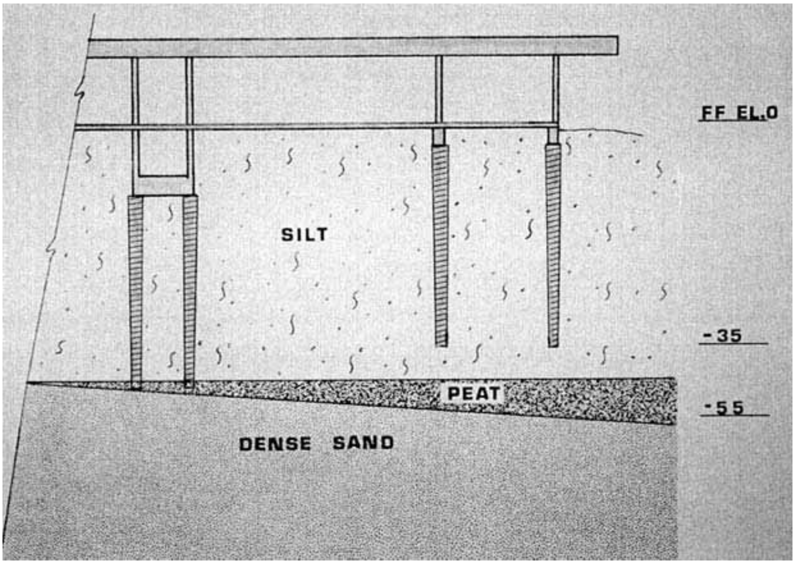


**FIGURE 6B.63** Cross section of building showing peat layer underlying pile tips.

**FIGURE 6B.64** Drilling in restricted interior area with compact hand-held drills.

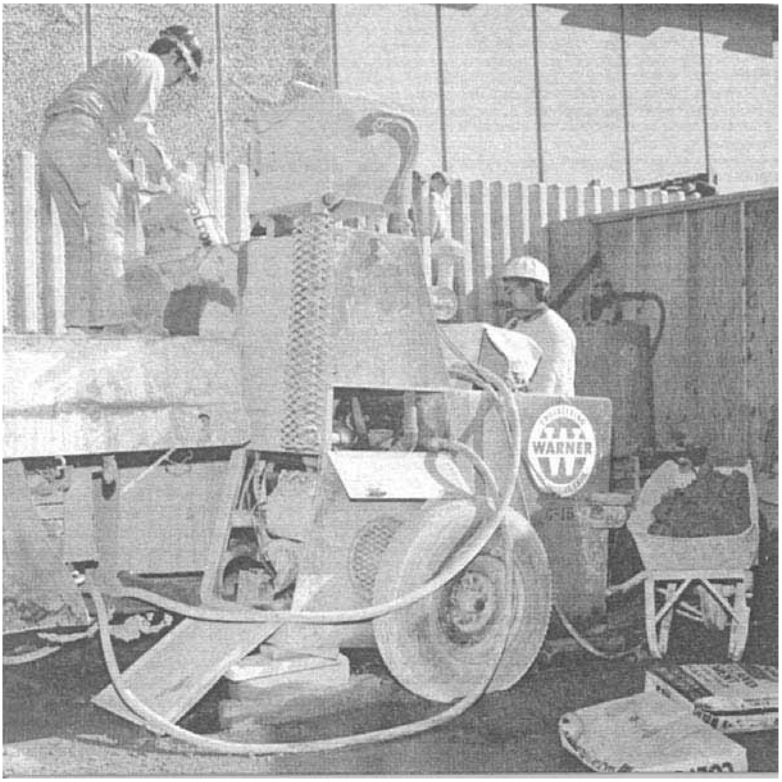


**FIGURE 6B.65** The grout pump remained outside the building.

Early in the settlement investigation, survey monuments were placed into the roof slab, over each pile location. They were monitored throughout the repair program and three, six, and nine months after completion. Although the original intent was to monitor them for several years following the repair effort, as no detectable movement occurred, further monitoring was canceled after the nine month survey. A visual inspection by the writer in 1998, 22 years after the work was performed, found no evidence of distress or the prior major settlement damage. In fact, none of the occupants had been there when the work was done, and all expressed surprise at the idea that the building once had a serious problem.

In 1978, a newly constructed, cantilevered concrete sheet pile sea wall (Figure 6B.66) began to fail as backfill was initiated. The wall was composed of pretensioned, interlocking concrete piles, twelve inches thick by four feet wide. They had been jetted into the ground, underlying the water, utilizing a barge-mounted crane. The wall was to function as a continuous cantilever structure, as illustrated in Figure 6B.67. Investigation determined that a silt layer under the sea floor had been badly disturbed during the jetting operation, along the entire length of the 11/2 mile wall.

Because the work was underwater, and any cracks or displacements that developed on the sea floor could not be readily seen, a very conservative grouting program was conducted. Grout holes were located adjacent to the existing bulkhead faces, at a spacing of 2.4 m (8 ft). Grout conforming to the Brown and Warner (1973) criteria was injected into vertical stages of 0.6 to 1.8 m (2 to 6 ft) from the top down, at a maximum rate of 0.03 $m^3$ (1 $ft^3$) per minute. Grout pressures varied from about 0.34 to 1 MPa (50 to 150 psi) but were most often within a range of 0.34 to 0.48 MPa (50 to

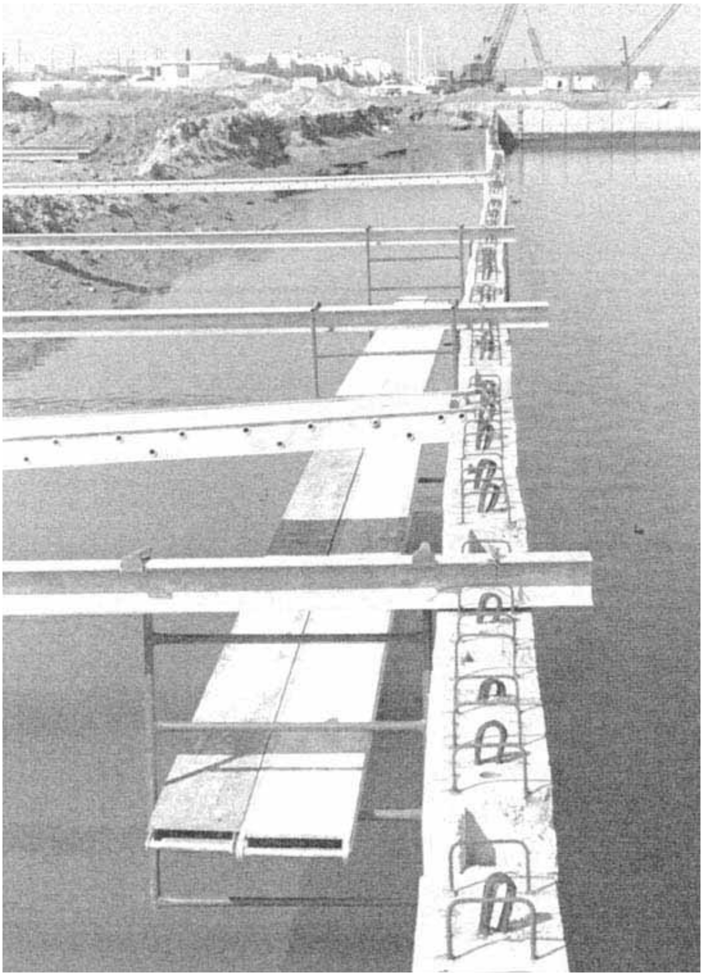

**FIGURE 6B.66** Cantilever sea wall tilted outward when backfill was initiated.

70 psi). Approximately 0.3 m$^3$ (10 cu ft) of grout was injected for each linear meter (3.0 ft) of bulkhead wall.

Two rows of cone penetrometer test probes were placed on the water side of the wall, where dense soil was required for the cantilever to function. They were made both before and after the grouting. The first row was approximately 1.5 m (5 ft) from the face of the bulkhead, with probes being made every 4.6 m (15 ft). The second row was 4.6 m (15 ft) off of the bulkhead, with probes at 7.6 m (25 ft) intervals. The work was accomplished using a 10 ton capacity penetrometer secured to a floating barge, and the total penetration force included that portion of the weight of the barge lifted by the force acting on the penetrometer rods. When the rods could no longer penetrate, the barge was lifted partly out of the water, which established refusal.

Cone tip resistance prior to grouting was less than 40 kg/cm$^2$ (41 T/ft$^2$) to a depth of 7 m (23 ft),

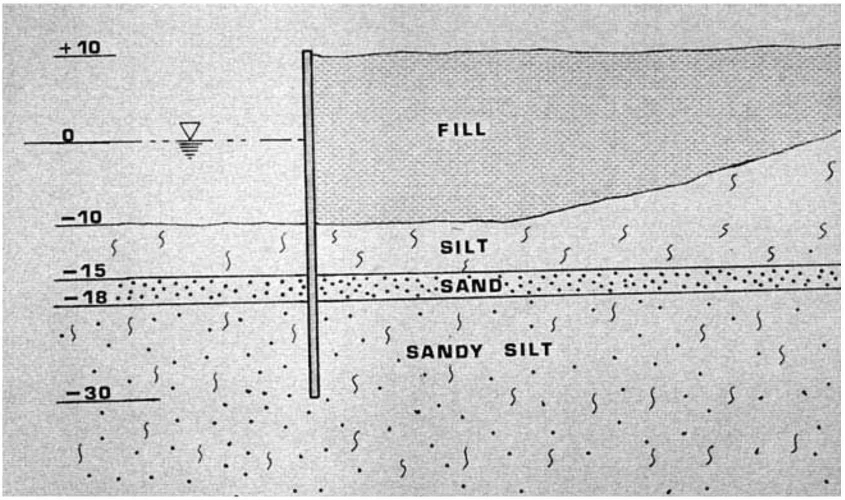


**FIGURE 6B.67** Cross section of sea wall installation as designed.

and decreased to about 20 kg/cm$^2$ (20 T/ft$^2$) below that depth in the first row. In the second row, beginning values were less than 5 kg/cm$^2$ (5 T/ ft$^2$) to a depth of 7 m (23 ft), and about 20 kg/cm$^2$ (20 T/ft$^2$) below that depth. The postgrouting value increased with depth in both rows to nearly 50 kg/cm$^2$ (51 t/ft$^2$) at about 6 m (20 ft), below which refusal was reached at around 80 kg/cm$^2$ (82 T/ft$^2$) at about 8 m (26 ft).

Compaction grouting was an especially suitable method of repair, as it did not require the use of heavy equipment at the wall location, which was surrounded by water. Grout holes were drilled with hand-held pneumatically powered drills (Figure 6B.68). The grout pump remained on the adjoining grade, with the delivery line supplying a standard header (Figure 6B.69). Extremely close control of the work was required, as deformation of the piles and resulting cracks could not be tolerated. Cracks in the piles would result in rapid corrosion of the prestressing tendons, due to the salt water environment. Load cells, anchored to land-side deadmen, were used to monitor forces on the wall. A diver was used continually during grout injection to monitor the sea bed for any displacements or grout leakage. Because of the sensitivity of underwater injection, the pumping rate was limited to less than one cubic foot per minute. Grout pressures varied between 150 and 300 psi. Following the remedial work, the wall was backfilled, and the planned construction of a commercial center progressed. The writer has visited the site several times since the work was done, and has always found it to be performing well, with no indication of the former distress.

Compaction grouting is especially well suited for the densification of backfill material that was not properly compacted when originally placed. Many projects have been completed where faulty fills behind retaining walls, or around, and over buried pipelines have been improved. Figure 6B.70 illustrates such a case, where settlement of the fill behind the basement wall of a shopping mall is being densified. As can be seen, the operation is fairly orderly, and there is relatively little interference to the shoppers, as the work is being done while the stores remain open.

Shortly after the original opening of the Japanese Pearl Divers attraction at Sea World in San Diego, California, unacceptable leakage of a man-made lagoon threatened closure of the facility. The problem was the result of insufficient compaction of backfill materials surrounding the underwater viewing areas during original construction. Compaction grouting was used to densify the faulty soils (Figure 6B.71). Most of the work was under raised decks and shops over the lagoon, with only about three feet of overhead clearance. Two inch diameter holes were drilled through the

**FIGURE 6B.68** Drilling of underwater grout holes.

wood decks to allow drilling of grout holes to proceed from above. Once the hole at a given location was completed, wood plugs were placed to restore the deck. The grout mixer and pump were located outside the park, about 600 feet distant from the work (Figure 6B.72), and the grout delivery hose line was picked up at the end of each work shift, which was at night and the early morning hours prior to opening of the attraction. With exception of the absence of water in the lagoon, there was thus no indication of a problem to the visitors of the attraction, which remained open and fully operational during the work. The project was monitored for several years following the work, with no further leakage or other distress noted.

The 1991 failure of a large sewer conduit in Houston, Texas, resulted in collapse of a road. Investigation established that the backfill around and overlying some 8000 feet of the pipe was of very low density. The silty and clayey sand backfill was found to have Standard Penetration Test "N" values averaging 12, with many below 10. Compaction grout was injected through rows of holes on each side of the pipe, at ten foot centers. Verification that the specification requirements of improvement to an average Standard Penetration Test N value of 20 with no test below 15 were easily

**FIGURE 6B.69** Grout injection header.


**FIGURE 6B.70** Grouting faulty backfill in shopping center.

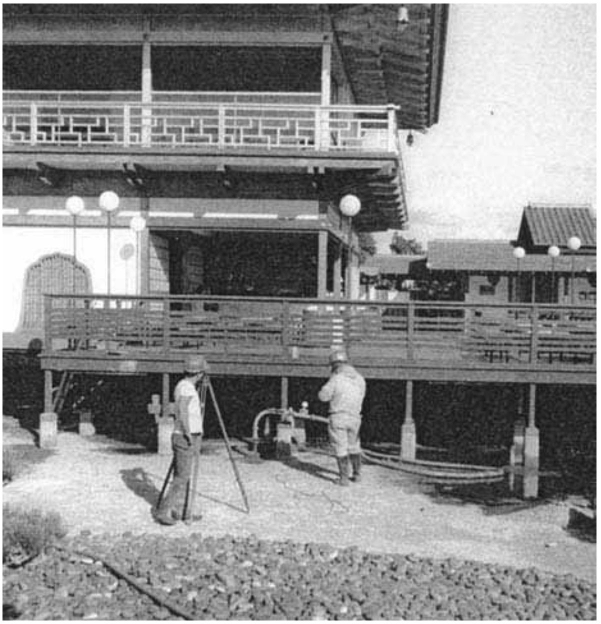

**FIGURE 6B.71** Grouting under tourist attraction with little evidence of the ongoing work visible.

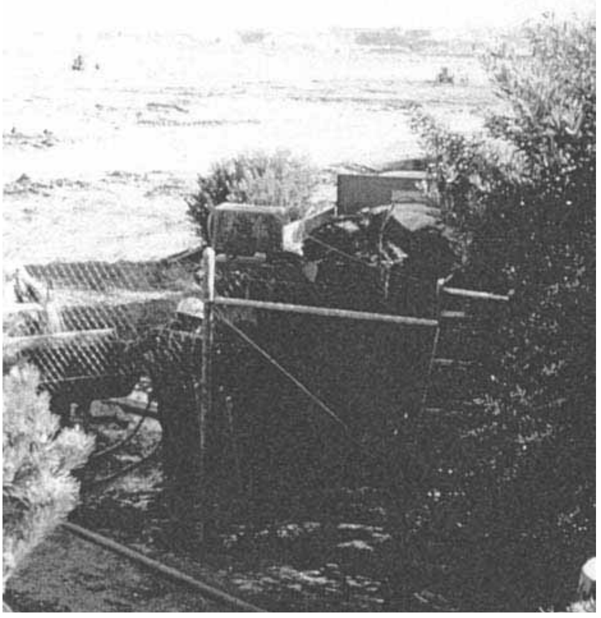

**FIGURE 6B.72** Grout pump and big equipment are out of sight.

met. One hundred thirty-four postgrouting SPT tests, made between the grout holes by the same drill rig and operator, produced N values averaging 22.3.

In another case, CPT testing was used to evaluate the appropriateness of compaction grouting to densify loose backfill soil around and underlying several miles of distressed storm drain pipe. The backfill materials were fine to medium sand and silty sand, with silt and clayey silt lenses. The drains were under the pavement of a major freeway, and had resulted in surface settlement so great that lanes had to be closed to traffic. Cone penetrometer testing disclosed cone tip resistance (Qc) of the defective soils, generally less than 30 tons/ft$^2$ (29 kg/cm$^2$). Resistance on the order of at least 50 tons/ft$^2$ (49 kg/cm$^2$) would be required to prevent settlement. The test section involved injection of 9.5 m$^3$ (318 ft$^3$) of grout into 32 injection points spaced at 2.1 m (7 ft) on center, using a maximum grout pressure of 2.8 MPa (400 psi). Details as to the aggregate gradation and pumping rate of the work, which was done on an emergency basis, are not known. The required, after grouting, minimum CPT values, of 50 tons/ft$^2$ (49 kg/cm$^2$) were easily met, as illustrated in Figure 6B.73.

As a result of the successful test application, a contract was let to improve the soil around several miles of the storm drains. Bottom plugged, two inch, proprietary, flush wall casing was driven on seven foot centers (Figure 6B.74) in rows on each side of the drains. Prior to injection, the casing was raised about one foot and the plug knocked out. Grout was then pumped at a rate of less than 0.04 m$^3$ (1.5 ft$^3$) per minute. As in the previous example, CPT tests were taken both before and after the grout injection. The required degree of soil improvement was easily achieved.

Compaction grouting is often used to raise the density of granular soils, for the purpose of mitigating the risk of liquefaction during earthquakes. In 1983, compaction grouting was used to improve the site for an addition to an existing pile-supported hospital building. Because the work was immediately adjacent to the existing structure, which remained fully operational, excessive noise or

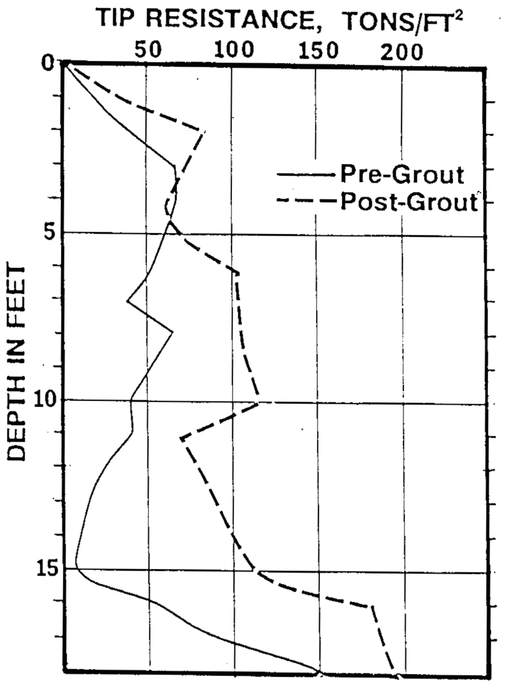


**FIGURE 6B.73** CPT values before and after grouting of faulty trench backfill.

**FIGURE 6B.74** Driving grout injection casing.

vibration could not be tolerated. Of the several candidate methods evaluated, compaction grouting was the only one that met these requirements. The upper level of the soils from 7 to 17 feet in depth was first grouted. Then the remaining soils were treated from the bottom up, starting at the bottom of the liquefiable sands at a depth of 34 feet.

Prior to construction of the addition, grout holes were drilled on a square grid over the entire site (Figure 6B.75). The holes were spaced eight feet apart, and average grout take was 2.7 cubic feet of grout for each foot of hole. The grout was composed of silty sand, cement, and water mixed to a stiff consistency, with slump between 25 to 50 mm (1 to 2 in). Postinjection testing indicated an increase in the density of the in-place soils of about 20%. The site was in an area that experienced widespread damage from the Loma Prieda earthquake in 1989. The hospital site performed well and no indication of liquefaction was evident.

In 1988, compaction grouting was used to densify foundation soils underlying Chessman Dam, near Helena, Montana. This was the first of many similar projects in which compaction grouting has been employed to upgrade the seismic resistance of dams and other large civil works. In 1999, a massive project was started to improve the soil underlying the Narita airport, which serves Tokyo, Japan, in order to mitigate the risk of liquefaction. That effort will involve one of the largest undertakings to date, in terms of amount of grout used and volume of soil treated.

Another area, in which compaction grouting has been found advantageous, is prevention of surface damage as a result of soft ground tunneling. A phenomenon is well understood wherein a trough of soil overlying such tunnels occurs as the tunnel shield passes. This zone of disturbed soil starts directly over the tunnel and extends up and out until it reaches the surface. It will often extend beyond the foundations of adjacent structures, and can be very damaging, not only to the structures, but also to underground pipes and utilities within the zone of disturbance.

In 1980, a large evaluation program was conducted in the early stages of construction of the Bolton Hills tunnel, part of the Baltimore Metro project. A test application was conducted in a section of the actual tunnel where the adjacent structures were of minimal value. Compaction grouting

**FIGURE 6B.75** Small track-mounted rig used for drilling grout holes.

was performed immediately over the tunnel shield as it progressed. This was accomplished through a single row of grout holes, which were drilled from the surface of the street above and extended to within about five feet of the crown of the tunnel (Figure 6B.76). A single stage of grout was injected immediately after the shield passed, in the tunnel below. The trial, which included extensive instrumentation and evaluation of the adjacent soils, was so successful that the method was adopted for protection of the historical and irreplaceable structures on the remainder of the tunnel route.

Since that early experience, compaction grouting has been used in a similar manner on a number of other significant tunnels constructed in both North and South America. It was also specified for use in construction of the Taipei Metro, in Taipei, Taiwan, Republic of China.

Another extensive use of compaction grouting is in connection with distressed buried pipelines. In 1976, a 96 inch drainage conduit, which was badly overstressed and deformed as a result of excessive loading, was repaired with compaction grouting. Because access to the overlying area was not available, all work was done from within. Twelve foot long, radial holes were drilled with handheld equipment from within the conduit (Figure 6B.77). Grout was then pumped in stages of about one foot, from the bottom out. Excessive deformations of the pipe were removed during the grout injection.

In 1977, the heading of a flexible liner plate tunnel, which was under construction, began to sink uncontrollably. Compaction grout was injected to stabilize a highly organic silt layer found to extend from the invert of the tunnel, a depth of about 12 feet. Once the faulty ground was sufficiently improved, the heading of the tunnel was grout jacked back to its proper elevation. All of the work was done from within the tunnel (Figure 6B.78), as no disturbance of traffic on the major highway above was allowed.

In 1991, the process was used to stabilize defective soil underlying an 84 inch diameter conduit, connecting to a water treatment plant in Des Moines, Iowa. The plant had been underpinned with micro-piles; however, such piles could not be readily tied to the round pipe section. The

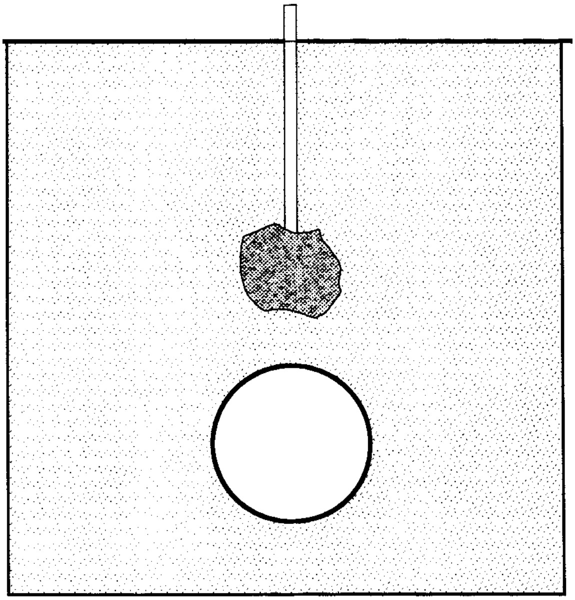

**FIGURE 6B.76** Injection of grout to densify soils as they loosen.

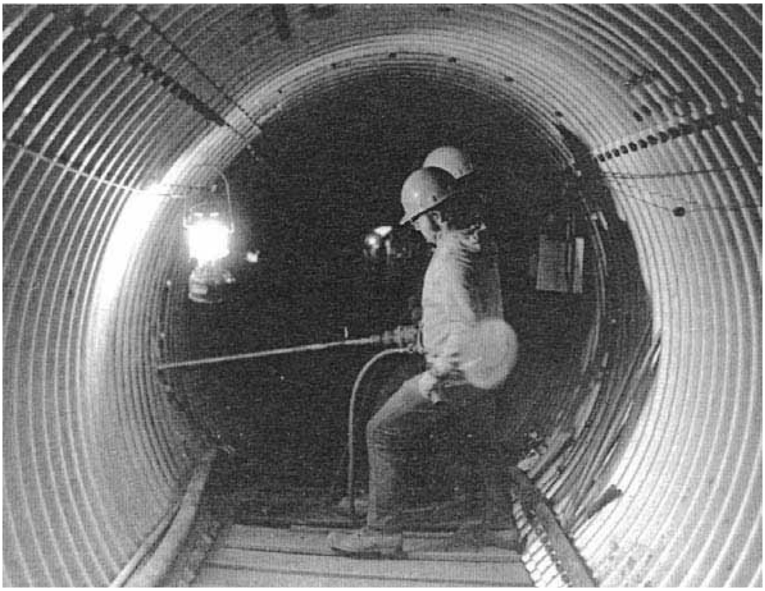

**FIGURE 6B.77** Drilling of 12 foot long radial grout holes from inside drainage conduit.

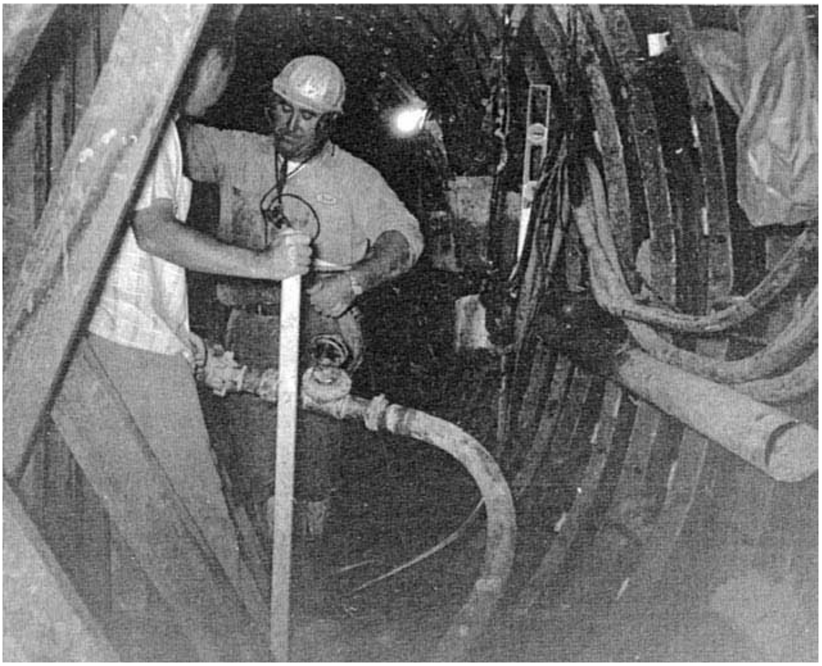

**FIGURE 6B.78** Settlement correction to heading of liner plate tunnel.

grouting was used to not only stabilize the underlying soil, but to jack the pipe up to its correct level as well.

Perhaps the most sensitive of any geotechnical remedial applications involves work in water-retaining embankments. Should excessive pore pressures develop, uplift leading to complete failure of the embankment could occur. Obviously, should hydraulic fracturing occur, leakage through the embankment would present an extreme risk, which could also lead to complete failure. Because compaction grouting allows for close control of the grout deposition location, when properly performed, it is one of the safest remedial procedures for correcting defects in such embankments. In order to achieve the maximum assurance against creating hydraulic fracturing, careful design and rigid control of the grout mix to be used is of crucial importance. In this regard, a minimum of 25% of the grout aggregate should be larger than a number 4 sieve. Additionally, the aggregate should be completely free of any clay component.

In 1996, a sinkhole developed in the core of Bennett Dam (Figure 6B.79), which is in the northern part of British Columbia, Canada. Bennett is one of the largest embankment dams in the world. Extensive investigation determined the disturbed zone to be generally less than 20 feet across, but about 400 feet deep. The investigation also uncovered a second sinkhole of somewhat less magnitude. After much deliberation, compaction grouting was selected as the best method for remediation of the defect.

This was a critical problem and warranted the very best design and remedial implementation. Extensive investigation was made to accurately map the sinkholes. To monitor any changes of condition within the embankment or surface profile, an extensive installation of instrumentation was installed. Requirements for continuous computer monitoring (Figure 6B.80) of the parameters of both the drilling and the grout injection were adopted. Continuous analysis of the retrieved data was made, along with appropriate adjustments in the conduct of the work.

**FIGURE 6B.79** Bennett dam in British Columbia, Canada, one of the worlds largest earth embankment dams.

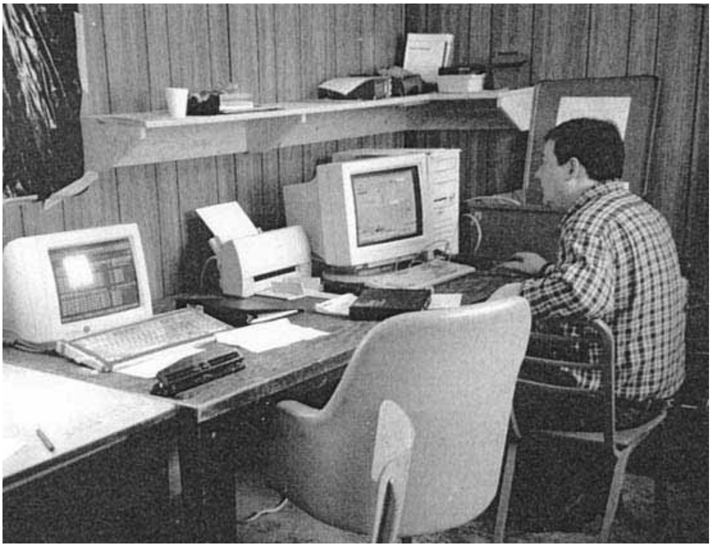

**FIGURE 6B.80** Continuous real time computer monitoring.

Compaction grouting had not been previously accomplished to such depths or under such sensitive conditions, and huge losses would result if the embankment were breeched during the repair. Additionally, the work was to be done in severe cold weather, during the winter months. This would require protective, heated enclosures (Figure 6B.81). Thus, a full-scale test hole was drilled and injected, at a site of similar soil materials in Vancouver, prior to any work being accomplished on the dam itself. Among the objectives of the test hole were to assure that the proposed drilling and grouting methods, which were to be continuously electronically monitored, could meet the specification requirements. The hole was required to be within 1% percent of verticality over its entire length. The drilling method also had to prevent any positive liquid head from being imposed within the embankment.

Further objectives of the test hole were to confirm pumpability of the proposed grout mix, which was to contain a minimum of 25% of aggregate retained on a number 4 sieve. The aggregate was also to be essentially free of clay. Creation of a hydraulic fracture signature, by intentionally pumping too fast and/or adding clay to the grout, was also an important objective. This would allow the personnel who would be monitoring the actual work on the dam to become more familiar with areas deserving special alertness. All objectives of the test hole were realized and additional shortcomings of the contractor's operation were identified. Although it represented a considerable cost, the test proved to be extremely valuable to the overall operation, and is credited with enabling the successful completion of the work.

To accurately drill the grout holes, a powerful dual rotary drill rig, such as might be used for water well drilling, was used (Figure 6B.82). It was capable of simultaneously turning both the drill string and the casing in opposing directions. A heavy 6 inch steel casing, was used, and the joints were welded to assure tightness and proper alignment. The completed holes were checked for verti-

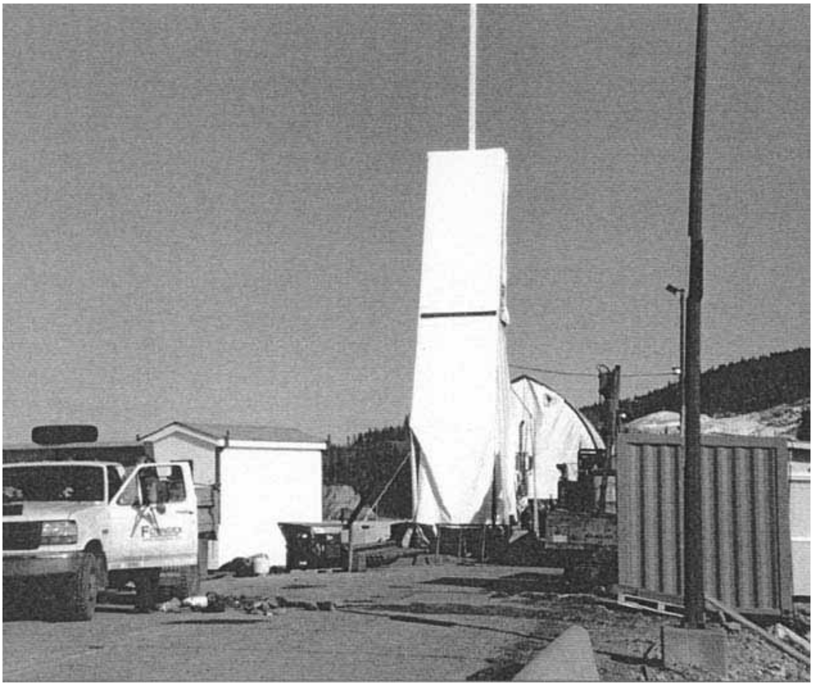

**FIGURE 6B.81** Cold weather enclosures.

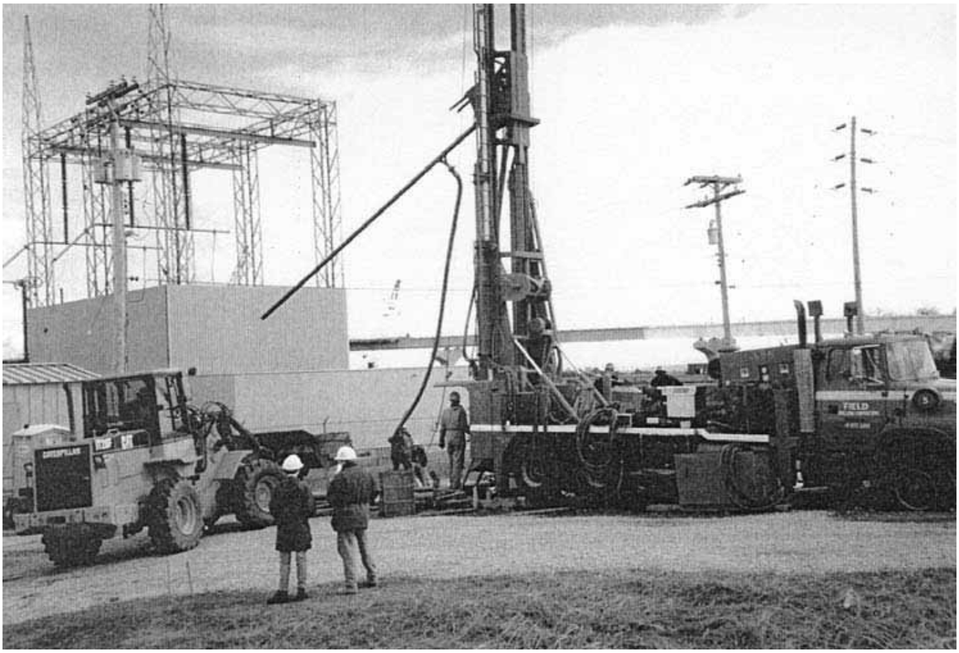

**FIGURE 6B.82** Powerful rotary wash drill rig handled casing easily.

cality with a gyro instrument, and found to be well within the required 1% tolerance. The contractor had originally proposed to inject the grout directly through the 6 inch casing, however it was found that the grout would exert a positive pressure of about 0.3 psi for each vertical foot. This could result in unacceptable pressures at the bottom of the hole, so a smaller HWL 4 inch ID, flush wall casing was inserted and sealed off at the bottom of the hole with a packer. With the smaller casing, a negative pressure of about 0.3 psi per foot was experienced.

