GROUND STABILIZATION
REVIEW OF GROUTING AND FREEZING TECHNIQUES FOR UNDERGROUND OPENINGS

AUGUST, 1975

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16. Abstract

The ground stabilization techniques of grouting and artificial freezing are reviewed. General grouting considerations are discussed including selection of grouts and techniques of injection. Materials for both particulate and chemical grouts are described along with their influence on ground properties and advantages and disadvantages. Artificial ground freezing is discussed in terms of techniques of freezing, strength-deformation and thermal considerations, and advantages and disadvantages. Selected case histories which illustrate the application of grouting and freezing to tunnel and shaft construction are briefly summarized.

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PREFACE

This review was performed by the Department of Civil Engineering of the University of Illinois at Urbana-Champaign, Urbana, Illinois during the period August 1974 to August 1975. The investigation was sponsored by the Federal Railroad Administration, Department of Transportation, through contract No. DOT FR 30022 under the technical direction of Mr. William N. Lucke.

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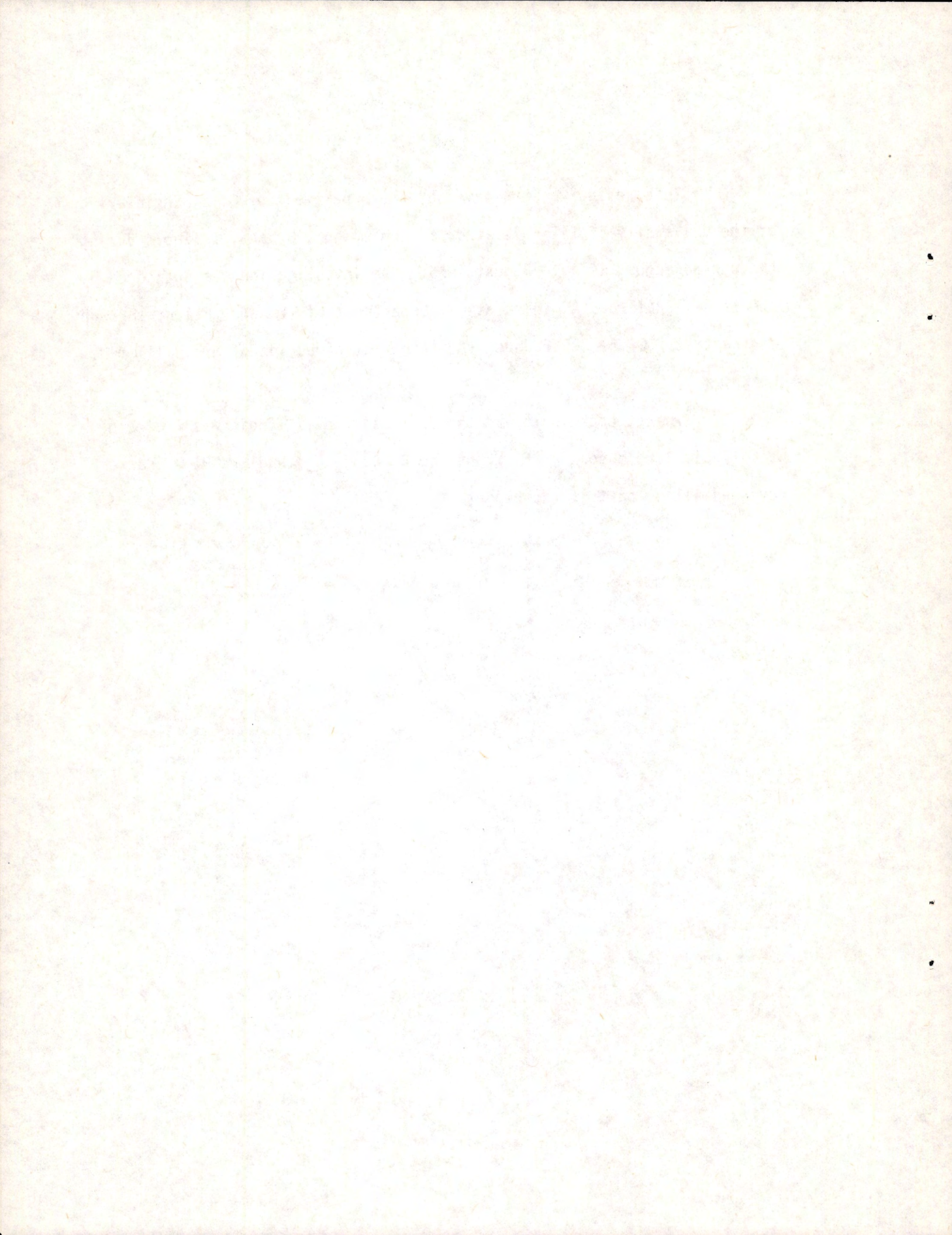
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CHAPTER 1

INTRODUCTION

This report presents a review of the techniques of grouting and artificial freezing for ground stabilization for use in construction of underground openings. These ground stabilization techniques may be used in special instances to make feasible a given construction method or to reduce adverse effects of construction on adjacent existing facilities. Feasibility of certain construction methods under soft ground and poor rock conditions may be determined primarily by the interrelated factors of ground stability and water inflow. Grouting or artificially freezing the ground in such circumstances may reduce or eliminate the need for such procedures as pilot drifts, full face breasting, forepoling, compressed air, etc. Dewatering is frequently a more economical means of ground stabilization and water control; however, in some cases, the formation in question cannot be successfully dewatered, or dewatering may actually be more expensive than grouting or freezing. The report focuses on those conditions where grouting or freezing can be successful and economical.

A major problem in construction of soft ground tunnels in urban areas is settlement of the ground surface and subsequent effects on existing facilities. The amount of settlement is closely related to construction procedures as well as ground conditions. In some cases expected settlement will dictate the need for underpinning adjacent structures and utilities at substantial cost. The construction of any tunnel produces a change in the state of stress in the surrounding ground and the resulting strains and

displacements may cause surface settlements in certain instances. In some cases grouting or freezing may improve the stress-deformation properties of the ground so that this portion of the settlement may be reduced. Generally, the major portion of surface settlement is related to construction procedures and loss of ground. In a dense sand for example, the settlement due to the changed state of stress around the tunnel may be insignificant. However, intolerable settlements in such ground can develop from "runs" in the heading, from voids behind the shield tail if improperly filled or from an inadequate expansion of the primary lining. In some cases grouting or freezing may improve face stability and may increase the ground's standup time so that the primary lining can be installed and expanded or the annular space filled before significant loss of ground occurs.

Even though both grouting and freezing are relatively expensive techniques, they may frequently be of great benefit. Both techniques are often of a proprietary nature with specialized firms performing the operations. As a result, general contractors and designers are often unfamiliar with the techniques and thus reluctant to include them in the original design or planned construction procedures where they may in fact be quite appropriate.

This report describes techniques of grouting and artificial freezing including the materials, construction procedures, applicability to different ground conditions, influence on soil properties, basic design considerations, and advantages and disadvantages of the techniques. Applications of the techniques to various tunnel ground conditions are briefly discussed and a few selected case histories are summarized. This report is based primarily on a review of published literature and manufacturer's information supplemented by conversations with contractors and limited field observations of applications.

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Chapter Two deals with general considerations for grouting soil and rock including selection of a particular grout and methods of injection. Chapters Three and Four discuss the particulate cement and clay cement grouts and the silicates, acrylamide and other chemical grouts, respectively. Chapter Five treats the application of grouting to tunnel construction and includes illustrative case histories. Artificial freezing is covered in Chapter Six including application to tunnel and shaft construction and case histories.
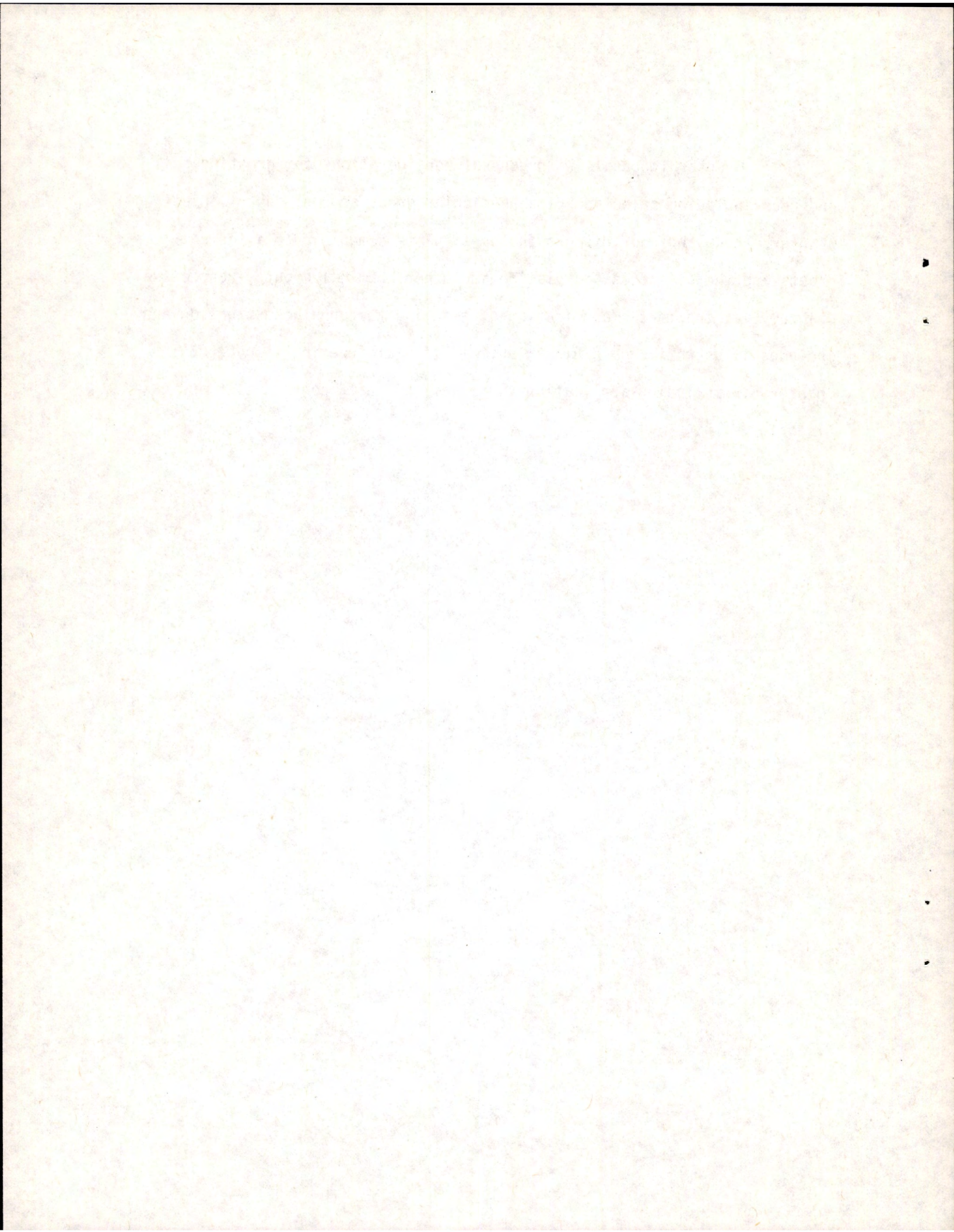
CHAPTER 2

GENERAL CONSIDERATIONS FOR GROUTING

2.1 HISTORICAL DEVELOPMENT

It is believed that injection grouting was first used in 1802 by Charles Berigny (1772-1842) in the course of repairing a scouring sluice at Dieppe, France, using grouts of clay and hydraulic lime injected with a piston plunger pump. This method proved to be successful and was used on other hydraulic works in France during the mid 19th century. Cement grouting was introduced in England in 1856 by Kinipple and was used from 1864 for filling the voids behind the lining of shield driven tunnels.

The application of chemical grouting dates to 1887 when Jeziorsky obtained a patent for the injection of soils with sodium silicate solutions. The Belgian engineer Albert Francois in 1914 used a process which he called "silicatization" that consisted of injecting solutions of aluminum sulfate and sodium silicate to act as lubricants before the injection of cement grouts. This technique was used to successfully sink large mine shafts through difficult ground water conditions (Glossop, 1960).

In 1925 Hugo Joosten, a Dutch mining engineer, patented a process for treating sandy alluvial soils using successive injections of a concentrated sodium silicate solution followed by a saturated solution of calcium chloride. This calcium chloride/sodium silicate injection process has been called the "two-shot" or "two-solution" process.

In 1934 Charles Langer developed a "one-shot" or "one-solution"

process using sodium silicate and a metallic salt reagent to control the set time of the gel. The search for an improved one-solution method using sodium silicate continued and in the 1950's ethyl acetate, an organic ester reagent, was developed to control the set time of the silicate, causing the silicate to gel by changing to acetic acid at a controlled rate. Other chemicals that have been used change the pH of the silicate solution to control the set time and maintain the high strength of the silicate gel.

A non-silicate one-solution process with a controlled set time, developed in the early 1940's, was the use of lignochromes such as lignosulfites or lignosulfonates that formed gels when mixed with bichromate in solution.

A rapid advancement of polymer science in the 1950's focused attention on applying these chemical polymer systems to soil impregnation. Acrylamide is a chemical system which forms cross linked polymers from an initial monomer solution at a controlled rate, when the monomer solution with a catalyst and an oxidizer initiator is injected into the soil. Acrylamide chemical grouts have proven successful in grouting fine sands and silts because the chemical solution has a low viscosity and a wide range of controlled set time.