Analysis indicated that in order to preclude excessively high pore pressures developing in the embankment, as very slow pumping rate of less than 0.5 cubic feet (14 L) of grout per minute, was required. The grout was injected from the bottom up, in stages of one foot (0.3 m). The 6 inch casing and the contained HWL grout tube were pulled by the drill rig for each stage. As each 20 foot increment was withdrawn, the 6 inch casing was cut with a pipe cutter, the threaded HWL casing joint broken, and the header rejoined by welding for the next pull. As expected, the grout take was quite variable, due to the variation of the sinkhole boundaries. Required grout pressure for these very deep holes was on the order of 1400 psi (9660 kPa).

An extensive array of piezometers was installed around the sinkholes prior to grout injection, and pore pressures closely monitored throughout the work. Occasional excessive pressure did require a pause of grout injection in the early stages of the work. This was the result of the contractor exceeding the specified pumping rate. Once he obtained a proper grout pump, and his crew gained experience, the injection proceeded at a rate of about 0.33 cubic feet per minute (8.6 L/min), without further problems. As part of the final remedial work, the soil cap material was excavated down to the top of the injected grout. The grout was found to exist in a perfect columnar mass, as expected (Figure 6B.83). The success of this project proved the engineering validity of a properly designed and controlled compaction grouting program.

Another interesting application of compaction grouting in a water-retaining embankment was

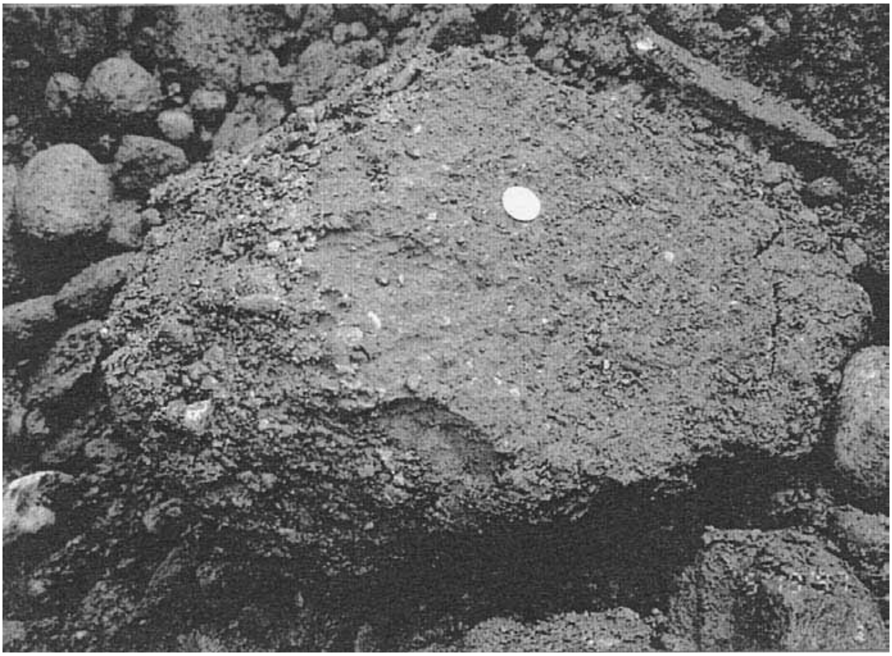

**FIGURE 6B.83** Near-perfect columnar mass of grout exposed by excavation of overburden.

the emergency repair at mile 55 of the California Aqueduct (Figure 6B.84). Massive leakage was observed at the base of a substantial embankment (Figure 6B.85), and complete failure appeared imminent. A sinkhole had developed about 300 feet upstream of the leak, and a section of the concrete lining had fallen into it.

The massive leakage was initially controlled by rapid insertion of about 80 burlap-encased, 50 pound paper bags of bentonite, followed by the rapid pumping of 10 cubic yards of ready-mixed concrete into the sinkhole. The concrete had about 30% minus ½ inch gravel and contained 940 pounds of cement per cubic yard. It would have been very advantageous to include an antiwashout admixture; however, this material was not immediately available from the only concrete plant in the area of the work. Although this work was done on a panic basis, and with little control, it was successful in substantially reducing the leakage. Because it was likely that the blockage of flow was in a cavernous void near the sinkhole, and substantial piping voids probably remained in the embankment, further work was required to assure against possible failure of the embankment

A method to "find" any such voids, and fill them, was needed. Compaction grouting was selected as the safest method, because it would provide for the greatest control of the grout deposition area and, properly performed, would not result in hydraulic fracturing of the embankment. This was a particular concern, because of the bagged bentonite, which had been inserted during the emergency. Neat bentonite gel is known to act as a fluid in soil, and would likely initiate a hydraulic fracture if subject to a sufficient pressure, which could result from grout injection. A very carefully controlled compaction grouting program was thus adopted.

Initially, a single row of grout holes, spaced at 16 foot increments, was established adjacent to the top of the channel for the 300 feet segment. Following injection of these primary holes, secondary holes were placed midway between, for a final spacing of 8 feet. Once this first row of holes was completed, four additional parallel rows were established across the embankment. These rows were

**FIGURE 6B.84** Mile 55 of the California Aqueduct during the emergency grouting repairs.

**FIGURE 6B.85** Leakage was at the base of the embankment.

about 8 feet apart, with holes located at a spacing of 8 feet. They were grouted using split spacing, with a primary–secondary order of injection.

Grout was prepared using aggregate of the preferable gradation, containing about 25% gravel and about 10% cement. It was injected from the bottom up, in one foot stages, at a conservatively low rate of one-half cubic foot per minute. Because of the sensitivity of the work, which was done with the aqueduct in continuous operation, continuous monitoring of the embankment was carried out. In addition to careful observation and recording of the grout injection parameters, this monitoring included a manometer level control system and continual observation of the ground surface and exposed top portion of the canal lining by optical survey, which is evident in Figure 6B.86, showing the work in progress.

Because of the emergency conditions, the grouting crews arrived within one day of discovery of the problem. They worked 24 hours a day until the work was essentially completed. The rapid mobilization precluded continuous computer monitoring, which would have been preferred. Very carefully made manual records were kept, however, which were entered into a computer database immediately upon completion of injection of each hole. Graphical printouts thereof were assembled on the wall of the trailer office (Figure 6B.87), so that the actual conditions as dictated by the grout injection behavior could be easily observed. The work was a success in that it allowed the aqueduct to function for several months until a scheduled shutdown. A permanent repair consisting of a continuous membrane protected by a shotcrete overlay was then made.

Concurrently with the start of grouting, a subsurface geological investigation, which eventually included 12 carefully logged bore holes and one test pit, was made. It was determined that the embankment had been founded on soil materials underlain by a gypsiferous formation. The piping leakage was determined to most likely be the result of solution of the gypsiferous materials from water permeating through an overlying sand stratum, which was under the embankment fill. This

**FIGURE 6B.86** Remedial grouting in progress. Note survey activity to ensure against any surface heave or other movement.

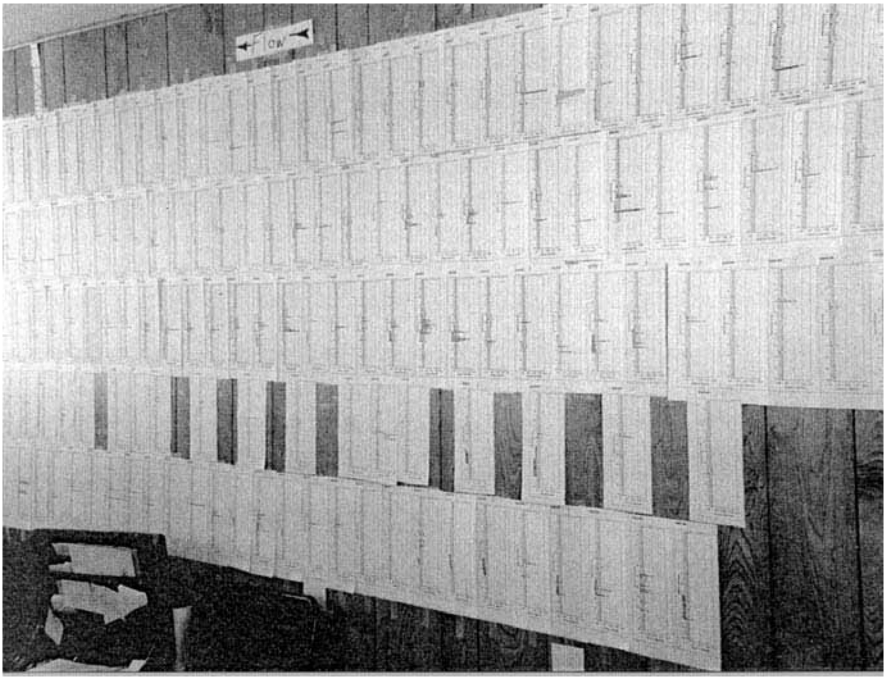

**FIGURE 6B.87** Posted records facilitated analysis and evaluation.

conclusion was supported by the results of the grouting program. Therein, the majority of the grout was injected at depths greater than those of the fill and extended throughout a plan area much greater than could be reasonablly represented by typical piping leakage. A total of 4523 cubic feet (125 $m^3$) of grout was injected into 281 holes extending through the embankment. Careful records of grout take and behavior were kept and promptly reviewed. They confirmed that the voiding was much more extensive than originally thought and that, indeed, it was in the original soils underlying the embankment itself. Figure 6B.87 shows the records for the individual holes, assembled in a collage, on the wall of the office trailer. Thereon, the grout take is indicated by a horizontal bar at each respective depth, such that the actual configuration of the voiding can be readily visualized.

As a final note, *compaction grouting need not be messy*. Figure 6B.88 shows a grout hole casing, within inches of sensitive laboratory equipment in the Spalding Research Laboratory at the California Institute of Technology in Pasadena, California. With reasonable care (note that the equipment is wrapped in plastic sheeting), compaction grouting can be successfully conducted under even the most sensitive environments.

## 6B.6.3 PERMEATION GROUTING

Permeation grouting is the longest established and most widely used grouting technique. In soils, the procedure involves permeating and filling the soil pore spaces, without any significant disturbance to, or movement of, the individual soil grains. The structure and dimensions of the soil pore spaces dictate the type of grout that can be effectively used. The particles of most fluid suspension type grouts containing ordinary portland cement and are too large to penetrate any but the coarsest

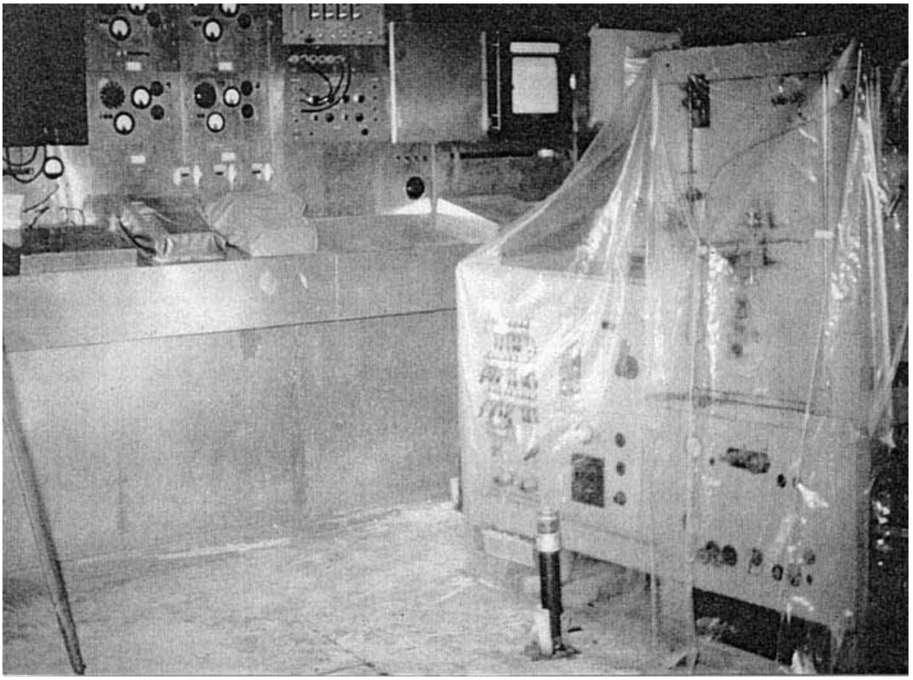

**FIGURE 6B.88** Work around sensitive equipment in a research laboratory.

soil materials, limiting their use to reasonably clean coarse sands and gravels. Accordingly, permeation grouting of most soil is performed with either highly penetrable chemical solutions or ultrafine cement grouts. Outside the United States, principally in Europe, clay, combinations of clay and cement, or chemical solution grouts have also been extensively used.

This type of grouting is generally limited to sands and sandy soils containing minor amounts of finer particles. Whereas suitable grouts can be injected into fine sands and silts, slower injection rates are required than in the more permeable coarser-grained materials. The slower injection adds to both the required time for placement and cost of the work, which sometimes preclude its use. Permeation grouting of soil is generally more expensive than is compaction grouting. Its use is thus commonly limited to applications where a considerable increase in the *cohesion*, rather than the density, of the granular material is required, or in clean, coarse materials, which are not appropriate for compaction grouting.

The largest single use of permeation grouting is for temporary solidification of low-cohesion sandy soils as an aid to construction, or to reduce or eliminate soil disturbance or failure resulting from excavation. Figure 6B.89, illustrates such a case; here a self-supporting buttress of solidified soil enabled a nearly shear cut to be made for the basement of a new building. Permeation grouting is also seeing ever-increasing use for mitigation of the liquefaction potential of in situ soils as a result of earthquakes. The procedure is also occasionally used to permanently increase the load-bearing capacity of appropriate soils.

Another significant area of use for permeation grouting is for reduction of permeability or control of the flow of water through a soil formation. Both ultrafine cement and chemical solution grouts are so used. Fine fillers, such as silica fume and ultrafine cement, are sometimes mixed into solution grouts to increase their strength, alter their properties, or lessen their cost. Expanding

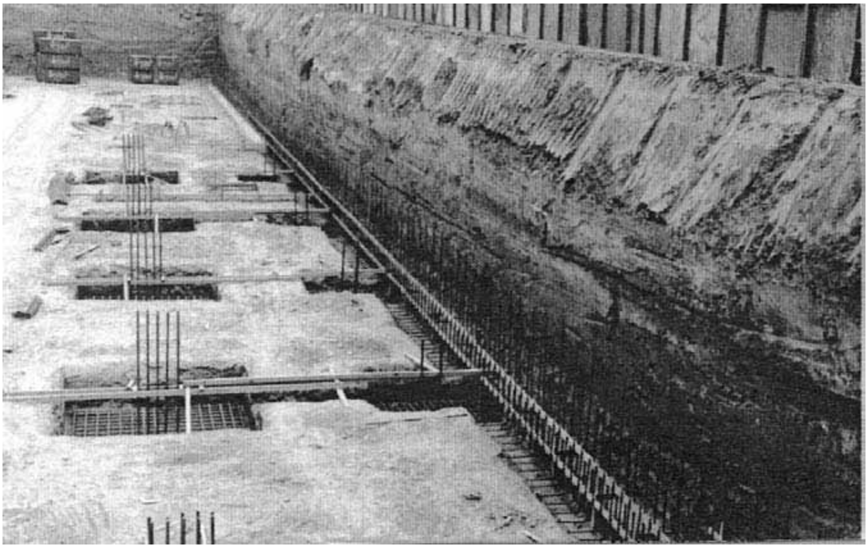

**FIGURE 6B.89** Solidification of in situ soils into a self-supporting buttress negated the need for any other support of this excavation in wind-deposited, cohesionless fine sands.

knowledge, more reliable and durable strength, and increasing availability of ultrafine cements have resulted in their increased use for permeation grouting, and they are now preferred over the more traditional chemical solution grouts by many professionals.

#### *6B.6.3.1 Fluid Suspensions for Strengthening*

Fluid suspension grouts were the first grouts to ever be used and continue to be widely employed. The largest area of use for these grouts is in the filling of cracks and joints in rock, although their use has been extended greatly and includes controlled injection into soil. In the simplest form, a suspension of ordinary portland cement and water is mixed. High-shear mixing is required to thoroughly disperse the cement particles, and continuous agitation of the mixed grout is required to maintain the individual grains in suspension.

Grout mix design is expressed as the ratio of water to cementitious solids, either by weight or by volume. Volume measurements have been the most often used traditionally, as they were very easy to make when using bagged cement that contains one cubic foot of dry powder. Where bulk cement is used, proportioning by weight is more convenient. In the United States, grout mixtures as thin as 10:1 by volume have been extensively used historically. Many, including the writer, consider such thin grouts to be little more than dirty water, and extensive testing and research by the author and others (Houlsby, 1982; Weaver, 1991) have shown such thin mixtures to provide little durability. Current thinking suggests that thicker mixtures (maximum W/C 3:1 by volume) are far more appropriate for most applications. In a well-documented discussion of appropriate water to cement ratios, Houlsby (1982) concluded that W/C 2:1 by volume has been used on thousands of projects and was optimal for general use.

Although unmodified common cement–water suspensions continue to be widely used, they suffer from settlement of the solids, resulting in accumulation of mix water at the top of unagitated grout. This is known as bleed and is an important factor to consider in any grout formulation. In this regard, the amount of shear subjected during the mixing of the grout has an important bearing upon the resulting bleed. When cement and water come together, the cement grains tend to clump or floc together. To minimize the amount of bleed of a particular grout, high-shear mixing is required so as to break up and separate the individual cement grains. This factor was thoroughly discussed by Kravetz (1959). The essence of his conclusions, as reported by Houlsby, (1990) are:

1. Cement grains, when mixed with water, tend to aggregate and form clumps. This slows the wetting process, as does air attached to the grains. The effect of high-speed shearing or laminating, plus the centrifugal effect, is to thoroughly break up the clumps and to separate air bubbles. As a result, each individual grain is rapidly and thoroughly wetted and put into suspension.
2. During cement hydration, needle-like or spring-like elements of hydrates form on the superficial layer of each wetted grain of cement. In a high-speed mixer, the laminating effect and high-speed rotation keeps breaking these hydrates away from the grain of cement, thus exposing new areas to the water and consequently bringing the formation of new elements. These hydrate elements are of colloidal size, and as the amount of these elements in the mixture increases, the grout becomes colloidal in character.

In reality, virtually no common cementitious suspension grout is truly colloidal, as even well-dispersed cement grains will settle in water unless special admixtures are used, as will be subsequently discussed. High-shear mixing will greatly reduce and can in thick grouts nearly eliminate bleeding. In some places, but generally not in the United States, grout that exhibits little or no bleed is regarded as an "stable." Stable grouts are generally defined as those that exhibit a total of no more than 5% bleed. European practice commonly calls for "stable" grouts, which typically include a few percent of bentonite, to act as a suspension agent. Whereas bentonite inclusion will tend to lessen the cement grain settlement and resulting bleed, there are significant disadvantages to its use. Modern admixture technology offers a variety of formulations that can minimize bleed without the undesirable aspects of admixed bentonite. This will be subsequently discussed in more detail.

Suspension grouts of ultrafine cement and water are significantly more penetrating than those made with Type 3 cement, which is the finest of the common cements, and some ultrafines can form nearly colloidal mixes. These can be readily injected into sandy soils, as illustrated in Figure 6B.90. Because of the minute grain size of ultrafine cements, they provide very large specific surface areas. Inclusion of a high-range, water-reducing admixture in these grouts is virtually required, and will result in much better dispersal of the individual cement particles. This will substantially reduce the propensity for pressure filtration in the soil. It will also dramatically increase the specific surface area of the cement available for the water to contact, resulting in much increased hydration activity, and higher ultimate strength.

There are two main types of ultrafine cement—those based on slag and those based on portland cement. Although the two are comparable in many respects, there is a fundamental difference in the span of hydration and the resulting setting time. Because of the very high specific surface area of ultrafine cement based on portland cement, hydration activity is very high, resulting in rapid setting and strength gain. For this reason, retarding admixtures are sometimes included in the cement to delay the otherwise rapid setting. Portland cement combined with some common retarders can be extremely sensitive to temperature, so the proportions of the additive must be accurately dosed and appropriately matched to the temperature of both the injection environment and the formation being grouted. Problems with flash set and difficulties with both setting time and rate of strength gain have been reported when using some of the earlier ultrafine cements based on portland cement.

Cements based on slag tend to set and gain strength much slower than their portland cement cousins. Thus, retardation is seldom a problem, but accelerating admixtures may be required in some instances. Well-established accelerators are available and their behavior is quite predictable, so time of setting is not much of a problem with slag-based ultrafines. It is only in recent times that ultrafine cements have become commonly available. With their ever-increasing use and accumulated field experience, continuous improvement in the available products is occurring, and continual monitoring is required to keep up with the latest technology.

Most ultrafine cements are composed of a range of differently sized particles. Providing they are

**FIGURE 6B.90** Ultrafine cement grout used to solidify cohesionless sand allows excavation under the foundations of a structure.

well dispersed, some larger particles are acceptable for grouts that will be used to fill obvious fractures and voids, as long as the largest particles are smaller than the thickness of the defect. In the permeation of granular soil, however, because the soil itself acts as a filter, the larger-sized particles of the grout can build up as they are pushed into the soil, initiating the formation of a filter cake. The size, and the proportion, of the larger grains of ultrafine cement used for permeation grouting of soil is thus of great importance. Obviously, use of a cement with the smallest mean particle size and a minimal amount of larger particles is desirable for soil grouting. The particle size distribution of several different ultrafine cements as well as Type III common cement are shown in Figure 6B.91.

It should be noted that not all type III cement has the same grain size distribution. ASTM C 150 "Specification for Portland Cement" calls only for Blaine fineness of the specific surface area. A compliant Blaine value can be obtained with a fairly wide range of grain size distribution. The Type III curve provided in Figure 6B.91 is for an actual batch of cement produced at one given plant.

### *6B.6.3.2 Chemical Solution Grouts for Strengthening*

Another form of strengthening is adhesion of the individual soil grains with a chemical solution grout. There are a variety of chemical solution grouts available, but they all work in essentially the same manner. A base material, which usually represents the majority of the final mix and is often diluted with water, is combined with one or more reactant materials. The base part is usually fluid, whereas the reactants can be either liquid or powder depending upon the individual grout formula-

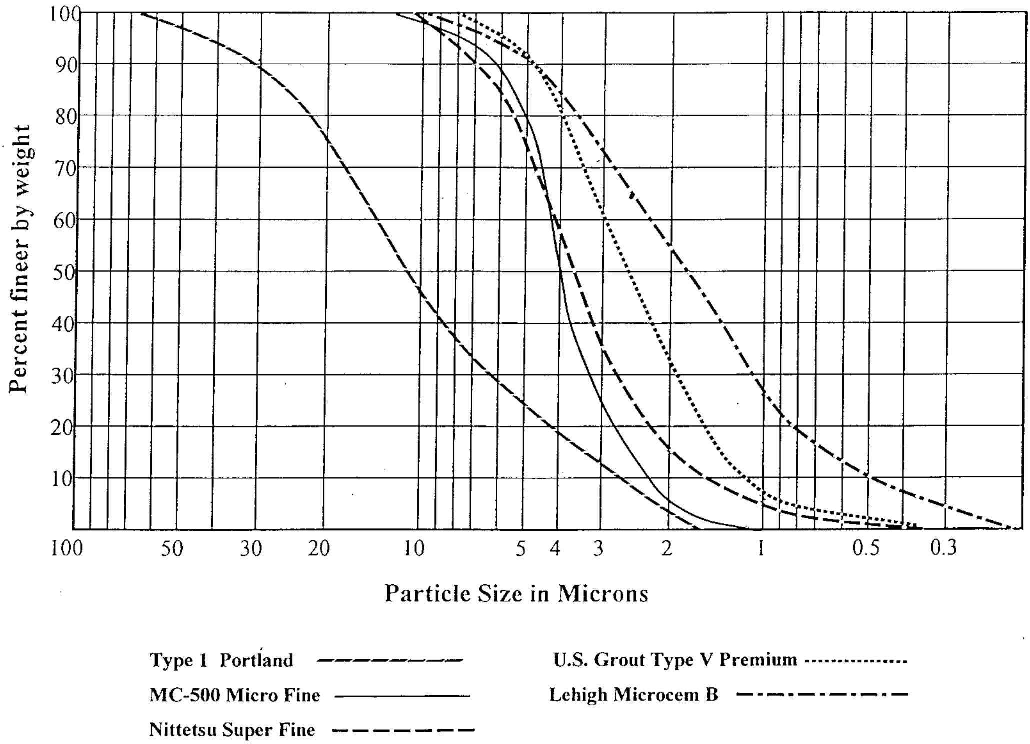


**FIGURE 6B.91** Grain size distribution for various cements.

tion. At some period of time, after mixing the base and reactants together, setting occurs and the injected solution turns either to a foam, gel, or solid state. The setting time can be instantaneous to several hours or even days, depending upon the individual system used and the proportion of the individual ingredients thereof.

Many chemical grouts are mixed and pumped as single solutions, whereas the different components of others are individually pumped and stream mixed at the injection point (Figure 6B.92). There are many manufacturers of proprietary chemical grout formulations, especially those used for control of water movement within the soil or into underground substructures. For strengthening applications, especially where large quantities of grout are required, most specialty contractors tend to purchase the individual chemical components separately and mix them on the job site near the point of injection. Although some specialists represent their formulations as something very special and "proprietary" (a word greatly overused, in the writers opinion), the vast majority of grouts used for strengthening of soil throughout the world involve sodium silicate as the base material. There are, however, a wide variety of different reactant systems, most of which are well known and readily available.

For the strengthening of soils, the most important parameters for selection of a chemical solution grout are injectability, strength, safety of use, and cost. Hard, rigid gels are desired, as they provide the greatest stiffness as well as providing good strength. Cost can also be an important factor because this type of work is often performed on a massive basis, where very large quantities of grout are involved. With these factors in mind, grouts based on sodium silicate are particularly suitable and most frequently used for strengthening applications. Appropriately, the technical literature is

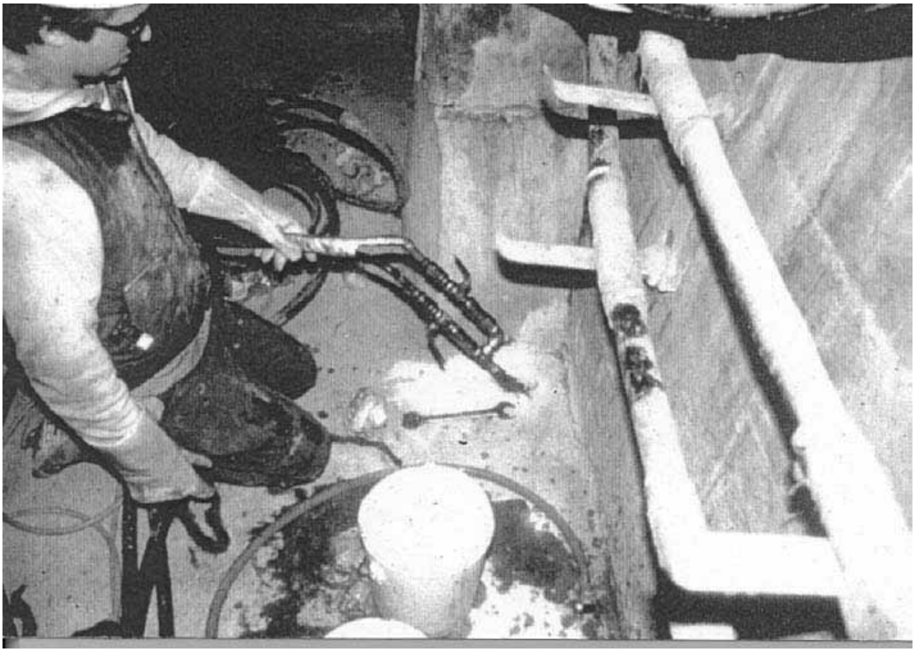

**FIGURE 6B.92** Two-component water-control grout being stream mixed at the injection point.

filled with publications relative to the use of sodium silicate base grouts. Unfortunately, much of it is quite academic, and some downright misleading.

As an example, several publications present strength data for "sodium silicate grouts" but make no reference whatever to the reactant system. While sodium silicate may represent the largest component of a chemical solution grout, it requires a reactant component to harden. There are many different reactant systems, which may consist of one or more components, as will be shortly discussed, and the properties of the resulting grouts will vary greatly with the different systems. The combination of sodium silicate with some reactant systems results in a grout that will break down and lose strength with time, whereas other mix combinations have proven to be durable over long periods of time. The only available guide for the selection of chemical solution grouts for strengthening soil is ASTM D 4219, "Standard Test Method for Unconfined Compressive Strength Index of Chemical-Grouted Soils." The preamble of this document states that it applies only to the "short-term" strength of the grouted mass. No time definition of short-term is given, but at best, grouts conforming to the standard should be considered for temporary strengthening only.

Graf, Clough, and Warner (1982) reported on the long-term aging effect of chemically grouted soil specimens that were aged from 9 to 11 years. They concluded that "Except for the very weakest samples, the strengths of the stabilized soils tested with no environmental change, showed no change or a modest increase over those measured after one to two years aging." The specimens on which they based their conclusions were composed of either 50% or 60% sodium silicate and reactant systems that are still available but not frequently used today. The reactant systems most frequently now used with sodium silicate, consist of dibasic or tribasic esters. There are no long-term data available as to the permanence of grout made with these reactants.

The strength of the sodium silicate concentration will also affect the final properties of the grout, including long-term durability. Some publications fail to recognize this crucial factor or in some cases report incorrectly that sufficiently strong solutions either cannot be injected or cannot obtain a satisfactory hardening time with a given reactant system. Bad results have been experienced in several instances where strongly diluted solutions were used. This notwithstanding, such poor results are virtually always the result of faulty mix design, and not indicative of the behavior of properly designed sodium silicate formulations. There are many factors that influence the long-term strength and durability of chemical solution grouted masses and these will be dealt with subsequently.

### *6B.6.3.3 Grouts for Water Control*

Historically, cementitious suspension grouts have been used to stop the flow of water through the cracks and joints of rock. The pore size of most soils, however, is significantly smaller than the width of typical rock joints, and except in the case of coarse sands and gravels, it is insufficient for intrusion of a common cementitious, particulate grout. Thus, water control in soils is almost always accomplished with chemical solution grouts or suspensions based on ultrafine cement.

Whereas hard, rigid chemical gels are preferable for strengthening applications, some flexibility is usually desirable in water control work, and thus different chemical grout systems are in order. Dilution of the grout with groundwater is always a concern, and where rapid flow of the groundwater occurs, the grout must be resistant to being washed away as well. This usually means use of a rapidly setting grout. A variety of materials that can provide instantaneous setting are widely available. These grouts are usually more chemically complex than those used for strengthening of soils, and some formulations present health risks and require special handling. They are thus usually obtained as proprietary systems from specialty grout producers.

With the increasing availability of ultrafine cements, many of which are able to readily penetrate the pores of even the finest sands, suspension grouts of these cements are increasingly being used for soil solidification. They usually result in a stronger solidified mass, and are often less costly than chemical solution grouts. Also, unlike many chemical solution grouts, they do not lose strength over time.

### *6B.6.3.4 Grout Injection*

Permeation grouts can be injected from the top down, bottom up, or selectively at any depths or locations desired, through the use of sleeve port pipes. The proper sequence of injection is dependent upon the relative permeability of the different strata of the particular formation being treated. A common mistake, which has resulted in poor performance on many projects, is to inject upstage in a deposit that is significantly more permeable at depth than at the higher elevations. In such cases, the grout being injected, even at the higher levels, will tend to pipe down into the more permeable zones. For uniform permeation of soil, it is important to inject the proper quantity of grout in each stage of each hole. Except in the case of sleeve port injection, this usually means injection of the zones of lowest permeability first.

### *6B.6.3.5 Hole Layout*

Grout holes are generally placed at regular spacing of about two to six feet. In rare instances, holes might be as close as about 0.5 feet where very low permeability soils are to be penetrated, or where very precise injection is required. Where the thoroughness of the grout saturation is of little importance, greater spacing might be used. When injecting ultrafine cement suspensions, filtration of the grout particles can occur, especially in finer-grained soils. Thus, closer hole spacing might be required. Where several rows of holes are used over a large area, they are usually staggered in alternate rows, as shown in the top of Figure 6B.93. Alternate *primary* holes should be injected first. Once the primary (P) holes have been injected, grout can be placed in the secondary (S) holes. If the optimal spacing has been used, the secondary holes should require slightly less grout than the primary. In those cases where extremely thorough grout saturation is required, a pattern as shown on the bottom of Figure 6B.93 can be used. Here the tertiary (T) holes are grouted after the adjacent primary and secondary holes have been completed.

In water control grouting, a reduction of the water flow is an indication that the grout is being placed in an effective manner. In strengthening applications, however, the only assurance of com-

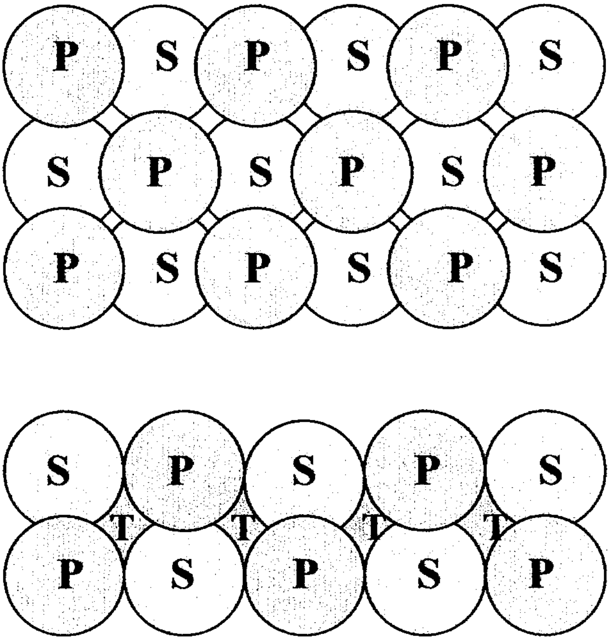


**FIGURE 6B.93** Typical grout hole layout. P = Primary, S = Secondary, T = Tertiary.

plete saturation of the intended soil is accurate preinjection appraisal of the soil's porosity, and precise placement of the proper quantity of grout at each intended location. In this regard, continuous observance of the injection pressure behavior is mandatory, as any sudden drops of pressure are indicative of the occurrence of hydraulic fracturing. Obviously, grout that is allowed to follow fractures will not solidify the intended soils. Excessive hydraulic fracturing is usually indicative of an excessively high injection rate.