2.2 SELECTION OF GROUTS FOR SOILS

Terzaghi (1936) stated that the limit of effective grain size for cement grouting was 1.4 mm for compact sands and 0.5 mm for loose sands. For sands with smaller grain size the cement grout would merely displace the sand and not fill it. Cambefort in 1967 also presented soil grain size curves illustrating limits of application of the various grouts. Figure 2.1 shows the

limits of grain size for successful injection of various grouts. The relationships shown in Figure 2.1 are based on cases of successful injection where at least 90 percent of the particles in the soil being grouted were larger than the limits shown for the particular grout type.
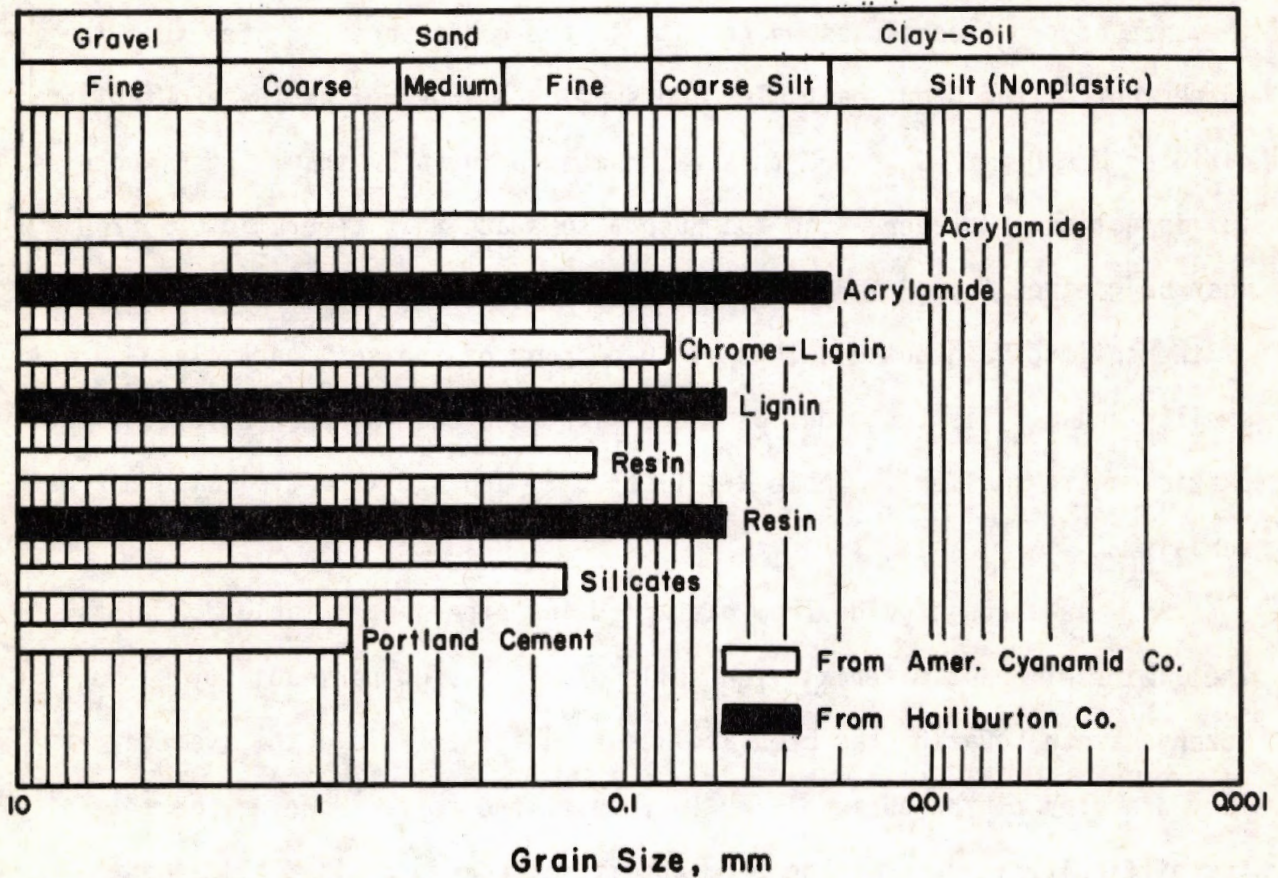
FIGURE 2.1 COMPARISON OF METHODS FOR STABILIZING
SOILS AND RELATIVE PENETRABILITY
(FROM CORPS OF ENGINEERS, 1973)

For particulate grouts, King and Bush (1961) relate the maximum size of grout particles to the size of pores in the soil to be grouted for successful injection. They use D_{15s} as a measure of the pore size of the soil to be grouted where D_{15s} is the particle size of the soil such that 15 percent of the soil particles are smaller. Similarly, D_{85g} is a measure of the size of grout particles where D_{85g} is the grout particle size such that 85 percent of the grout particles are smaller. They define the groutability ratio as $N = D_{15s} / D_{85g}$ and this ratio must be greater than 15 for successful injection. However, King and Bush also state that the ratio D_{10s} / D_{95g} must be greater than 8 for successful injection (D_{10s} is the particle size of the soil to be grouted such that 10 percent of the soil particles are smaller and D_{95g} is the grout particle size such that 95 percent of the grout particles are smaller). Table 2.1 lists D_{85g} and D_{95g} for typical grout materials.

As a rough guide from past grouting experience, Table 2.2 lists the limits of grout materials for soils of decreasing permeability. The Kozeny Carman Equation has been applied to soils to relate the average capillary diameter to the permeability (water) of the soil. However, because of the difficulty in determining permeability of coarse grained soils and assumptions and approximations in the Kozeny Carman Equation, the use of permeability in determining groutability should be considered only as a rough guide.

2.3 SELECTION OF GROUTS FOR ROCK

Most rock grouting is done using cement grouts because of their

TABLE 2.1

D_{85g} AND D_{95g} FOR TYPICAL GROUT MATERIALS

(AFTER KING & BUSH, 1961)

Grout	D_{85g}	D_{95g}
Type I cement	67 microns	74 microns
Type II	40	52
Type III	25	34
Type III scalped	17	26
Fly ash	33-52	47-100
Ground slag	18	27
200 mesh silica flour	36	51
Clays	26	50
Bentonite	1.5	7
Asphalt emulsion	1.8	2.2

TABLE 2.2

LIMITS OF GROUTABILITY AS A FUNCTION OF PERMEABILITY

Grout	Limiting permeability of material to be grouted
Cement	$k = 3 \times 10^2$ ft/day (1×10^{-1} cm/sec)
Clay	$k = 2 \times 10^2$ ft/day (8×10^{-2} cm/sec)
Silicates	$k = 1$ ft/day (5×10^{-4} cm/sec)
Acrylamide AM-9	$k = 3 \times 10^{-2}$ ft/day (1×10^{-5} cm/sec)

high strengths, low cost, and relatively large size of rock discontinuities. Various numbers have been given as to the minimum fissure size that could be injected with cement grouts. Terzaghi in 1936 spoke of experiences at Bon Hanifia Dam, Algeria, in grouting fissured rock, indicating that a fissure smaller than 0.004 in (100 microns) could not be grouted with cement. Kennedy (1961) stated that tests showed it possible to grout effectively a joint having a width of 0.005 in (125 microns) using a cement screened through 200 mesh. The Corps of Engineers indicate the ratio of crack width to grout particle size should be at least 3 for successful injection (Corps of Engineers, 1956). Type I cement has a D_{95} of 74 microns. If effective grouting requires a fissure three times as wide as the maximum grain size of the grout, the smallest fissure which could be grouted with Type I cement would be 220 microns. For a Type III scalped cement with a D_{95} of 26 microns the smallest fissure that could be grouted would be 80 microns.

Grouting specifications commonly call for a maximum grout pressure of 1 psi per foot of overburden depth (22.6 kPa/m) assuming rock with no tensile strength. It is difficult to evaluate factors such as stratification, jointing, and changes in tensile strength with depth. As a result, field tests should be used to evaluate these factors. A graph of flow vs. pressure from water pressure tests can be used to decide the critical pressure before excessive hydrofracturing takes place during grouting. For shales, Morganstern and Vaughan (1963) found that the injection pressure varied from 2.4 to 1.2 times γh where γ is the unit weight of the rock and h is the overburden depth. For these shales the unconfined compressive strength varied between 2000 and 6000 psi (13.8 and 41.4 MPa). It was found that the apparent cohesion decreased

with depth of hole so that P/hγ (P is grouting pressure) approached 1 when grouting was carried out at increasing depths. The allowable injection pressure depends primarily upon the strength of the rock, on the in situ stresses, on the pore pressure existing in the rock prior to injection, and on structural discontinuities (joints, foliation, bedding, etc.). As a rough guide, Fig. 2.2 shows safe rock grouting pressures.
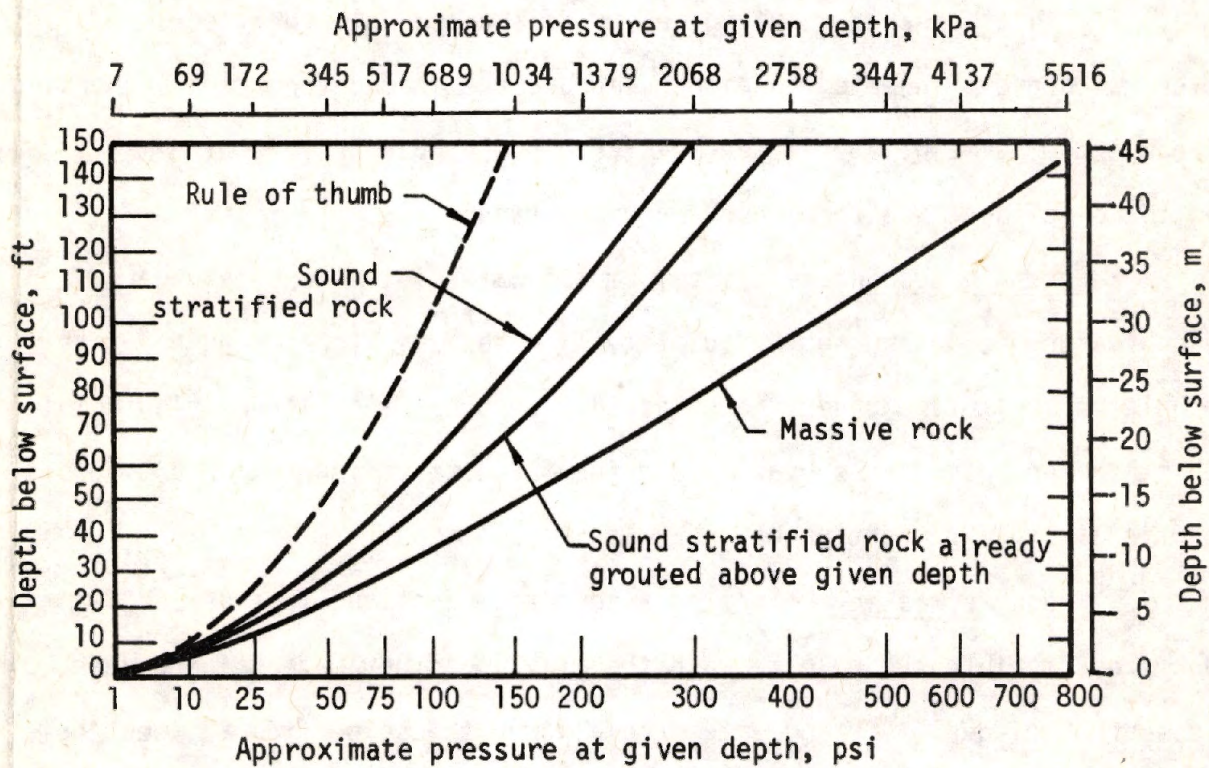
Approximate pressure at given depth, kPa

FIGURE 2.2   ROUGH GUIDE FOR ROCK GROUTING PRESSURES
(AFTER CREAGER, JUSTIN AND HINDS, 1945)

2.4 GENERAL GROUTING DESCRIPTION

2.4.1 GROUTING TERMS

Curtain grouting refers to drilling and grouting a linear sequence of holes to construct a curtain or barrier. The grout curtain may be made of a single line of holes or of multiple parallel lines.

Blanket or area grouting refers to shallow holes drilled on a grid pattern with grout injected to either improve the load-deformation properties or reduce the permeability of the fractured rock. Blanket grouting may be used to consolidate fractured rock around a tunnel.

Pressure testing refers to pumping water into the hole under a given pressure to determine the quantity of water which can be pumped into the hole per unit time under the selected pressure. This information combined with other geologic data serves as a guide for determining the need to grout a particular hole or portion of the hole.

Pressure washing refers to alternate injection of water and air under pressure into the hole to wash the clay and other material from the hole or fissure surfaces. Air and water are allowed to escape from adjacent holes carrying the loose material with it.

Split spacing or closure grouting is a technique of grouting whereby initial (primary) grout holes are located at wide spacings, usually 20 or more feet and grouted to refusal. Secondary holes spaced midway between the primary holes are then drilled and grouted to refusal. Tertiary holes are then drilled dividing the interval between the primary and secondary holes in half and are grouted until refusal. The grout take per foot of hole usually decreases as the holes are drilled and grouted at successively closer intervals.

2.4.2 GROUTING METHODS

Stage grouting, series grouting, circuit grouting, and packer or stop grouting are common grouting methods in use. In all of these methods the split spacing or closure grouting technique is used to determine final grout hole spacing.

For stage grouting the holes are drilled and grouted in depth increments until the final depth is reached. The depth increment to be used is determined based on the known geologic conditions and on optimum depth for grouting modified during drilling if drilling fluid is lost or gained or if rod drops occur. After the first stage is grouted the hole is jetted and cleaned before the grout hardens so that redrilling will not be required. This stage grouting is carried out in all primary holes for the first depth increment. The depth of the primary holes is then increased and the process of washing, pressure testing, pressure washing, and grouting at higher pressures is repeated. This cycle is repeated for increasing depth of the primary holes until the ground is tight (grout take minimal) to the required depth. The entire procedure is then repeated for the secondary grout holes, and then for the tertiary holes.

In series grouting each successive deeper zone is grouted from the top of the hole using newly drilled holes. There is no washing out of former drilled holes as in stage grouting. The first upper zone holes are drilled and grouted using successive split spacings of secondary and tertiary holes until the zone is tight. The next lower zone is then grouted using primary, secondary, and tertiary holes split spaced until that zone refuses grout. The process continues to the full depth required.

In <u>circuit grouting</u>, supply and return lines are connected to the grout hole at a header and the injection pipe extends to the bottom of the hole. The grout is pumped to the bottom of the hole through the injection pipe and the excess grout not taken by the formation returns in the annular space between the injection pipe and the hole wall. The return line empties into a holding tank and the circuit is complete. In circuit grouting the total length of the hole is exposed to the circulating grout. A feature of circuit grouting is that the circulation of the grout prevents the sedimentation and segregation of solid particles of the grout slurry. Premature plugging of the smaller fractures is minimized.