#### *6B.6.3.6 Soil Strengthening Examples*

Permeation grouting of soil has been successfully used on a wide range of different applications. These have included both temporary as well as permanent solidification of a variety of sands and sandy soils, as well as shutoff of water leakage, both into substructure and through the soil. Perhaps the largest single area of permeation grouting in construction is increasing the cohesion of in situ soils prior to the excavation of soft ground tunnels. On a worldwide basis, huge quantities of grout have been used in such work. The most common objective is to prevent soil runs during tunneling, as well as to improve support for any structures lying adjacent to or over the tunnel alignment. The most commonly used materials for this type of work have been sodium silicate based chemical solution grouts. As the ground improvement need last only long enough for the construction to take place, long-term durability of the grouted formation is usually not a concern. This type of work can be performed either from the tunnel heading using fan shaped arrays of grout pipes, as shown in Figure 6B.94, or from the surface, as shown in Figure 6B.95.

Where practical, working from the surface is preferred, as it does not interfere with the tunnel excavation and can be done well ahead of the tunnel advance. Although some cohesion is imparted

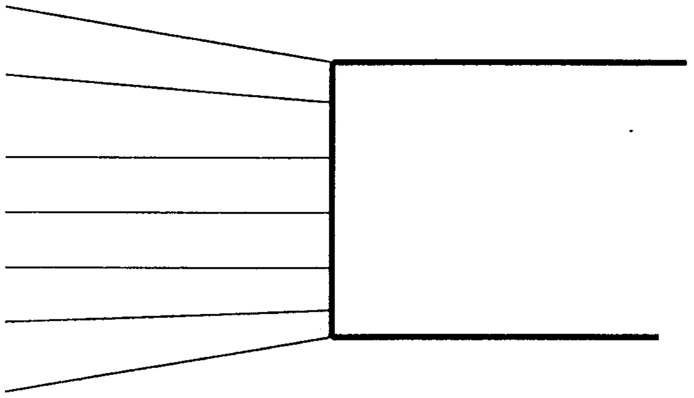

**FIGURE 6B.94** Grout holes in fan array.

to the soil within hours of injection, substantial strength does not occur for a minimum of several days. Thus, a time lag is required between grout injection and tunnel excavation. In some instances however, injection from the surface might not be possible, due to excessive tunnel depth or limitations of surface access. In such cases, where work must proceed from the tunnel heading, selection of a grout system that rapidly gains strength is essential.

Permeation grouting for temporary support during construction has also been used on a wide variety of different types of excavation and structures. As part of the expansion of Los Angeles International Airport, it was necessary to underpin an existing concrete retaining wall, which was found-

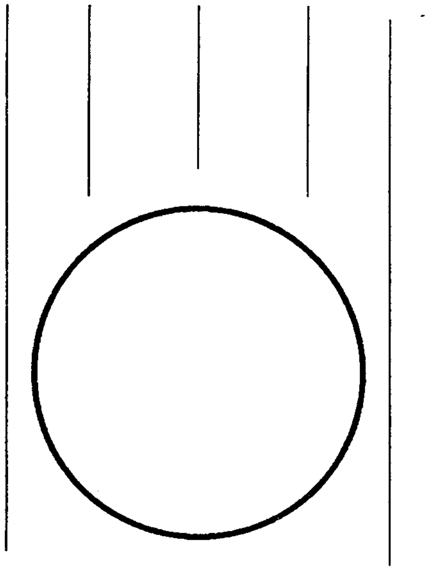

**FIGURE 6B.95** Grout holes from surface.

ed on wind-deposited fine dune sand. The wall was to be undermined and extended by a depth of 18 feet. A sodium silicate based grout was injected from the top down using a special injection needle, which was advanced in one foot increments (Figure 6B.96). Two rows of holes, were used, as indicated on Figure 6B.97. The lower portion of the grouted mass was injected from a row of holes inclined about 10 degrees from vertical, and the upper portion from holes inclined at about 20 degrees. The quantities of grout injected at each level was adjusted so as to provide the required lateral saturation needed, considering the inclination of the injection. The existing sands were improved to an unconfined compressive strength of about 150 psi. Excavation, which proceeded about two weeks after the injection, required pneumatic tools, due to the strength of the mass.

The rehabilitation of a side-hill structure supporting a road required the excavation of an immediately adjacent vertical face. Because normal support was not possible, due to the existing downslope, permeation grouting was performed to solidify the existing soil under the structure. In order to make the shear vertical cut, it was necessary to actually remove a portion of the footings of the structures supporting pillars, as can be observed in Figure 6B.98.

A sodium silicate based permeation grout was injected into the wind-deposited fine sand. This was accomplished through a single row of grout holes 15 inches behind the face of the structure. The holes were spaced 30 inches apart. A quantity of grout that would solidify a theoretical 36 inch diameter column based on the soil's porosity was injected into each hole. Holes were injected on an alternate primary/secondary split spacing order. Because the sand became less permeable with depth, the grout injection pipes were driven to the final depth (Figure 6B.99) prior to injection,

**FIGURE 6B.96** Grout injection from the top down with special drive needles.

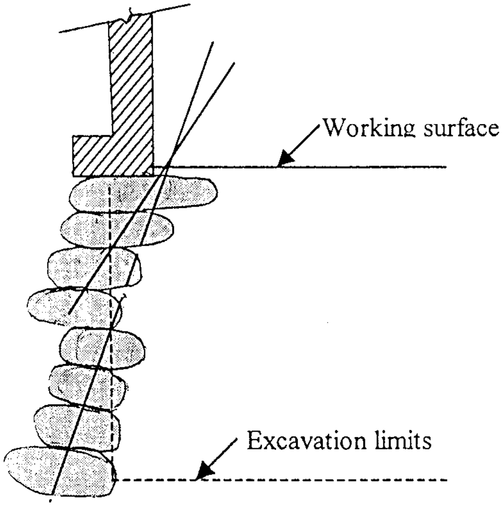


**FIGURE 6B.97** Two rows of inclined injection probes were used.

**FIGURE 6B.98** Solidified mass allowed excavation of shear vertical cut, including removal of part of the supporting concrete footings.

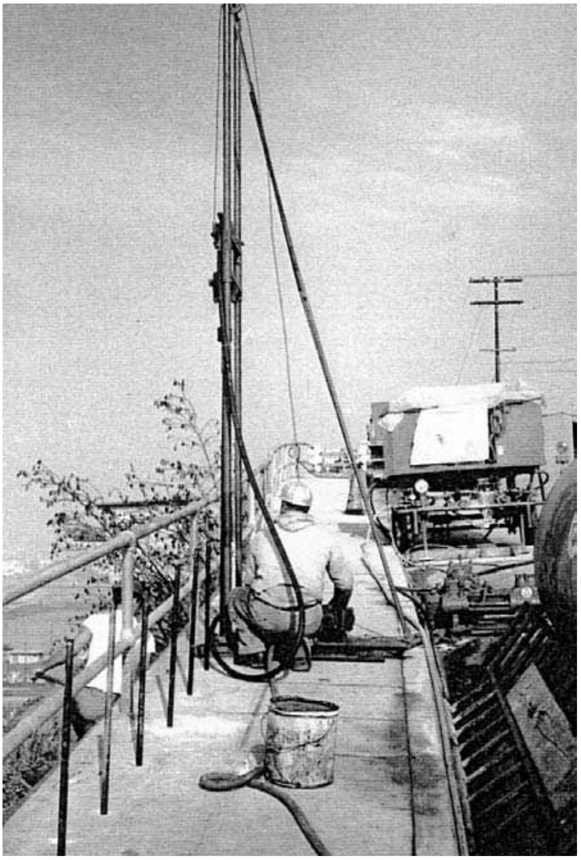

**FIGURE 6B.99** Special needles were driven to the depth of injection, which proceeded from the bottom, up.

which was then made in one foot stages, from the bottom, up. As can be seen in Figure 6B.98, pneumatic tools were required to trim back the solidified mass to the specified alignment. The flush cut was 240 feet long and varied in height from 6 to 24 feet, as shown in Figure 6B.100. For those sections which exceeded a cut height of 12 feet, a second row of holes was injected 27 inches behind the cut face, so as to provide a stabilized wall approximately five feet thick.

The foundation for new equipment in a steel mill required excavating to within four feet of the shallow concrete pad foundation of a column, which supported a large traveling crane. In spite of the dynamic forces that would be transferred into the column during operation, the crane was required to be in service 24 hours a day, seven days a week. The foundation was underlain with cohesionless sand and gravel, into which the required excavation would extend some 11 feet.

The entire area under the footing was solidified with a chemical solution grout, which was injected from the top down. This was done with hand-jetted, open-end ¾ inch pipe injectors (Figure 6B.101). Injection probes were on a grid of 24 inches, and a quantity of grout sufficient to create an equivalent 30 inch column was placed. The nearly vertical excavation was then made (Figure 6B.102) with continual operation of the crane. Due to the dynamic forces exerted into the footing by the passing crane, it was important to provide maximum possible stiffness, as well as

**FIGURE 6B.100** Vertical cut face varied from 6 feet to 24 feet high and extended for 240 feet.

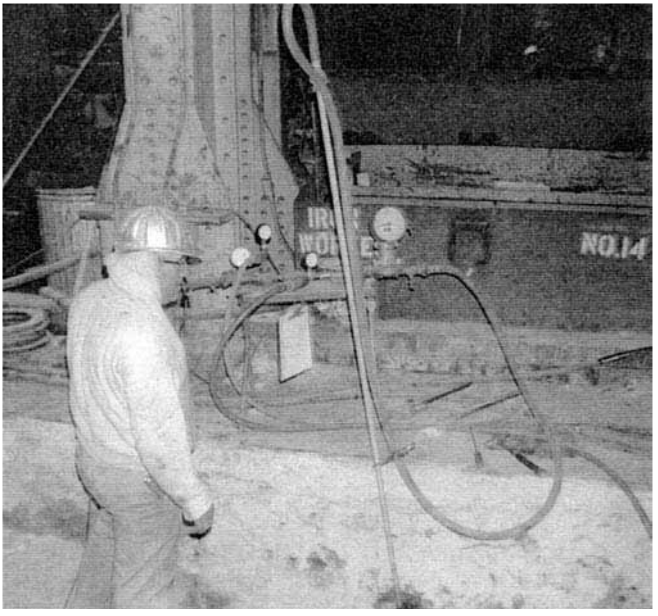


**FIGURE 6B.101** Grout injection in progress through standard ¾ inch schedule 40 pipe.

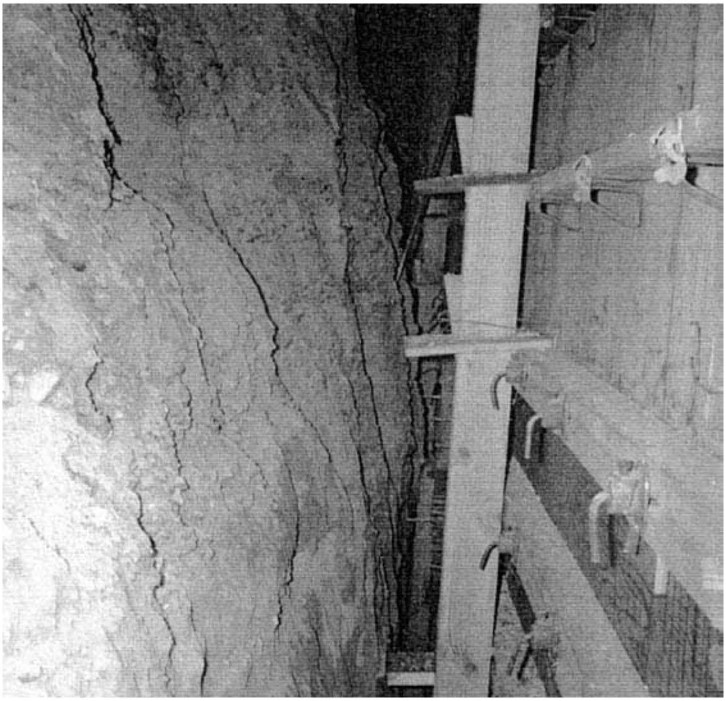

**FIGURE 6B.102** Formwork for the new foundation, adjacent to near vertical cut of cohesionless soil.

strength to the underlying soil. This would have been an excellent candidate to be grouted with ultra-fine cement grout; however, that material was not readily available at the time the work was performed.

Seismic retrofit of the San Francisco Civic Center required undermining of existing foundations by as much as nine feet. The foundation soils are basically round-grained beach sands, with virtually no cohesion. The ground, which was to remain under the foundation elements, was solidified with an ultrafine cement grout. As can be seen in Figure 6B.103, this allowed for otherwise unsupported excavation, as required for the new extended footings. In order to maximize the penetrability of the grout, it was mixed in a high shear colloidal mixer (Figure 6B.104).

When the terminal building at the Kansas City International Airport was originally built, the excavated space behind the basement retaining walls was backfilled with a granular fill sand. A contractor building four access tunnels connecting to the building in 1997 encountered this sand fill. It caved into his excavation, exposing utilities and undermining adjacent pavement. To mitigate the problem, ultrafine cement grout was used to solidify the cut face of the required excavations. The grout was injected from the top down, through jet pipes that were marked at one foot intervals. The pipes were positioned at a 15 degree angle toward the proposed excavation. They were jetted with grout into the formation, with a pause at each foot interval, as required to place the predetermined amount of grout (Figure 6B.105).

For most of the work, the holes were placed on three foot centers. The calculated amount of grout to form a four foot column was injected. Where the excavation was greater than 10 feet deep,

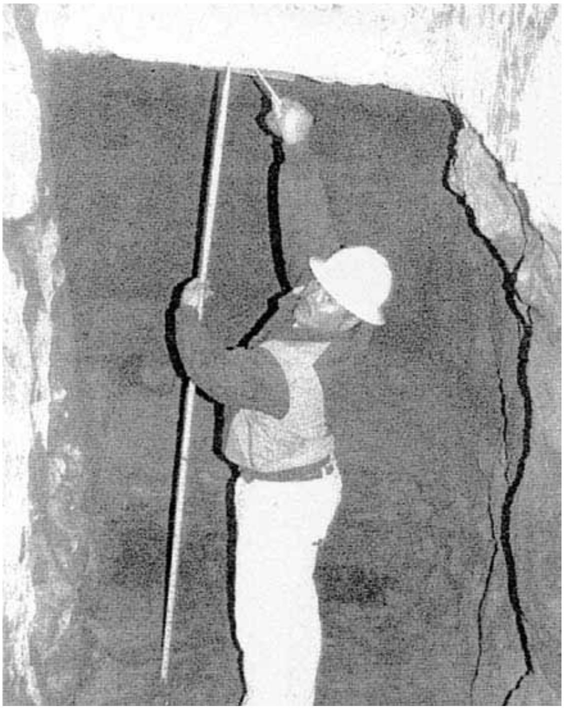

**FIGURE 6B.103** Nine foot vertical cut under foundation in cohesionless beach sand.

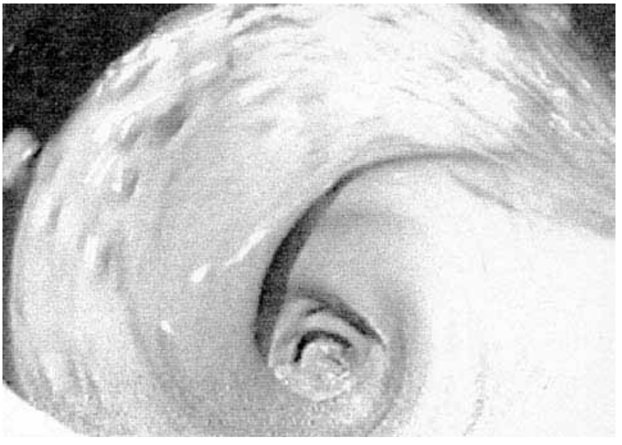

**FIGURE 6B.104** Ultrafine cement grout was mixed in high-shear mixer. Note vortex in grout.

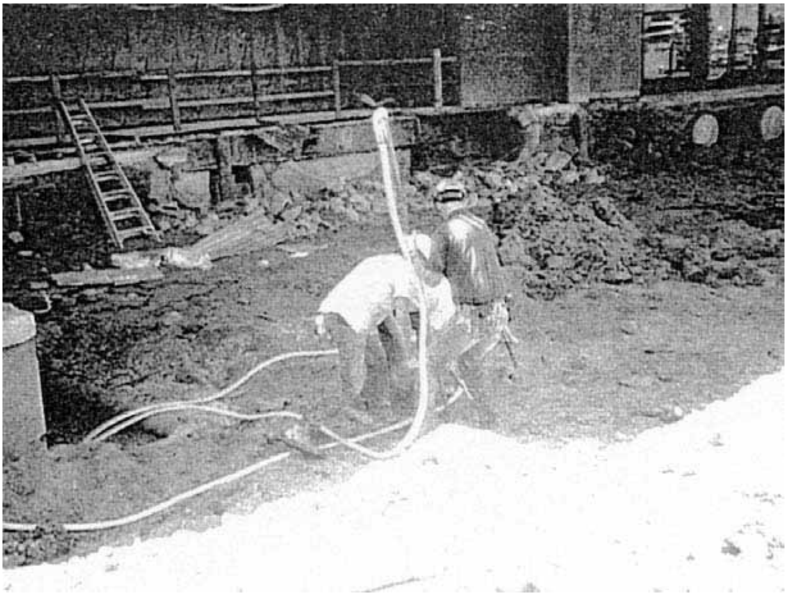

**FIGURE 6B.105** Jetting grout into the sand backfill.

a second row of holes was provided, split-spaced and three feet behind the initial row. The grout was composed of:

One bag Nittetsu Superfine cement

3.46 cubic feet (26 gallons) water

0.44 pounds high-range water reducer

It was mixed in a high-shear colloidal mixer and transferred into an agitator tank, which supplied a 2L8 Moyno pump. The pump was set for an injection rate of 2.6 cubic feet (20 gallons) of grout per minute.

Upon excavation, the sand fell freely from the injected columnar shaped masses (Figure 6B.106). They withstood the elements, including several cycles of freeze and thaw, for a period of several months. Both vibration and impact forces were created during demolition of the adjacent concrete wall but had no apparent effect on the solidified mass.

To save time and money, NASA used permeation grouted piles rather than conventional piling for the foundation of a 350 ton missile launch gantry at Vandenberg Air Force Base in California. A tight schedule for launch of a Physics Interplanetary Monitoring Platform satellite into orbit required very rapid construction of the launch facility. There simply was not sufficient time to construct conventional piling. Studies found the underlying soil to consist of about 20 feet (6.09 m) of fine to medium dune sand, underlain by shale bedrock. Groundwater was present, at a depth of about 15 feet 4.57 m).

The gantry traveled on sets of rails mounted to two parallel pile caps. The heavily reinforced concrete caps were supported on two rows of piles, as shown in Figures 6B.107 and 6B.108. The pile design was based on a permanent, chemically solidified mass, with an unconfined compressive strength of at least 100 psi (689.5 kPa). This was considered conservative, as strengths nearly twice that value were found to exist after seven years in a nearby installation, which used the same grout formulation. Accordingly, the 188 piles were designed, with a minimum diameter of 30 inches (0.76

GROUTING TO IMPROVE FOUNDATION SOIL 6.441

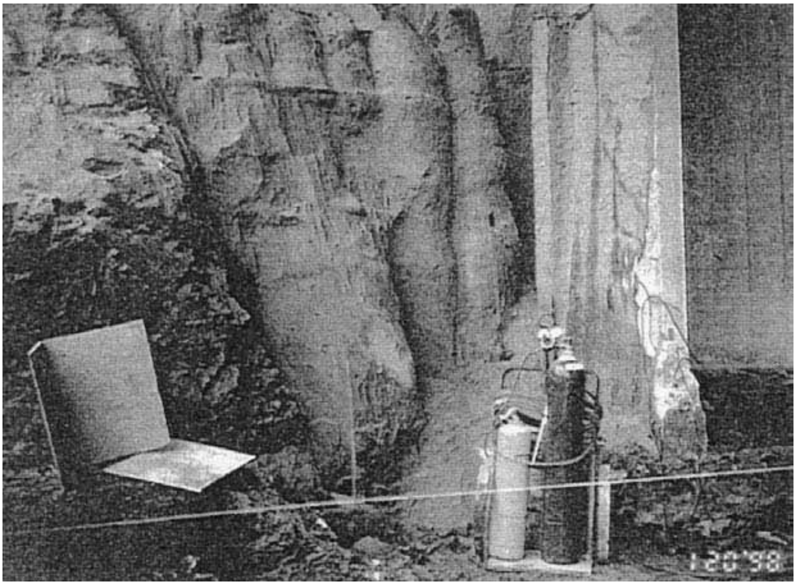


**FIGURE 6B.106** Self-supporting solidified sands.

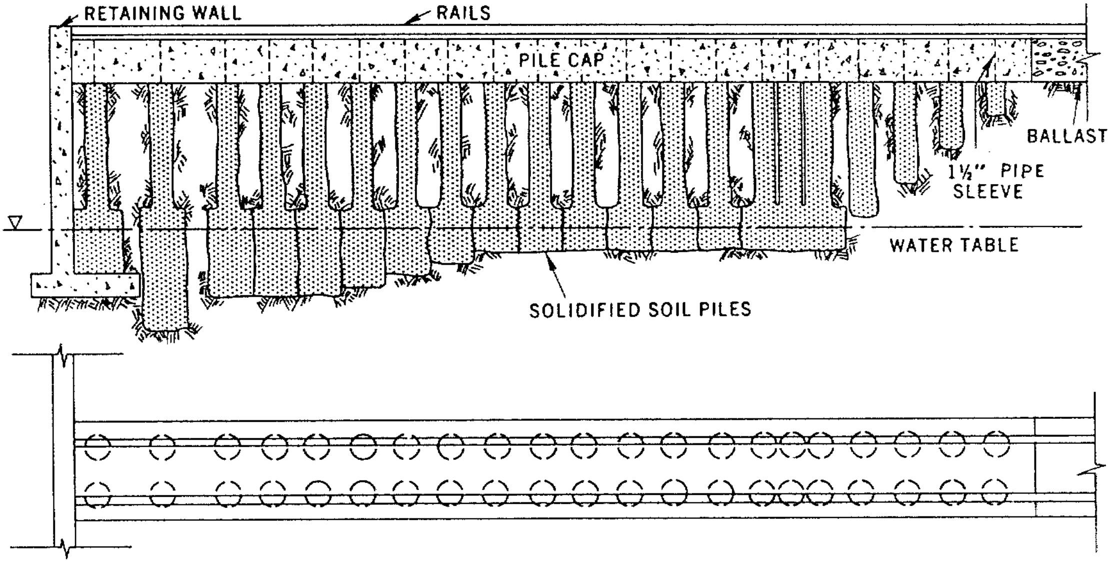


**FIGURE 6B.107** Two rows of piles supported each parallel cap.

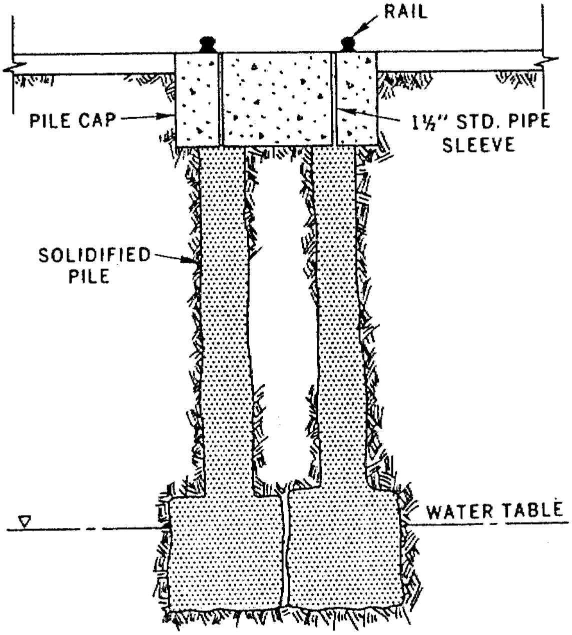


**FIGURE 6B.108** Cross section of pile cap and solidified piles.

m). The diameter was increased to 60 inches (1.52 m) for the portion below the water table, as chemically grouted masses are known to obtain lower strengths under water. The piles extended full depth to the underlying shale formational deposit.

One distinct advantage of injected piles for the project was the ability to form them either before or after the pile cap had been cast. This greatly reduced the total time required for the work. Initially, piles were injected prior to casting of one of the parallel concrete pile caps, while the other cap was being simultaneously constructed. Sleeves of 1½ inch (3.8 cm) pipe, as shown in Figure 6B.108, were cast into the second cap at each pile location. The remaining piles were then injected under the completed pile cap, using the pipe sleeves for access.

A three-component, sodium silicate based chemical solution grout was prepared in batch mixing tanks (Figure 6B.109). The base tank combined sodium silicate, water, and a surfactant. The reactant tank combined ethyl acetate and calcium chloride reactants, with water. These were accurately proportioned by way of properly calibrated metering pumps, which fed a "flash mixer" located immediately ahead of the main grout pump. The flash mixer was simply a three inch squirrel cage blower enclosed in a small tank, turning at about 1800 rpm, so as to provide very high shear mixing. The final grout mixture contained 55% sodium silicate.

Injection was from the top down, through special drive needles (Figure 6B.110). The needles were advanced in one foot increments, followed by injection of the grout. A calculated amount of grout, sufficient to completely saturate a 36 inch column, was injected for the 30 inch sections, and 66 inches (1.67 m) for the 60 inch (1.52 m) sections. Tests carved from the solidified masses, both during the work and after completion, attained fundamental strengths somewhat greater than 200 psi.

A large, horizontal piston-type air compressor was installed on a massive reinforced concrete foundation. After several years of operation, excessive vibrations resulted, which not only affected the foundation block, but the surrounding building elements as well. Investigation revealed that the

**FIGURE 6B.109** Grout mixing and pumping plant.

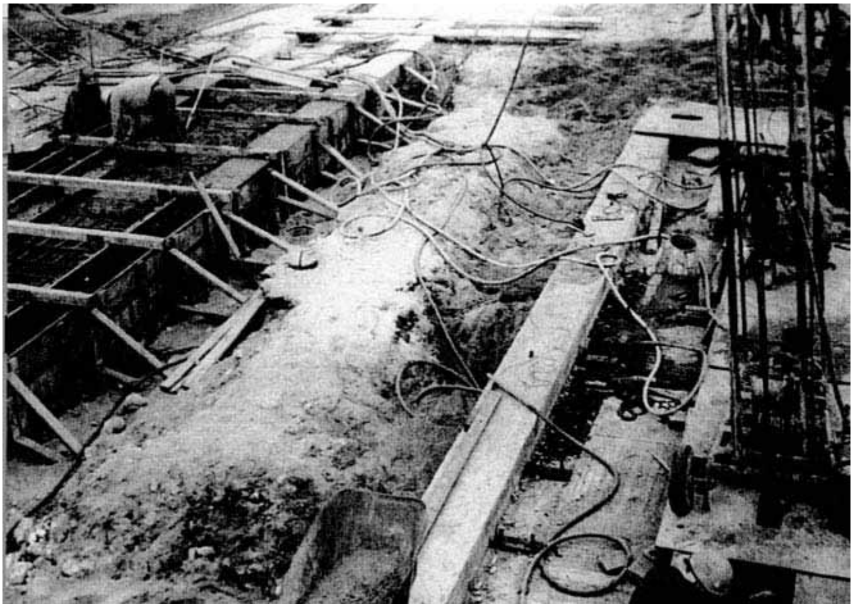

**FIGURE 6B.110** Special rig drives injection needles through pipe sleeves in the completed pile cap.

foundation block was founded within a deposit of silty fine sand. The sand had a natural frequency, which was in harmony with that of the compressor. It was concluded that an increase of the stiffness of the sandy soil was required to eliminate the excessive vibrations.

Accordingly, a grouting program was formulated with the intent of stiffening the sand through solidification with a chemical solution grout. The chosen formulation contained a 70% solution of sodium silicate. The penetrability of a grout containing such a high concentration of the base component is significantly less than that of more conventional formulations; however, it was used for this application, in order to provide the greatest degree of additional stiffness practicable. With a grout of such low penetrability, very slow pumping rates are necessary, and it is desirable to space the grout holes closely. The machine resulted in severe restrictions of space for the layout of grout holes. A single row was thus placed on 1.5 foot (.46 m) centers, immediately adjacent to and around the periphery of the foundation (Figure 6B.111). Additional holes were placed within this boundary containment, as allowed by the existing conditions. The grout was injected at an unusually slow rate of less than one gallon per minute, as shown in Figure 6B.112.

Although the cost of injection at such low injection rates can be high, there are situations such as this where they are warranted, and the resulting benefits are very cost effective. The grouting was completely successful, in that the excessive movements of the machine were completely eliminated.

#### *6B.6.3.7 Permeation Grouting for Water Control*

The procedures used to create an impermeable barrier in soil are essentially the same as those used to inject grout for strengthening applications. Whereas the grouted mass cannot be seen as in those cases where the adjacent soil has been excavated, the degree to which the objectives have been achieved will usually be obvious by way of reduction of the leakage. This is usually observed in the

**FIGURE 6B.111** The machine presented a severe restriction to hole location and drilling.

**FIGURE 6B.112** Grout injection rate was very slow due to low grout penetrability.

drainage system, if existing, or from pump drawdown tests conducted in drainage holes placed downstream of the curtain.

It is extremely difficult to effect an impermeable barrier with a single row of grout holes. Accordingly, two or more rows of holes should be placed with split spacing, as illustrated in the top of Figure 6B.93. Where an especially tight barrier is required, hole layout with tertiary injections, as shown on the bottom of Figure 6B.93, can be used. In such cases, a quantity of grout that exceeds the theoretical quantity required should be placed, unless significantly high injection pressure exists, which would indicate that all available voids have been filled.

Stoppage of water leakage adjacent to a solid object such as the wall of a structure is somewhat easier in that one boundary of the work is established. This can usually be accomplished with a single row of grout holes placed adjacent to the wall. An alternative would be to drill holes through the wall, on a regular spacing, followed by grout injection. Such work is best done during periods of active leakage, such that the effectiveness can be observed by way of a reduction of the leakage.

Water was penetrating the concrete masonry unit wall of a subsurface equipment room (Figure 6B.113). A hydrophobic urethane grout was injected in a single line of holes, at 24 inch (0.6 m) intervals, and about six inches (15 cm) outside the wall. Injection was started in those areas that had the greatest leakage. As can be seen in Figure 6B.114, some of the grout penetrated the areas of leakage, reacting on the wall surface. This gave confirmation that the culprit leakage paths were being plugged, although it also created a nasty cleanup job.

In another instance, a seal around a pipe penetrating a concrete wall had failed, allowing water to leak into subsurface space (Figure 6B.115). A hydrophilic urethane grout was injected into a hole adjacent to the penetration, and extending to the back of the wall. As can be seen in Figure 6B.116, the grout, which was formulated to cure as a somewhat flexible foam in order to allow differential

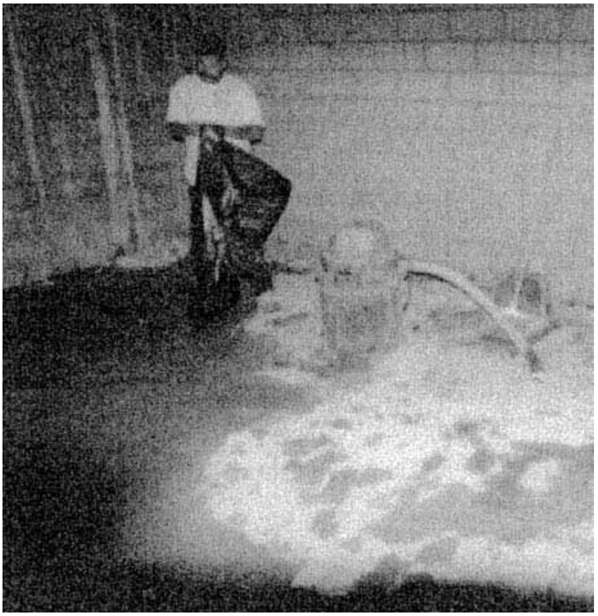

**FIGURE 6B.113** Water leakage into equipment room.

**FIGURE 6B.114** Reacted grout on wall.

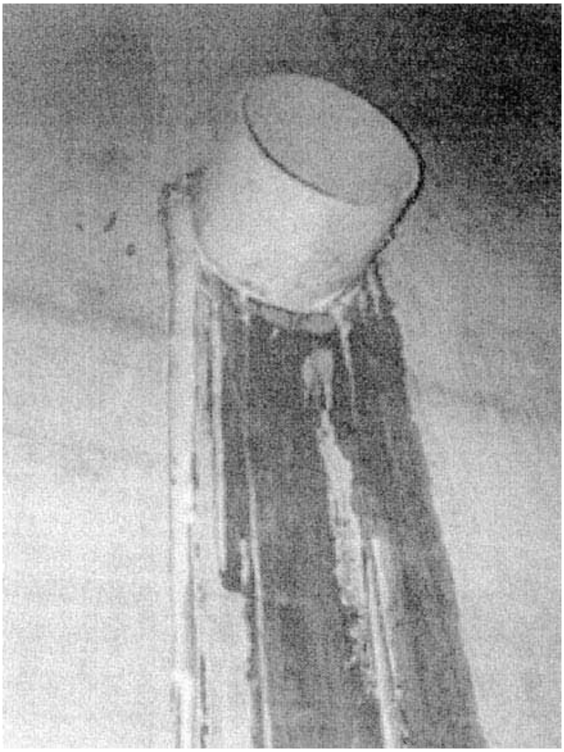

**FIGURE 6B.115** Water leakage through defective pipe seal.

movement, completely filled the annulus around the pipe. A small amount of grout also ran down the wall, reacting thereon. Whereas such deposits of grout can be very hard to clean up, they do give assurance that the leakage has been completely stopped.

Excessive water was flowing into a gallery of an embankment dam. The water was under a positive head of about 300 feet (91.4 m) at the gallery level. Both hydrophobic and hydrophilic urethane grouts were injected into the zone behind the leaks. Holes were drilled through the concrete section, packers installed, and the grout injected. Because the exact source of the leakage was uncertain, grout holes were angled in several different directions (Figure 6B.117). The injection was carefully monitored, and grout returns, or diluted returns, carefully recorded. The leakage was reduced by more than 95%, although some minor seepage continued. One advantage of grouting for water control is that more holes can always be drilled and further grout injected until a satisfactory amount of leakage reduction is achieved. As this work site was located far into the gallery, it is fortunate that neither large equipment nor great quantities of material were required. Figure 6B.118 shows the small pneumatically powered proportioning pumps that were used.