In <u>packer or stop grouting</u>, a packer is used to isolate part of the hole for grouting. The hole is drilled to the full depth and a packer is used to separate the hole into segments or zones for grouting purposes. Three types of packers are common--the leather cup packer, the mechanically expanded packer, and the pneumatic packer. Figure 2.3 shows these three packers, discussed in detail by Lippold (1958). A leather cup packer is best suited to fairly hard rock where the drilled hole is not oversize and the walls are relatively smooth and true. This packer has been used successfully at grouting pressures up to 750 psi (5200 kPa). The mechanically expanded rubber ring is adapted to somewhat poorer rock but it may be difficult to seal if the hole is too much oversize. The pneumatic packer is suitable in poor rock where the drill hole is oversize. The length of rubber sleeve in the pneumatic packer should not be less than 18 in. (45 cm). This type of packer can be used with pressures up to 100 psi (700 kPa) in poor conditions and up to 200 psi (1400 kPa) in good conditions.

3/4" Grout
supply pipe

1-1/2" Hard
leather cup

Leather cup packer

Grout supply
pipe

Air supply
line to
packer pipe

Grout supply pipe

Adjusting nut

EX grout hole

1" pipe

2" pipe

4" to 6" soft
rubber hose

Mechanical packer

Rubber tube
expanded by
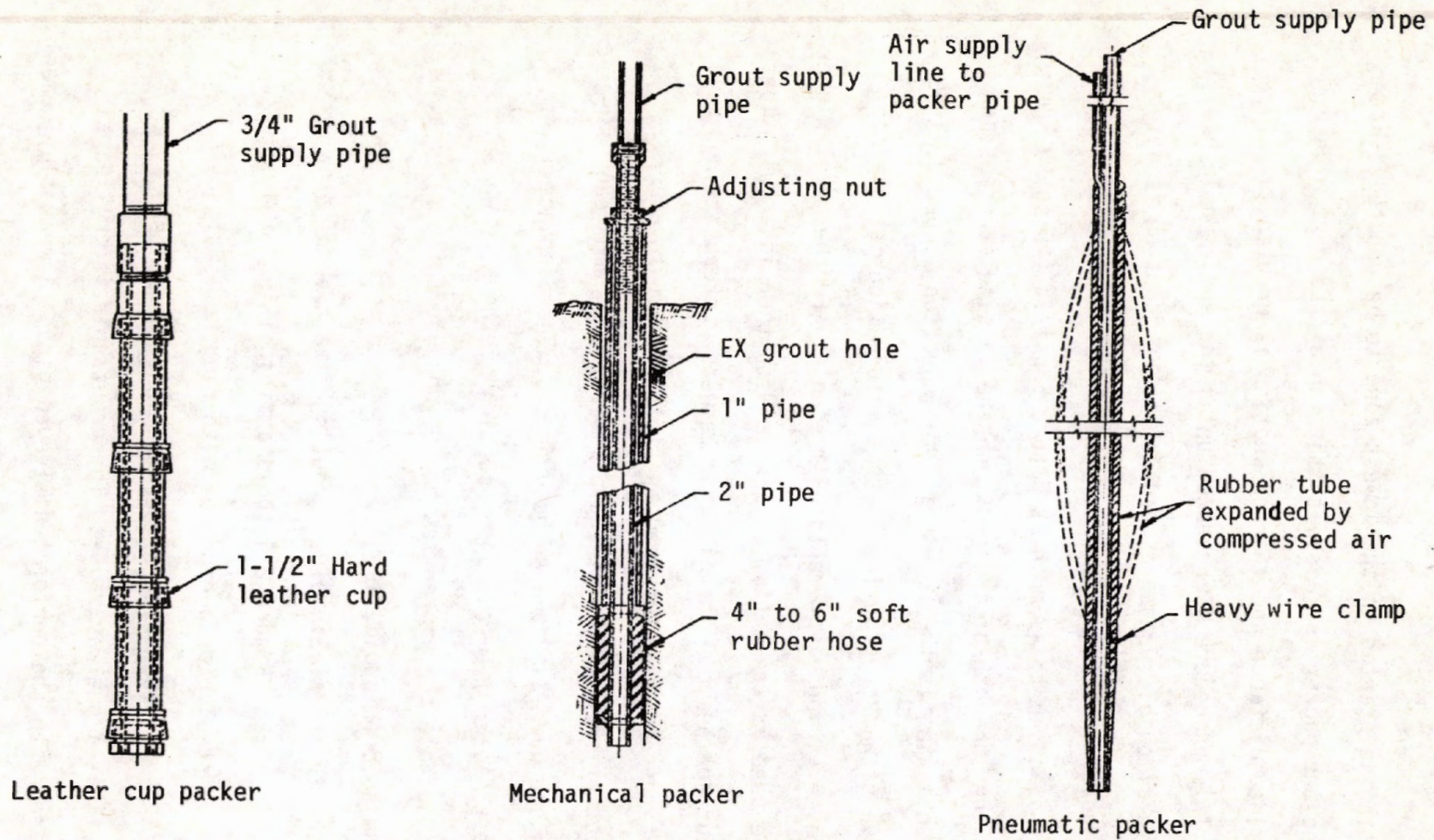compressed air

Heavy wire clamp

Pneumatic packer

FIGURE 2.3 TYPES OF PACKERS
(AFTER LIPPOLD, 1958)

Packer grouting is started in the lowest zone to be grouted. After completion of each zone the packer is raised to the top of the next higher zone or to a predetermined distance and the new section is grouted, usually at a lower pressure. This procedure is repeated until the entire hole has been grouted. The safe allowable pressure used in packer grouting is determined by the depth of cover over the packer and will vary from as low as 0.75 psi/ft (20 kPa/m) of cover to as much as 2 to 2.5 psi/ft (45 to 55 kPa/m) of cover depending on the strength of the rock, on the in situ stresses and on the pore pressures existing in the rock prior to injection (Lippold, 1958).

One advantage of packer grouting is that information is obtained as to the grout take over various increments of the hole. Furthermore, higher grout pressures can be used at the bottom of a hole with the packer with less chance of producing surface heave. The distance between packer settings will vary with local conditions and is determined by field trial. In general, packer settings are established so that the available grout pumps can maintain the desired pressure over the zone to be grouted. The distance is not less than 10 to 15 ft (3 to 5 m) in the upper section of the hole while for the lower section of the hole the packer setting may be 50 ft (15 m) or more from the bottom of the hole (Lippold, 1958).

Tube a Manchette or "sleeve pipe" grouting is a type of stop grouting invented by Ischy in 1938. Figure 2.4 shows a typical *Tube a Manchette*. The device consists of a steel pipe perforated with rings of small holes at intervals of 12 in. (30 cm). Each ring of perforations is enclosed by a rubber sleeve that acts as a one way valve. The grout hole is drilled to the full depth and cased with the perforated sleeved pipe. A weak clay cement

Double packer

Wall of grout hole

Clay-cement mixture
(sleeve grout)

Rubber manchette (band)

Grout orifice

Tube a Manchette (sealed
into grout hole)

Grouting pipe
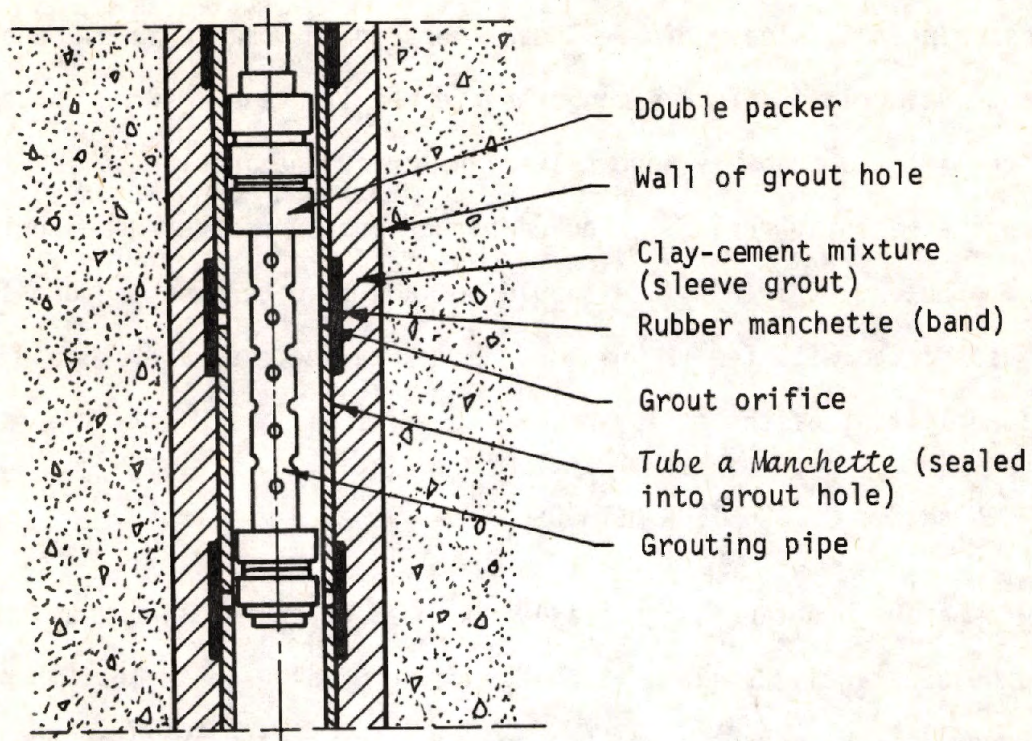
FIGURE 2.4   *TUBE A MANCHETTE* (AFTER ISCHY, 1948;
ISCHY AND GLOSSOP, 1962)

mixture is pumped into the annular space between the ground and the perforated
sleeve pipe. A small diameter injection pipe fitted with two opposite leather
cup packers is lowered into the sleeve pipe to the zone to be grouted. Grout
is pumped into this inner pipe and out the perforated holes of the sleeve pipe,
raising the rubber sleeve valve and rupturing the clay cement annular grout to
penetrate into the soil. The rubber sleeve acts as a one way valve preventing
the return of the injected grout. Clay-cement grouts can be pumped first into

the large voids and then chemical grouts can be injected into the finer cavities using this sleeve pipe. These sleeve pipes remain in place so further grouting in the future can be performed if needed. Grout pressures needed to lift the rubber sleeve valves and crack the annular solidified grout before reaching the soil require higher pump pressures than for other types of grouting. Field trials should be made to determine the pressure losses associated with the use of the *tube a manchette* so as to avoid fracturing and lifting of the soil.

2.4.3 COMPARISON OF GROUTING METHODS

Series grouting is generally the most expensive since new holes have to be constantly drilled. Stage grouting usually costs less than series grouting but more than packer grouting, since the drilling equipment must be moved and set up over the hole several times. Stop grouting or packer grouting is usually the cheapest and fastest grouting method. It is the method least likely to produce lifting of the rock (since grout pressures can be reduced near the top of the hole, yet deeper sections can be grouted under higher pressures if required). Holes drilled for packer grouting are larger than for stage or series grouting in order to place the packer into the hole. Problems with packer seating are common.

CHAPTER 3

CEMENT AND CLAY-CEMENT GROUTS

3.1 TYPES OF GROUTS AND FILLERS

3.1.1 NEAT CEMENT GROUTS

Any of the ASTM standard Portland cements or special purpose cements (such as expansive, regulated-set, etc) having the desirable setting and strength characteristics may be used in cement grouts. Portland Type I cement is the general purpose cement most often used. Type II is the sulfate resistant low heat of hydration cement that develops its strength at a slower rate than Type I. Type II cement is used where the ground water contains moderate amounts of soluble sulfates. If the ground water has a very high concentration of sulfates, then Type V should be used. Type III cement produces high early strength and has a finer grind than Type I or Type II and thus may be desirable in certain formations. Air-entrained cements Type Ia, IIa, IIIa, should be avoided as the Corps of Engineers has reported difficulties in pumping grouts containing air-entrained cement and that grouting may be unsatisfactory (Corps of Engineers, 1966). Portland cement is most often used with water in a mix of cement: water of 1:(0.75-4.0) by volume.

A wet-ground, water-quenched blast furnace slag will produce a very fine cement (slag cement) that has a Blaine specific surface of 4000 to 6000 cm^2/g, and has superior sulfate resistance properties. This cement

is catalyzed by sodium hydroxide in the proportion of sodium hydroxide: cement of 0.5:100 (weight), added just before grouting.

Resin gypsum cements are available for extremely fast set times. The initial set is achieved in 30 to 90 minutes and a compressive strength of 1500 psi (10 MPa) is achieved 10 minutes after initial set (King and Bush, 1961).

3.1.2 SAND-CEMENT GROUTS

The sand in the sand-cement grout acts as a filler and extends the cement. Sanded grouts are commonly used for grouting foundation rock containing large voids; they should not be used to grout rock that contains small cracks as the sand will not be able to penetrate the fissures. Sand-cement mixes should not be used in holes that do not readily accept thick neat cement grouts. Any finely divided mineral admixture such as fly ash or diatomite may be used to keep the sand in suspension.

The sand used should be well graded. A mix of cement:sand of 1:2 (weight) can be successfully pumped if at least 95 percent of the sand passes the No. 16 sieve and 10 percent or more passes the No. 100 sieve. (Corps of Engineers, 1966). For filling of large voids, cement:sand proportions of 1:(5-10) weight have been used. (King and Bush, 1961).

3.1.3 CLAY-CEMENT GROUTS

Clay-cement grouts are used to seal off water flow in gravels, coarse sands, and rock fissures. Any finely divided, inert mineral filler may be used to extend the cement. In many cases natural clays may be available locally and thus are usually the cheapest filler. Depending

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on the grain size or fissure size of the material to be grouted, commercially prepared bentonite may be required or natural clay may have to be processed to remove the fine sand and silt fraction and ensure successful injection. Clay-cement grouts give good adherence to clay coated surfaces of rock fissures but develop lower compressive strengths than neat cement or sand cement grouts. Typical proportions for clay cement grouts are cement:clay: water of 1:(2-3):(3-6) on a weight basis (dry weight of cement and clay) (Leonard and Grant, 1958; Mayer, 1958; Kravitz, 1958; Johnson, 1958).

Clays are stable in alkaline suspensions but tend to flocculate when the pH is lowered to acidic. Small amounts of additives called peptizers may aid in keeping the clay in suspension. Bentonite, sodium silicate, or colloidal silica added to the grout mix in 0.01 to 1 weight percent of the grout mix will keep the particles from settling out. Other additives that prevent the clay from settling out and lower the viscosity of the clay suspension when used in small amounts are sodium hydroxide, sodium phosphate, sodium tripolyphosphate, sodium hexametaphosphate, and quebracho extract (Van Olphen, 1963; Rebinder, 1967).

3.2 TYPES OF ADMIXTURES

American Concrete Institute Committee 212 has prepared an extensive report on admixtures for concrete which is the basis for this section. Some of the more important points related to grouting are summarized in the following paragraphs.

3.2.1 POZZOLAN ADMIXTURES

Pozzolans are siliceous or siliceous and aluminous materials which chemically react with cement and moisture to form compounds with cementitious

properties. Fly ash, diatomite, and pumicite are typical pozzolans used in grouting. Fly ash is a residue from the combustion of coal and generally has a fineness about the same as Portland cement. Diatomite consists of minute fossiles which when processed has a fineness several times that of cement. Pumicite is finely pulverized volcanic ash having a fineness about that of fly ash or slightly finer.

3.2.2 EXPANSIVE ADMIXTURES

Aluminum powder reacts with cement to form gas bubbles which cause expansion of the grout and tend to reduce segregation. Aluminum powder is added in concentrations of 0.005 to 0.02 weight percent of cement to offset the shrinkage which takes place as the grout sets.

3.2.3 SET-MODIFYING ADMIXTURES

Accelerators lower the set time of the grout and increase the rate of early strength gain. Calcium chloride is the most commonly used accelerator and is added to the mixing water in concentrations of up to 2 weight percent of cement. Calcium chloride is the least expensive of the accelerators (Corps of Engineers, 1966). Triethaniolamine is an organic compound which is also used in small quantities to lower the set time.

Retarders increase the set time and decrease the rate of early strength gain which may be useful if grout has to be pumped long distances or grout holes redrilled. Calcium lignosulfonate is a retarder and has the property of lowering the viscosity and reducing the water content of the cement grout. Two or more admixtures of different types may be used

to achieve the desired characteristics of the grout. However, tests may be necessary to evaluate the effects of the admixtures on the grout properties unless such information is already available.

3.3 TECHNIQUES OF INJECTION

Much of the information in Chapter 2 on techniques of injection is applicable to all types of grouting, including clay and cement grouts.

If the hole is primed before grouting with lubricants or chemicals that lower the surface tension and have a lyophilic effect, then the grout take may be increased. Sodium silicate, colloidal silica, sodium alkayl aryl sulfonate, and sodium lauryl sulfonate are examples of such materials.

In formulating clay-cement grouts the clay slurry should be made first, with the cement then added to the clay slurry. The proper electrolyte or additive should be added to the water to keep the clay and cement in suspension. High velocity mixers should be used for cement, sand-cement, or clay-cement grouts as the high rate of shear produced by the mixer will increase the stability of the grout and keep the solids in suspension.

Bleeding should be minimized by using the lowest possible water-cement ratio, as bleeding reduces the strength and impermeability of the grouted mass. Calcium lignosulfonate and the alkyl aryl sulfonates decrease bleeding by lowering the water-cement ratio required for the grouts. These chemicals also act as retarders, so that accelerators such as calcium chloride may also be needed to achieve desired set times.

3.4 INFLUENCE ON SOIL PROPERTIES

Neat cement grouts or sand-cement grouts injected into soils produce solids with compressive strengths comparable to that of concrete and also decrease the permeability of the soil. Clay-cement grouts have compressive strengths of only a few hundred psi depending on the amount of clay used in the mix and are also effective in reducing the permeability of soils.

3.5 ADVANTAGES AND DISADVANTAGES

Sand-cement and clay-cement grouts are less expensive than neat cement grouts. Chemical grouts (next section) are more expensive than neat cement. The hazards of using cement grouts are minimal and the beneficial effect of the cement grout is permanent. Cement grouts have the disadvantage of having particles of large diameter which are in suspension and thus have a limited range of penetration. The control of set time is more difficult than for the chemical grouts, and it may be difficult to successfully grout a formation with sand-cement or clay-cement grouts if ground water is flowing.

CHAPTER 4

CHEMICAL GROUTS

4.1 SILICATE SYSTEM

4.1.1 LIMITS OF APPLICATION

Figure 2.1 shows the limits of injecting fine sands with sodium silicate solutions. Sodium silicate solutions can be injected into sands having permeabilities of about 1 ft/day (5×10^{-4} cm/sec) or greater.

4.1.2 DESCRIPTION OF THE CHEMICAL SYSTEM

Sodium silicate is made by fusing sand and sodium carbonate (soda ash) in an electric resistance furnace at 1200 to 1400 degrees Centigrade. Various glasses of $SiO_2:Na_2O$ composition can be made but only those glasses with less than a 4:1 weight ratio of $SiO_2:Na_2O$ can be dissolved in water. The most common glass made is of a 3.3 $SiO_2:Na_2O$ composition which is then dissolved in water (Vail, 1952; Iler, 1955).

J. N. von Fuchs in 1825 coined the word "waterglass" in his studies of sodium silicate solutions. A typical commercial sodium silicate solution is of a 3.22:1 $SiO_2:Na_2O$ weight ratio and contains 8.9% Na_2O and 28.7% SiO_2 by weight in solution. The solution at a pH of 11.4 has a specific gravity of 1.394 (41 degree Baume) and a viscosity of 180 centipoise (.18 Pa·s). (Philadelphia Quartz Bulletin).

The silica is in solution as a negatively charged silicate ion at pH above 11. When the pH is lowered to 10 the silicate ion changes to

partially polymerized silicic acies and colloidal aggregations. Gelling of the silicate to a hydrous silica network is most rapid at pH of 5-7. Chromate ions have a retarding effect on the rate of gelation as they form soluble complexes with the silicate ions.

The addition of a salt of a polyvalent metal to a solution of a soluble silicate results in the formation of an insoluble silicate precipitate. Calcium chloride is the principal salt reagent used to effect precipitation. This method is called the "two-shot" or "two-solution" process invented by Joosten in 1925. The calcium chloride is available in type 1 solid flakes of 70-80 percent purity or in type 2 flakes of 94-97 percent purity. Calcium chloride can also be obtained in 40 or 55 percent weight concentrated liquids.

Sodium silicate has also been combined in a one solution injection system with controlled set time. Sodium bicarbonate, hydrochloric acid, with copper sulfate are reagents used with sodium silicate to form gels which are weak and more susceptible to disintegration. The concentration of added acid controls how fast pH is lowered and thereby how fast gelling occurs. The bicarbonate and hydrochloric acid produce gels that are washed away in a few months time due to the leaching action of the ground water (Cambefort and Caron, 1957).

Several reagents have been developed that undergo chemical changes to produce acids which cause the silicate to gel. Ethyl acetate reacts with the alkaline sodium silicate solution to produce acetic acid and ethyl alcohol. Glyoxal when added to a sodium silicate solution forms glycolic acid

which makes the silicate gel. Formamide is hydrolyzed to ammonium formate when mixed with the alkaline sodium silicate solution and gels the silicate. The three reagents listed above are used with accelerators. Small additions of sodium aluminate or calcium chloride are added to control the set time of the gel. The formamide used may vary from 5 to 21 percent (weight) of the silicate (SiO_2) in the solution. Sodium aluminate is used as an accelerator in concentrations of .6 to 10 percent (weight) of the SiO_2 in solution.

When injecting sands the sodium silicate solution is often diluted to lower the viscosity and improve the degree of penetration. Solutions with the SiO_2 of less than 10 percent should not be used as they form weak gels.

Colloidal silica is also available with the silica in colloidal size of 50-150 Angstroms and in concentrations of 30 to 50 percent, at viscosities of 5-20 centipoise (5×10^{-3} to 2×10^{-2} Pa·s). These solutions are stable at pH of 8 to 10 with small amounts of sodium ion present (Iler, 1973). The colloidal silica should increase the range of penetration as the viscosity is lower than that of sodium silicate in solution.

4.1.3 TECHNIQUES OF INJECTION

For the two solution sodium silicate-calcium chloride system, piston pumps or progressing cavity pumps (Moyno pumps) may be used. The sodium silicate and calcium chloride are not corrosive so that the mixing tanks and lines can be fabricated of mild steel. The pump chamber should be of stainless steel for long life.

In the two solution process separate mixing tanks and pumps are

required for the sodium silicate and calcium chloride solutions. The calcium chloride obtained in solid form is dissolved to form a saturated solution. A separate water pressure line is needed. The three lines carrying sodium silicate, calcium chloride and water are connected to a three way valve near the head of the injection pipe.

The maximum depth of penetration of the pneumatically driven injection pipes used in the two solution process is 60 ft (18 m). The grouting pipe can have a lost point tip which is pushed out when the proper depth is reached so that the chemical grout can be injected from the end of the pipe. Another technique is the driving of a pipe with a fixed cone at the end. When the depth is reached grouting is carried out through perforations along the perimeter of the pipe near the cone.

After the pipe is driven to the proper depth and the tip pushed out, pumping of the grout is started with the silicate solution being injected first, followed by a slug of water, then a calcium chloride solution. A three way valve at the head of the pipe is used to alter the flow of the solutions. The more viscous silicate should always be pumped first followed by the less viscous calcium chloride solution which penetrates the silicate in the ground. The slug of water that separates the two solutions clears the pipe and prevents solidification to take place in the pipe. The injection pipe is pulled out of the hole in stages with grouting performed in each stage.

In the one-solution grouting method the sodium silicate solution and the reagent solution are separately pumped to the mixing chamber, then mixed and injected down the grout hole. The proportioning pumps are typically progressing cavity pumps connected to a variable speed motor and drive.

The pumps, mixing tanks, valves, and lines should be constructed of stain-less steel as some of the reagents are corrosive and as the equipment can also be used to inject other chemical systems (acrylamides) which are much more corrosive.

The sodium silicate is placed in one tank and pumped to the mixing nozzle near the head of the injection pipe. The reagent and accelerator are dissolved in the other tank and pumped to the mixing chamber. By changing the speed of the proportioning pumps the flow rate of the reagent solution or silicate solution can be increased or decreased thus altering the gel set time. As an alternative, three tanks and proportioning pumps may be used with the third tank holding the accelerator for finer control of set time.

Rotameters should be placed on the solution lines to indicate the pumped flow rates. All gages should be protected from direct contact with the grout fluid to avoid corrosion and plugging. Rubber diaphragms are used to separate the grout fluid from the gage pressure transmitting fluid. Glycerol or oil may be used for the gage pressure transmitting fluid.

Grout injection can be made either by using pneumatically driven pipes which are withdrawn in stages, or by using drilled holes set with packers. Whatever the mechanical system used to inject the grout, the use of propor-tioning pumps to feed the silicate and reagent solutions to the mixing cham-ber near the head of the injection pipe provides for flexibility of set time for the varied flow conditions that may be encountered.

The one solution three component silicate grout can be combined with a neat cement, sand cement or clay cement grout to control the set time in areas where the external hydraulic gradient is large and tends to wash away the cement grout.

4.1.4 INFLUENCE ON SOIL PROPERTIES

The compressive strength of sands grouted with the sodium silicate system will depend on the relative density of the sand before grouting, the concentration of sodium silicate, and the reactant used to gel the silicate. In general, compressive strengths up to several hundred psi have been achieved (Corps of Engineers, 1973).

If drying and shrinkage is prevented, sands grouted with sodium silicate will have very low permeabilities (on the order of 10^{-7} cm/sec or less). The decrease in permeability and increase in strength will be of a more or less permanent duration depending on the concentration of silicate and groundwater conditions. The Corps of Engineers recommends grouts containing less than 30 percent silicate by volume be used only where the grouted material is in constant contact with water or only for temporary stabilization (Corps of Engineers, 1973).

4.1.5 ADVANTAGES AND DISADVANTAGES

The silicate grouts can be used to penetrate fine sand with effective grain size of 100 to 70 microns, and sands having permeabilities down to 3 x 10^{-1} ft/day (10^{-4} cm/sec). The hazard of using the silicate chemical grouts are minimal. Silicate grouts have the least health hazard of all the chemical grouts but rubber gloves and eye goggles should be worn when mixing the solutions.

Grouting with silicates produces a strong solidified mass of permanent duration provided suitable concentration of silicate is provided. The silicate grout with a controlled set time has the advantage of flexibility in stopping water flows and strengthening the soil. The cost of the materials used for the silicate system is the lowest of the various chemical grout systems. Silicate grouts can be combined with cement type grouts to control the set time.

If the silicate gel, especially those gels formed using the bicarbonate or hydrochloric acid reagents, are exposed to air for long periods of time syneresis takes place. Syneresis is the polycondensation of silicates into macromolecules with the resulting shrinkage that increases the permeability. Syneresis is stronger if there is no supporting matrix that limits contraction and thus is substantial in coarse soils, but practically non-existent in fine sands.

4.2 ACRYLAMIDE SYSTEM

4.2.1 LIMITS OF APPLICATION

See Fig. 2.1 for the limits of application using Acrylamide. Acrylamides can be used in soils having a permeability of 3×10^{-2} ft/day (10^{-5} cm/sec or greater). Acrylamides have a viscosity of 1.5 centipoise, (1.5×10^{-3} Pa-s), similar to that of water, so that they can easily penetrate fine sands.

4.2.2 DESCRIPTION OF CHEMICAL SYSTEM

In the acrylamide system a powder mixture of acrylamide and methylene-bisacrylamide organic monomers, sold under the trade name AM 9, is dissolved in water and polymerized with the aid of a catalyst activator and oxidizer initiator.

The catalyst activator normally used is dimethylaminopropionitrile (DMAPN), while the initiator is ammonium persulfate, a strong oxidizer. The ammonium persulfate reacts with the DMAPN to form free radicals in solution that polymerize the organic monomers. The ammonium persulfate is the last material to be added. The induction period (gel time) starts with the presence

of the ammonium persulfate in solution. Potassium ferricyanide is an inhibitor that may be added in concentrations of .001 to .04 percent to delay gelling. The concentration of the DMAPN in the final solution ranges between .8 to 1.6 percent (weight), while the concentration of the ammonium persulfate ranges between .5 to 1 percent (weight).

Buffers such as disodium phosphate heptahydrate are sometimes used in acid waters to bring the pH of the acrylamide solution to 8. The pH of the acrylamide solution should be between 7 and 11. If the pH is lowered below 7 the gel time will be considerably extended.

Soluble salts have an accelerating effect on the rate of gelation. Clays slow down the rate of gelation and require more of the catalyst to be used.

4.2.3 TECHNIQUES OF INJECTION

Acrylamide chemical grout injection is made using a one solution proportioning pump system. The acrylamide (A) and methylenebisacrylamide (M) powder is dissolved in water in a mixing tank and the catalyst activator DMAPN is added to the solution. The ammonium persulfate initiator is dissolved in water in another tank. Two separate progressing cavity type pumps (Moyno pumps) are connected to their respective variable speed motor drives and pump the separate solutions to a mixing chamber. Usually a large pump is used for the A&M solution and a small pump is used for the ammonium persulfate solution. Sometimes two equal size pumps are used when a large volume of grout is required. The two streams combine in a mixing chamber near the entrance to the injection pipe. If delay in the set time is desired, an inhibitor, potassium ferricyanide is added to the A&M solution.