## 6B.6.4 JET GROUTING

Jet grouting involves erosion of the soil with a high-pressure jet of grout, water, or air-enshrouded water and the simultaneous injection of grout into the disturbed soil by means of a rotating drill

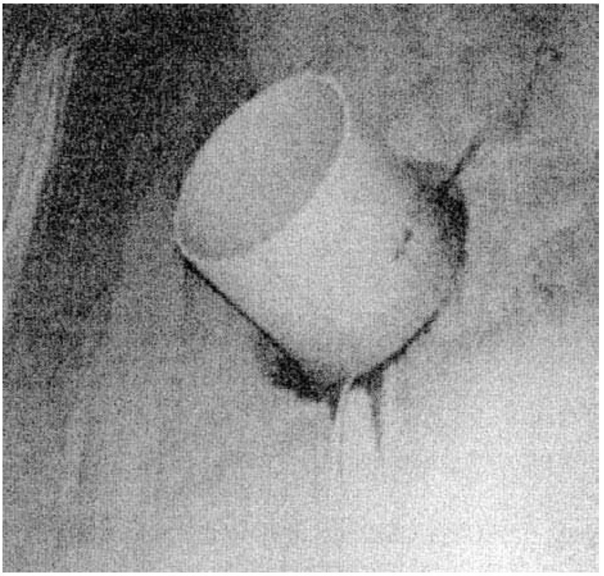

**FIGURE 6B.116** Leakage completely stopped.

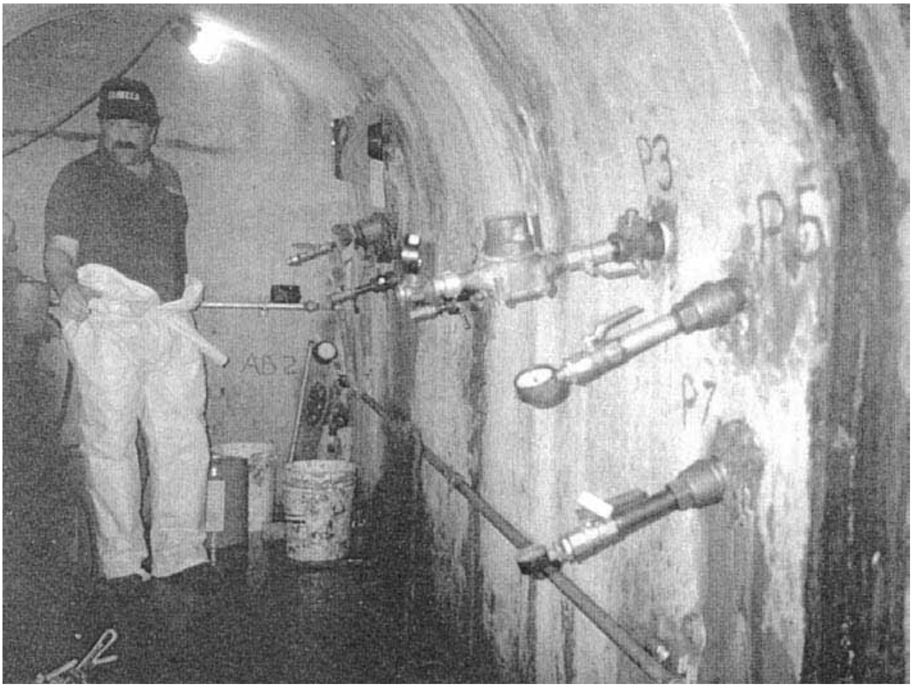


**FIGURE 6B.117** Grout holes at various angles so as to affect a large area of leakage potential.

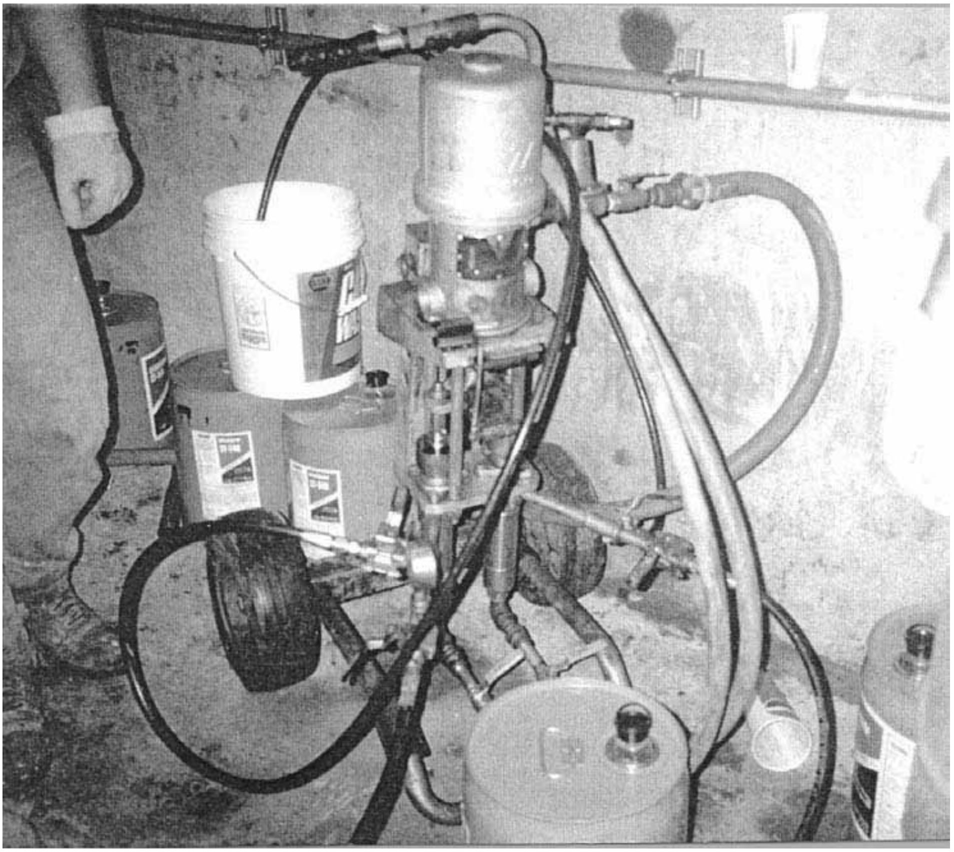

**FIGURE 6B.118** Small pneumatically powered grout pump could be carried to injection location.

stem and special jet *monitor* (Figure 6B.119). The drill stem and monitor are simultaneously raised and rotated so as to mix the grout with a portion of the original soil to form *soilcrete* or, if required, it is possible to replace almost all of the soil with grout. By raising the drill stem absent any rotation it is possible to form soilcrete panels. The procedure has the advantage of effectiveness in virtually any type of soil, although the efficiency and thus strength of the resulting mass is soil-dependent. It is lowest in clays and cohesive soils, and increases as the soil becomes more granular. Not surprisingly, the greatest strength is obtained in clean sands and gravels, which result in a concrete-like mass. Jet grouting can be used in the presence of large rocks or other obstructions, although a shadow effect can interrupt the continuity of the grout solidification.

In application, a hole, typically about 4 inches in diameter, must be accurately bored to the desired depth of improvement. The initial hole can be bored separately or by a drilling or jetting tool mounted onto the jet grouting monitor, which contains the nozzles. There are three major variations of the jet grouting operation. In the simplest form, known as the *one-fluid system*, a special hollow drill rod equipped with a monitor that contains horizontal jet nozzles at the tip (Figure 6B.120) is lowered into the hole. A cement suspension grout is pumped down the drill rod at very high pressure (up to 9,000 psi) (52 kPa) while the drill rod and monitor are simultaneously rotated and withdrawn (Figure 6B.121). The grout, which exits the jet nozzles at high velocity, disintegrates the soil and

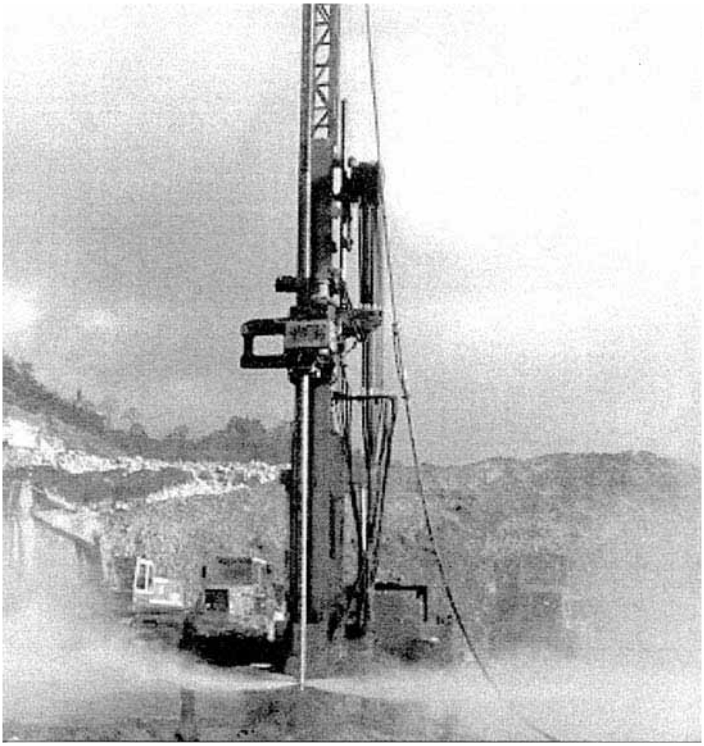

**FIGURE 6B.119** Jet grouting monitor ejects water and/or grout at high pressure.

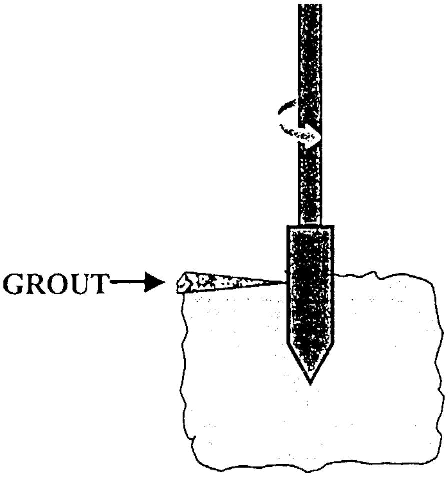


**FIGURE 6B.120** One-fluid jet grouting system.

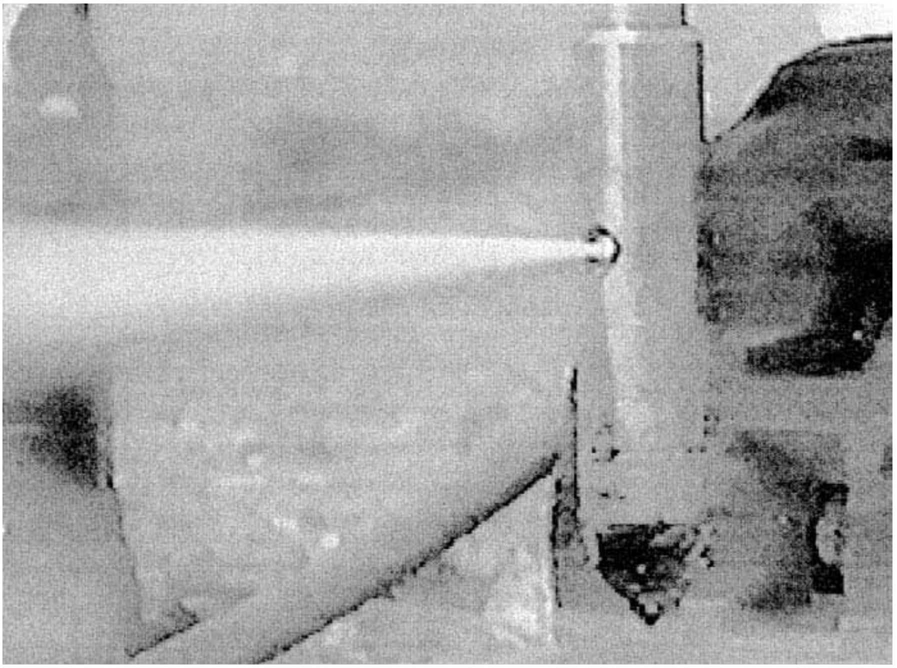

**FIGURE 6B.121** Jet grouting monitor for one-fluid system.

mixes with it to form soilcrete columns. The effective radius of the grout jet is dependent upon the properties of the particular soil which is to be treated. Column diameters on the order of 15 to 30 (0.38 to 0.76 m) inches are typically obtained in cohesive soils, and diameters as great as four feet can be achieved in granular soils (Figure 6B.122).

In the *two fluid system* (Figure 6B.123), the grout is encased within a shroud of compressed air. This is facilitated by use of a special coaxial drill string and monitor. The air shroud acts as a buffer between groundwater and the grout, which greatly increases its cutting radius. The air also creates turbulence in the drill cuttings as they rise to the surface, greatly increasing the spoil removal efficiency. Diameter of the columns resulting from the two-fluid system are on the order of 1.5 to 3 feet (0.45 to 0.9 m) in cohesive soils and 3 to 6 feet(0.9 to 1.8 m) in granular soils.

The *three-fluid system* is the most complicated in that it requires a triaxial drill stem and monitor, with appropriate nozzles. In this system, an air-enshrouded jet of water erodes the soil as grout is simultaneously injected through separate nozzles (Figure 6B.124). The cutting jets are always located above the grout supply, allowing for a nearly complete replacement of the soil with grout as the monitor is withdrawn. The triple-fluid system enables formation of the largest diameter columns. Effective diameters of two to five feet in clays, and up to 10 feet (  m) or more in sands (Figure 6B.125), have been experienced. It is, however, the most complicated system to use, and requires a very costly and complex drill string and monitor.

Jet grouting can be employed to depths of 200 feet or more. Its application is fast, and it can be performed in a wide range of soil types. Strength of the resulting soilcrete is dependent upon the original soil, and the jetting parameters (rotation and withdrawal rates) used, but can be of structural strength [2,000 psi (13.8 kPa) or more]. By overlapping probes, a nearly continuous curtain can be formed. Generally, large and sophisticated equipment, such as the rig shown in Figure 6B.119, is required which severely limits the availability and competitiveness of qualified contractors. The effec-

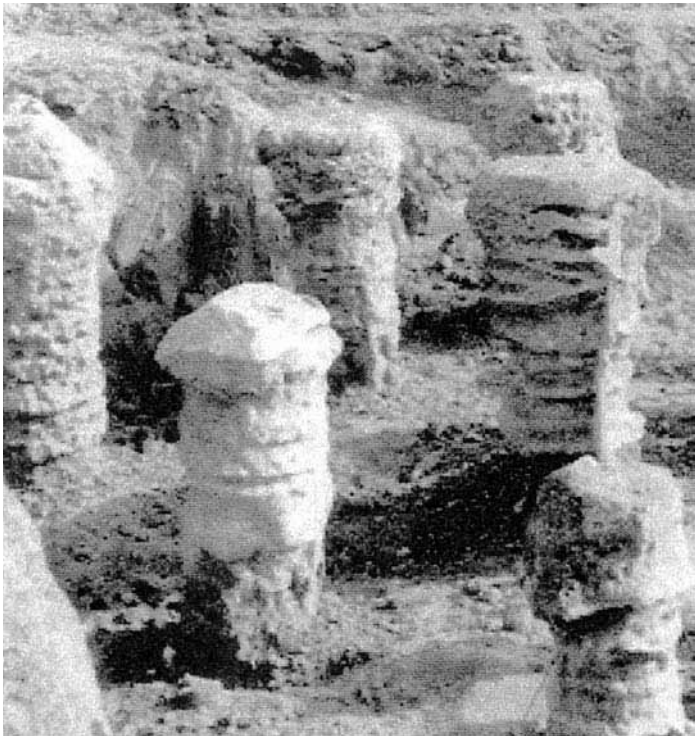

**FIGURE 6B.122** Solidified columns resulting from one-fluid system.

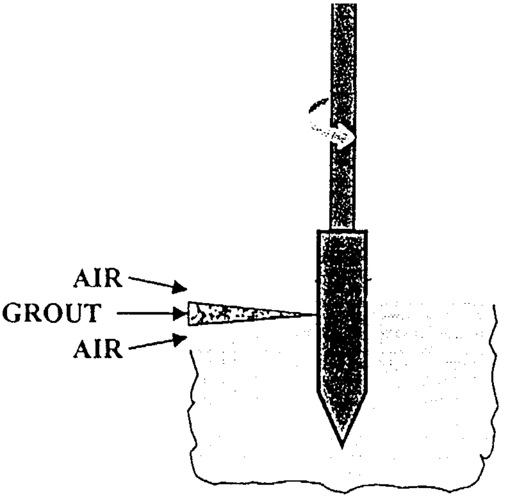


**FIGURE 6B.123** Two-fluid system where the grout is enshrouded in air.

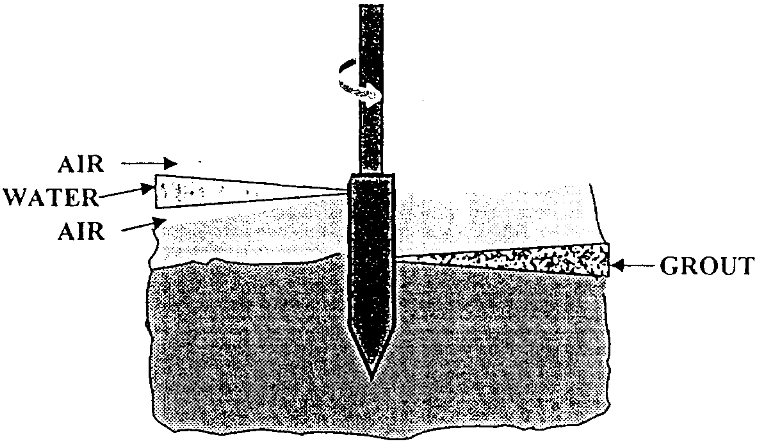


**FIGURE 6B.124** Three-fluid system.

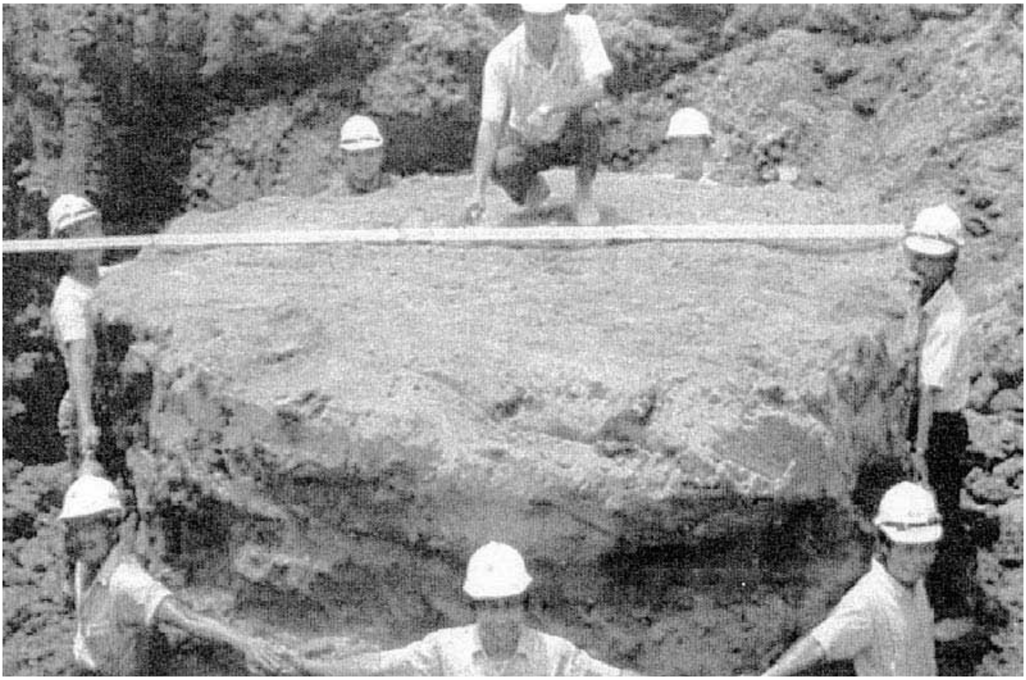

**FIGURE 6B.125** Very large columns of soilcrete are possible with the three-fluid system.

tive diameter of the resulting mass is dependent on the individual system used, and the jetting and injection parameters, such as grout pressure, rate of rotation, and withdrawal speed of the jet. Considering these, and the influence of the particular site soil, trial injections are required to establish the optimal hole spacing and operating parameters.

Perhaps the most significant limiting factor, however, is the production of large amounts of spoil, which must be contained, handled, and disposed of. Also, the very high pressures used at the jet can be imparted into the soil at the nozzle depth if passage for the spoil should become blocked. This is a particular risk where the holes penetrate soft clays. Such high pressures could severely damage the adjacent soil and/or structures, and there are situations where the results could be catastrophic. All things considered, the practicability of the procedure is virtually limited to use in relatively large applications. Details of the optimal procedures to be used will vary with individual project requirements and the site soils. Consultation with available specialist contractors is thus advisable when the procedure is contemplated.

## 6B.6.5 Replacement/Compensation/Fill Grouting

Grouting is often used to fill voids within the soil. These might be large subsurface voids in the ground resulting from erosion, sinkhole activity, or solution cavities, or the result of old mine workings. Conversely, they might be small or thin planar voids resulting from leaking pipelines or similar events, or voids resulting from subsurface construction or tunneling. Filling of thin planar voids adjacent to a hard surface such as concrete (Figure 6B.126), usually referred to as *contact grouting* or sometimes as *backpack grouting* if in connection with a tunnel (Figure 6B.127), is usually performed with a cementitious fluid suspension or slurry grout. Any grout used to fill obvious voids should have a low water to cement ratio or be formulated to have little bleed or shrinkage. In this regard, defects thicker than about an inch are best filled with very thick slurry or plastic consistency mortar grout.

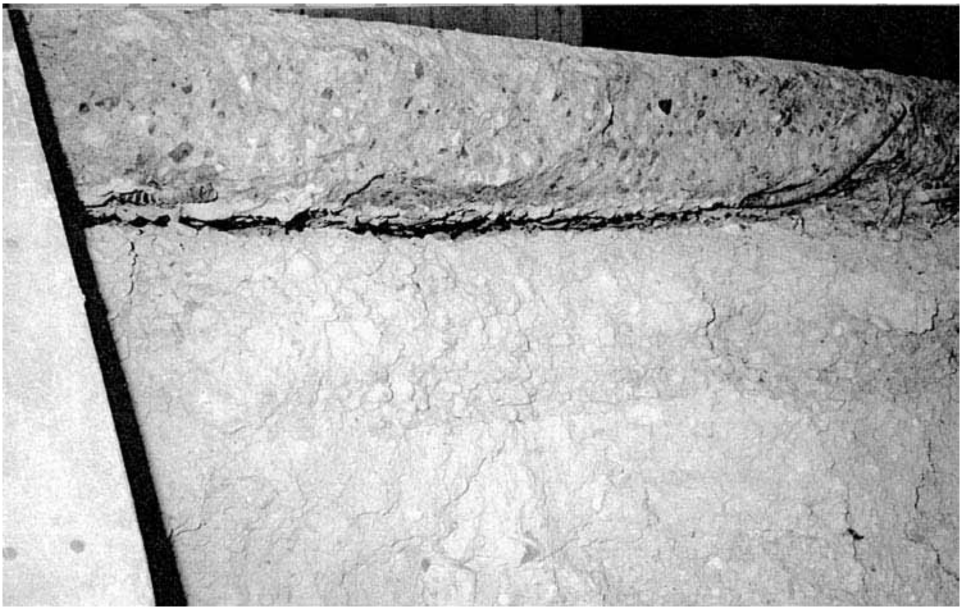

**FIGURE 6B.126** Void under concrete foundation resulting from adjacent excavation.

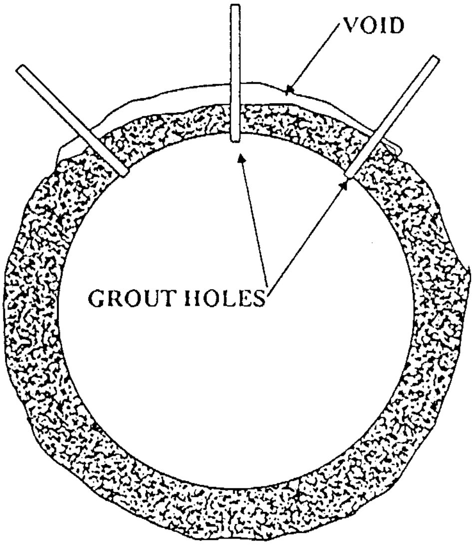


**FIGURE 6B.127** Typical void remaining after concreting of a tunnel lining.

Very large voids, those with a volume of more than about a cubic yard, are usually best filled with commonly available concrete or mortar, utilizing ready-mixed material and standard concrete pumps for placement (Figure 6B.128). Unlike in earlier times, modern cement and concrete technology allow design of mixes with virtually any desired properties. As an example, otherwise common mixes can be made to be very cohesive by inclusion of *antiwashout* admixtures. The setting time can be accelerated to less than an hour, or delayed for up to several days.

For those applications where added weight is a concern, foaming agents can be introduced into the mixer so as to provide low-density, hardened compositions. As the density of a cementitious composition lowers, the strength will also be reduced. This is seldom a problem in geotechnical grouting, however, as strengths greater than that of the adjacent soil are seldom needed or justified. Typical density and strengths for readily available cellular concrete are on the order of 50 pounds per cubic foot (800 kg per $m^3$) for an unconfined compressive strength on the order of 300 psi (2 MPa), and 100 pounds per cubic foot (1600 kg per $m^3$) for a strength of about 1500 psi (10.3 MPa). An area where low-density materials have been found particularly advantageous is grout filling of the annular space around pipes and liners in tunnels (Figure 6B.129). Not only is the cost of the rather large amounts of material often used in such applications reduced, but also the risk of *floating* of the liner is greatly reduced due to the lower density of the fill.

For the filling large voids, rapid injection rates are usually quite acceptable, and relatively little pressure is normally required. Care must be taken however, to not develop too great a pressure when the defect is nearing complete filling. A rapidly increasing pressure will nearly always indicate this, as closure occurs. For this reason, pressure gages are required and must be monitored, as in all other grouting.

Although it is possible to design ready-mixed compositions with low water to cement rations, and thus minimal shrinkage, more often than not, such mixes will be subject to shrinkage. This can become significant where the depth of such masses is great. Therefore, where the filling of such large voids is to be complete and tight, injection of a fluid suspension or slurry contact grout can be made after the main body has been filled. Where done, this should be delayed as long as possible,

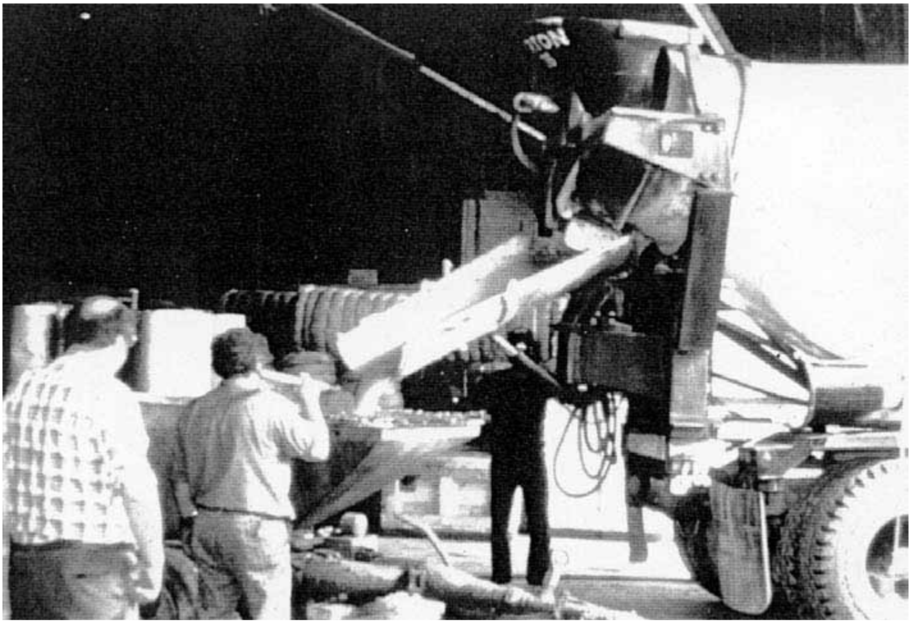

**FIGURE 6B.128** Fill grouting of large voids with ready-mixed mortar and a standard concrete pump.

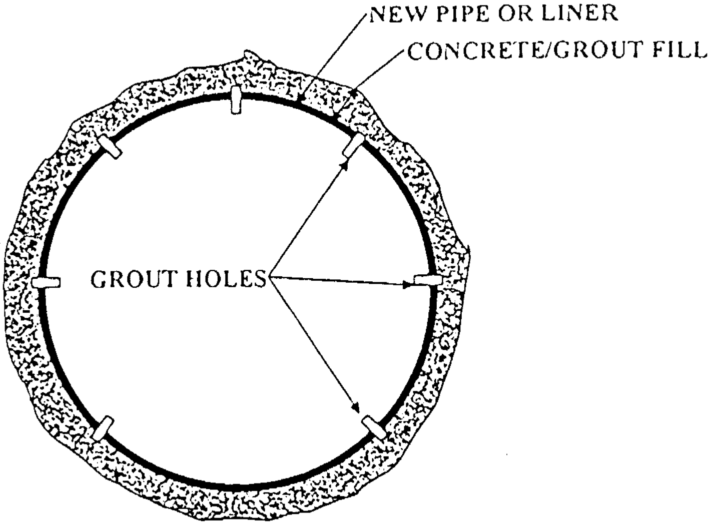


**FIGURE 6B.129** Low-density fill for the annular space around linings provides economy as well as lessening the risk of floating the lining.

and at least several days, to allow primary shrinkage of the mass placement. In those situations where a strong grout is not wanted, it is possible to inject a low-mobility compaction-type grout with a very low cement content or even completely absent of cement; however, this will be much more expensive than simply filling with ready-mixed material.

### 6B.6.6 Groutjacking

It has long been recognized that settled pavements could be accurately raised by slab jacking, which is simply the controlled injection of grout under them. (Refer also to Section 7B.2.) If a void exists under the slab, which allows the grout to immediately spread, no further preparation is required. If such is not the case, a void can be created by jetting through the grout holes with water or compressed air (Committee on Grouting, 1977). The same principal is involved when groutjacking a structure, except that the void must extend over a larger area, due to the higher loads involved. The area of void required is a function of the weight of the structure to be jacked times the available grout pressure, and must include provision for any frictional resistance between the structure and the adjacent soil. Thus, for heavy structures, considerable pressure and area is needed. Such lifting has sometimes been facilitated by horizontal boring or hand mining under the structure. Whereas light pavements can be raised with a wide variety of grout materials and only nominal pressure [generally less than 100 psi (690 kPa)], once lift has been initiated, very low mobility grouts such as those used for compaction grouting and generally much higher pressures (500 to 2,000 psi) (3.4 to 13.8 MPa) are required under heavy structures.

If compaction grouting is being used to improve the soils under a structure, it is possible to lift it during the grout injection. If sufficient lift has not been obtained during the compaction, however, or perhaps the underlying soils are not in need of improvement, it is possible to jack a structure by creating a *controlled*, horizontal hydraulic fracture in the soil. The fracture is then expanded or *jacked* until the desired lift occurs. Uncontrolled hydraulic fractures that occur during grouting, usually follow the orientation of the minor principal stress of the soil, which is normally a vertical plane. A horizontal fracture can usually be created, however, by injecting a small amount (3 to 5 gallons) of a highly mobile grout into a casing that extends to within about one inch of the bottom of the hole. A low-mobility grout, such as that used for compaction grouting, can then be injected so as to open the fracture and raise the ground and overlying structure. Raising of both pavements and structures to very close tolerances is possible. Although many grouting contractors lack the knowledge and ability for this type of work, those who are qualified will routinely raise a structure to within about 0.1 of an inch of the specified elevation.

One side of an aluminum smelter, which was in full operation and filled with molten metal, had settled several inches. The differential settlement resulted in fracture of some of the steel framing element connections, which restrained the firebrick lining (Figure 6B.130). The very high heat in the furnace area made investigation of the problem or remedial work extremely difficult. However, because the owner had a commitment for delivery of a highly specialized alloy, several months of further operation were required, and something had to be done to temporarily relieve the distress of the steel containment structure.

Grout holes inclined about 60 degrees off of vertical were drilled under the settled side. They were fully cased to the edge of the shallow foundation elements, and extended about another eight feet under the base. About one cubic foot of slurry-consistency grout was injected into the holes, followed by low-mobility grout, such as would be used for compaction grouting. Although, the objectives of the work were to raise the settled portion of the structure only, and no compaction of the underlying soils was intended, an unexpectedly high grout take revealed that the silty soils immediately under the foundation were quite loose. Although the intent of the groutjacking was limited to raising the settled portion of the foundation about one inch, in order to temporarily relieve the structural distress, the foundations were raised nearly four inches (20.1 cm) to near their original level.

Based on the grout takes and the history of the site, it was suspected that localized soil distur-

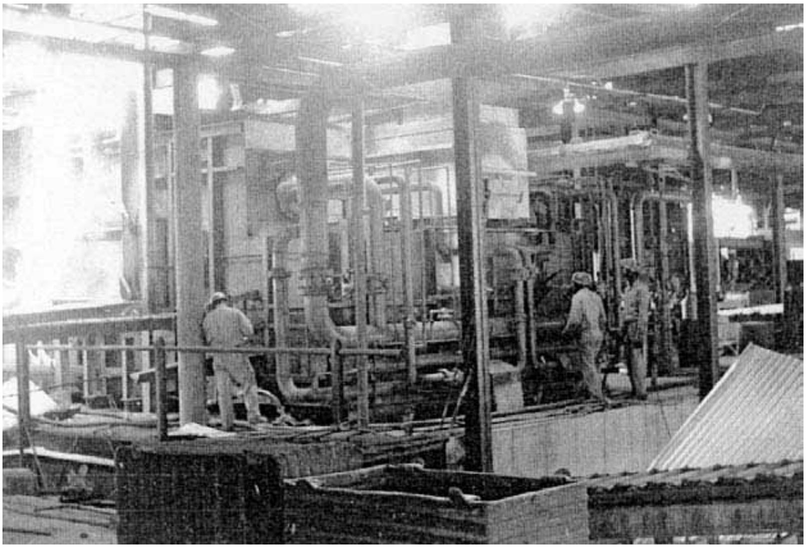

**FIGURE 6B.130** Metal furnace full of molten aluminum during groutjacking.

bance resulted when a drainage sump was removed from the area prior to construction of the furnace. As no further settlement occurred over the next several months, the originally contemplated investigation and permanent remedial program were cancelled.

Work on a new parking structure in Texas had to be stopped due to differential settlement of the pile-supported foundation. Groutjacking was used to precisely raise the individual settled columns and their supporting piles (Figure 6B.131). A two inch (5 cm) ID casing, such as that commonly used for compaction grouting, was placed to slightly past the pile tip elevation. A five gallon bucket of cementitious slurry was dumped into the casing, immediately followed by a compaction grout mix that incorporated the preferred aggregate gradation shown in Figure 6B.57. The columns were all raised to within about 1/16 th of an inch (1.5 mm) of their original elevation.

## *6B.7 SPECIAL CONSTRUCTION TECHNIQUES INVOLVING GROUTING*

There are a number of specialized foundation construction methods that involve grouting of one sort or another. The development of such systems has advanced rapidly within the last several years. This has been greatly facilitated by the ready availability of both high-capacity cranes and modern concrete and mortar pumps, as well as significant advances in cement and concrete technology.