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All tanks, valves, and pumps should be of stainless steel. Two rotameters that indicate flow rates are placed on the solution lines leading to the mixing chamber. The pumps have both bypass automatic relief valves and manual bypass valves, with the return going to the mixing tanks.

When using the large pump for the A&M solution with the small pump for the initiator solution the set time is controlled by regulating the amount of catalyst solution pumped to the mixing chamber. The variable speed drive of the pump allows the flow rate to be easily changed.

When large injection flows are required, two equal volume pumps are used with the set time determined by the careful addition of weighed amounts of ammonium persulfate and DMAPN to the separate tanks and inhibitors added as desired. Two additional small pumps may be used with two separate tanks holding a concentrated solution of ammonium persulfate and a solution of potassium ferricyanide for a fine control of the set time. Field trial is necessary when using either system.

In handling the A&M powder, respirators, rubber gloves and proper clothing should be worn so that the powder or solution does not enter the body or make contact with the skin. The acrylamide is toxic, can penetrate unbroken skin, and with prolonged contact will affect the central nervous system causing muscular dysfunction. Unreacted acrylamide should not be allowed to enter the ground water as it will form a health hazard to those people dependent on drinking well water.

4.2.4 INFLUENCE ON SOIL PROPERTIES

The increase in unconfined compressive strength of materials grouted

with acrylamide solution will also depend on the concentration of the acrylamide and the relative density of the soil before treatment. The Corps of Engineers report a slight gain in unconfined compressive strength for fairly loose sands treated with 10% acrylamide solution but the strength behavior of dense sand in consolidated-undrained triaxial tests was not affected (Corps of Engineers, 1973). The Corps of Engineers also report a decrease in permeability from about 10^{-3} cm/sec to 10^{-10} cm/sec for a sand treated with 10 percent acrylamide solution.

4.2.5 ADVANTAGES AND DISADVANTAGES

The principal advantage of the acrylamide grouting system is the ability of the grout to penetrate fine grained soils with effective grain size of 10 to 20 microns and transform the soil to an impermeable solid. Gel time can easily be changed using the proportioning pumps so that gelling can occur before flowing ground water removes the grout.

Except for epoxy and polyester resins, the acrylamide is the most expensive of the chemical grouts. The acrylamide is toxic and contact with the skin should be avoided. A respirator should be worn when handling the acrylamide powder. Rubber gloves should be worn and the skin covered when handling the acrylamide solution.

Japan has banned the use of all chemical grouts except silicon-based grouts on public works projects after several cases of water poisoning were reported (Engineering News Record, 1974). The water poisoning was reportedly linked to the use of acrylamide grout on a sewer project. Originally only acrylamides were banned but the ban was extended to other chemical

grouts (except silcon-based) as well. Even though gel times of acrylamides can be shortened so that theoretically the grout would gel before being removed by flowing groundwater, it should be emphasized that it may be difficult, if not impossible, to determine if any grout has been removed before gelling.

4.3 OTHER CHEMICAL GROUT SYSTEMS

4.3.1 LIGNOCHROMES

Lignosulfonate is a byproduct of the sulfite process used in the paper industry. Lignosulfonate is oxidized by sodium dichromate and forms a heavy metal precipitate. Lignochrome in dilution of 1:4-5 has been used to grout sands with effective grain size of 100 microns. Usually the sodium dichromate is added in the amount of 15 to 25 weight percent of the lignosulfonate. Ferric chloride is used to adjust the set time. The pH has to be adjusted with acids to be about 4- 3.5 for best results. The dichromate is toxic and limited in drinking water to 5×10^{-6} percent by the US Public Health Service. Respirators should be worn when handling the sodium dichromate powder as it is toxic if inhaled. Diphenylcarbazole is used as an indicator for chromium ions in water.

4.3.2 RESINS

Epoxy resin is a two component grout system formed when a base epoxy such as bisphenol A epichlorohydrin is catalyzed by an amine with polyamide modifiers. The proportions of base to catalyst are 5:1 by weight. The epoxy

resin grout when solidified generally has a compressive strength in excess of 10,000 psi (69 MPa) and a tensile strength in excess of 4,000 psi (30 MPa) (Corps of Engineers, 1973; Erickson, 1964 and 1968). It has a high viscosity, long set time which can be varied by selection of components and is very expensive.

The polyester resin grout is formed by a polyester resin base and a peroxide catalyst.   The proportions of the base to the catalyst are 15:1 by volume. Compressive and tensile strengths of the solidified grout are generally at least equal or greater than those of the epoxy resins. The cost of the grout is high. Rubber gloves and respirators should be worn when handling these two resins.

4.3.3 FOAMS

Polyurethanes have been injected and foamed in sands to increase compressive strength and decrease permeability. The polyurethane foam is pro- duced by mixing polyisocyates (toluene diisocyanate) with polyols (triethylene glycole, castor oil, or pentaneidiol). Blowing agents (diacetone alcohol or adipic acid) and surfactants are added to produce the foam. The closed cell foam will reduce the permeability. For the two shot process the polyisocyanate and the polyols are premixed together and later foamed by reacting it with water or carboxylic acid.

The isocyanate is toxic and inhalation of the vapor is injurious to the lungs. Gas masks should be worn. This chemical system is the most hazard- ous of all the chemical systems and should not be used in confined spaces.

CHAPTER 5

APPLICATIONS OF GROUTING TO UNDERGROUND OPENINGS

5.1 GENERAL CONSIDERATIONS

The basic reasons for grouting in construction of underground openings are to either: 1) reduce water flow into the excavation, thereby increasing ground stability, and reducing water pumping requirements; 2) improve strength-deformation characteristics of the ground thereby increasing ground stability and/or reducing the amount of surface settlement. Because of grain size and permeability limitations which dictate whether a soil can be successfully grouted, most soils which are grouted in underground opening construction are granular materials. The most common purpose of grouting these types of soils is to reduce permeability thereby controlling water inflow into the tunnel and associated loss of ground. Certain grouts also increase soil strength to provide more resistance to seepage pressures.

The range in grain size and permeability of materials which can be successfully grouted roughly corresponds to the limits for successful dewatering (including dewatering by vacuum but not electro-osmosis); these types of soils can also often be stabilized by compressed air. Where applicable, dewatering or use of compressed air are usually the most satisfactory and least expensive means of groundwater control. However, in some cases grouting is a more favorable stabilization technique than either dewatering or compressed air. Dewatering may itself cause settlement in some cases and usually is not permanent. Settlement due to dewatering may require expensive underpinning of adjacent structures and utilities and, if dewatering is not of a permanent

nature, the final tunnel lining may require waterproofing. In the case of
cohesionless granular soils, dewatering may control flows into the tunnel but
problems may still be encountered with loss of ground due to short stand-up
times. The use of compressed air may not be feasible for reasons such as
physiological effects on workers, the likelihood of blow-outs where depth of
cover is small, etc.

Problems in grouting for tunnel construction relate directly to the
particular site geology. If the ground conditions include stratified coarse
and fine materials, the grout may penetrate the coarser zones and successfully
reduce their permeability while the finer zones may remain untreated. Serious
runs into the tunnel may still occur in these finer zones. In extreme cases
this can lead to undermining and collapse of the grouted coarser zones.
This problem may be partially overcome by multiple stage grouting first using
grout materials capable of consolidating the coarse zones (such as cement or
clay cement), followed by chemical grouts which can penetrate the finer zones.
Use of an injection technique such as *tubes a manchette* provides some control
over which zones the grout will penetrate. However, it may be difficult to
even distinguish or accurately locate variations in permeability sufficiently
great to require such techniques.

Groundwater flows also create potential problems of successful
grouting. If flow velocity is high, the grout may be washed away before it can
set or gel. This may be partially overcome by controlling the set or gel time;
however, even if the set or gel time is short enough to prevent loss of
the grout, the hardened grout will be somewhat displaced in the direction of
groundwater flow. If this displacement is not accounted for by close grout

hole spacing "windows" may be left in the treated soil which will permit concentrated flows into the excavation. This concentration of flow may seriously magnify problems of erosion of soil materials. These problems with water flows often can be reduced if grouting is done sufficiently in advance of any excavation. High gradients induced close to an opened excavation may make successful grouting more difficult. In tunnel construction grouting may be performed from the ground surface, from pilot drifts, and in some cases from the heading. Grouting from the ground surface may not be feasible under some conditions such as excessive interference with existing utilities or structures. Grouting from the heading has the disadvantage of interfering with other tunneling operations, but this interference may be of less cost than separate pilot drifts. When grouting from within the tunnel, it is important to maintain a proper distance of grouted soil ahead of the face. This distance will depend on type of grout, nature of the ground, and hydraulic conditions, and can usually be determined only by field trial. Anderson and McCusker report a case where the estimated required distance of grouted soil beyond the face was 5 ft (1.5 m), but in actuality 15 ft (4.5 m) of grouted material ahead of the face was required to insure stability.

In several cases in Europe (e.g., Perrot, Janin and LeSciellour) grouting has been used not only to prevent water inflows, but also as a pre-support technique. A ring of soil surrounding the proposed tunnel was grouted either from the ground surface or from pilot drifts before any tunneling began. The thickness of the ring was varied at the crown, invert and sides of the tunnel depending on the strength of the treated soil and on the loads to be resisted. Excavation then took place in the core which can be left ungrouted,

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except for transverse diaphragms to reduce the amount of seepage into the heading. Two cases where this procedure was successfully applied are discussed in more detail in the following section of this report.

5.2 ILLUSTRATIVE CASE HISTORIES

5.2.1 NEW BLACKWALL TUNNEL (Perrott, 1965)

During the final period of uplift of the London Basin, the River Thames cut a deep trench from above London to the sea. At Blackwall this trench is about 600 ft (180 m) wide and 60 ft (20 m) deep. This trench cut through the London Clay into the underlying Woolrich Beds below, and was filled with sands and gravels known locally as the Thames Ballast. Finally a layer of mud was laid down in the present river channel above the Thames Ballast.

The New Blackwall Tunnel would encounter the Thames Ballast, which ranged in permeability from about 3×10^3 to 3×10^0 ft/day (10^0 to 10^{-3} cm/sec), over a distance of about 700 ft (210 m), of which about 500 ft (150 m) of the tunnel was entirely in Thames Ballast. The excavated diameter was 31 ft (9.5 m) and the depth of the crown below the Thames River bed was as small as 16 ft (4.5 m) and seldom more than 20 ft (6 m).

The River Thames is tidal at Blackwall. In order to balance the hydrostatic pressure at the invert under high tide, an air pressure of 44 psi (0.3 MPa) theoretically would have been required. Even if blow-outs could have been avoided, serious air losses through the gravels were anticipated. When the original Blackwall Tunnel was driven, this problem was overcome by dumping clay in the river to form a blanket about 15 ft (5 m) deep, but modern navigation requirements prevented such a solution for the new tunnel.

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Thus, the solution adopted was to form a grouted annulus 10 to 15 ft (3 to 5 m) thick around the periphery of the main tunnel to reduce the permeability of the Thames Ballast thereby reducing water inflow and loss of air. The grouting was also designed to strengthen the arch above the tunnel.

Grouting was carried out from two 7 ft (2 m) diameter pilot tunnels, one located near the crown and the other near the invert of the main tunnel. The specification required the Thames Ballast permeability to be reduced to not more than 8.5×10^{-1} ft/day (3×10^{-4} cm/sec). This was verified during grouting by performing in-place permeability tests in grouted material; the same type of permeability test was used as an exploratory tool prior to grouting to determine appropriate types of grout.

A clay-chemical grout was used for the main annulus treatment and in addition a pure chemical grout was injected to form a 120 degree arch above the crown, within the clay-chemical grout annulus. The clay-chemical grout consisted of a 7 percent concentration of commercial bentonite, sodium silicate and an alkaline dispersant. Since nearly all commercial bentonites contain 6 to 8 percent (by weight) of particles at least as large as cements, a hydro-cyclone process was used to remove these particles thereby improving penetrability; in the more open lenses, a 10 percent clay concentration was used and hydrocycloning was not necessary. The chemical grout used in the arch was a chrome-lignin type grout and was selected because of the improvement in strength which would result.

Grouting was carried out by injection in radial directions from the upper and lower pilot tunnels at hole spacing of 4 to 6 ft (1.2 to 1.8 m). Two in. (5.1 cm) O.D. injection rods were driven to full depth of the hole

before injection. Primary mixing and preparation of grout was done at the surface and the grout was delivered into the tunnel through pipes in two solutions where they were combined in a mixing chamber on the end of the injection rod. Injection rods were driven full depth and then withdrawn 5 in. (12.7 cm) at a time as grout was continually injected until the desired quantity, based on estimated porosity of the soil, had been pumped or until no further grout could be injected at the design pressure of 50 psi (0.35 MPa) (fissuring pressure in pilot tunnel determined from on-site tests).

After completion of grouting, in-place permeability tests were performed in the grouted annulus. Of 416 post-grouting tests, only 25 indicated permeability greater than the specification limit of 8.5×10^{-1} ft/day (3×10^{-4} cm/sec) and only one of these was greater than 1.7×10^{0} ft/day (6×10^{-4} cm/sec).

In this case a large tunnel was successfully driven in waterbearing, cohesionless soils with only 15 to 20 ft (5 to 6 m) of cover between tunnel crown and Thames River bed. Through the use of in-place permeability testing, it was possible to both determine grout type, hole spacing, etc., in advance of injection and also to check that specified permeabilities had been achieved after grouting. In this case history, grouting was a planned part of tunnel excavation and support, performed in advance of mining, and eliminated the necessity of high air losses and the risk of blow-outs through the permeable Thames Ballast.

5.2.2 AUBER STREET AREA - PARIS RAPID TRANSIT (Janin and LeSciellour, 1970)

The Auber Street Station is 130 ft (40 m) wide by 750 ft (230 m)

long by 60 ft (18 m) high. The deepest point is 130 ft (40 m) below street level and 60 ft (18 m) below groundwater level. The central arch of the station spans 80 ft (24 m). An existing subway line parallels the station directly above the arch and both sides of the station extend 28 ft (8.5 m) laterally beyond adjacent building lines. In addition to the main station, the construction included transition structures 50 ft (15 m) high by 30 ft (9 m) wide at each end as well as a 2300 ft (700 m) long siding.