### 6B.7.1 Auger-Cast Piles

In the late 1940s, the Intrusion-Prepakt Company began experimenting with novel pile installation methods. An early result was the auger-cast pile. Therein, a hollow-stem, continuous-flight auger of the pile design diameter (Figure 6B.132), is drilled into the soil to the intended pile tip elevation. A

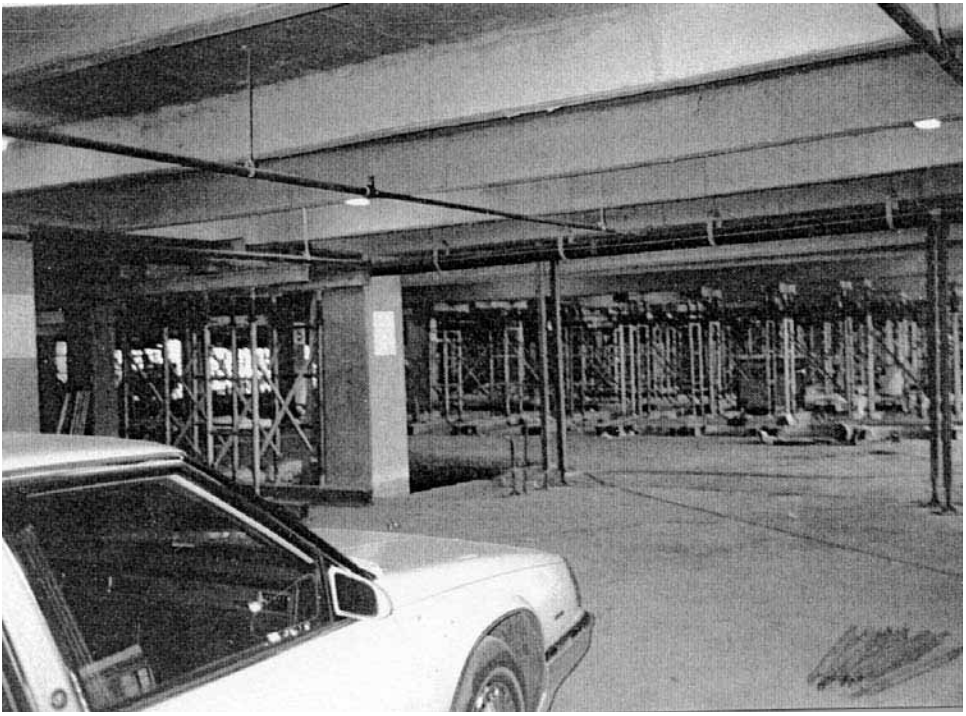

**FIGURE 6B.131** Groutjacking was used to precisely level pile-supported columns of parking structure.

cementitious grout or mortar is then injected as the auger is withdrawn. The injection pressure of the grout can actually aid in the withdrawal of the soil-laden auger, and contributes significantly to the realized skin friction. As originally conceived, equipment limitations controlled the size of piles that could be practically constructed, and thus their capacity.

With the advent of the modern concrete pump in the 1960s, it became practicable to rapidly inject high-strength mortar or small-aggregate concrete, rather than the original particulate grout. This allowed virtually complete filling of the hole with the higher strength replacement material, which greatly widened the obtainable pile capacities and the range of soil for which the technique is suitable. This period also saw significant advancements in hydraulic power transmission technology, which made possible very high torque drilling machines and high-capacity cranes and carriages on which to mount them.

With presently available equipment and technology, auger-cast piles can be installed in virtually any granular soil that is relatively free of large rocks or boulders. Piles with diameters up to 36 inches and lengths on the order of 130 feet (39 m) have been constructed. They are routinely placed to withstand a design load of 200 tons, (182 metric ton) or more. The auger-cast method can be used to construct individual piles, including batter piles, or any combination or groups. In fact, the method has been used extensively to install secant piles so as to form a continuous wall for earth retention (Figure 6B.133). The technology for auger-cast piles is now well developed and several specialty contractors regularly install them. The Deep Foundations Institute (1993) has available a reference, *Augered Cast-in Place Piles Manual*, which discusses the process in detail and includes a guideline specification.

**FIGURE 6B.132** Casting of an auger-cast pile.

## 6B.7.2 Mixed-in-Place Piles

Mixed-in-place piles involve rotating a hollow drill stem into the ground to the proposed pile tip elevation. The stem is equipped with suitable outlet nozzles and mixing blades of the desired pile diameter. A cementitious suspension grout is pumped down the hollow stem while it is slowly inserted and simultaneously rotated so as to initially blend the grout into the in situ soil. Rotation continues as the apparatus is slowly withdrawn from the hole, further mixing so as to enhance the initial blending. As originally conceived in the early 1950s, the finished pile would be composed of a mixture of the native material with a cement suspension grout. The piles can be formed in a range of soil materials, as long as they are not very stiff or very dense and are free of large rocks or boulders. Strength of the resulting piles is dependent on the amount of grout blended and the gradation of the in situ soil.

In essentially clean sands that are reasonably free of silt or clay fines, strengths similar to those of conventional mortars are routinely obtained. As the amount of fines increases, however, the resulting strength levels decrease. Auger-cast piles have many advantages. They can be installed quickly, and the installation is relatively free of noise and vibration. Because the hole is always filled with either soil or the finished pile, neither groundwater nor caving soil is a problem. Mixed-in-place-piles were developed by the Intrusion-Prepakt Company in the early 1950s and were the forerunner of the much larger-scale process now known as DMS (deep soil mixing). Traditional mixed-in-place-piles were commonly 16 inches in diameter with depths up to about 50 feet (15 m). Because of the method of formation, they provide a very tight and irregular interface with the surrounding soil, and capacities are thus often greater than equivalent driven piles.

**FIGURE 6B.133** Shoring composed of cantilevered auger-cast secant piles.

### 6B.7.3 Deep Soil Mixing

Deep soil mixing involves the simultaneous injection and mechanical mixing of a cementitious suspension grout with in situ soil. It is applicable to any soil that is not very stiff or dense and is free of boulders. The process requires very large and sophisticated equipment and overhead clearance that must be greater than the depth of the mixed soils. This limits its application to generally large projects. The process, as now practiced, was developed in Japan in the early 1970s and has been used in the United States since 1986. The grout typically used is a cement suspension, which might contain a small amount of bentonite (less than 5%).

Deep soil mixing can be used for the construction of individual elements such as columns or short walls, although its most common application is to construct continuous walls. These may be either structural, such as for earth retention, or hydraulic for cutting off the underground flow of water or other liquid. The size of the element produced varies with the equipment used, but thickness is usually between three and five feet (0.9 and 1.5 m), with the width of a single probe on the order of three times the thickness. Common depth limits are on the order of 100 to 150 feet (30 to 45 m), although more than 200 feet (60 m) have been produced. The process is highly mechanized and employment of high-capacity, fully automated grout plants is common. It is thus performed only by a limited number of specialty contractors, most of which have their own proprietary equipment and methodology.

### 6B.7.4 Micropiles

Micropiles are formed by drilling of the pile hole, usually with simultaneous placement of a stay-in-place casing. Structurally, they are thus a composite of the casing and a grout fill. They are typically designed as friction piles, and their intimate contact with the surrounding soil is thus critical. Micropiles are of relative small diameter, usually less than 10 inches (25 cm), and have been placed to depths of about 200 feet (30 m), although more typical depths are on the order of 100 feet (60 m) or less. The casing is usually composed of appropriate lengths of threaded flush wall sections. In order to accommodate the flush threaded joint and the structural requirements of the particular design, fairly thick casing walls are typical. Additional reinforcement can be embedded into the grout fill before it hardens, if required.

Once the pile is drilled to depth, a cementitious grout is injected, oftentimes through the drill rig mud swivel. Although cement suspension grouts are the most often used, ready-mixed mortars have also been employed. Typically, the drill bit is of greater diameter than the outside diameter of the casing, resulting in an annular space, which must be filled with grout, so as to provide the intimate contact with the soil required of a friction pile. The grout is usually injected into the casing, with injection continuing until it has returned up the annulus to the surface. The casing is nearly always custom fabricated, and can be of any length, which allows the piles to be placed in areas of restricted headroom. Figure 6B.134 shows an installation of closely spaced, 200 foot deep micropiles in an area of restricted access. Because of the requirement of special equipment and skills, micropiles are generally constructed by a limited number of specialty contractors, many of whom have their own proprietary systems. The work is often performed on a design–build basis, with performance load testing of the design pile capacity the primary acceptance requirement.

### 6B.7.5 Posttensioned Soil Anchors

The pullout capacity of soil anchors can be greatly enhanced by injection of grout in the anchorage zone. This is the result of both the increased intimacy at the anchor–soil interface and, often, an improved soil in the anchorage zone. Although cement suspension grout is most commonly used, chemical permeation grouting has been employed in sandy soils. The grout is usually injected to the tip of the anchor, either through a separate injection tube or the anchor itself, in the case of hollow stem anchors. Construction of ground anchors is somewhat specialized, and grouting in connection therewith is usually performed by the anchor contractor.

## *6B.8 INJECTION FUNDAMENTALS*

Regardless of the type of grout being used or the purpose of its injection, there are several important parameters that will always be present and must be considered. A good understanding of these factors and their relationships are thus fundamental to proper design or field performance of any grouting program. The pore pressure of the soil into which grout is being injected comes under increased pressure and, oftentimes, the soil is subjected to mechanical disruption as the grout is introduced. It is important to understand these factors, and to make prompt adjustments in the injection parameters as required, if excessive disruption or damage is to be precluded.

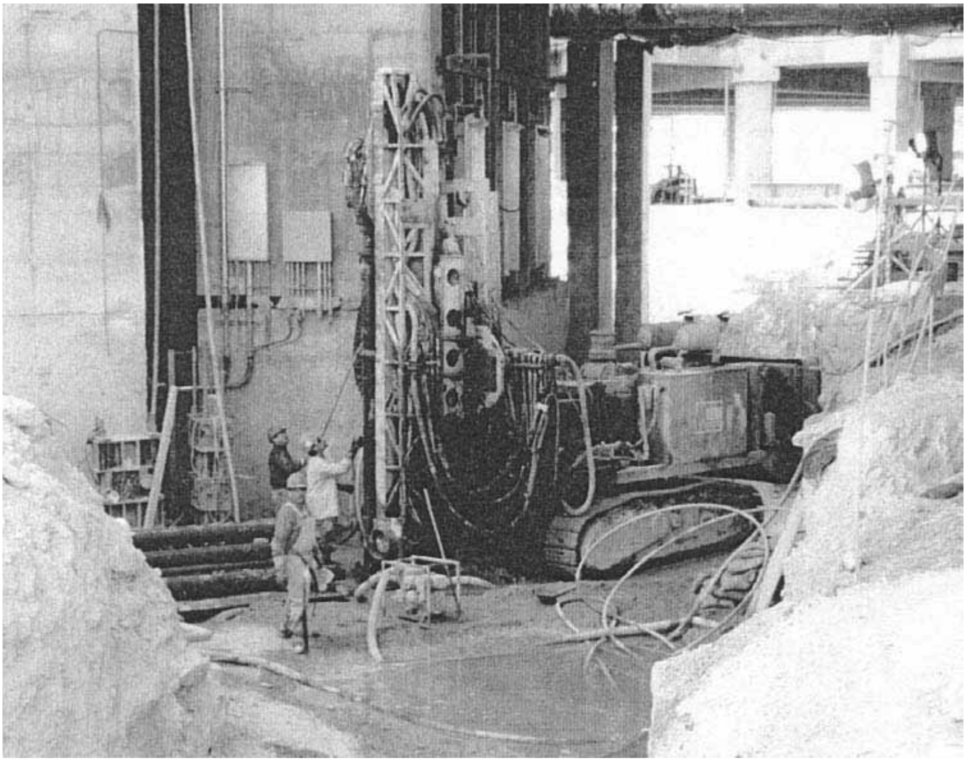

**FIGURE 6B.134** Construction of micropiles in an area of restricted access.

Except in those cases where continuous monitoring of previously installed geotechnical instrumentation occurs, the only parameter that provides timely information is the injection pressure, and more particularly, its behavior during injection. Although a given pressure level in itself is of little value, knowledge of the pressure variations at a constant pumping rate usually reveal much valuable information as to the constantly changing conditions in the soil at the point of injection. The pressure is directly effected by both the grout rheology and the pumping rate. Whereas changing the grout rheology during injection usually requires a time delay, adjusting the injection rate can be done instantly, providing proper equipment is being used. Continuous observation of the grout pressure behavior and its control through appropriate variation of the pumping rate is necessary if optimal effectiveness of the grout injection is to be realized.

## 6B.8.1 Injection Pressure

Setting of the maximum injection pressure is perhaps one of the most contentious issues in geotechnical grouting, especially in the United States. A widely recognized rule of thumb says that the maximum pressure in psi should be limited to twice the injection depth in feet. Well-documented experience, in the United States as well as abroad, however, has proven this idea to be faulty. The developed pressure is directly proportional to the rate of injection. For reasons of economics, use of the highest *safe* injection rate, and thus highest pressure, is desirable. This pressure level will be variable and is dependent upon many factors, including the type and rheology of the grout being used, the purpose of the injection, particulars of the site soil, hole geometry, etc.

The pressure used in grouting can be divided into two distinct modes: that which results from friction and any restriction within the delivery system (pump output to lower end of casing), and that which results from resistance to permeation or displacement of the formation or other medium being filled. Where injection is through a void of uniform cross section, such as a pipe or injection hose, the total pumping pressure will be dependent upon the length thereof, and will be the product of the unit length resistance times the number of units of length. Unless a void is filled with water or isolated such that air cannot escape, pressure at the leading edge of the grout will always be zero. Where injection is made into a porous mass such as soil, or voids of varying size and configuration, the pressure distribution within the grout mass will be ever variable. It will, however, always be greatest at the point of injection, which is typically the lower end of the injection casing, and will decrease with distance therefrom.

Because the effective pressure on the formation will invariably start at the casing outlet, it is important to consider the pressure differential between that point and the top of the casing, where pressure readings are typically observed. This value may be either positive or negative, depending upon the particulars of both the grout and the delivery system. As an example, on a recent compaction grouting project, the contractor proposed use of a six inch (150 mm) diameter injection casing for some very deep holes. Because of concern that the static head pressure might be excessive, a trial injection using the six inch casing was made. A positive grout pressure of 0.45 psi (0.3 kPa) per vertical foot (0.33 m) of casing resulted. Because the total gravity pressure that would result at the bottom of the hole would be excessive, a smaller three inch (75 mm) casing was required for the production work. This resulted in a head *loss* of about 0.45 psi (0.3 kPa) per vertical foot.

With the above in mind, it is possible, through experimentation and calculation, to determine the *approximate* pressure differential between the pressure gauge at the point of injection and the casing outlet, where the grout makes first contact with the formation. The term approximate is used because some variation will occur within a delivery line, depending upon the smoothness of its interior, number and sharpness of bends, and any restrictions at couplings or other fittings. Because it is the pressure at the bottom of the injection casing that is pertinent, the gauges for pressure evaluation should always be located as close as possible to the point of grout entrance into the formation, and the injection pressure corrected for any loss or gain.

Many factors contribute to the total pump pressure required for injection. Major contributors to that pressure are frictional resistance within the delivery system, rheology of the grout being pumped, rate of pumping, size of the lines and resulting grout velocity, and frictional resistance at the grout conduit wall interface, which is a function of the wall smoothness and its affinity for the particular grout being used. Once the grout is in the medium to be filled, both the size of the individual fractures or pores and their surface condition are major factors. Obviously, the rheology of the grout and its affinity for the formation surfaces are also significant contributors.

### 6B.8.2 Pumping Rate

As aforementioned, the pumping rate and developed pressure are directly related. An increase in pumping rate will *always* result in an increase in pressure, as will a reduction result in lower pressure. This is well illustrated in Figure 6B.135, which is an actual printout of a computer-generated record of pressure behavior on a recent project where the grout pump malfunctioned. The piston speed varied such that one piston was traveling nearly twice as rapidly as the other. The actual pump strokes can be observed by their pressure differentials, and it will be noted that the pressure on the short stroke (higher pumping rate) was more than 100 psi greater than that on the slower long stroke. In this figure, it is also of note that the pressure raised slightly during each of the short strokes, which is indicative of a generally optimal pumping rate, whereas there was a slight pressure decay during the long strokes, which indicates a slower than optimal rate.

Because the resistance developed within the formation to be treated is beyond our control, the in-

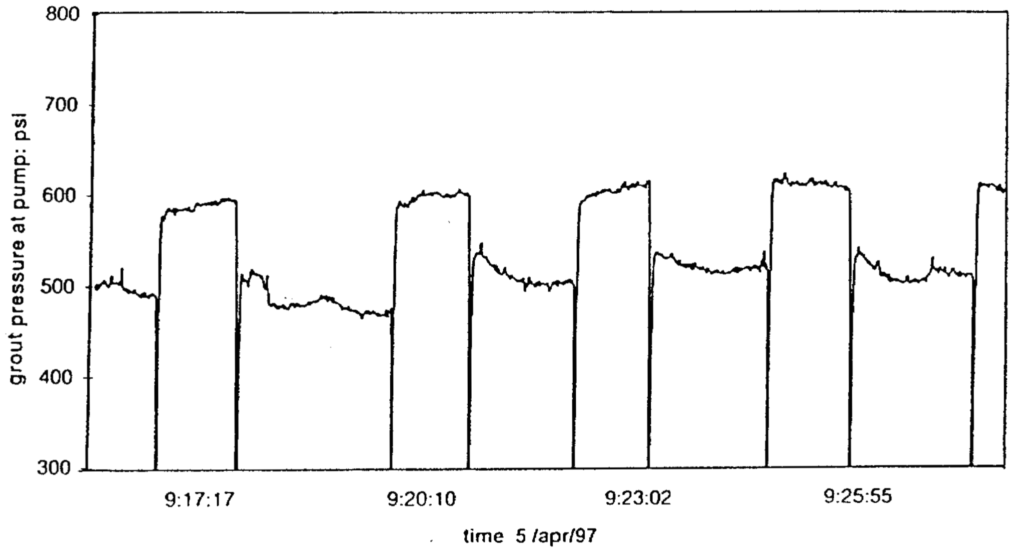


**FIGURE 6B.135** Computer-generated printout showing variation of pressure concurrent with changes in pumping rate.

jection rate selected must be limited, such that excessive pressure will not be developed. Because wide variation of formation resistance is common, flexibility of the output capacity of the pump is usually required. This is no doubt the reason for the past widespread use of circulating grout injection systems. In such systems, the grout is continuously circulated in a route from the pump, past the grout hole, and back to the pump, as shown on Figure 6B.136. A connection to the grout hole is made with a "T" fitting in the line. A valve is located on the return side of the "T" fitting and a valve and pressure gauge are located between the "T" fitting and the injection hole casing, as illustrated in Figure 6B.137. The amount of grout that is allowed to go into the hole, and thus the injection rate, can be varied by "throttling" the valves. Because economy dictates use of the highest practical injection rate, the valves are adjusted so as to allow the maximum amount of flow possible, without exceeding the maximum allowable pressure.

Single-line direct delivery systems, wherein the grout flow is directly from the pump to the grout hole (Figure 6B.138), should be used only with variable output pumps. Therein, the speed of the pump, and thus the rate of the grout output, is regulated so as to maintain injection rates that will not exceed the allowable pressure. Varying the rate in discrete increments provides the advantage of observation of even slight changes of pressure, facilitating optimum evaluation of the grout movement within the formation.

Grout pumps and delivery systems should be sized for reasonable injection rates, so as to preclude the development of excessive pressures during injection. All other things being equal, system pressures will usually diminish as the pipe or hose size increases. The size should not be so large, however, as to preclude complete emptying of the system within a reasonable time. In this regard, the set time of the grout must be considered so that it is not allowed to react within the system. Many grout materials, and especially cementitious mixes, will tend to build up on the wall of the delivery system if sufficient grout velocity is not maintained. Such build-up results in an ever decreasing opening for the grout to travel, and will thus increase both the grout velocity and resulting pressure. Figure 6B.139 delineates the velocity of grout through various size delivery lines.

The importance of the relationship of injection pressure to pumping rate cannot be overempha-

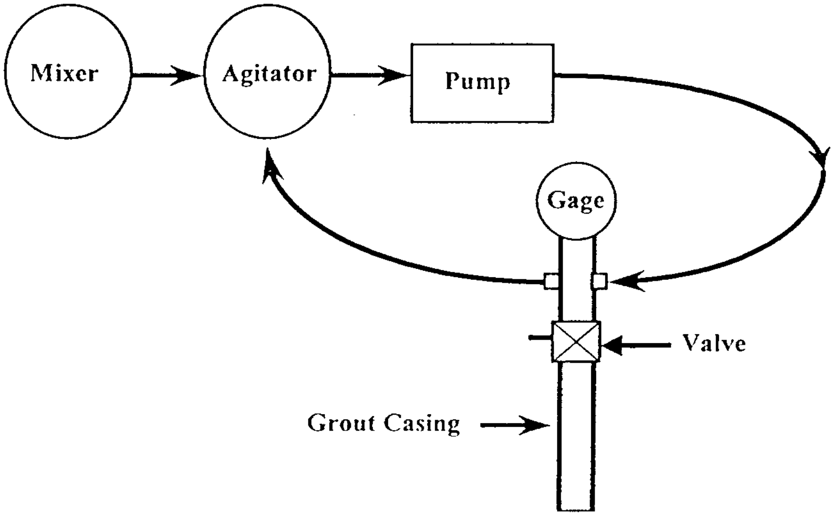


**FIGURE 6B.136** Circulating injection system with grout return to the agitator.

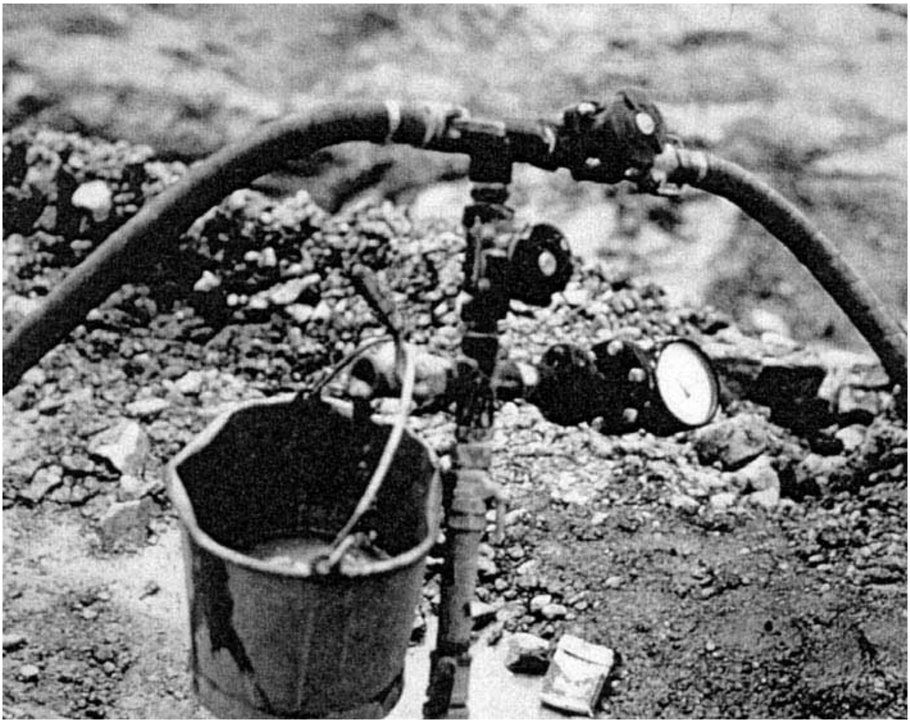

**FIGURE 6B.137** Valves are manipulated to maintain proper pressure.

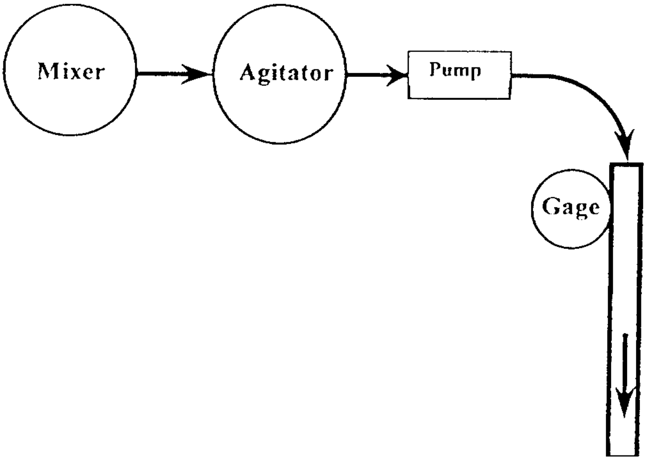


**FIGURE 6B.138** Direct injection system where grout flows from the pump directly to the grout hole collar.

## GROUT VELOCITY IN VARIOUS SIZE LINES

| FLOW Cubic Feet/Minute | FLOW Gallons/Minute | VELOCITY Feet/Second | FLOW Cubic Feet/Minute | FLOW Gallons/Minute | VELOCITY Feet/Second |
|---|---|---|---|---|---|
| **1/2 Inch Line** | | | **1 1/4 Inch Line** | | |
| | 0.5 | 0.8 | 0.5 | 3.8 | 0.7 |
| 0.1 | 1 | 1.6 | 1 | 7.5 | 1.4 |
| 0.3 | 2 | 3.3 | 1.5 | 11.3 | 2.0 |
| 0.4 | 3 | 4.9 | 2 | 15 | 2.7 |
| 0.5 | 4 | 6.5 | 3 | 22.5 | 4.1 |
| | | | 4 | 30 | 5.4 |
| **3/4 Inch Line** | | | **1 1/2 Inch Line** | | |
| 0.1 | 1 | 0.7 | .50 | 3.8 | 0.6 |
| 0.3 | 2 | 1.4 | 1 | 7.5 | 1.3 |
| 0.4 | 3 | 2.2 | 1.5 | 11.3 | 2.0 |
| 0.5 | 4 | 2.9 | 2 | 15 | 2.7 |
| 0.7 | 5 | 3.6 | 3 | 22.5 | 3.5 |
| 0.8 | 6 | 4.4 | 4 | 30 | 5.3 |
| 0.9 | 7 | 5.1 | | | |
| 1.1 | 8 | 5.8 | | | |
| **1 Inch Line** | | | **2 Inch Line** | | |
| 0.3 | 2 | 0.8 | 1 | 7.5 | 0.7 |
| 0.5 | 4 | 1.6 | 1.5 | 11.3 | 1 |
| 0.8 | 6 | 2.5 | 2 | 15 | 1.4 |
| 1.1 | 8 | 3.3 | 3 | 22.5 | 2.2 |
| 1.3 | 10 | 4.1 | 4 | 30 | 2.9 |
| 1.6 | 12 | 4.9 | 5 | 38 | 3.6 |
| 1.9 | 14 | 5.7 | 6 | 45 | 4.4 |

**FIGURE 6B.139** Velocity of grout through various sized delivery lines.

sized. Many practitioners seem to fall under the *erroneous* impression that the maximum grout pressure level alone indicates the degree of soil improvement obtained. In many cases of investigating the cause of grouting program failures, the writer has found records that indicated only the hole number, grouting depth, and *maximum* pressure reached during injection. Such records are usually short of useless for obtaining an understanding of what actually happened in the ground, and thus the cause of the failure. The pumping rate/pressure relationship is so important that it warrants special attention. The specifying or recording of injection pressure is of absolutely *no* use unless the injection rate at which the pressure level occurred is also specified and/or recorded.

### 6B.8.3 Pressure Behavior

A basic law of physics tells us that an increase in pressure *always* indicates greater resistance to flow, whereas a reduction indicates less resistance. Although somewhat subjective, knowing the value and nature of the pressure movement allows one with experience to make an informed prediction as to the cause. As an example, if we were to assume that a pressure of 100 psi (700 kPa) was required to pump a grout material through a given length of hose, should a connection located in the middle of the length become broken, the pressure would sharply drop to approximately half of its original value. Similarly, were a restriction placed on its outlet end, a sudden increase in pressure would result. Applying this theory to the real world of grout injection, it is widely recognized that a sudden reduction in pressure indicates a likely disruption, in the interior of the soil or rock mass being grouted (Houlsby, 1990), (Warner and Brown, 1974).

Within the writer's experience, sudden changes in pressure, or increases of injection rate at a constant pressure, *always* indicate the occurrence of a significant event. In more cases than not, such events lead to negative results, which can be greatly minimized, and in many cases completely averted, through quick response, usually the immediate lowering of the pumping rate or complete cessation of injection.

Typical events that are accompanied by a loss of pressure include hydraulic fracturing of the soil; displacement or heaving of the formation; grout loss into a subsurface pipe or other substructure; outward displacement of a downslope or retaining wall; grout entering a much larger fracture or void, or encountering a much softer or more permeable formation; thinning of the grout or other change in the grout rheology, which increases its mobility; leakage of the grout; or pump malfunction.

An interesting example was observed during the injection of a very deep compaction grouting hole. A sudden drop in pressure was noted. Consideration of the value of the pressure drop, combined with the depth of injection and the injection rate, ruled out hydraulic fracturing of the formation as the cause. The particular circumstances of the injection led to the conclusion that some sort of break in the injection line was the most plausible source of the sudden pressure loss. The writer assumed that a casing joint at a fairly shallow depth had failed. After withdrawal of forty feet of casing, however, a vertical split, as shown in Figure 6B.140, was found. Whereas there were no underground pipes in the area of the injection, if such had existed, the indicated pressure behavior could have indicated leakage of the grout therein. The importance of meticulous monitoring of the injection behavior cannot be overemphasized.

Events that are preceded by an increase in pressure include plugging or restriction in the injection system or formation, thickening or lowering the mobility of the grout, completion of the filling of a fracture or void, or binding or wedging of a structure or slab that is being raised, (Committee on Grouting, 1977)

The compaction grouting procedure employs relatively high pressures, even at shallow depths. This results in relatively large pressure variations that are especially instructive. Significant pressure movements develop during injection, as conditions within the zone of influence of the growing grout mass change. Where the defective soil is of a uniform though inadequate condition, such as in the case of very loose wind-deposited granular materials, the pressure buildup will generally be constant, as illustrated in Figure 6B.141.

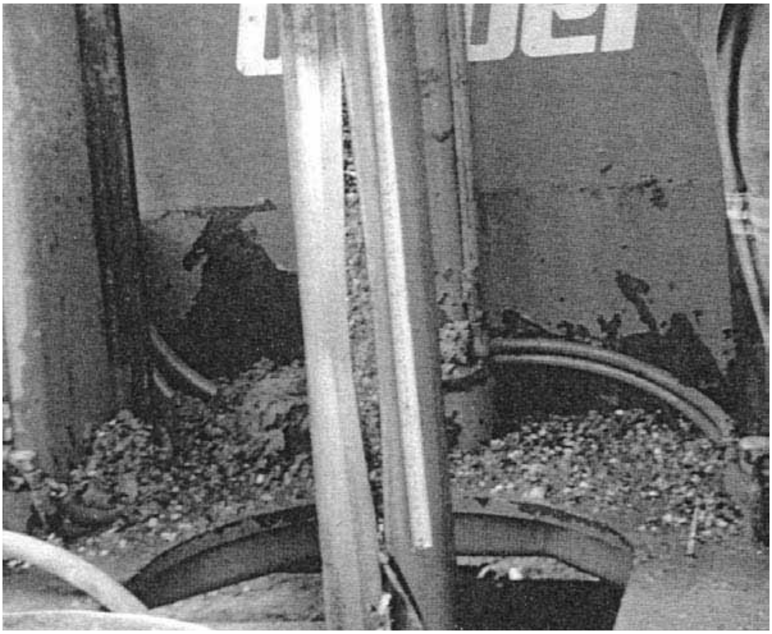

FIGURE 6B.140 Split of casing that resulted in sudden drop of injection pressure.

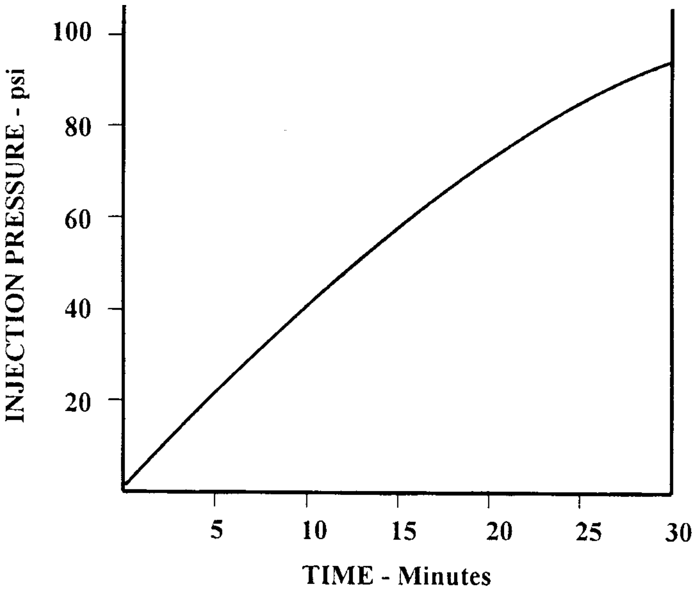


FIGURE 6B.141 Steady pressure buildup indicates generally uniform soil.

Often times, however, settlement problems develop as a result of defects such as masses of buried brush or large boulders in or under a fill, uncompacted soil resulting from initial clearing or development of haul roads, or insufficient removal of slopewash or other weak surface materials prior to fill placement. Both the existence and approximate volume of such subsurface defects can be determined through continuous pressure behavior monitoring. In such cases, a nonuniform pressure buildup will be experienced (Figure 5.B.142). In the figure, the approximate volume of the apparent defect can be ascertained by determining the amount of grout that was injected from the time of the initial pressure drop until the previous pressure level is regained.

As soil grouting is frequently performed under or in close proximity to existing structures, underground piping and other substructure are often nearby. There is thus inherent with the procedure always a risk of grout leakage into such utilities. The occurrence of grout entering substructures will nearly always be preceded by a significant drop of the injection pressure. Whereas such drops commonly indicate other events, in cases where grout has entered an underground pipe a slow but steady increase of the initial entrance pressure will typically occur. In such cases, the level and behavior of the pressure will usually be markedly different from that which was experienced prior to the occurrence of leakage, signaling the need for corrective action.

Whereas major events can be detected by substantial pressure movements or changes of injection rate, more subtle events often result in only minor changes. Such minor changes cannot easily be observed in circulating injection systems or those systems that are subject to pulsations from pump stroking. Thus, where detection of minor events is important, use of a constant output pump, free of any significant pulsation, combined with careful and continuous pressure monitoring is recommended. Piston pumps operating at rates greater than about one hundred strokes per minute, combined with a minimum hose length of about 100 feet (33 m), have been found satisfactory, as at high rates the stroking results only in a flutter of the gauge needle, and dampening occurs in the flexible hose. Pressure pulsations resulting from stroking of piston or diaphragm type pumps can

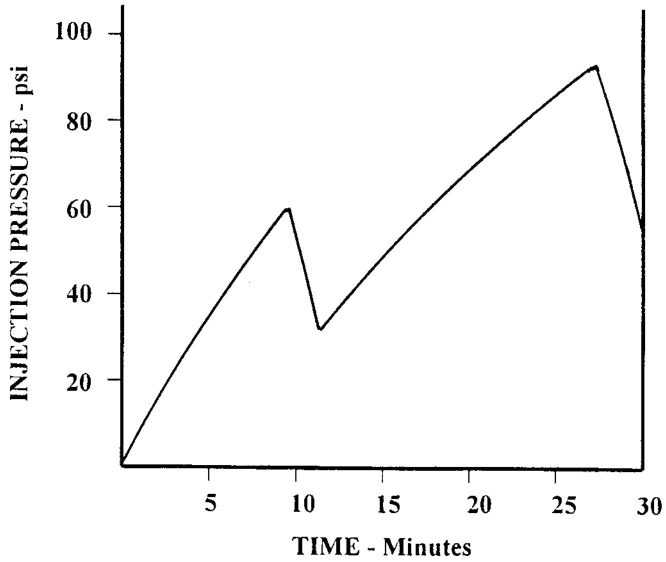


**FIGURE 6B.142** Erratic pressure buildup indicates subsurface voids.

also be lessened through use of a hydraulic accumulator near the pump outlet and/or an increase of at least 50% in the diameter of the injection hose for a distance of about 25 feet (7 m), starting at the pump outlet. Conversely, as illustrated in Figure 6B.135, the actual pressure of individual pump strokes can be observed in the case of piston pumps operating at very slow stroke rates.

Pressure behavior can be obtained through continuous readings of a pressure gauge and manual entries onto an appropriate form, or with fully automated equipment employing either a continuous disk or strip chart recorder, or with computer processing. Regardless of the recording method, the actual time of each entry should be included, and when feasible, continuous real time should be included in the record. When manual monitoring is performed, it is good practice to record entries at predetermined uniform increments of pressure. Alternatively, entries can be made at regular time intervals. The magnitude of pressure increments or time intervals employed will vary according to the individual application and type of grout being used, but should be of sufficient frequency to allow plotting of a curve that accurately displays all significant pressure movements.

Although disk recorders are commonly used with automated systems, especially in Europe, and are satisfactory for monitoring pressure behavior, continuous strip chart recorders can include other injection parameters that are more easily compared and interpreted and are thus favored by the writer. Regardless of the method used, the records produced must include real time and facilitate immediate interpretation on the job site, as well as provide a permanent record for later reference.

Computer monitoring systems are now readily available that provide for instantaneous readout of all injection parameters. They have the advantage of providing a permanent record on disk and the ability to print hard copies as desired. When using traditional practice employing circulating systems, the constant variation of injection rate virtually precludes accurate manual recording. The use of real time, continuous computer monitoring is thus especially beneficial in such cases.

### 6B.8.4 Pressure–Volume Relationship

The quantity of grout pumped at any one location and the shape that it forms in the soil have a significant effect upon the soil reaction. A given pressure on a very large mass of grout will affect a greater area, and thus exert a much larger *total* force within the soil, than will a small mass (Figure 6B.143). Depending upon the particulars of the individual injection, this might affect the pressure level that can be safely used. Where large quantities of grout are placed, the pressure used for initial injection may be too high once a significant mass of grout has been placed. Therefore, reduction of the pumping rate, and thus pressure, is often in order during injection as the injected quantity of grout grows.

The shape of the injected grout mass is also important. As an example, if the grout were to form an essentially horizontal lens, a large surface area would be affected, and the likelihood of displacing the surface upward would be great. If a vertical fracture were created, any effect of the grout would likely be adjacent to the plane of the fracture, rather than in the planned zone adjacent to the hole. As previously discussed, initiation of *controlled* groutjacking usually requires a relatively low-mobility grout. Jacking can occur with any grout, however, if the injection rate exceeds the rate at which the soil is able to accept it. Much damage has been done as a result of *uncontrolled* jacking resulting from too great a pumping rate and resulting excessive pressure, especially when combined with a large or adversely shaped grout mass.

## 6B.9 PLANNING A GROUTING PROGRAM

All too often, grouting is considered only after a problem has developed, with the work proceeding hastily and lacking rational planning. Whereas grouting can often solve soil problems effectively, it is an established technology, and should be planned just as any other geotechnical construction. The purpose of the grout injection, and clear objectives, should precede any injection. Pumping of grout

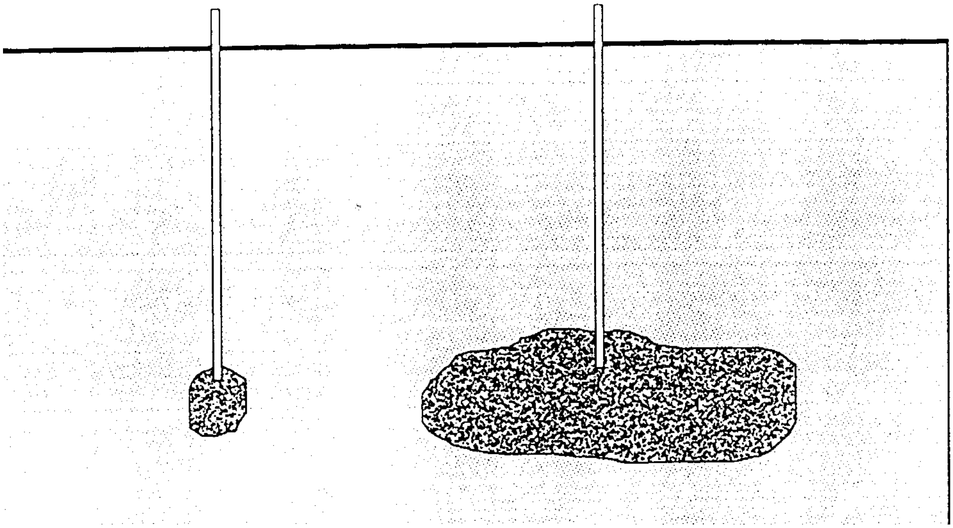

**FIGURE 6B.143** A large grout mass exerts a much greater total vertical uplift force than does a small mass.

without a good understanding of the particulars of the site is risky at best. This is particularly pertinent on improved sites and especially where extensive subsurface improvements exist.

### 6B.9.1 What are the Objectives?

Clear objectives should be established as part of any grouting program. Why is the grouting being done, and what properties are expected of the improved soil? Is the required improvement to be permanent, or is it temporary, such as solidification of a cohesionless soil to enable tunneling or other construction? If the grouting is being done to increase the existing soil density, what density is required? How will it be confirmed? Should the grouting be done to reduce the flow of water through the soil, how much reduction is required, and under what conditions? The precise purpose and objectives of the proposed grouting program should be thoroughly considered and expressly stated prior to commencement of the work.

Where used under emergency conditions, such as reducing or correcting piping of water through an earth embankment or settlement around an active sinkhole, both the time available for consideration and the selection of methods and materials is often limited. The objectives and associated risks should nonetheless be considered. Blindly pumping any grout into the ground can have adverse results, and such risks and ramifications should be recognized and considered prior to the start of the work.

Unlike most construction, the planning of a grouting program should not end upon start of the work. A great deal of information, relative to the existing soil conditions, can be obtained during both the drilling and injection. In a sense, each grout hole can act as an exploratory hole, complimenting the original exploratory program. This will often suggest beneficial changes to the originally conceived program and will provide a better finished product or possible cost saving. In cases of remedial grouting, the writer has in many instances determined the actual cause of the problem for which the grouting was being done solely as a result of close observation of the drilling and grout injection behavior. In this regard, grouting contract provisions should always allow for and facilitate possible changes in the work, as dictated by the information gained during progress of the work.

### 6B.9.2 Access/Surface Improvements

Of all the available soil modification methods, virtually none can be performed with less disturbance than grouting. Although many, including a number of grouting contractors, opine that the procedure is inherently messy, such need not be the case. The writer has been involved in more than 100 projects where grouting was done inside structures that remained fully occupied and operational throughout the work. In many instances, normal operations were ongoing in areas only a few feet from the grout injection location. An extreme case is shown in Figure 6B.88, where compaction grouting is being done immediately adjacent to sensitive equipment in the Spalding Laboratory at the California Institute of Technology. Unlike most other remedial methods, large equipment is not required at the injection site, and that which is can be free of excessive noise, vibration, dust, and other annoying elements. Access or other restrictions will, however, affect the manner in which the work progresses.

It is important to consider the layout, construction, and condition of any existing structures when contemplating a grouting program. Obviously, any requirements for occupancy or continued use of the affected areas, must be considered. Drilling of grout holes in soil need not require large drill rigs, and in fact, some contractors routinely use hand-held drills for holes to depths up to 100 feet (33 m) or more. Most grout mixtures can be pumped a distance of at least several hundred feet, so the grout mixer and pump need not be located very near the area of injection. Provision must be made for routing of the grout delivery lines; however, they can be run over or under any obstructions or through areas that must be maintained in operation. Where walls or other solid elements block access to the injection site, small holes can be cut through them for access of the grout hose and other utilities.

Work inside structures or where cleanness is of particular importance does result in increased costs, but the increase should usually not be great. When grouting is being considered under such conditions or in other sensitive locations, consultation with experienced grouting contractors with a proven reputation for orderly work is advisable. The condition and especially any defects within the structure should obviously be observed and noted prior to commencement of the work.

### 6B.9.3 Subsurface Structures and Utilities

Good field practice dictates that a careful inspection of the area, and the proposed grout hole locations in particular, be made prior to the start of drilling. Grout hole layout might disclose conflicts with existing improvements or underground substructure. In this regard, it is especially important to locate any existing conduits, pipes, or drains. Electrical and communication conduits usually run in fairly straight lines between switches, outlets, or other exposed features. Placing of grout holes in such locations is not advisable unless the exact line locations have been positively identified.

The location and proper functioning of sewers and drain lines should also be established. Even where the location is known, it is a good idea to run water into the system, from upstream of the area to be grouted, and observe its flow at a downstream location, such as a clean out or manhole. In some cases where downstream access is not readily available, excavation to the top of the pipe and selective opening of a hole therein might be advisable. Where grout holes are located near such lines, continuous running of the water during grout injection, combined with occasional observation of the downstream opening, is prudent. Should grout be entering the line, a change in the color of the water will be observed prior to any extensive filling with the grout. This will allow relatively easy clearing of the grout by immediate flushing with water.

Drilling should also be carefully monitored as it can disclose unanticipated objects, especially if the work is in or near a structure. Gravel is often used to provide drainage or as bedding for pipes. Penetration of such gravel could indicate existence of a subdrain or pipeline. Encountering unantic-


**FIGURE 6B.144** Much can be learned from observation of the drill cuttings.

ipated concrete is always of concern. It could be an underground structure or encasement for conduit or piping. Encountering organic matter at unanticipated depth is usually significant and can often indicate the cause of settlement, leakage, etc. By careful observation of the project site and structures and retrieved drill cuttings, as illustrated in Figure 6B.144, and continuous vigilant observation of the injection behavior, a great deal can be learned about the foundation soil. In cases where faulty conditions are being remediated, such diligence often provides a greater understanding of the cause of the problem that the grouting is intended to correct.

## 6B.10 CONDITION ASSESSMENT/GEOLOGICAL CONSIDERATIONS

Injection of grout into soil, absent a good understanding of the existing geology or soil conditions, is risky at best, and in some cases can have serious negative consequences. Grout injection will often affect movement of water through the zone of treated soil, and can in some cases, especially where chemical solution grouts are used, change the groundwater levels. This can result in changes of the state stresses within the soil, which might affect their future performance. Additionally, all grouting adds weight to the treated soils. The magnitude of the increased body load can be significant with some grouting methods such as compaction grouting. It is thus crucially important to assure that any soils underlying the proposed grouted mass are capable of supporting the additional weight of the grout.

Where grouting is being considered in regard to structural settlement, a level survey should be provided, with sufficient entries to enable a good understanding of the settlement patterns and differentials. Where the structure has a slab on grade floor, it is especially useful to provide relative elevation contours thereof, as illustrated in Figure 6B.145. This can be readily accomplished with use of a manometer system (water level). In such an operation, a reservoir of water or other fluid is placed at a generally central location and appropriate elevation (Figure 6B.146). One end of a small flexible tube, usually ⅜ or ½ inch (10 or 13 mm) in diameter is connected to the reservoir, with the other end attached to a rigid rod that is provided with a ruler, as illustrated in Figure 6B.147. Fitting of a hardened sharpened point to the bottom end of the rod, as shown in Figure 6B.148, greatly reduces the time required for such level surveys, in that it can be poked through carpet or between sections of floor covering. This obviously negates the requirement for elevation correction calculations.

Information relative to cracking or other signs of structural distress should also be provided. Where significant cracking exists, it should be mapped prior to the grouting. Obviously, original construction documents, including any geotechnical reports, plans, photographs, and such, if available, should be reviewed and also be made available to the grouting contractor. Record or "as built" drawings can be useful; however, their accuracy should not be relied on. These documents are often produced only after construction has been completed, as a requirement for receiving final payment. The writer has witnessed a large number of instances where information provided on as built drawings proved to be grossly wrong. As a general rule, it is reasonable to hold the grouting contractor responsible for any damage to underground lines for which the accurate location has been provided. Conversely, it is not reasonable to for the contractor to accept responsibility for

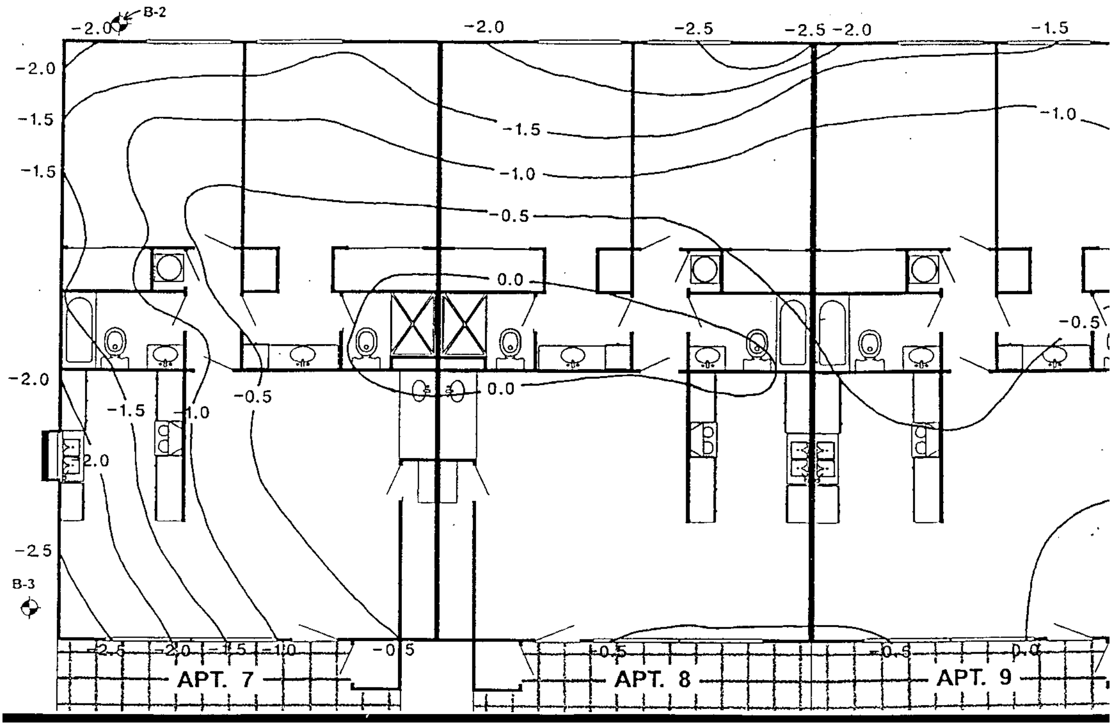


**FIGURE 6B.145** Contour map of the mode and magnitude of settlement of a typical structure.

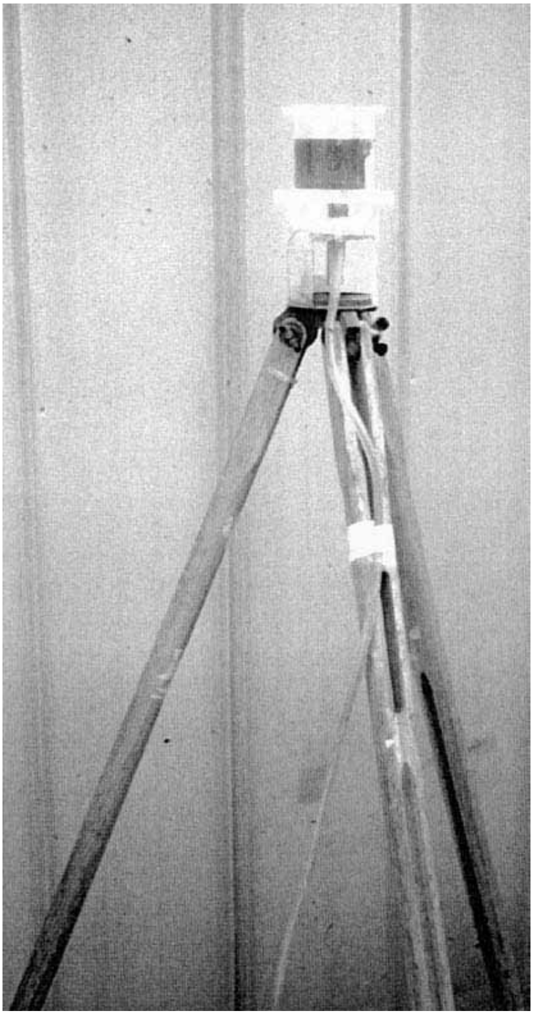

**FIGURE 6B.146** Manometer reservoir is centrally located in the survey area.

damage to such improvements if their location is unknown or incorrectly indicated. Refer also to sections 7B.7, 7B.D, and 9A.

### 6B.10.1 Soil Type and Properties

Grouting is performed in soil for one of two reasons—either to repair some fault of the soil, or to change its performance. In either case, a rational grouting program cannot be formulated without a good knowledge of the existing soil properties, boundaries of the zones requiring improvement, and any faults therein or adjacent thereto. Selection of the grouting method, the particular grout formu-

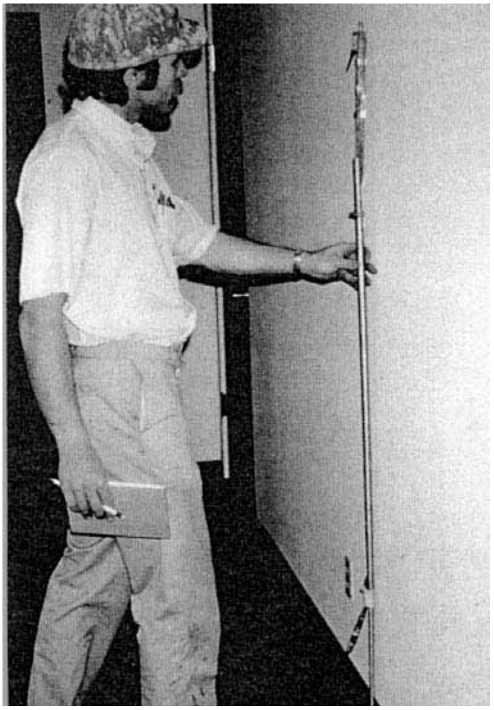

**FIGURE 6B.147** Elevation readout is from ruler attached to rod.

lation to be used, and the parameters of injection are dependent upon the soil structure and condition. In some instances, more than one grouting method and/or material, will perform equally satisfactorily for a given condition, whereas in other cases only one will be clearly superior or appropriate. Effect of the grouting on the underlying and adjacent formation can only be determined with knowledge of the particulars thereof. Thus, unless the properties of the soils are well understood by prior experience, exploratory borings or excavations, penetrating not only the areas to be improved, but those adjacent as well, and most importantly, those underlying the zone to be grouted, should always be assessed. A thorough investigation for a grouting program is an activity that is almost always beneficial to proper planning and engineering. Such performance nearly always *pays*, in lower ultimate cost and greater effectiveness of the work. In the long term, *it does not cost*.

A sufficient number of borings or excavations should be made to enable a reasonable, basic understanding of the site soils. Reported data should include the field soil classification, consistency, moisture content, groundwater elevation, sampling method, and Standard Penetration Test (SPT) or Shelby tube blow count. The frequency of sampling will be dictated by the amount of soil variation within the borings, the objectives of the proposed grouting, and the degree of advanced certainty as to grouting costs required by the client. As previously discussed, once injection commences, every grout hole provides further information about the in situ conditions. Sufficient sampling must be provided before grout injection begins, however, to enable determination of the most appropriate grouting program and the general method and materials to be used. Obviously, the location and dimensions of the borings and the drilling and sampling methods used should also be provided.

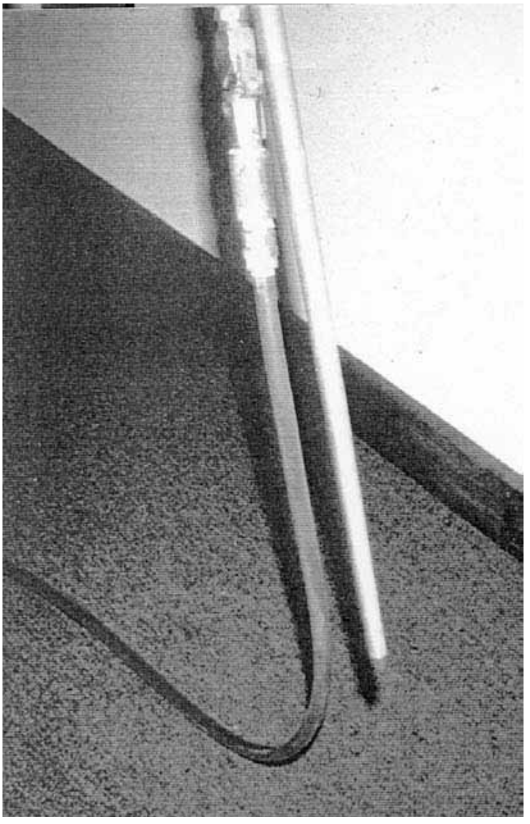

**FIGURE 6B.148** Sharpened steel point eliminates correction calculations necessary for carpet and many other floor coverings.

Where the purpose of the grouting is to strengthen the soil or increase its bearing capacity, Cone Penetration Tests (CPTs) are often faster and less costly than SPTs in conventional borings. They can provide good data and are very useful. On most sites however, at least one conventional boring should be provided to allow for visual inspection and classification of the soil and provide specimens for laboratory evaluation. CPTs do provide an excellent means of evaluating the comparative condition of the soil, before as well as after the grout injection, and are especially useful for verifying the increase in density achieved by compaction grouting.

The purpose of the grouting will dictate the necessary laboratory tests. Where permeation grouting is anticipated, both dry density and grain size analysis should be provided. Where the quantity of minus 200 mesh fraction is significant, hydrometer tests should be performed so as to further classify that material. Permeability determinations are useful but it has been the writers experience that laboratory measured permeability is seldom reflected by actual field injection results. He attributes this to the inability to reasonably duplicate the in-place soil structure in remolded laboratory specimens and thus does not rely on them much. In this regard, full-scale field permeability tests should be performed in those instances where the penetrability of the soil must be known. When densification of fine-grained soil is expected, both Atterberg limits and consolidation test data are useful. In those instances where significant variation of the soil properties are found, frequent sampling and abundant laboratory testing are in order.

## 6B.10.2 Formation Stratigraphy

When permeation grouting is contemplated, it is important to understand the detailed stratigraphy of stratified soils, as this factor can have a dramatic impact on the grout distribution and penetration. The effect of stratigraphy is vividly shown in Figure 6B.149, which shows an extricated solidified mass resulting from the test injection of ultrafine cement grout into a sandy soil. The clear boundaries of the various strata and decreasing radial penetration of the grout with depth can be clearly observed in the lower portion of the grouted mass.

Grout will tend to flow to and through the more permeable soils, often completely missing those of lower permeability, unless special injection efforts are made. Also, grout penetration can be retarded and even completely stopped by layers of clay or other very low permeability soil. Even very thin seams of clay can block movement of the most penetrable of grouts. When injecting grout below the water table, the groundwater must be able to escape at a rate at least equal to that of the grout placement, and at the driving pressure of the grout.

It is not unusual for faulty fine or silty sands in need of grouting to exist over deposits of much more permeable clean and/or coarse-grained materials, which although quite adequate and not in need of improvement are significantly more permeable. There have been many instances where a chemical solution grout intended for permeation of the upper layer actually traveled to the underlying more permeable material and failed to solidify the soil zone needing improvement. Well-recognized injection techniques are available that can prevent the misapplication of grout in such cases, but the stratigraphy must be understood prior to the start of injection.

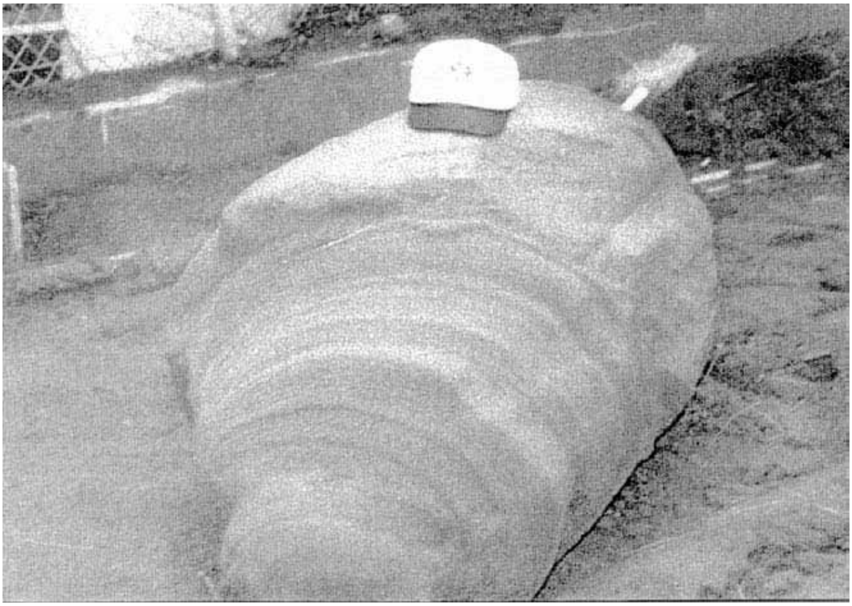

**FIGURE 6B.149** Extracted solidified mass from test injection showing effect of soil stratigraphy on the effective radius of grout injection.

### 6B.10.3 Groundwater

As previously discussed, the level of the groundwater can be important in the design of a grouting program. Chemical solution and fluid suspension grouts can be diluted by existing water in the formation. If that water is moving, the grout can be washed away from its intended zone of deposition. The setting time of most grouts is affected by temperature, and differences of temperature can exist above and below the groundwater elevation. In rare instances, unusual chemistry of the groundwater can affect the setting or hardening of the grout.

Depending upon the grout injection rate and pressure, the pore pressure in the vicinity of the injected mass might increase, which could raise the groundwater level, at least temporarily. This is why the ability of the water to drain from the zone of injection often dictates the acceptable injection rate. When grouting in sensitive situations, it is sometimes useful to monitor the groundwater level during and following the grout injection. If the water level is permanently raised as a result of the grouting, water movement at depth can be affected. As an example, a substructure that was leak free at the pressure head exerted from the original groundwater elevation before grouting began might leak at the increased head resulting from the higher water table elevation.

Compaction grouting is relatively unaffected by the presence of groundwater, as long as the soil is able to drain at least as fast as the grout is being placed. There could be an increase in the localized pore water pressure, which is entirely satisfactory as long as the soil continues to exhibit drained behavior. Should the pore pressures become so excessive that undrained behavior of the soil occurs, loss of control of the injection and hydraulic fracturing of the soil, as shown in Figures 6B.42 and 6B.46, would likely occur.

## *6B.11 INJECTION MONITORING AND EVALUATION*

The effectiveness of the grouting is often made obvious by the stoppage of visible leakage of water or jacking of a settled structure to proper grade. Verification of the results of much grouting, however, is not so obvious. As an example, where otherwise running soils have been solidified by permeation grouting to facilitate tunneling, it is desirable to know the completeness of the solidification prior to the tunnel heading reaching the grouted zones, which may not have been satisfactorily treated. Likewise, where the purpose of the grouting is strengthening of the soil at depth, where it cannot be directly observed, verification of the grouting effort is needed.

The completeness and quality of a grouting program begins with appropriate preinjection planning and a clear knowledge of the soil to be improved. It continues with careful and accurate monitoring of the work at all times during injection and indicated modification of the work as dictated by the observed behavior of the grout and the site of the work during grout injection. If these matters have been conscientiously attended to, the likelihood of a high-quality finished product are virtually assured. If, on the other hand, as is all too often the case, injection begins without a good knowledge of the soil, and little or no monitoring of the work occurs, less than quality performance is virtually certain.

In earlier times, monitoring of grout injection behavior was often made with methods and systems that lacked accuracy, and the reported results were usually subject to unavoidable delay. This made quick corrective action difficult at best and virtually impossible in many instances. Further, in the United States, designers and owner representatives have often relied on the grouting contractor to monitor and report on the work. This results in a delay of reporting and, sadly, experience has produced often questionable data. With modern computer technology, the earlier limitations have been eliminated. With present technology, there is no excuse for not properly monitoring the work and making timely adjustments so as to obtain a predictable quality product.

It must be realized that subsurface conditions do vary, and practicality and economics will virtually limit our understanding of the details of the beginning soil. Unknown or changed conditions, however, will be indicated by the grout behavior. If timely and appropriate monitoring and reporting procedures are employed, required changes can be made in an effective manner, without extraordinary increased costs. These can assure that the intended purpose of the grouting is being achieved and increase the knowledge of the starting soil conditions. It is recognized that there are many smaller projects where computer monitoring may not be practicable. In such cases, careful manual records should be kept and evaluated in a timely manner.

Continual observation of the ground surface and both overlying structures and substructures is imperative when grouting methods are used that might cause soil deformation or upheaval. With, compaction grouting in particular, a slight heave of the surface is often a criterion for limiting grout injection at a particular location. It is important in such instances that the upward movement in any one stage be very small, as the accumulation of many grouting stages can be considerable. In all grouting that is performed in urban areas, or where substructure might be present, careful observation is imperative to avoid leakage into or displacement of the substructure.

## 6B.11.1 Injection Behavior

Much has been said previously, relative to the importance of continuous monitoring, of the grout pressure behavior during injection. As the cost of the work is directly linked to the rate of grout injection, it is advantageous to all parties involved with a project to use as high a pumping rate as practicable. With the knowledge that too high a rate will result in hydraulic fracturing of the soil, however, it is quite acceptable to slowly increase the pumping rate at the beginning of injection, until a sudden pressure drop occurs, which indicates the initiation of a fracture, and thus the practical maximum pumping rate that can be used safely. The rate should then be reduced slightly, to a point that no further sudden pressure drops occur. Ideally, the pressure at each grouting stage should increase slowly and steadily, as shown on Figure 6B.150, until injection of that stage is complete. Should the rate of pressure rise be too fast, or should it increase abruptly, the pumping rate should be immediately reduced and adjusted so as to maintain a slow uniform rise, as illustrated in Figure 6B.150.

Conversely, should the pressure level remain static or slowly deteriorate during injection, it is likely that the pumping rate is too low. Of course, if the pressure drop is sudden, fracturing of the soil or leakage of the grout to an unwanted location is indicated, and immediate reduction of the injection rate or complete cessation of injection is in order. Although occasional rapid pressure reductions should be expected on most grouting jobs, their occurrence should be infrequent, and immediate corrective actions should be the norm. When very sensitive work is being performed, such as injection into a water-retaining embankment where even minor hydraulic fracturing of the soil cannot be tolerated, abnormally low pumping rates are in order. On such sensitive applications, ample instrumentation and monitoring of the grouting site should also occur. Of special importance in this regard is monitoring of the pore water pressure. Unacceptable increases thereof can be controlled by slowing the pumping rate.

Figure 6B.151 is an actual record of the injection behavior of a continuously computer monitored compaction grouting operation. The pressure spike occurring at the beginning of the record is the elevated pressure required to start the flow of grout, following a short time at rest for removal of a section of casing. This is normal and the result of thixotropy (cohesion) of the grout, as illustrated on Figure 6B.17. The nearly complete pressure losses indicated are the result of pulling of the casing to the next higher injection stage, as is clearly indicated by the rise in total depth of injection shown on the bottom curve. The smaller dips of pressure during individual grout stages are indicative of the pump strokes. Note that these line up with the increases in grout quantity shown on the grout volume curve.

Real time records that show all the injection parameters, as in Figure 6B.151, provide a good knowledge of the grout deposition and potential effectiveness. Hydraulic fracturing, loss of control of the grout, or other undesirable events would likewise be indicated by a sudden pressure change, assuming the pumping rate was uniform.

In permeation grouting, it is usually prudent to place a limitation on grout to be placed at any

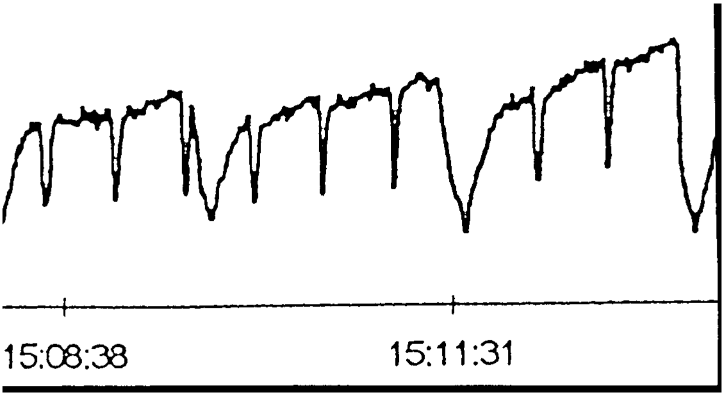


**FIGURE 6B.150** Proper slow, steady increase of injection pressure from actual project record.

one stage. In those rare instances where unreasonably large quantities of grout are taken at a number of holes, or stages of holes, the spacing is likely too close. For economic reasons, the greatest spacing of grout holes that will allow the intended results should be used. Thus, firm establishment of the grout hole spacing is best delayed at least until initial injection takes place. Even on the same site, it is not unlikely that more than one hole spacing will be found to be optimal.

Whereas the pressure behavior at a constant pumping rate discloses a huge amount of invaluable information relative to the formation being treated, knowledge of the pressure, absent that of the injection rate or variations therein, is of absolutely no value. The widespread practice of reporting only the highest injection pressure reached on a given grout hole, without any information relative to the injection rate or pressure variation, is unfortunate, although many professionals who should know better have blindly accepted such data.

Experienced grouters, and especially grout pump operators, know that if it is desired to reach a given high pressure, the only action that is required is a sudden increase in the injection rate. This being so, high pressures are often inappropriately created and are reported far too often, usually to the economic benefit of the contractor. Relative pressures and injection rates are very instructive however, and should be routinely monitored and reported in real time. This can be effectively done either manually or with automated recorders or computer processing.

### 6B.11.2 Site Surveillance

Virtually all grouting into soil imparts elevated pressures into the formation. This results in a continual risk of possible hydraulic fracturing, upheaval, and lateral spreading. Such deformations can continue into surface improvements or substructure, resulting in their damage. It is thus imperative that the ground surface and all improvements thereon be continuously monitored while grout is being injected. Likewise, underground structures, and especially sewer and drainage piping, must be monitored to assure that they have not been displaced or entered by the grout. Most damaging movements of the soil or associated structures are immediately proceeded by a significant loss of pressure, as has previously been discussed. It is thus imperative that the grouting personnel are particularly observant of the work area upon any such sudden losses.