Because the construction would be below existing building, tunnels, and utilities which could be damaged by movements associated with the construction, a control system was installed which consisted of benchmarks, vibrating wire strain gages, uplife gages, and inclinometers.

The subsurface conditions in the station area consist of 10 to 12 ft (3 to 4 m) of fill; 30 to 40 ft (9 to 12 m) of alluvial sands and gravels; 35 to 60 ft (11 to 18 m) of calcareous marl; and fissured coarse limestone. In addition, a lens of waterbearing silty fine sand intersects one side of the station. Except for the sand lens, the station is entirely within the calcareous marl. Although the marl is not described in the paper, this material could be assumed to behave as a cohesive granular soil. The water table at the station site is about at the station crown.

Because of the proximity of existing structures, displacements had to be minimized. Since most of the excavation had to be carried out below the water table, there was concern about settlements resulting from dewatering. As a result, grouting was chosen to control loss of ground and to eliminate the need for dewatering.

The grouting design consisted of a grouted arch which extended 26 ft (8 m) both above the station crown and beyond the side walls and a 10 ft (3 m) impervious lining below the invert and base of the sidewalls. The same basic design was planned for transition structures and siding except the thickness of the grouted arch was recduced to 10 to 13 ft (3 to 4 m).

Grouting was carried out from a pilot tunnel near the crown, above the water table, from which the arch and a portion of the sidewalls were grouted. An additional pilot tunnel was excavated along each sidewall (below water table) after completion of grouting from the crown pilot tunnel. From these lower pilot tunnels the remainder of the sidewalls and invert were grouted.

Grout was injected by the *tubes a manchette* method. Grout pipes to be left in place were of polyvinyl chloride so as not to impede later excavation. Grout holes were spaced not more than 10 ft 6 in. (3.2 m) apart in the marl, 6 ft 6 in. (2 m) in the alluvial sands and gravels and 4 ft (1.2 m) in the running sand lense. Clay/cement grout was used in the alluvial sands and gravels (no details given as to mix); silica gel (one-shot) was used for the marl; and phenoplastic resin was used in the running sand lense. The crushing strength of the treated soils was kept at 285 psi (2 MPa). The grout station was outside the pilot tunnels and the grouts were delivered to the injection joints through piping.

Instrumentation consisted of benchmarks, read daily, in the existing subway tunnels, on the street and in adjacent building; vibrating wire strain gages (Telemac acoustic gage) installed in basements of adjacent buildings; uplift gages consisting of a deeply buried rod which extended to the surface

and whose movement could be measured relative to a surface point; and inclinometers installed along station sidewalls. During grouting operations, general uplift did not exceed 0.4 in. (10 mm) with a few isolated points reaching 0.6 in. (15 mm). During excavation of the station, settlements of this same order of magnitude resulted.

As in the New Blackwall Tunnel, grouting was a part of the overall construction procedure on the Auber Street Station and not merely a remedial measure. Adjacent facilities including large buildings, subway tunnels, and utilities were successfully protected by the grouting, without use of under-pinning. Careful control was maintained by remote monitoring of several of the instrumentaion devices which provided quick warning to the grouting station if movements exceeded the allowable.

5.2.3 SEWER TUNNEL - MANHATTAN (Anderson and McCusker, 1972)

This interceptor sewer tunnel ranges from 10 ft 6 in. to 14 ft 6 in. (3.2 to 4.4 m) horseshoe almost entirely in Manhattan Schist except where it crosses major faults at shallow depths in mixed face and earth conditions. Soil conditions in these areas consisted of interlayered silty fine sand, silt (known locally as "bull's liver"), and in some areas silty fine to coarse sand with cobbles. Permeabilities ranged from 3×10^0 to 3×10^{-3} ft/day (10^{-3} to 10^{-6} cm/sec) (predominantly at lower end) and the tunnel was below the water table.

Because a relatively small portion of the tunnel was expected to be in mixed face conditions, successive set-ups of compressed air equipment wherever these conditions were encountered would have been extremely costly.

Furthermore, extra compressed air crews (which work only 4 hour shifts) would have been difficult to recruit. Freezing and dewatering were also considered but because of overlying utilities, trees and other encumberances, it would have been difficult to establish an effective pattern of freeze pipes or ejector wells. Specifications prohibited dewatering in some areas because of potential for detrimental settlements. It was also doubted that deep wells would be effective in the stratified soils. Thus, chemical grouting with compressed air as a back-up was selected as the stabilization technique.

The bull's liver is a particularly difficult material to tunnel. It was anticipated that chemical grout would penetrate the more permeable layers and that by reducing the permeability and strengthening these materials, piping in the bull's liver would be prevented and thus breasting could control these materials.

When the tunnel approached the mixed face section, probe holes were drilled to locate soil-rock interface and a small drift was advanced in the invert, maintaining 5 ft (1.5 m) of rock cover above the drift. This drift was extended until bad rock was encountered at the face. The purpose of the drift was to provide a guide for the shield to be used in the mixed face, and also to provide a means of grouting the ground to be excavated. Because of surface restrictions, it was originally planned to grout from within the tunnel to a distance of 5 ft (1.5 m) beyond excavation lines and ahead of the face. However, it soon became apparent that the necessary thickness of grouted material ahead of the face was a minimum of 15 ft (4.6 m). This thickness was impractical to achieve solely by grouting from within the tunnel, so surface grouting was employed with better results.

Most of the grouting done from within the tunnel was with AM-9 chemical grout which reduced permeability. From the surface Terranier A was used to increase soil strength. Grout holes at the surface were located 8 ft (2.4 m) each side of the tunnel centerline and spaced 10 ft (3 m) on the centers. In mixed face areas grouting began at the top of rock and in soil areas 10 ft (3 m) below invert; in both cases grouting continued to 8 to 10 ft (2.4 to 3 m) above the crown.

Grouting did not prevent surface settlements in the major area described above. A subsidence trough followed the tunneling operation with a maximum settlement of about 1 ft (30.5 cm) occurring over the centerline 40 to 50 ft (12.2 to 15.2 m) behind the trailing edge of the shield. Ahead of the shield, settlements were typically on the order of 0.1 ft (3 cm) 20 ft (6 m) ahead and 0.06 ft (1.8 cm) 40 ft (12 m) ahead. In other mixed face areas than the one described above, when chemical grout pressures failed to reach 50 psi (0.34 MPa) cement grout was then injected to make sure voids would be filled. In these areas, surface settlement was reportedly negligible.

In this case history, the bull's liver was successfully stabilized by reducing the permeability of the interlayered coarser materials thereby preventing water flow through these materials and subsequent inflow of the bull's liver. Surface settlements, however, still resulted. Where overlying facilities might have been damaged, these techniques would not have been successful.
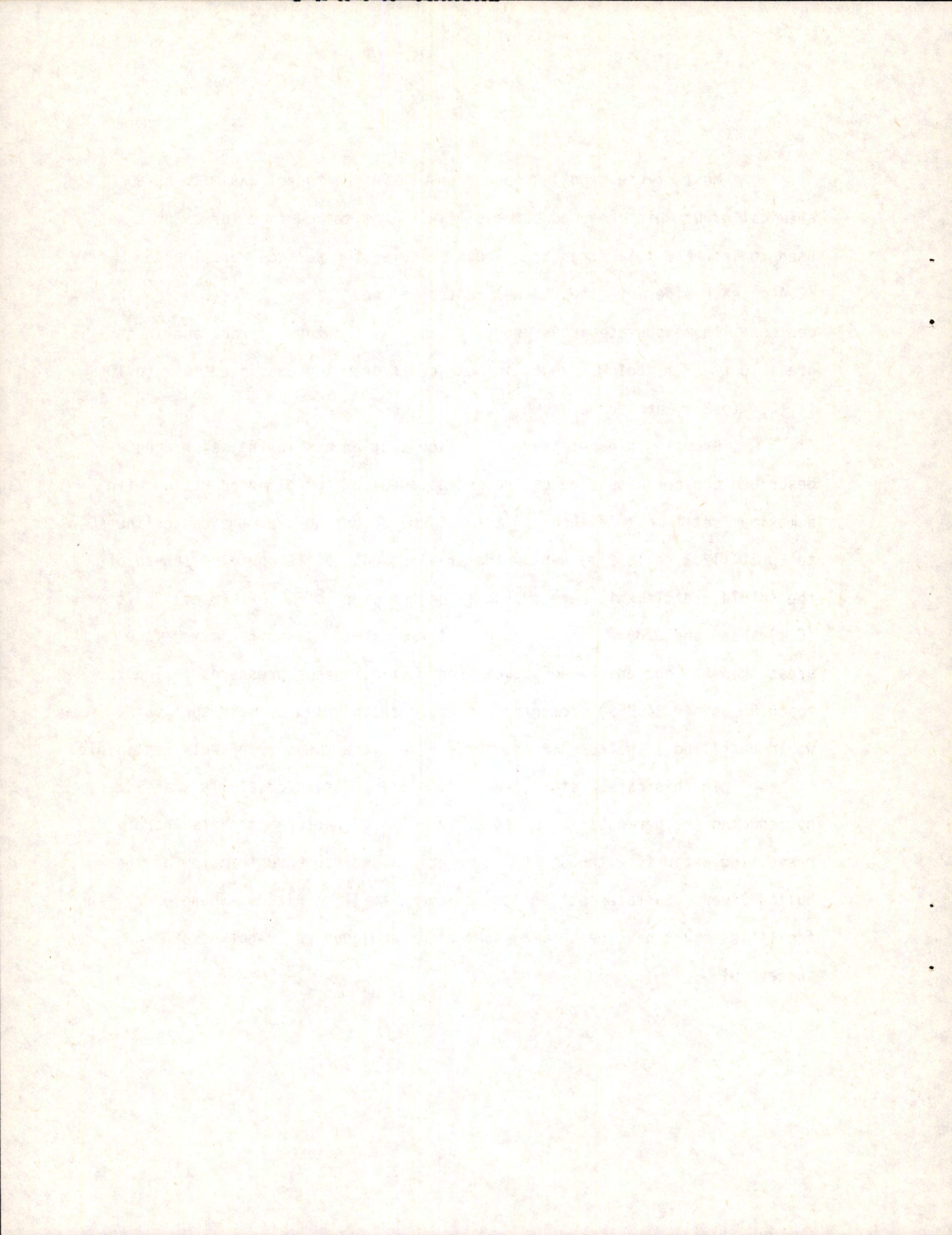
CHAPTER 6

FREEZING

6.1 HISTORICAL DEVELOPMENT

The use of artificial ground freezing for the support of excavations and cutoff of water flow dates to the mid 19th century. In 1883, F. H. Poetsch developed a freezing process to use for sinking a mine shaft in Germany, in which a conventional refrigeration plant was used to cool brine circulated through freeze pipes embedded in the ground to be frozen; the use of brine in the refrigeration process is still the most common method today.

The major use of artificial ground freezing has been in shaft construction. Typically, freeze pipes through which brine is to be circulated are placed around the periphery of the proposed excavation, and freezing proceeds outward from each pipe around the proposed shaft until the frozen zones from adjacent pipes overlap. At this point, a continuous cylinder of frozen soil has been formed which protects the area to be excavated from water inflows and instability of the walls. Similar procedures have been used in civil engineering particularly for construction of pump stations, shafts for tunnels, underpinning of buildings and in a few cases for tunnel construction. In the civil engineering field, particularly in the United States, freezing has usually been regarded as a last-resort, remedial measure, employed where grouting, dewatering, or compressed air either were not feasible or failed to perform adequately. This is probably due to the relatively high cost and the specialized nature of the technique.

6.2 LIMITS OF APPLICATION

Freezing theoretically can be used in any type of soil which has pore water present. One contractor who specializes in freezing techniques has stated that any soil with a degree of saturation of at least 8 percent can be frozen (Mile High Deilman Brochure). Actually, freezing has nearly always been used below the watertable; so, in practice, artificial freezing usually is in fully saturated soils. The most common type of soils which have been successfully stabilized by freezing are those such as water-bearing silts and fine sands which cannot be efficiently dewatered or stabilized by grouting and for which compressed air may not be feasible because of the possibility of blow-outs, labor considerations, etc.

Freezing may be ineffective or impractical under certain ground water conditions. If the groundwater is saline, the freezing point may be low and a large refrigeration capacity or long periods of freeze time may be required, adding greatly to the cost of the process. If groundwater is flowing, heat may be added to the zone to be frozen at a greater rate than can be removed by the freeze elements, in which case a continuous frozen zone may not be formed. Shuster (1972) has reported that groundwater flows of the order of 3 to 6 ft/day (1 to 2 m/day) can be successfully frozen using a conventional circulating coolant refrigeration system, and that flows as high as 165 ft/day (50 m/day) have been stopped with liquid nitrogen freezing systems.

6.3 TECHNIQUES OF FREEZING

The basic technique of artificial freezing is to install freeze

elements in and around an area to be stabilized and to remove heat from the
ground at the freeze elements, producing a frozen zone around these elements.
This frozen zone eventually expands and merges with adjacent frozen zones
until a continuous frozen mass is formed. The individual freeze elements
typically consist of concentric pipes through which the coolant is circu-
lated. The inner pipe is typically 1 to 2 in. (25.4 to 50.8 mm) in diameter,
extends to within 1 to 2 ft (0.3 to 0.6 m) of the bottom of the outer pipe,
and carries coolant from the refrigeration plant. The outer pipe is typically
2 to 6 in. (50.8 to 152.4 mm) diameter with the lower end closed so that
coolant travels down the inner pipe and is returned through the annulus to
the refrigeration plant. Freeze elements are usually steel pipe and are
placed outside the future excavation lines. Typical spacing of freeze elements
is 2 to 4 ft (0.6 to 1.2 m) and the elements usually extend a short distance
into an underlying competent soil layer. Thermocouples are often installed
in the freeze pipes to monitor coolant temperature and provide information on
freezing progress. In the case of shaft construction it has been common prac-
tice to install an observation well in the center of the freeze element pat-
tern and to observe the water level in this well. A rise in water level in-
dicates closure of the freeze wall.

The Poetsch process is still the most common freezing process in
use today. In this process, coolant (typically calcium chloride brine,
glycol-water, methanol, etc.) is chilled by a conventional ammonia or freon
refrigeration plant and circulated through the freeze elements, which are
connected to a coolant supply manifold. The coolant is returned through the
outer freeze pipes to a return manifold and then back to the chiller. This
system is depicted schematically in Fig. 6.1. Although this process is well

Prime
Mover

Compressor

Condenser

Evaporator

Chiller

Coolant supply manifold

Inner
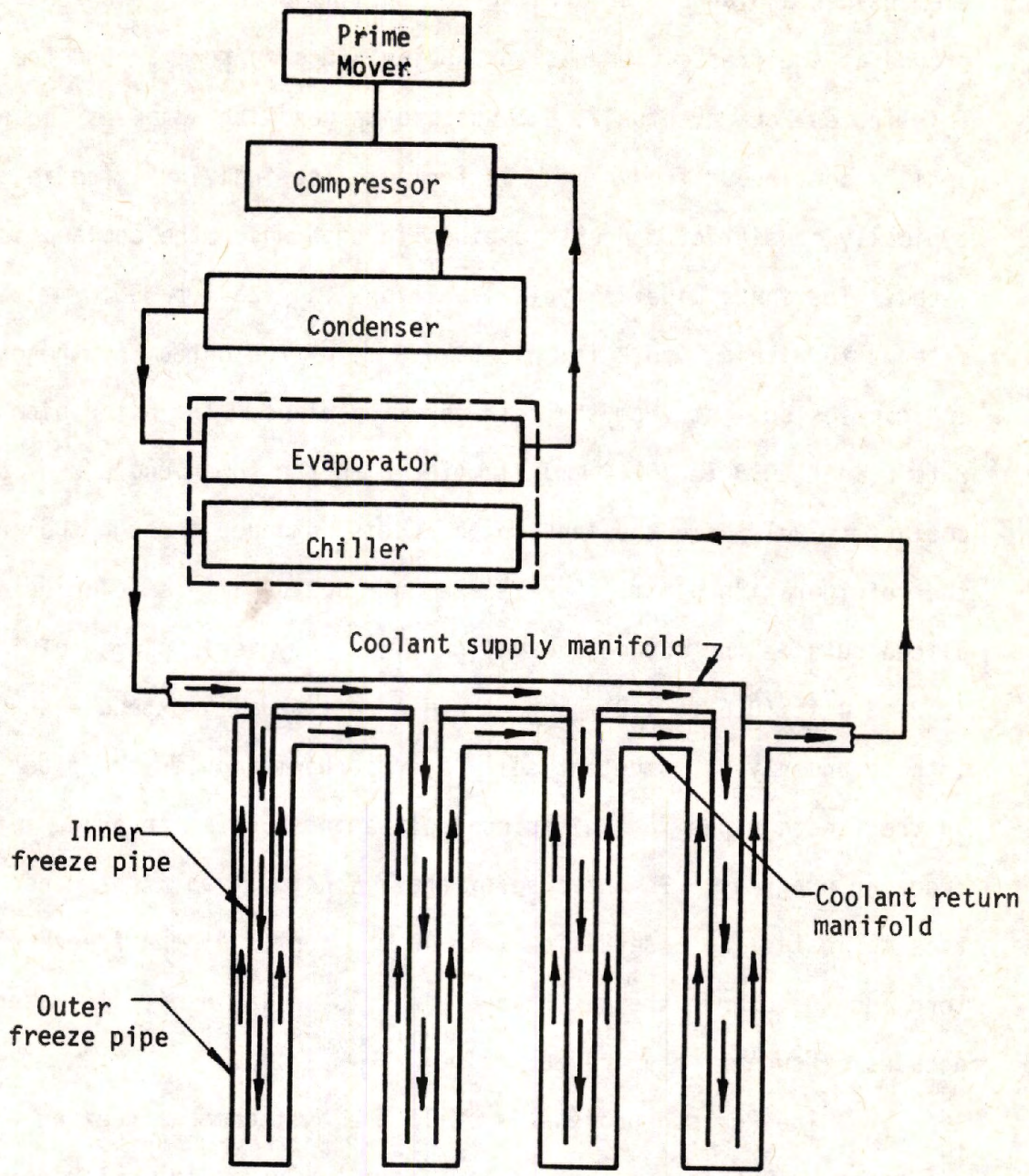freeze pipe

Coolant return
manifold

Outer
freeze pipe

FIGURE 6.1   SCHEMATIC DIAGRAM OF POETSCH PROCESS
(AFTER SCHUSTER, 1972)

understood and has been used very often, it lacks flexibility to adjust the capacity of individual freeze elements without changing the entire system. This flexibility may be desirable in the case of concentrated water flows.

Schuster describes four other possible freezing processes. The primary plant with in situ evaporator differs from the Poetsch process in that the secondary coolant is eliminated and a smaller volume of refrigerant is circulated directly from the condenser (see Fig. 6.1) to the freeze elements where evaporation occurs. This process has been used in the past but not in recent years. The reliquefaction plant with in situ second stage also uses a primary refrigeration plant with a secondary distribution system, but heat transfer in the freeze elements is by phase change rather than convection as in the Poetsch process. This is a much more efficient method of heat transfer and as a result much smaller volumes of coolant need be circulated. Furthermore, since each individual freeze element is an evaporator, any individual element may be operated at a lower temperature than the others which might be advantageous in the case of concentrated ground water flows. This process has not been used in artificial ground freezing, probably because of the high cost of refrigeration plants and their lack of availability; however, it is used in the liquefied gas industry.

Two other processes Schuster describes both involve expendable refrigerants. This type of process may use liquefied nitrogen or carbon dioxide either liquefied or in the form of solid dry ice pellets. Such a system may be attractive for very short term or emergency situations, but since the refrigerant cannot be recovered, costs are high and control is difficult.

In the more commonly used Poetsch process, the primary refrigeration plant may be operated to maintain a constant coolant temperature throughout the freeze element system. In this case the refrigeration load will vary with time. This procedure can be controlled moderately well by monitoring coolant temperature throughout the distribution system by thermocouples inserted in the freeze elements. Thermostats can be used to vary the quantity of flow into the distribution system. Alternatively, the primary plant may be run at full capacity in which case the coolant temperature varies with time. The latter procedure requires a longer time to achieve freezing, but may be necessary if a refrigeration plant of adequate size to maintain the desired coolant temperature is not available (Schuster).

For a given freezing process, material properties, and geometry, the size and spacing of the freeze elements are the most important variables affecting size of refrigeration plant and time to achieve freezing (Schuster; and Sanger, 1968). The refrigeration load (and hence cost), cost of drilling, and cost of materials will increase as the size and spacing of the freeze elements decreases; however, as the spacing decreases, the time required to freeze can be reduced. Lowest cost can be achieved for a given project by varying freeze element size and spacing, refrigeration plant capacity, and coolant temperature to select the most economical combination.

6.4 DESIGN CONSIDERATIONS

6.4.1 GENERAL DESIGN CONSIDERATIONS

Design of an artificial ground freezing project requires consideration of the strength-deformation characteristics of the frozen soil to

determine freeze wall thickness, and consideration of the thermal properties
of the ground to design the refrigeration system. Since strength-deformation
properties are temperature and time dependent, the two considerations are not
independent. Design is usually by trial calculations to converge on an ef-
ficient freezing scheme. In general, the lower the temperature of the fro-
zen ground, the more favorable the strength-deformation properties, requiring
a smaller frozen zone to resist the earth or water pressure around an excava-
tion. On the other hand, the lower the design temperature, the longer is the
time required to reach the design temperature and the larger the refrigera-
tion plant required.

6.4.2 STRENGTH-DEFORMATION CONSIDERATIONS

Vialov (1965) has performed extensive research in Russia on the
properties of frozen soils. His work is the basis for present design in
this country as well as in Europe and Russia. Frozen soil does not behave
elastically but creeps under constant stress, the strength-deformation be-
havior being strongly time and temperature dependent. If a specimen of
frozen soil is loaded with a constant stress greater than the yield value
under uniaxial or triaxial conditions, a plot of axial strain versus dura-
tion of loading will have the general form shown in Fig. 6.2, exhibiting
three stages of creep. The time to reach the beginning of the tertiary
stage is usually taken as ultimate failure since the point represents in-
cipient failure and it is not possible to predict the time at which actual
failure occurs beyond that. For a given applied stress, the lower the tem-
perature of the frozen soil, the greater the time to failure (beginning of

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T_f = Time to reach
tertiary creep

Axial strain (%)

Case 2

Higher temperature and/or higher
stress level than Case 1

Primary
creep

Secondary creep
(Strain rate constant and minimum)

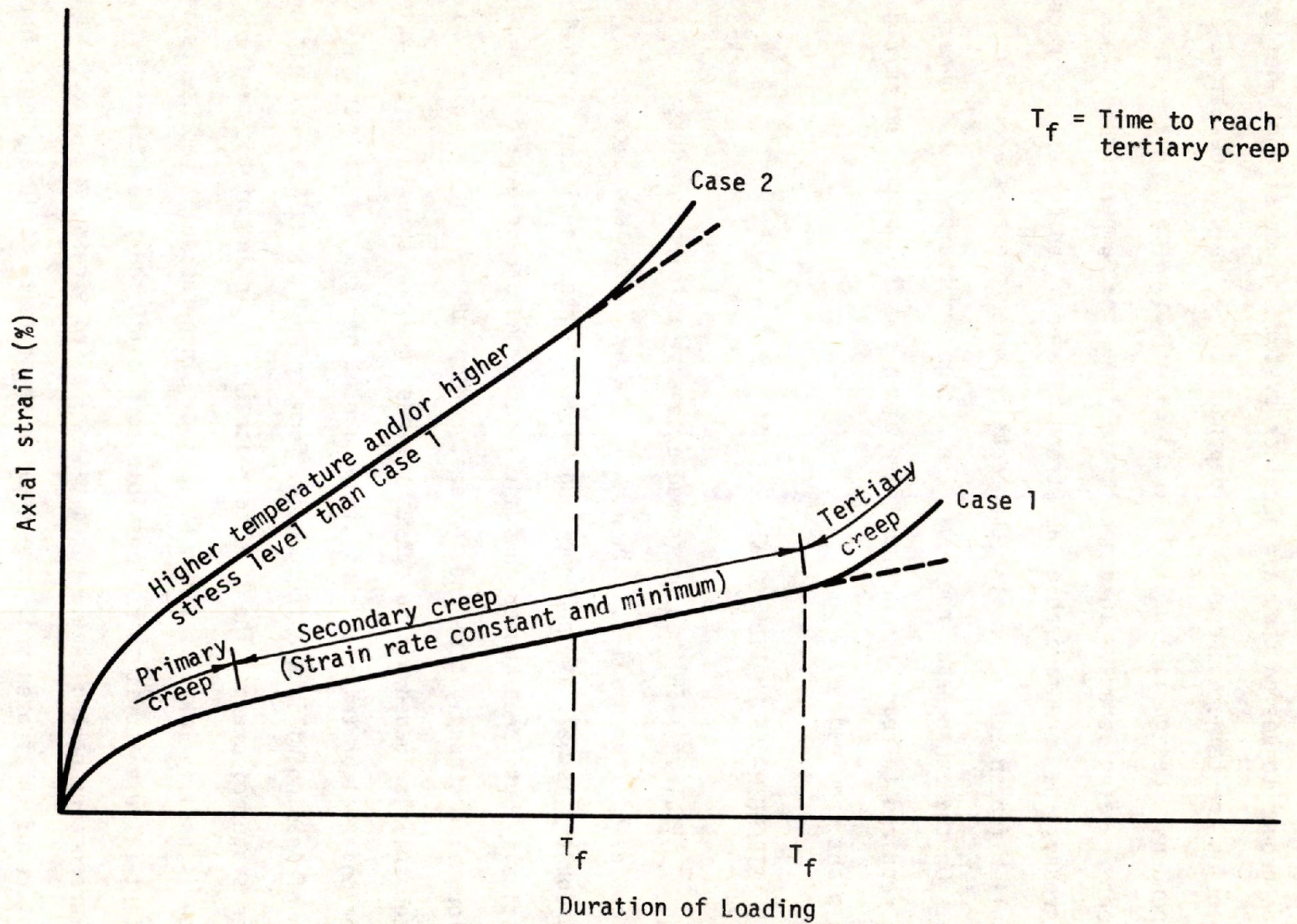Tertiary
creep

Case 1

T_f

T_f

Duration of Loading

FIGURE 6.2   TYPICAL CREEP CURVES FOR FROZEN SOILS
(SHOWING EFFECTS OF STRESS LEVELS AND
TEMPERATURE)

tertiary creep) and for a given temperature, the lower the applied stress the greater the time to failure.

Sanger states it is probable that the design of most frozen soil barriers have been according to elastic theory where frozen soil properties are determined from conventional laboratory compression tests using a factor of safety on compressive strength. This can be misleading since the actual strength and factor of safety depends upon temperature and duration of loading. It is likely that the successful use of such a procedure is due primarily to the experience of the designer with similar soils and excavations.

Vialov presents equations for the design of frozen soil cylinders which are applicable to shaft design. His procedures assume a uniform average temperature across the frozen soil wall. Strength-deformation parameters for use in the equations are determined from laboratory tests at this temperature and for a duration of loading corresponding to the time lapse between excavation and installation of supports.

The earth pressure which the frozen soil wall must be designed to resist will be time dependent since the pressure will be dependent on deformation of the wall. A few hours after excavation, the frozen soil may not have undergone enough deformation to significantly reduce the earth pressure. Schuster suggests that design earth pressures for this case may be considered to be between at-rest and passive; passive pressures can develop due to expansion of the frozen soil. If the delay between excavation and installation of support is more than about one shift, Schuster suggests that

design earth pressures may be between at-rest and active since by this time, secondary creep will have produced enough deformation to cause a reduction in earth pressures. Vialov has derived equations for the time-dependent earth pressure, considering deformation of the frozen soil cylinder, and assuming that the soil in the frozen zone exhibits creep properties, while that outside the frozen zone may have either constant or time-dependent stress-strain properties. His derivation assumes a Rankine-Mohr zone of plastic equilibrium around the frozen cylinder.

In some cases where an excavation is to be open for a relatively long period of time before installation of permanent support, deformations of the frozen-soil wall must be considered. Deformations will develop due to creep in the frozen soil wall and can lead to movements of the adjacent ground. Vialov has also derived equations for the inward displacement of frozen soil cylinders; Sanger has found that in practice the equations give a high value for displacements but are on the safe side if frozen soil walls are designed, as is usual practice, on the basis of strength.

6.4.3 THERMAL CONSIDERATIONS

In the design of the refrigeration plant and freeze element layout, the major items of concern are the time required to produce a frozen soil barrier of the desired geometry, the temperature at which the frozen soil is to be maintained, and the amount of energy (load on refrigeration plant) necessary to produce the desired frozen temperature and geometry in the given time. The problem is usually analyzed in two stages: Stage I is the formation of a frozen zone around the freeze element and the Stage II is the

merging of the frozen zones around individual elements and gradual thickening of the frozen soil barrier.