There are many methods available to monitor the elevation of the surface or any overlying structure. To be effective in grouting, however, the selected methods must be capable of surveying very

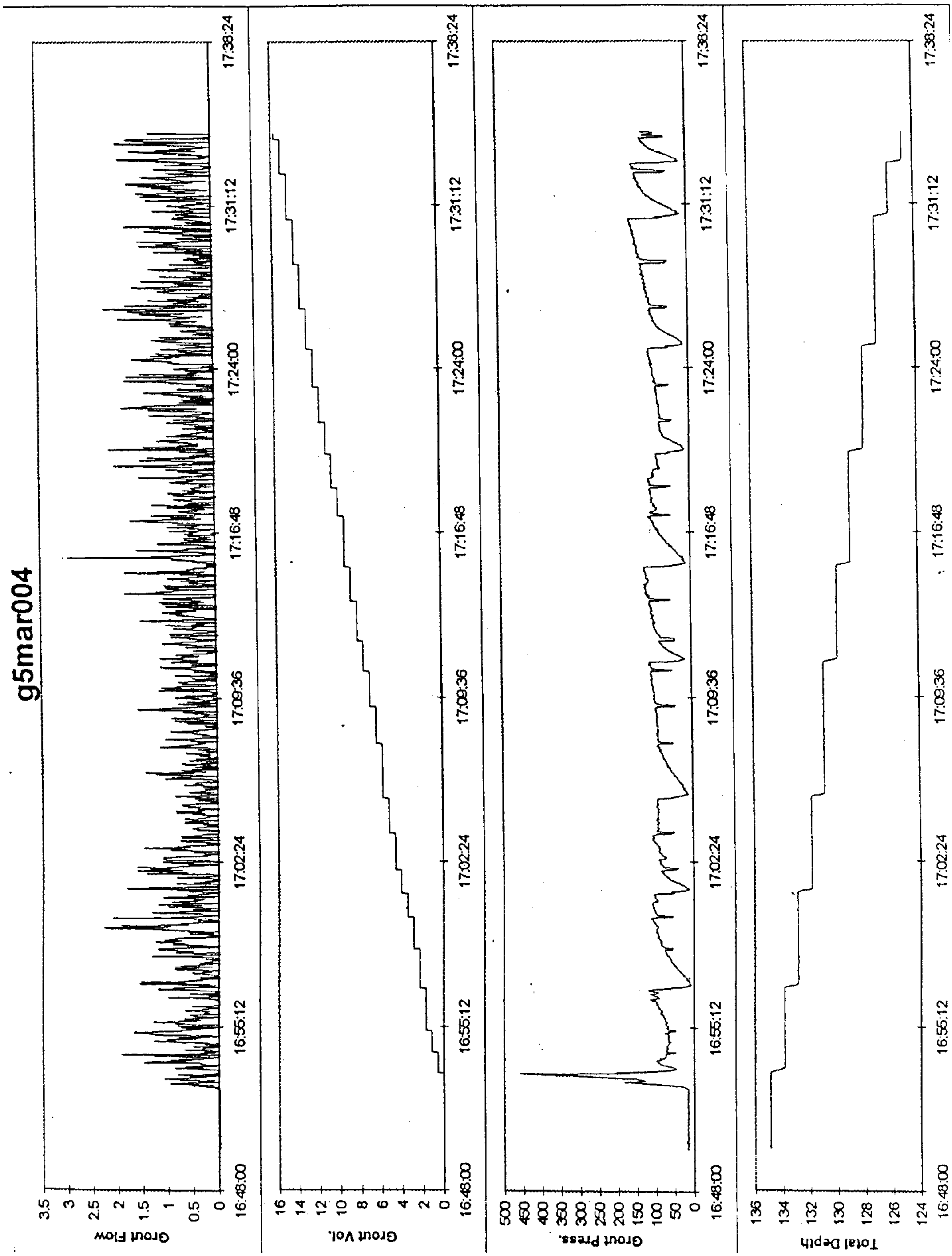


**FIGURE 6B.151** Computer-generated record of grout injection parameters. Note generally constant but slowly rising grout pressure.

large areas and providing immediate notice of any movements. Thus, whereas conventional surveying can be useful, it usually is not satisfactory for the primary surveillance. Rotating laser instruments (Figure 6B.152), combined with multiple targets (Figure 6B.153) mounted on stands that are distributed throughout the work area are quite effective.

Another method, which has been used on literally thousands of projects, is the multistation manometer. Therein, one or more reservoirs, such as illustrated in Figure 6B.146, are used. Any number of tubes can be connected to a single reservoir, with the terminal end fixed to the ground surface or on structures as desired. The precise water level is marked on the tube or adjacent mount prior to grout injection (Figure 6B.154). Any vertical movement that occurs will cause a change in the water level, which is readily observed. In order to enhance visibility, the water can be colored, which is easily accomplished with a few drops of food color in the reservoir.

Multistation manometers are extremely flexible, as is the manner in which they are used. Figure 6B.155 shows one of a number of tube terminals that have simply been attached to posts driven into the ground so as to be easily observable at normal eye level. A similar terminal, attached to small stands that can be distributed over the ground surface, is illustrated in Figure 6B.156. Note that the tube is fixed at an angle to the vertical, so as to facilitate reading from a standing position. Figure 6B.157 illustrates tubes from several different reservoir locations that are attached to a common terminal board, enabling surveillance of an extended area from one location. In Figure 6B.158 the terminal end of a tube is simply fixed to the wall of a structure with duct tape.

**FIGURE 6B.152** The signal from rotating laser instruments can cover a large planar area.

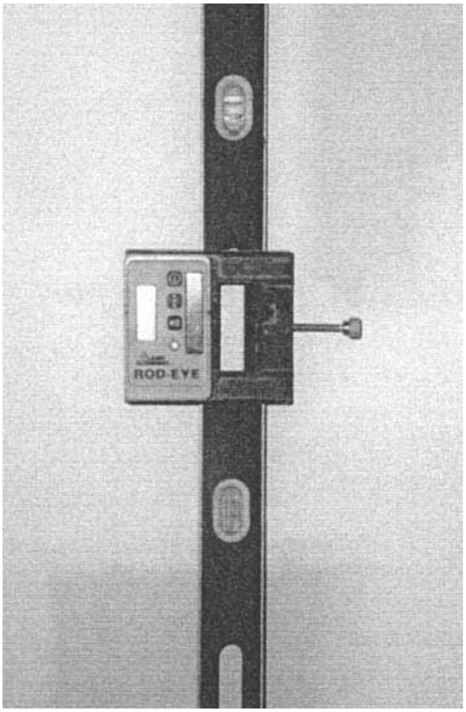


**FIGURE 6B.153** Any number of laser targets can be mounted on stands distributed throughout grouting area.

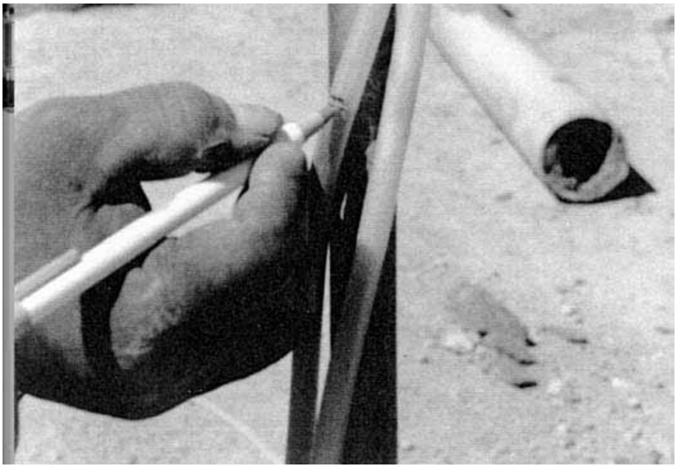

**FIGURE 6B.154** The precise water level is noted when the instruments are set up.

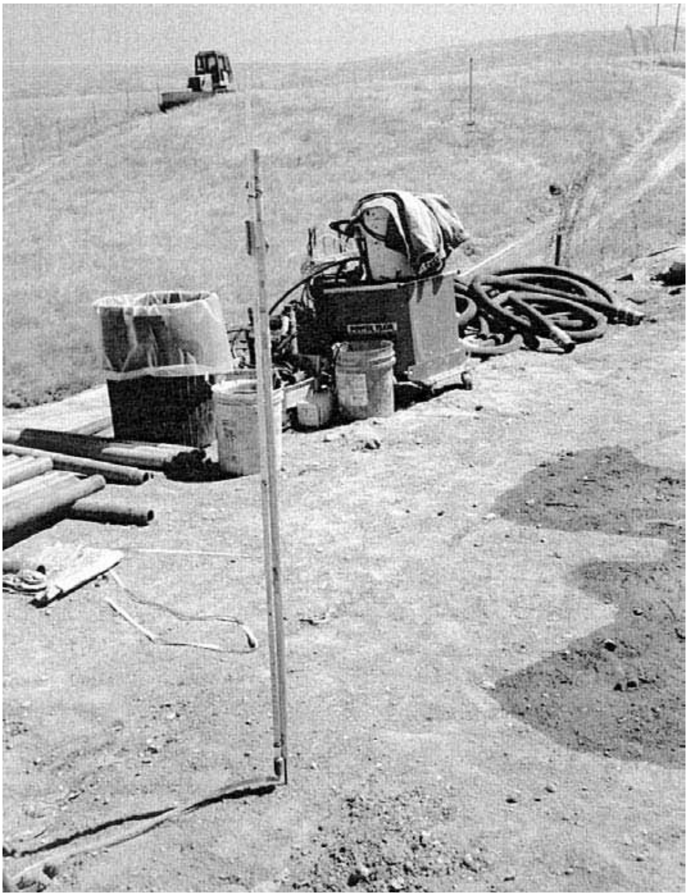

**FIGURE 6B.155** Manometer tube terminal attached to fence post driven in ground.

Note the ruler that has been fixed with it, which will enable easy reading of the exact magnitude of any movement.

Although they do not necessarily provide immediate surface movement measurements, conventional survey instruments can be quite useful, especially for the scanning of large areas. Figure 6B.159 shows part of a large array of prisms that have been set into the slope of a dam. They are being constantly swept by a total station located in the small protective shelter shown in Figure 6B.160, in order to detect any movement of the slope.

Simple, inexpensive devices can also be very effective. A string line tightly stretched across an injection site can be used for reference of the surface elevations. Small pieces of masking tape fixed over cracks will either deform or break if there is movement of the crack. For evaluation of the exact amount of movement, rulers or crack monitor gauges can be secured over cracks in structures (Figures 6B.161 and 6B.162).

Nothing, however, replaces the eyes and ears of competent and alert grouting technicians. Figure 6B.163 shows a worker "sweeping" the ground surface of loose material so as to expose a hardened

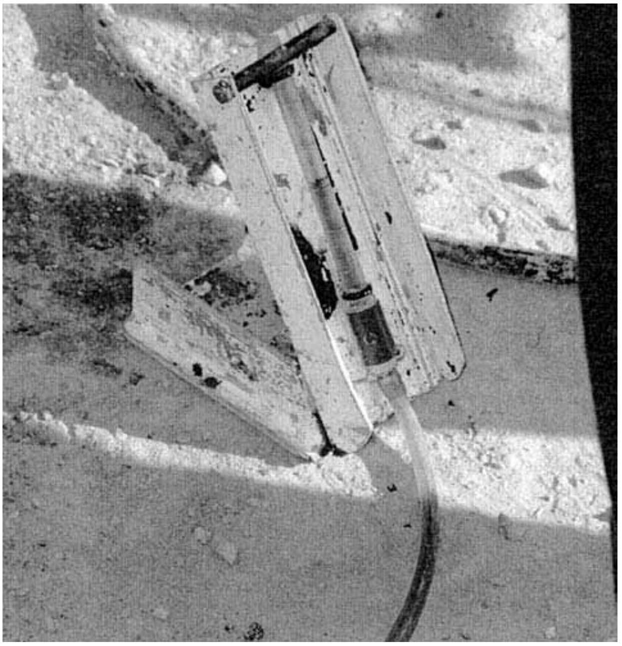

FIGURE 6B.156 Readout terminal mounted on small stand.

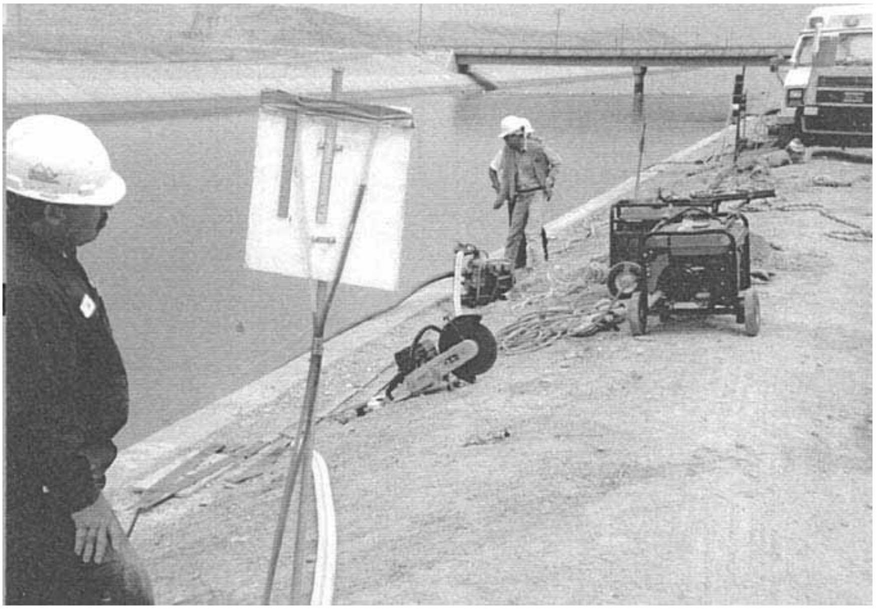

FIGURE 6B.157 Terminals from several reservoirs are placed together for easy readout.

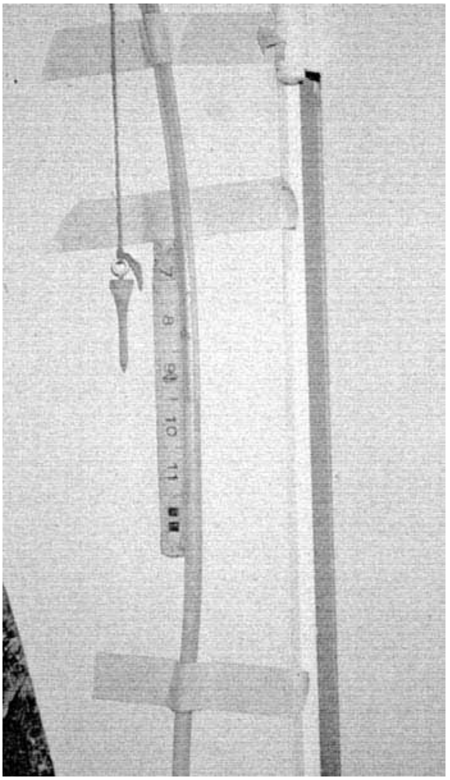

**FIGURE 6B.158** The terminals can be attached to walls or other members of structures.

surface. By very close visual observation of the surface over the injection zone, members of the unusually observant crew were able to detect surface movements, even before they were indicated by the multiple instrument systems used.

## 6B.11.3 Injection Records

Maintenance of complete and appropriate records of all grouting operations is imperative if optimal results are to be achieved. Their greatest value is during the actual work, to provide a timely appraisal of the injection behavior, which indicates the actual conditions existing in the formation. This allows quick evaluation by the grouting personnel on site, as shown in Figure 6B.164, allowing appropriate changes for the execution of subsequent work to be made. As discussed throughout this chapter, flexibility of the work as it progresses is imperative, so as to allow adjustment of the grouting parameters, as required, for optimal achievement of the work objectives. It is also important to maintain the records long term, in order to provide pertinent information in the event of future work. And sadly, with litigation so common in the real world in which we all work, a good record of any operation is important, should that unfortunate event occur.

At a very minimum, the records should indicate grout pressure, the volume injected, and the


**FIGURE 6B.159** Part of a large array of survey prisms covering the face of a dam during grouting.

**FIGURE 6B.160** Prisms were constantly scanned from a survey total station inside protective shelter.

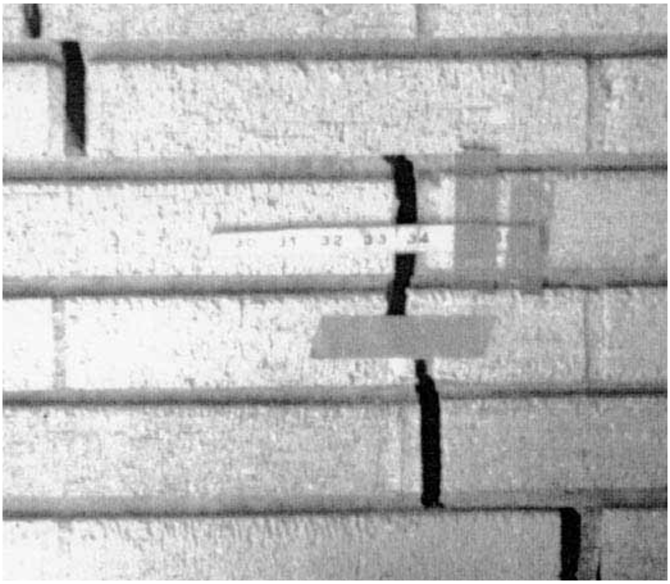

FIGURE 6B.161 Ruler fastened over crack in a wall to monitor any changes in its width.

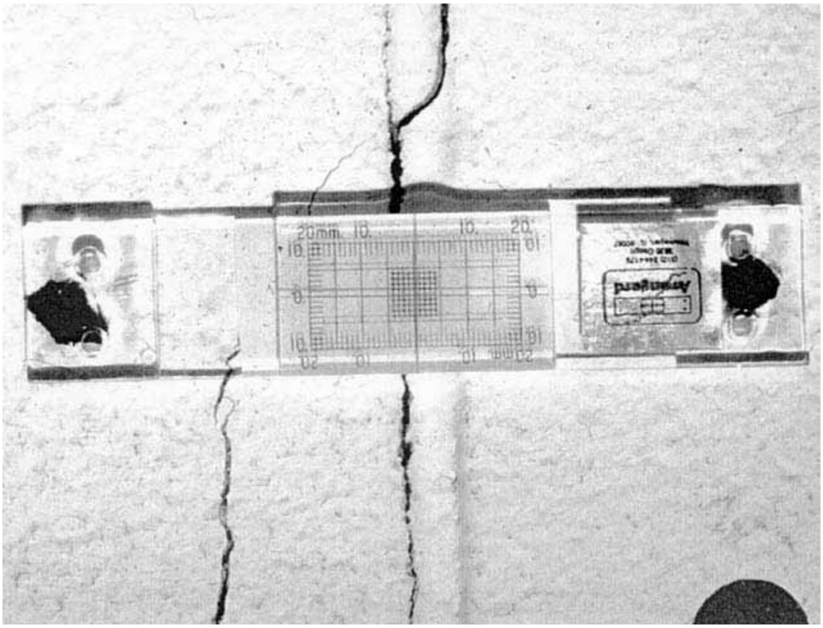

FIGURE 6B.162 Crack monitor can give very accurate measurements of any crack movement.

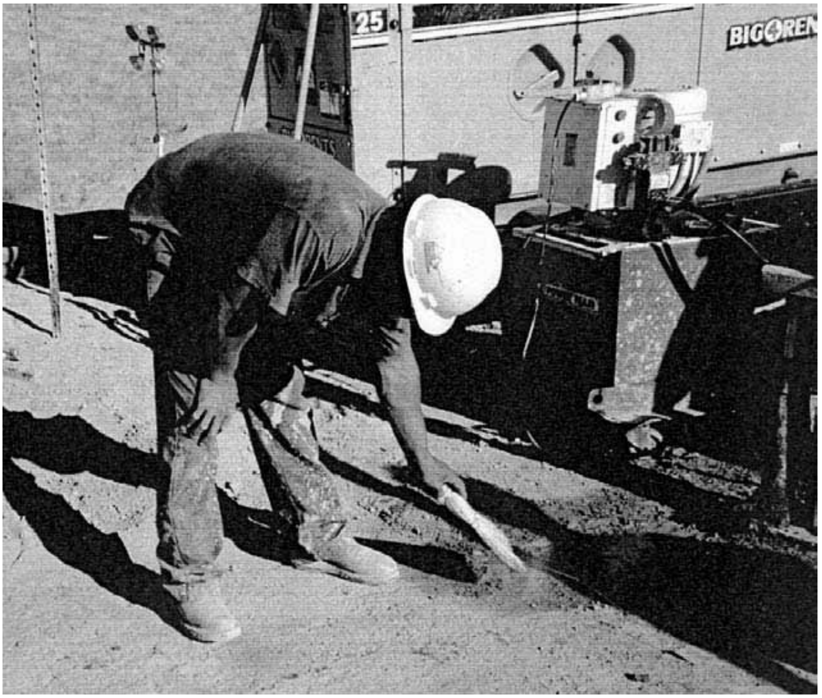


**FIGURE 6B.163** Sweeping loose material off of earth surface to enable closer visual inspection of surface.

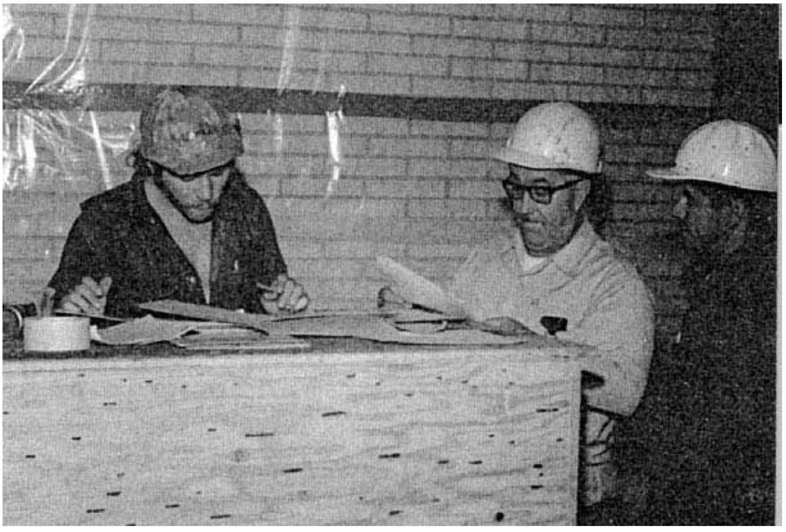

**FIGURE 6B.164** Supervisors analyzing injection data as it is procured.

time at regular intervals. Obviously, the date, time, hole number, and grout stage depths should always be provided. Although it is convenient to also have the grout flow rate recorded, this can be back calculated if the time and volume are noted. Where suspension grouts are being used, continuous monitoring of the grout density will provide precise values of the ratio of water to solids. This can thus be an excellent quality assurance procedure.

If the records are being generated manually, entries should made at all pressure movements, such that the pressure behavior can be plotted against time. Figure 6B.165 shows a typical form for manual recording of individual grout holes. It is also important, to consider the highlights of all holes and their interrelation. This is conveniently done on a form such as that provided in Figure 6B.166, where the overall grout performance can be viewed. It has been the writer's experience that many grouting professionals tend to analyze in great detail the activity of individual grout holes, but sometimes fail to observe the trends of the overall performance. It is important to occasionally stand back and view the overall grouting operation. Understanding how the individual holes behave and interact often clarifies scatter in the individual hole data.

On large or important projects, either continuous chart recorders or automated data acquisition systems with direct connection to a computer should be used. Computer software programs are now available, and others are being developed, especially for the monitoring of grout injection. Also, some grouting specialty contractors have their own proprietary programs. A substantial advantage of computer monitoring is the ability to immediately output the data in a suitable form for closer observation or further analysis. Also, data formats that are computer generated can usually be adjusted in exaggerated scale or otherwise manipulated so as to aid in interpretation of specific events.

Even where the data acquisition is manual, it is advantageous to enter it into a computer database, such as Microsoft Excel, so that graphical output is facilitated. Figure 6B.167 illustrates bar graphs from one of the first holes grouted on an actual project. As can be observed, the grout take and pressure are shown side by side as a function of depth. It is noteworthy that grout takes were quite low at depths greater than about 28 feet, even though relatively high injection pressures were used. Following similar behavior on a number of distributed primary holes, a decision was made to reduce the depth of the drilling and grouting for the remainder of the work.

Similar data for a grout hole on another actual project is shown in Figure 6B.168. Here it will be noted that in spite of the injection rate and pressure remaining generally uniform, very large grout quantities were injected in a region at about mid-hole depth. By magnifying the grout volume figure, as in Figure 6B.169, the areas of high take are even more obvious. When several such records are assembled on the wall according to their relative positioning on the site, as illustrated in Figure 6B.170, it becomes very easy to recognize the relatively limited zone of high take and thus faulty beginning soil. The writer has found this format especially useful for displaying the data as it is developed on the jobsite. Records so posted, for each hole or hole stage, allow for even the lower level technicians to visualize the position of the overall grout deposition zones. This ability to quickly comprehend the overall project status is very helpful in determining how best to proceed. As an example, the posted data shown in Figure 6B.170 would suggest that depth of future holes need not be as great, as long as they extended through the zone of large take.

### 6B.11.4 Be Prepared for Changes!

Aside from the fact that geology and geotechnical engineering are not perfect sciences, from a practical standpoint, it is never possible to have a complete understanding of the subsurface conditions into which grout will be injected. As has been amply mentioned throughout this chapter, flexibility of a grouting program is thus mandatory if the optimal performance is to be realized. It is not unusual to find large grout takes in only a portion of the soil zone to be treated, as was the case in the project for which the grouting records are depicted in Figures 6B.168, 169, and 170. The ability to change both the grout hole spacing and grout deposition stage lengths and depths can be of great advantage in optimizing any grouting program. This is particularly true when working in older urban areas that have historically experienced considerable underground construction, for which the de-

PREPARED BY ______________ JOB ______________ JOB NO. ______

| DRILLING | HOLE NUMBER ______ STAGE ______<br>DATE DRILLED ______ | | GROUTING | | MATERIAL ______<br>SEQUENCE ______ | | HOLE NO. ______<br>DATE GROUTED ______ |
|---|---|---|---|---|---|---|---|
| Depth | Material | REMARKS | Quantity | Rate | Press. | Time | REMARKS |

FIGURE 6B.165 A typical form for manual recording of grout injection data.

**SUMMARY OF DRILLING & GROUTING**

PREPARED BY______________ DATE ______________ JOB ______________ JOB NO. ______________

| DRILLING | | | | | GROUTING | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| HOLE NO. | DATE DRILLED | STAGE | PROCESS | REMARKS | DATE GROUTED | SE-QUENCE | DEPTH / FLUID | PRE-TREAT | C.F. PLACED | MAX. PRESSURE | REMARKS |
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PLATE ______________

PLATE ______________

**FIGURE 6B.166** Placing all data on a summary form enables overall interpretation of operation.

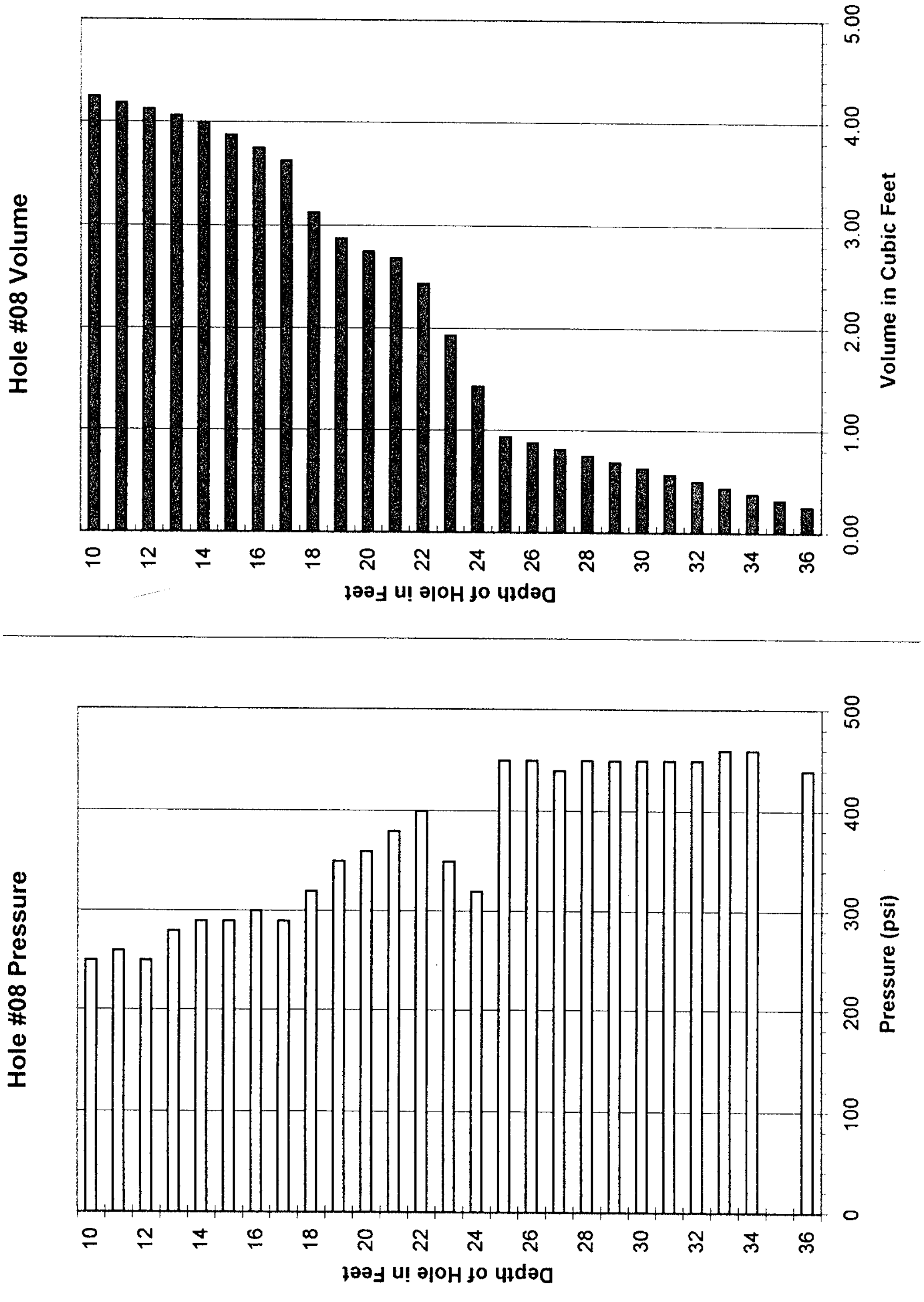


**FIGURE 6B.167** Computer database-generated graph showing both grout volumes and pressures for a grout hole.

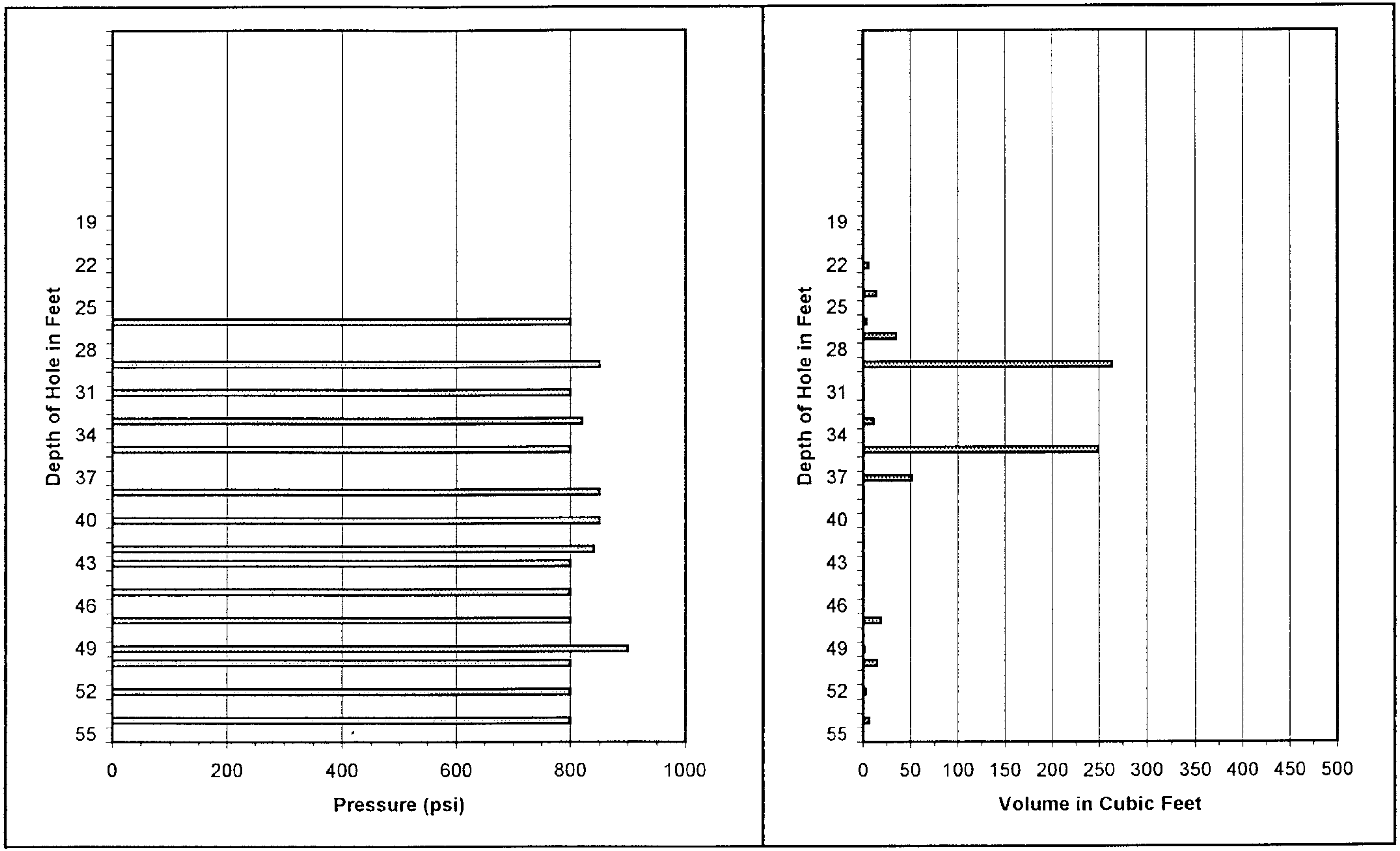


**FIGURE 6B.168** Similar output graphs. Note large grout takes are limited to mid depth of holes.

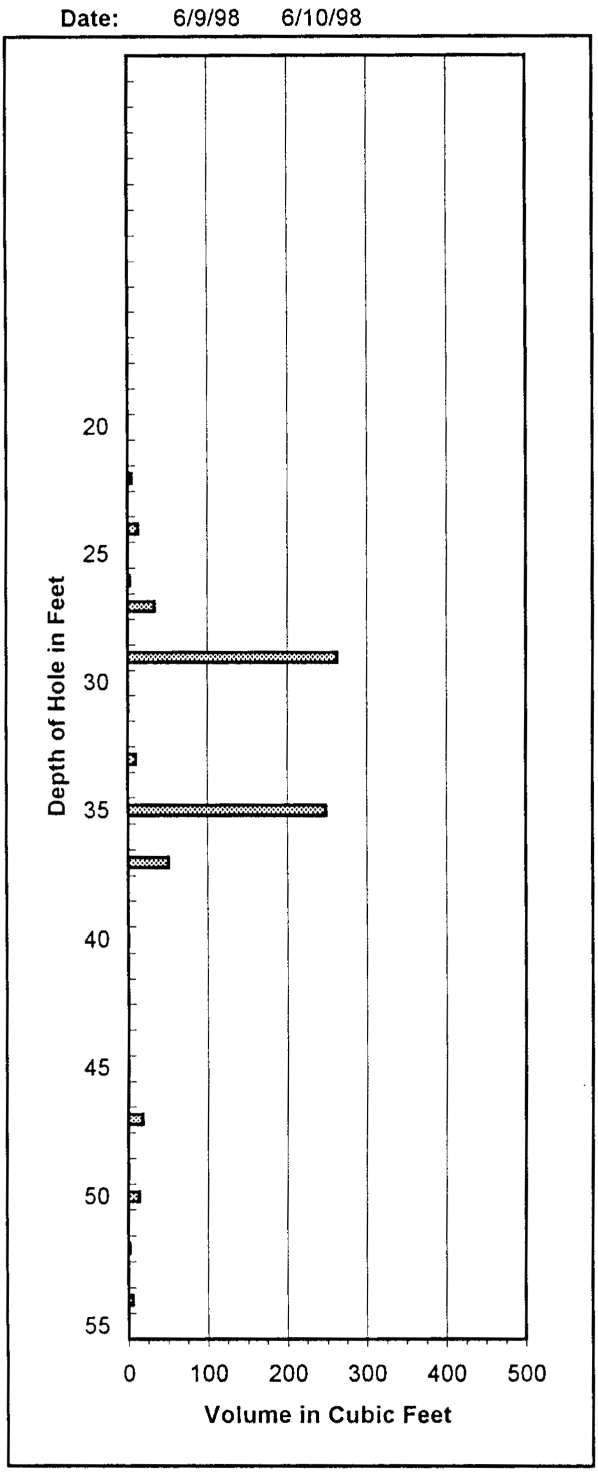


**FIGURE 6B.169** Blowup in exaggerated scale of the right portion of Figure 6B.168 highlights grout distribution in the grout hole.

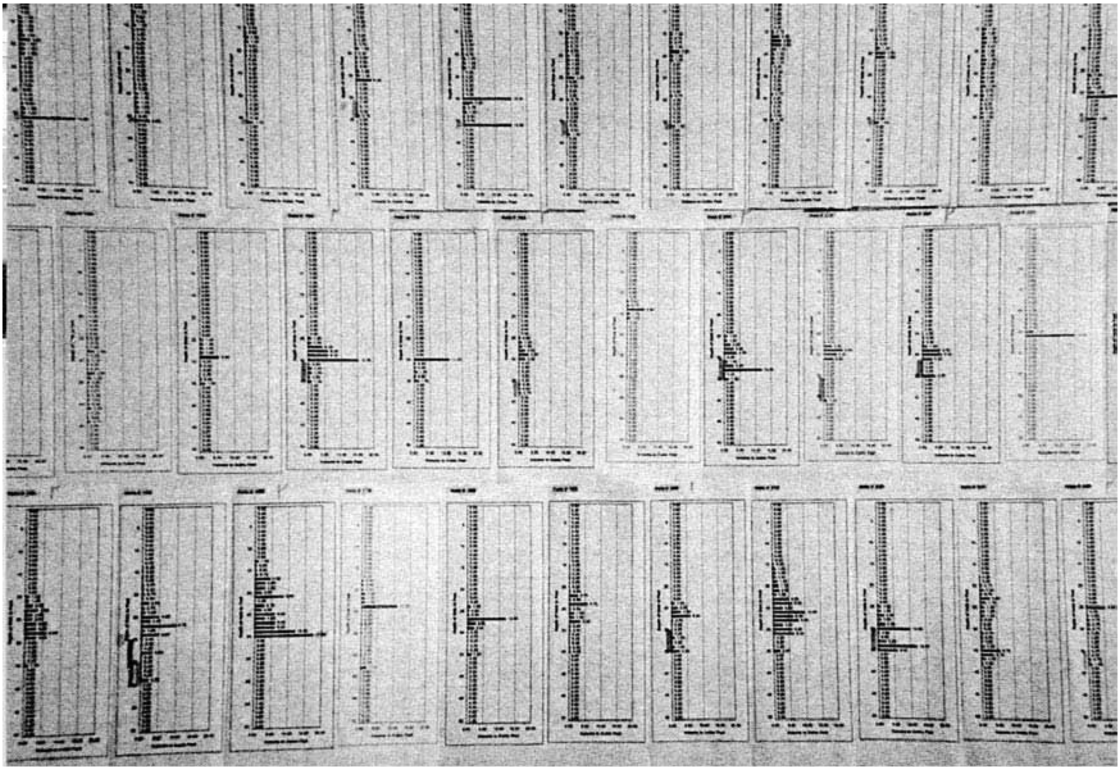

**FIGURE 6B.170** Assembling volume graphs in the same layout as the grout holes allows easy recognition of the grout deposition locations.

tails are not known. Even in those cases where good records of prior underground works are available, important details may be missing.

As a very young contractor, in the mid 1950s, the writer had a job to re-level a settled swimming pool by grouting. The pool was constructed in clayey-silt soil, which filled the basement of a building that had previously occupied the site. Leakage from the newly constructed pool had saturated the fill soil, which had apparently not been properly compacted. At the time, grout improvement of such fine-grained soils was not considered possible, and the work was to be limited to leveling the pool shell. On commencement of grout injection, a steady flow of water entered the pool from adjacent grout holes. This continued for many hours before the pool finally began to rise as planned. About nine times the quantity of grout, which by calculation would have raised the pool as planned, had been required! And throughout the injection, what was thought to be a similar amount of water was bailed out of the pool.

The only logical explanation was that the force of the grout "squeezed" the water out of the soil, thereby densifying it. Although the job was a financial disaster for the contractor, who had agreed to do it for a lump-sum price, the geotechnical engineer considered it the most wonderful event of his career. You see, he explained, it's not supposed to be possible to squeeze water out of soil so easily; this is wonderful! And that was the beginning of the writer's research, which led to his current knowledge of the compaction grouting mechanism. Wonderful it was, though it didn't seem so at the time.

In another memorable project for the fledgling firm, remedial grouting in connection with the settlement of some railroad tracks was to be performed. Many years prior, a small tunnel had been excavated, about 30 feet under the tracks, in which a sewer pipe was placed. The soil was very sandy and although the records were sketchy, it was believed the tunnel was solidly shored with continuous sets of timber planks. The tunnel, which was only about 25 feet long, was said to have been backfilled with "jetted" sand. Payment for the grout, which was specified to be a cementitious suspension, was by the bag of cement. The contract quantity estimate was for 350 bags.

Penetration of about three inches (7.5 cm) of timber over the tunnel roof during drilling confirmed the expected shoring. A drop of the drill steel upon penetration of the timber, however, was a surprise. The contractor's people reported what they felt were voids, varying from a few inches up to more than one foot. The owner's inspector, who had not shown much regard for the novice grouter, however, insisted the contract estimate was correct, with a cryptic comment to the effect "when you get more experience, you'll know the difference between loose sand and voids."

Four thousand bags of cement later, the voids were filled. For the young contractor it was a very profitable job, and almost offset the losses on the pool! But more importantly, it offered a valuable lesson. Subsurface conditions are not always what we expect, and don't ever believe, you really *know* what you *think* you know (as did the arrogant inspector). Surprises do occur in the grouting business.

Another interesting example is in the failure of a four year old sewer, which had been constructed quite deep in an area of generally poor soils with a high groundwater level. Following the development of a sinkhole over the line, internal inspection disclosed some minor distortions of the invert alignment, suggesting questionable stability of the pipe bedding, which consisted of a 12 inch (30 cm) thickness of one inch gravel. The selected repair provided for injection of a cementitious grout into the pore spaces of the bedding gravel, to in essence solidify it into a rigid concrete. After a few days of uneventful injection, a large pressure drop occurred. The first thought was that the grout was entering the pipe, but inspection found that not to be the case. After careful evaluation determined that no other substructure existed in near proximity to the area of grout leakage, injection proceeded, but with very close observation of the pressure behavior. It was indicative of the filling of a small open pipe for about four cubic feet of grout, after which normal behavior returned. Subsequent discussions with the inspector of the original work disclosed that during construction the contractor occasionally embedded slotted pipe in the bedding rock, as part of the dewatering effort. No records existed of this detail, and had it not been for the memory of the inspector, the otherwise harmless anomaly would have remained a mystery.

This brings to mind a similar incident, which did not turn out to be so harmless. In the early 1960s, the writer's contracting firm was retained to do some remedial grouting under the main courthouse in Los Angeles. A tunnel had been excavated under the existing basement to provide access from a new underground parking structure. The tunnel passed under the heavily loaded slab-on-grade floor of an evidence storage area, by a depth of only about four feet (1.2 m). The floor slab had settled about three inches (7.5 cm) over the tunnel alignment. Slabjacking with a low-mobility grout was to be used to raise the floor to its proper grade, followed by compaction grouting of the soil down to the tunnel roof. Because this was in an active public building, all work was done at night. Grouting had a rather poor reputation at the time, and the owners' maintenance staff was concerned about possible damage to the existing facilities. They thus had a plumber present at all times during grout injection.

One night the grouting foreman observed pressure behavior indicative of the grout entering a pipe. He halted injection and informed the plumber that he thought the grout was getting into a pipe of some sort. On his manual record form was a comment, "stopped grout—told the plumber I think I'm in a pipe," with the time noted. About 10 minutes later, a further comment was entered, "Plumber says everything OK," with the time noted. Injection was resumed, but a further note after about ten minutes stated, "Told plumber know I'm into something—stopped grouting—hole abandoned." Following that were notes addressing further word from the plumber that he had checked all lines, they were clear, and there was no reason to stop grouting.

Some months later, when the first rain of the season fell, it was found that the grout was indeed in "something" and the plumber was wrong. A 10 inch (25.4 cm) cast iron pipe that served as a collector for the entire roof drainage system and crossed the tunnel alignment was filled with grout! In repairing that problem, it was found that the probable cause of the original settlement had been leakage from that pipe as a result of disturbance during tunneling. Fortunately for the contractor, the vigilance of an extremely competent foreman and his notation on the grouting records relieved him of responsibility for what turned out to be a very expensive event. As stated many times throughout this chapter, continuous pressure behavior monitoring and recording should be mandatory in all grouting work!

In another instance, differential settlement of several inches had occurred in a large commercial building. The building, which was only about a year old, had been built on one edge of a deep canyon fill. The geotechnical investigation did not reveal any obviously poor underlying soil, but as the fill was not up to the specified density, compaction grouting was called for. Primary grout holes were spaced about twenty feet on center, along the outside of the foundation of the wall that had experienced the greatest damage. Drilling indicated nothing noteworthy, and only minor amounts of grout, with very uniform pressure buildup, were injected in the first several holes. But then came a surprise! At a depth of about 40 feet (12 m), one of the holes exhibited very irregular pressure behavior, and took a very large quantity of grout. This was followed by similar behavior at a depth of about 35 feet (10.5 m) in another hole.

Reevaluation of the available site grading data suggested that a pioneering haul road might have treaded up the canyon under the building. Working on this assumption, grout holes were established over the suspected haul road location. Indeed, large quantities of grout were taken in only one or two stages, at various depths, in those holes. Plotting those locations indicated a uniform gradient, typical of a haul road. Apparently, loose slope-wash and fill existed on the downhill side of the haul road. With this knowledge, the grouting program was reevaluated. Further work was limited to the area of the suspected haul road, reducing the cost of the work to less than half of that which was expected going into the job. This resulted in a very happy owner. Although the grouting contractor had the job cut short, he more than made up for the lost revenue by referrals and future work from the owner and his geotechnical engineer.

## 6B.11.5 Verification of Grouting Effectiveness

Much has been written and discussed about verification of the effectiveness of grouting, and in fact the American Society of Civil Engineers GeoInstitute Committee on Grouting had an entire session on the subject at a 1995 meeting, the proceedings of which are available (Byle and Borden, 1995). This notwithstanding, although many grouters talk verification as part of their promotion, the sad fact is that relatively little effort has been shown by grouters to really understand the quality (or lack of quality) of their work, especially in the United States.

From a practical and economic standpoint, the most meaningful verification of the results of virtually all grouting operations occur as the grout is being injected. This is through continuous observation and control of the various injection parameters, especially the rate of injection and pressure behavior. It is the writer's considered opinion that no amount of postinjection investigation or testing can duplicate the benefit of careful monitoring and control during the actual grout injection. There are however, a variety of postgrouting tests that can, to a varying degree, confirm the completeness and the general quality of the grouted mass.

In water-control grouting, if the leaking areas are clearly visible, such as leakage into an underground pipe or structure, direct observation of the cessation of the leakage is certainly acceptable verification. If the site of the leakage is not clearly visible, such as an impermeable curtain under a dam, downstream boreholes will be required.

Where permeation grouting has been used for strengthening of the soil, test excavations or borings can be made. The completeness of the solidification can thus be readily observed, and specimens of the grouted soil can be obtained for laboratory evaluation of the strength. It must be remembered that the fundamental strength, which is the strength under a continuous load, of solution chemical grout solidified soils is only a small portion of that indicated by most standard laboratory compression tests. Such masses should thus be subject to long-term creep tests, as discussed in Section 6B.3.1.1

For compaction grouting, wherein the grout remains in individual globule masses, density testing of the improved soil between the grout masses can be performed. Although in the early days of the procedure, split spoon specimens were often procured for laboratory evaluation, this is a time-consuming procedure that is not terribly practical, and is thus now seldom used. Likewise, Standard Penetration Tests can be made both before and after the injection has been performed, but this also is somewhat disruptive and time-consuming.

Cone Penetration Testing of the soil following grouting has proven to be a reasonable method to determine postgrouting density. Its one limitation, is the necessity of access for the relatively large testing equipment, so it usually isn't suitable for use inside or under structures. Where the results of compaction grouting are being evaluated, probes or tests are usually made midway between injection points, which theoretically would be the zone of least improvement. It is thus imperative to preserve the exact locations of the injection points, which often requires special effort. If the injection is through a paved surface, the hole locations will be easily observed. If the holes are drilled through a dirt surface however, they can become obliterated quite readily. It is thus important to provide markers such as the long lag bolts being placed in Figure 6B.171 or the flags as seen in Figure 6B.172. The normal operation of equipment, and dragging of hoses over the surface, which is inherent in grouting work, can easily destroy hole markers, so they must be quite tough and resilient.

A variety of geophysical methods have been promoted, and some have been successfully used, for the verification of all types of grouting. The most commonly used involves evaluation of the ve-

**FIGURE 6B.171** Long bolt used to mark location of grout hole.

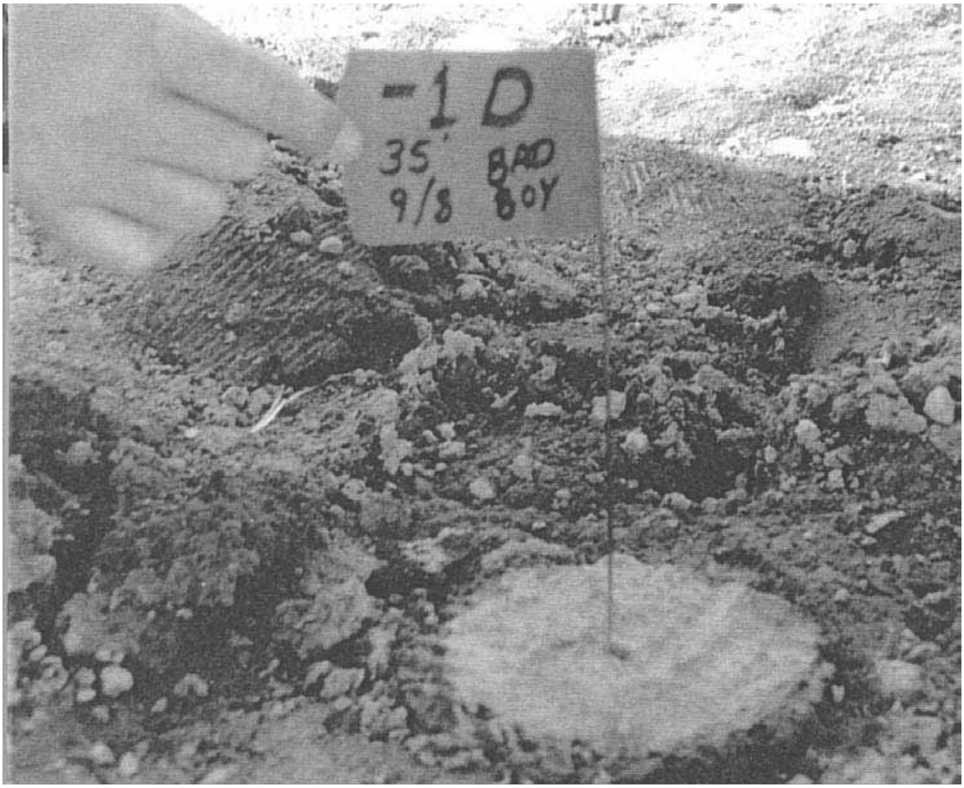


**FIGURE 6B.172** Small surveyor's flag to mark location of grout hole. Note that the hole identification and date grouted are also shown, as is the comment "Bad Boy," which apparently indicates some difficulty with the grouting.

locity of stress waves traveling through the formation, commonly referred to as seismic tomography. Simply stated, sound travels faster through a hard medium than a soft one. Thus, a densified or solidified mass will usually possess a higher velocity than adjacent soil that has not been grouted. By analysis and interpretation of the velocity and attenuation of a large number of sound waves driven at different inclinations through a profile of the soil formation, the relative density or stiffness of the various components of that profile can be determined.

In application, evaluation is usually preformed between two bore holes. A transmitter in one of the holes initiates a pulse, which is picked up by a receiver in the adjacent hole. Typically, the transmitter will be moved vertically in the bore hole, so as to direct pulses to the receiver at a variety of angles. The receiver usually remains at one location within its hole for a given set of data. By changing the depth of the receiver, multiple sets of data are obtained, with which a seismic profile can be developed. The waves can be either shear waves, which are perpendicular to the bore hole, or compressional waves, which are parallel thereto. Compressional waves are the most frequently used, as their analysis is simpler. Whereas stress wave velocities are usually determined with direct paths between two bore holes (cross-bore hole survey) or by analysis of reflective rays, some work has also been accomplished by directing waves from transmitters and receivers, which are both placed on the ground surface. The effective depth of the so-called surface procedure is a function of the distance between the transmitter and receiver. As the accuracy of the resulting plots, which are known as tomograms, is dependent upon the number of individual ray paths and resulting nodal intersections that are evaluated, the down hole evaluation is preferred.

To produce tomograms from the ray path data requires extensive data processing, which is practicable only with specialized computer software programs. For these reasons, such work is commonly performed only by firms who possess the highly specialized knowledge and equipment. This, plus the requirement of very large numbers of different ray paths to provide accurate determinations, makes the procedure somewhat expensive, and it is thus usually reserved for especially large or important applications. Where the budget allows for acquisition of sufficient data, however, the procedure has been found to accurately portray the existing conditions and provide valuable information.

Another geophysical method, as reported by Rehwoldt, Matheson, and Dunscomb (1999) is two-dimensional imagery, generated by electrical resistivity surveys. Therein, the conductivity of the formation is evaluated by placing a consistently spaced line of electrodes on the ground surface. They are driven into the soil to a depth of about 12 inches (30 cm) and connected to a computer-guided resistivity meter. The meter takes measurements along the electrode array, using four electrodes at a time. The depth of the evaluation is dependent upon the spacing of the surface electrodes.

Special computer software is then used to evaluate all possible combinations of current flow and analyze the data to locate areas of anomalously high or low resistivity. Soil and rock are basically nonconductive materials; however, both virtually always contain moisture. It is the moisture content that determines the conductivity of the various components that make up a particular formation. As examples, dry rock is quite resistive, whereas submerged clays are highly conductive. Accordingly, an area that was of very low density or perhaps an obvious void prior to grouting will exhibit different electrical behavior after being grouted.

Ground probing radar is another process that has been considered; however, with present technology, the accuracy and precision of its output is not as great as that of the previously described seismic methods. In radar evaluation, an electromagnetic pulse is generated on the ground surface. It is then reflected from the various interfaces at depth to a retrieval instrument on the surface. Equipment is available that can traverse the surface radiating and receiving repetitive electromagnetic pulses, so as to obtain a continuous record of the subsurface interfaces along the route traversed.

Ground probing radar instruments basically measure variations in the electromagnetic velocity of the different formation materials encountered. The procedure is thus especially useful for identifying clear interfaces, such as the presence of buried pipelines or subsurface concrete structures within a soil mass. Likewise, where the velocities of different soils in a grouted formation vary greatly, or have clearly different water contents and/or chemistry, identification of the boundaries may be viable. In most grouting applications, however, the variations of electromagnetic velocity between the grout and the soil are more subtle, and the applicability of the process is thus less advantageous. As with advanced monitoring in general, the process is in constant development and advancement. It may thus become more applicable to the verification of grouting work in the future.

### 6B.11.6 Grouting Specifications and Contracts

Although grouting is very much a technology, it is a specialized area of activity for most owners and their engineers, most of whom do not practice it regularly. There are however, many geotechnical engineering firms that have professionals with considerable experience, as well as several individual consultants. Perhaps those in the best position to maintain expertise in the technology, however, are the contractors who regularly perform the work. Many possess the specialized knowledge and equipment, and have the experienced personnel to optimally perform the work. Specifications should not be so limited or stringent that well-qualified contractors will be precluded from use of their innovations and special expertise.

Unfortunately, however, many grouting contractors completely ignore either sound engineering fundamentals or established grouting technology. And sadly, some continue to shroud their operations in mysticism. And then there the "high tech," black magic purveyors, who tend to use technical "mumbo-jumbo" espoused by well-educated engineers, to promote their activities, often in professional organizations and technical publications as well as through persuasive advertising. This latter group often sponsors seminars, where they provide supposedly nonproprietary handouts and guide specifications. Unfortunately, their field performance very often falls far behind the effectiveness of their marketing activities.

One must thus use extreme care in accepting advice from grouting contractors or, as many refer to themselves, "geotechnical specialty contractors." And especially, one should resist using "guide specifications" distributed from such operators for tentative projects. Such documents virtually always favor the supplying operator, and very often provide clever language that will eventually be used as a basis for a claim for extra payment. The author can often identify the origin of a specification upon viewing it. Tragically, very often such guide specifications are technically erroneous, and their use can result in very poor project performance, regardless of which contractor is awarded the work. As an example, a recent specification for compaction grouting, prepared by a well-known geotechnical engineering firm, stated that "additives such as . . . bentonite . . . can be included in the grout at the contractor's option." It has been well established, as extensively discussed in Section 6B.6.2, that inclusion of bentonite in a compaction grout will present a serious risk of hydraulic fracturing of the soil and loss of control of the grout deposition.

Specific areas that should be addressed in the specifications include:

- The clearly defined intent and limits of the zone to be grouted. A tentative layout of the grout injection holes is usually in order, although provision for changing either the number of holes or their spacing or location should be provided.
- Straightness or verticality requirements of the grout holes and the range of hole diameters that are acceptable.
- Grout mixer requirements. High-shear, colloidal mixers should be required for cementitious suspension grouts.
- Material requirements. This will vary with the type of grout being used. As an example, bentonite or other clay should be specifically prohibited from compaction grout, and an aggregate falling within the envelope presented in Figure 6B.57 should be required. Generic classes of solution grouts acceptable for water control grouting should be stated. The effects of shrinkage upon drying must be considered with any chemical solution grout.
- Acceptable range of pumping rates and a provision that the pump output will be uniform at all required pumping rates.
- Range of grout pressure that will be required.
- Requirements for monitoring and recording the various injection parameters during grouting. Consider mandated use of a record format, such as shown in Figure 6B.162, or a similar layout especially developed for the particular project or grouting methods being considered. Note that continuous pressure behavior monitoring should be mandatory in most grouting. (The only exception would be in some water-control applications.) Continuous, real time computer monitoring is preferable; however, many contractors lack either the knowledge or ability for its application, and it is thus usually reserved for large or important projects.
- Requirements for the workers and supervisors of the grouting crew. Some contractors, especially when working away from their headquarters location, send only a supervisor and/or perhaps one key worker to a project and hire locally to form the crew. This will result in less than good performance, due to the special skills required in grouting, and the required interworking of the different crew members, and should be prohibited by specification, in the writer's opinion.
- Levels of required improvement, acceptance criteria, and the methods to be used to establish compliance.

The use of open competitive bidding for most grouting work should be avoided, if possible. It serves as an invitation to lesser qualified and often shady operators, of which, unfortunately, there are many in the grouting business. Although public agencies are usually limited to the use of open bidding for their work, it is possible even for them to require meaningful prequalification of contractors for highly specialized work such as grouting, and many public projects have been so bid.

As a minimum, bidding should be limited to entirely prequalified contractors. Prequalification should not be given without a through appraisal of the individual applicants, which will include contacting the owners and/or engineers of *several* prior similar projects. Several is emphasized, as references given by some contractors in the past have turned out to be fraudulent. Additionally, even shoddy operators have an occasional project that has turned out well.

Continuity of a contractor's activity is also important. A contractor that has not preformed similar grouting work for an extended period of time is not likely to have working crews organized for that type of work. Use of a prequalification form such as that provided as Figure 6B.173 is strongly recommended. It is also a good idea to visit the contractor's headquarters, not only to confirm the statements made in the prequalification submittal, but also to see firsthand the manner in which he operates, such as orderliness of the equipment and facilities.

Where possible, rather than have competitive bidding, it is strongly recommended that a contract be negotiated with a thoroughly qualified and honest contractor. Oftentimes, these will be regional firms, which do not advertise but depend upon recommendation and repeat work instead. Locating them will thus require considerable effort, but that effort will be well placed, in that a better quality finished product, delivered in a timely manner and at a lower ultimate cost, will likely result. The importance of due diligence in selecting a contractor cannot be overemphasized. This should start with the completion of a prequalification submittal, including the information enumerated in Figure 6B.173.

In assessing the submittal, all entries should be independently verified, for very often, less than honest responses are made. The writer finds it distressing that so much dishonesty exists in the grouting business, but it does, and it must be recognized. As an example, one contractor responded in a submittal, that he was going to use a certain make and model of batching and mixing plant on a project. He further stated that he was going to rent it from Hertz Equipment Rental in Denver Colorado, and that he had a commitment that the equipment would be available to him should he receive the project. A simple phone call to the designated renter revealed that they did not own any such plants, but advised calling an 800 number, which connected to their national equipment inventory. A call there disclosed that the firm did not own any such equipment nationwide!

In another case, a contractor was awarded a project based on his proposal to use continuous computer monitoring, which involved a "proprietary software program specifically for the compaction grouting" that only he possessed. When the writer visited the project, a computer was indeed set up on a table in the injection area. Unfortunately, the only data that was being recorded was the hole number, which was entered manually, and pressure readings transmitted from a pressure transducer at the grout collar, which operated only sporadically. There was no mention of the injection rate and, of course, this rendered the pressure values of no usefulness whatsoever. The claim of use and setup of the computer on the work site did indeed look impressive. Unfortunately, this was simply a ruse to imply special competence, which the contractor did not possess. As for the proprietary software, it was nonexistent! And in the eyes of the writer, the entire operation was a sham.

The fact is, there are some brilliant advertising and proposal writers in the grouting industry. Sadly, they are not always so brilliant in carrying out the work, once it is awarded to them, and sometimes their presentations are downright dishonest. It is not unusual to find that once on the project, the formulation of often frivolous claims appears to take precedence over good performance of the work. One must be very cautious in accepting information from grouting contractors, and independent confirmation of the appropriateness and the accuracy of such is essential. Fortunately, there are some very good operators that are both honest and very knowledgeable, and therefore excellent sources of guidance.

Finally, unless the work to be done is quite routine, and the soil profile and the existing conditions are well understood, full-scale test injections are strongly recommended. Successful comple-

PREQUALIFICTION SUBMITTAL - GROUTING

Company Name________________________________Years in business_____

Address__________________________________________________________

Phone__________________Years in specialized Soil Grouting_________

Bank____________________________Contact__________________________

Surety Company____________________Contact________________________

List five projects of similar size, scope, and intent, as the instant for which this submittal is being made. Provide the following information for each project.

> Project name; Dates work started and completed; Location Owners name, address, and phone; Owners Representative, name, title, and phone; Description of work done and contract amount.

Key personnel which you agree to commit to this project should you be awarded the contract.

Provide the following information for each listed position, for both primary as well as backup employee:

> Name; Number of years with the company; Number of years in this position; Number of years of experience in this type of Grouting, Special education, training, or certification, if any; Is employment by contract or day-to-day?
>
> Superintendent
> Craft Foremen
> Drillers
> Grout pump operators
> Key workers

**FIGURE 6B.173** Recommended prequalification submittal requirements.

tion of a full-scale test, with exposure of the grouted soil as shown in Figure 6B.174, will not only confirm that the intended objectives can be met, but will also allow for a more precise understanding of the work to be done. Where more than one grouting method or material is being considered, they can all be evaluated in such test injections. This will also facilitate a more accurate cost estimate and, most importantly, allow appraisal of the contractor's field operations. One word of caution is appropriate, however. Unless the trials show needed changes from the anticipated methodology, allow no changes of the contractor personnel, equipment or methodology for the full-scale work.

PREQUALIFICATION SUBMITTAL - GROUTING, Page 2

Special equipment, including backup units which you will commit to this project:

List each unit; Provide make, model, and capacity, age and condition, location where unit can be inspected, and whether company owned or rental. If rental, provide name of renter and state whether you have a definite commitment that the equipment will be available for this project should you be awarded the contract.

Does your company maintain an employee training/qualification program? If yes, provide the scope and details thereof.

Does your company provide routine Quality Control on its projects? If yes, provide details thereof including any printed forms used therefor.

Does your firm maintain a testing facility for either quality control or research and development? If yes provide details thereof including a listing of equipment provided and tests performed.

Does your firm maintain a Safety Program? If yes provide a description thereof, including a listing of each accident, employee injury, or incident of property damage within the last five years.

**FIGURE 6B.173** (*continued*).

There have been instances where grouting contractors have performed brilliantly on test applications, so as to qualify for the production work, but then used less costly procedures, resulting in inferior performance once the production contract was received.

## 6B.12 SOME FINAL THOUGHTS

When the writer first entered the world of grouting in 1952, there was virtually no established technology for the grouting of soils. The large government agencies that managed major dams each had its own ideas on how grouting should be done, although, interestingly, there was not a great deal of agreement between them. The contractors generally operated behind a veil of mysticism, and there was virtually no sharing of knowledge. Most grouting in that era, was in rock, and grouting in soil was generally unheard of.

Because grouting in soil is of relatively recent origin, and some of the main players, including the writer, were not part of the established club of "magic grouters," development of the technology for soil grouting, and especially that of compaction grouting, has been much more openly discussed and orderly. The technology for grouting in soil is to a large degree better understood, and authentic information on the technology more readily available than for some of the earlier forms of grouting. There remain many, however, especially in Europe, who attempt to adopt earlier rock grouting technology to soil.

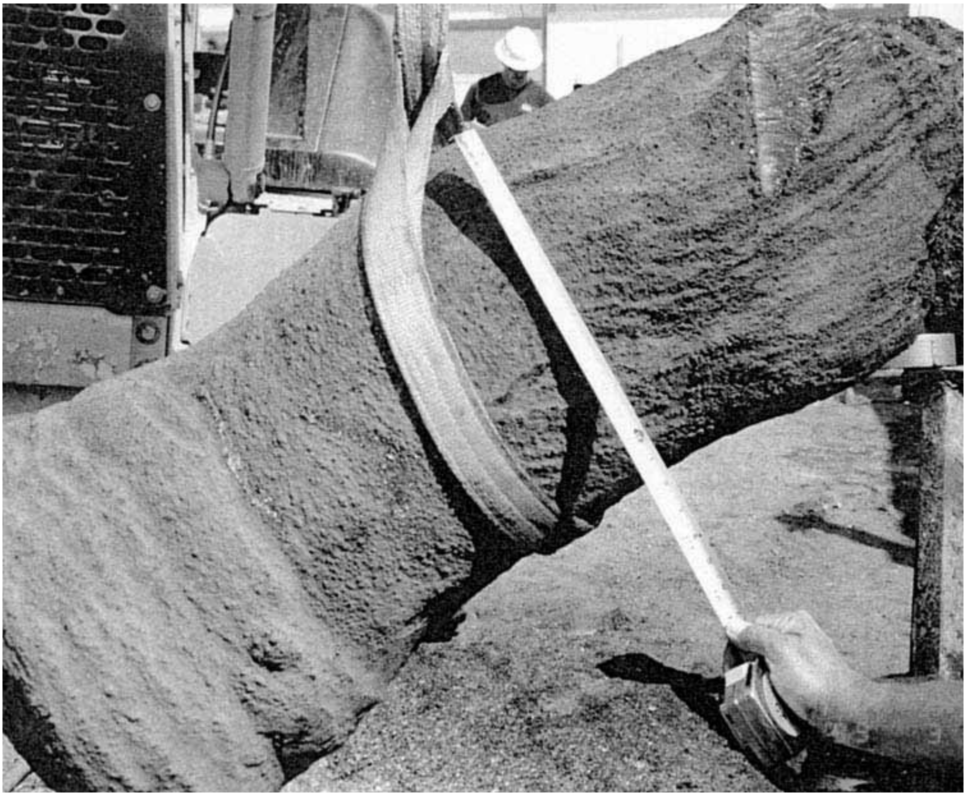

**FIGURE 6B.174** Grouted soil mass from a test injection.

It is unfortunate that to this day, with but little exception, there is not much cooperation between grouting contractors. In fact, although there have been several attempts to start an organization to represent the grouting industry, such efforts have never been successful, as the different players overtly refuse to share their "proprietary" knowledge. The fact is that there is now very little grouting technology that is not well established within the industry, and thus little to hide or protect.

Grouting contractors in general, and especially the very large ones with staffs of professional engineers, often intimidate the less informed with technical jargon that may or may not be appropriate. In one instance, the defendant contractor in a lawsuit retained a university professor to develop a complicated analysis that would presumably absolve him of responsibility for the virtual destruction of a structure in which his incompetent injection had blown out an adjacent down-slope. And it is not unusual to see very misleading and even technically incorrect information in published works, including that which has been peer reviewed.

The above notwithstanding, geotechnical grouting is a fascinating science, and there is now a huge amount of well-documented technology available. Additionally, informative grouting education is readily available, such as that presented at the Annual Short Course on Fundamentals of Grouting, sponsored by the University of Florida, annually since 1978. The result is an ever-growing number of well-informed grouting professionals actively engaged in a continuing effort to advance the technology.

Geotechnical grouting can benefit many soil deficiencies. Properly applied, it can solve innumerable and sometimes vexing problems to the benefit of all involved. Because of the confusion and misinformation promulgated by so many within the industry, enormous opportunity is available to informed professionals to advance the science. Although it is not possible to cover every detail of the many facets of geotechnical grouting in a single chapter such as this, it is the writer's fervent hope that the reader has become sufficiently informed on the technology, that he will never again become confused or intimidated by other grouters and will, rather, become part of the ever-expanding group of enlightened grouters. Grouting is a science, and anybody that says otherwise is either ill informed or a liar.

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