The important thermal properties of the soil according to Sanger's method of analysis are the volumetric latent heat (heat to be extracted to freeze the water in a unit volume of soil), the volumetric specific heat (heat required to raise the temperature of a unit volume of soil by one degree Fahrenheit) and the thermal conductivity (amount of heat transmitted in unit time across a unit area of unit thickness for a temperature change of one degree Fahrenheit). These properties are different for frozen and unfrozen soil. Sanger gives relations for the volumetric latent heat and volumetric specific heat which are both related to the water content and dry density of the soil. The thermal conductivity of various soils was investigated by Kersten who found that dry density, moisture content, mineralogy, temperature and gradation were important factors. Kersten presents charts which can be used to estimate thermal conductivity for various soils.

The common assumptions used in the analysis are that heat flow is horizontal and is a steady state provided constant temperatures are maintained at the boundaries (i.e., at the freeze pipe and at some radius from the freeze pipe at which the ground temperature is not affected by the freezing process, estimated by Sanger at three to five times the radius of the frozen zone based on field observations) (Mariupolskii). Sanger provides equations for the time required to complete State I and Stage II freezing for both straight and circular walls. For both stages, the time is a function of the thermal properties of the ground, the

difference in ambient temperature and the temperature at which the ground

freezes, the difference in temperature at which the ground freezes and the

temperature maintained in the freeze elements, and the geometry of the

problem including freeze element size and spacing. For given thermal pro-

perties of the ground, the time required to complete both stage I and II

freezing increases as the size of the frozen zone and the difference in

ambient and freezing temperatures increase, and decreases as the difference

in freezing temperature and freeze element temperature increases.

The quantity of heat to be removed to produce the desired freeze

element temperature and geometry of the frozen zone can also be estimated

by equations given by Sanger.   From this, the refrigeration load

can be estimated at any time during the freezing process. The refrigeration

load necessary to maintain the desired temperatures for a given geometry

will increase as the difference in freezing temperature of the ground and

freeze element temperature increases. The refrigeration load will also vary

with time since the volume of frozen soil to be maintained at the given

temperature varies with time. By preparing a plot of refrigeration load

versus time, the theoretical mean refrigeration load can be estimated and

the refrigeration plant sized accordingly.

The theoretical considerations described above do not consider

ground water flow. If ground water is flowing, energy will be added to the

zone to be frozen and required refrigeration load to producing a frozen

barrier will be greater than that required to freeze non-flowing water in

the soil voids. This problem cannot be treated analytically; Schuster

recommends close spacing of freeze elements, carefully controlled alignment

of freeze elements, and the coldest available refrigerant be used where
ground water flows are expected.

6.4.4 MISCELLANEOUS DESIGN CONSIDERATIONS

An important consideration in the design of a freezing project
is ground movement. Movements adjacent to an excavation supported by a
frozen soil wall, due to deformation, can be estimated by Vialov's pro-
cedure. In addition, movements resulting from volume expansion as the
soil freezes must also be considered.

In the case of a clean sand or gravel, volume expansion theoreti-
cally may be the result of a 9 percent increase in void volume due to
changing of the pore water to ice (Terzaghi and Peck). In actuality, in
these types of soils, the permeability is usually large enough that as the
porewater freezes and expands, unfrozen water is forced out of the voids,
resulting in expansion of less than the theoretical 9 percent of void vol-
ume. The amount of expansion will thus depend on the rate of freezing with
respect to the soil permeability. In general, volume expansion in such
granular soils will be negligible (Sanger). Even if freezing proceeds
faster than water is forced out of the soil voids, for a void ratio of
0.4 (which is typical for alluvial deposits), the volume expansion due to
changing of the porewater to ice would be only 2 to 3 percent.

Fine grained soils will undergo much larger expansion due to
the formation of ice lenses. Ice lenses form as water migrates toward
the freezing zone under capillary action and the excess water freezes into

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layers of segregated ice. Since this phenomenon is related to capillarity, the smaller the grain size, the greater the tendency for formation of ice lenses; but, on the other hand, the smaller the grain size, the lower the rate of ice lens formation (Terzaghi and Peck). Fine silts and silt-clay mixtures represent the worst case of ice lens formation since these soils are fine enough to have large capillary rise, and are of high enough permeability that ice lens formation is relatively rapid (Schuster). For rapid freezing, ice lens formation will not occur during active freezing. Thus, the total expansion due to ice lense formation will be reduced by quick freezing (Schuster, Sanger).

Schuster describes a procedure whereby potential frost expansion can be estimated. The amount of frost expansion will depend on the confining pressure; if the potential pressure due to frost expansion is less than the effective confining pressure, no frost expansion will occur. The procedure involves determination of the potential frost expansion pressure by laboratory tests on frozen samples and then comparing this pressure with the calculated in situ intergranular pressures.

Not only must frost expansion be considered in terms of ground movements, but soils which form ice lenses may have their strength-compressibility properties drastically altered upon thawing. Thawing of soil with ice lenses present transforms them into a supersaturated material of low shear strength and high compressibility (Terzaghi and Peck). Quick freezing thus will not only reduce ground movements due to frost expansion, but will also result in more desirable thawed properties.

Instrumentation is important to determine when freezing is complete and also to maintain desired frozen temperatures. In the case of shaft

sinking in free-draining soils, an observation well in the center of the freeze element pattern has been successfully used to determine when closure of the frozen soil wall has been obtained (Latz). When the freeze wall has closed, water level in the central observation well will begin to rise. In the case of fine grained soils such a simple indicator of wall closure is not functional and temperature measuring devices in test holes may be required (Sanger, Schuster). Costly delays may arise if excavation begins before the frozen soil wall is closed, particularly if incomplete closure is the result of windows arising from concentrated ground water flows. However, if the existence of the windows is recognized through instrumentation, additional freeze elements may be installed in the area or coolant temperature may be reduced prior to beginning excavation. The measurement of heave and lateral movements in soils susceptible to frost expansion is also important, particularly where adjacent existing structures would be affected by ground movements.

Heat losses through surface piping, the existence of utilities in the area to be frozen, and heat losses through the exposed faces of the excavation can all reduce effectiveness of the freezing process by producing larger than expected loads on the refrigeration plant. Good practice is to insulate all above-ground piping, typically with foamed plastic and, where large areas of the frozen soil are exposed to the sun, to cover them with reflective plastic or where necessary to spray them with foamed plastic insulation (Sanger, Schuster). Schuster describes the difficulties in freezing areas containing steam, water or sewer lines unless these facilities are properly insulated. Steam lines will be most susceptible to ill effects due to low ground temperatures which can cause undesirable condensation of the

steam. Insulation must also be effective in preventing thawing of the frozen ground around the utilities; Schuster describes an effective and economical approach to insulating these facilities by excavation around the lines and spraying in situ with foamed plastic.

Excavation of frozen soil has usually been by either pneumatic tools, clam shells, or drilling and blasting. Frozen soil has variously been described as behaving during excavation like a "weak rock or poor concrete" (Low) or as a "tough sandstone but not brittle" (Ellis and McConnell). Whether or not drilling and blasting will be required will be generally related to the frozen temperature as well as the type of soil which is frozen. An indication of what type of excavating procedures will be required, as a function of temperature and soil type, may be obtained from conventional laboratory unconfined compression tests on frozen samples. The type of excavation procedures will obviously affect overall project economy and this should be considered in selection of the frozen temperature.

6.5 ADVANTAGES-DISADVANTAGES

The major advantage of artificial ground freezing as an aid to excavation is that it may achieve success when other, more conventional, stabilization techniques cannot. In general, artificial freezing is not dependent on soil permeability, though ground movements resulting from frost expansion and the effects of ground water flows on the success of freezing are certainly related to permeability. Stratification and differences in vertical and horizontal permeability which can seriously affect the success of grouting or dewatering will have much less affect on the success of freezing since

the thermal properties of soil will vary over a much smaller range than will permeability (Sanger).

Freezing also can be successful in stabilizing soils of low permeability such as fine sands and silts which cannot be grouted or dewatered (except possibly by electro-osmosis). Such soils would typically be excavated under compressed air. Freezing eliminates compressed air problems such as the hazards of blow-outs and the inefficiencies of workers laboring under high air pressures. Gail describes a case where freezing was used and compressed air was ruled out for the construction of a tunnel because the noise of the compressed air plant would disturb nearby residents.

Artificial ground freezing has been utilized in most civil engineering projects mainly as a last-resort technique and as a result, experience with freezing is limited to a few specialized contractors and consultants. Furthermore, many of the details of the freezing process such as types of instrumentation, design of the freezing plant and freeze elements, etc., are of a proprietary nature and as a result, published case histories often omit such details. Freezing contractors who have successfully completed many projects are reluctant to share their experience with competitors. Published articles often omit details on costs, soil properties, the design procedures used on a given project, and many other details which are necessary both to plan and execute a successful project, and often to merely evaluate freezing as an alternative stabilization technique.

Based on the design considerations briefly outlined above, it is possible to evaluate the strength-deformation characteristics of frozen soil as a function of temperature, the time required to achieve freezing for the

desired temperature and freeze element pattern, the required freezing plant capacity, and the nature of ground movements to be expected. The analyses do require simplifications concerning strength-deformation and thermal considerations. As a result the design process requires substantial judgment and experience. Ground water flows and organic materials, in particular, have undesirable effects on time required to achieve freezing and can cause the time to increase substantially. It is difficult to control freezing temperatures closely because of unknown heat losses and simplifications in estimating the soil thermal properties. Since frozen soil strength is influenced significantly by temperature, a few degrees difference can drastically effect the behavior of the frozen soil wall (Vialov). On the other hand, excessively low temperatures increase the required size of the refrigeration plant and decrease required freeze element spacing, both of which increase costs substantially.

The direct cost of artificial freezing is relatively high and there are few published cases where costs of artificial freezing can be compared directly with alternative schemes such as grouting or dewatering. In general, freezing has not been used where other techniques are applicable but only as a last resort when local job conditions prohibit one of the other more commonly used techniques. Schuster suggests that freezing would be competitive cost-wise for supporting excavations more than 20 to 25 ft (7 to 8 m) deep in bouldery, soft, or running ground and for any tunnel in similar soils or mixed face conditions, particularly below the water table. Jackson feels freezing is competitive for ground support and control of water when an excavation is more than 20 ft (6 m) deep.

6.6 APPLICATIONS OF FREEZING TO TUNNELS AND SHAFTS

Table 6.1 summarizes some applications of artificial ground freezing in construction of excavations; several of these are discussed in more detail. Silt and fine sand below the water table are the most common ground conditions which have been successfully stabilized by artificial freezing. Chemical grouts or dewatering under vacuum or by electro-osmosis, both expensive techniques, would be required to stabilize these materials and the probability of success would still not be high. These materials may also be relatively loose so that dewatering, if successful, may lead to undesirable settlements. Compressed air is often used in such ground, but in many cases small cover or permeable strata above a tunnel increase the possibility of blow-outs, high air pressures decrease worker efficiency and increase safety hazards, or equipment and noise intefere with existing facilities so that the use of compressed air is precluded. Artificial freezing eliminates most of these problems and, although expensive, has been used where the more common stabilization techniques are not feasible.

Another common application for artificial freezing has been in the case of interbedded soils of varying gradation. Both grouting and dewatering are dependent on permeability. In these ground conditions, it may be difficult to grout the finer zones without use of expensive and time-consuming techniques to insure grouting of all the units. Dewatering may be difficult for similar reasons. It may also be difficult to plan a dewatering scheme in such conditions because of wide variations in permeability. Whether a soil can be frozen and the amount of time and energy required to achieve

freezing are functions mainly of thermal properties of the soils and temperature to which they are to be maintained. The variation in thermal properties between two different soil types or within a given soil type is much less than variations in permeability (Sanger). As a result, even though artificial freezing may be more expensive than grouting or dewatering, the probability of success may be higher with freezing.

Ground water flows have been the major nemesis to successful freezing. If the natural ground water flow in an area is small, freezing an area may be feasible in advance of excavation. If closure is not obtained in the frozen soil wall surrounding a tunnel or shaft in advance of excavation, flows directed into the excavation may make freezing very difficult, if not impossible.

Common practice in shaft construction is to form a frozen soil wall around the area to be excavated. In mine shafts, depths as great as 720 ft (220 m) have been successfully frozen (Construction Methods and Equipment, 1954). Artificial freezing has been used only a few times in tunnel construction, common practice being to install freeze elements from the ground surface. Normally the ground is frozen from the ground surface to an impermeable base or to below the invert a sufficient distance to resist uplift, and around and within the limits of the tunnel. Gail describes freezing for construction of a sewer tunnel where frozen walls were formed from the ground surface to rock below the invert parallel to the tunnel, with frozen transverse walls about 100 ft (30 m) apart.

Where depth of overburden is large or utilities present near the ground surface could be affected by freezing, it may be more

TABLE 6.1

TYPICAL FREEZING PROJECTS

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Reference	Project Description	Ground Conditions	Reason for Freezing	Freeze Elements Pattern	Size	Spacing	Depth	Coolant	Coolant Temperature	Approximate Refrigeration Plant Load	Approximate Time Freeze
Latz, 1952	15' φ mine shaft	350' sed. rock with water bear. silt & sand; sol. cavities	Failure of grout to con- solidate flowing sand	31'φ	2"in6"	3'	350'	CaCl$_2$ brine	+5°F to -10°F	60 to 100 tors	8 to 10 wks
Jackson, 1969	150' x 100' x 50' deep excavation for nuclear power plant foundation	50' of sand & gravel over shale; water table @ 20'	Elimination of heavy bracing req'd with sheet piles; quantity of exc. large with de- watering & open cut	Oval wall outside exc, inner wall around limits of exc.	1-1/2" in 6"	3'-9" and 5'-0"	56'	CaCl$_2$ brine	-13°F	250 tons	---
Low, 1960	Tunnel reconstruc- tion; removal of double arch & dividing wall under railroad traffic	Uniform silt, below water table; adjacent 3 & 9 story buildings	Silt too low perm. for grout; busy railroad tunnel precluded comp. air; depth great for vac. drainage	Rectangu- lar grid	1" to 2-1/2"	4'±	18' to 20'	Methanol solution	-10°F	150 tons	3 to 4 wks
Gail, 1972	Sewer tunnel, 20' φ, 80' ± cover above invert	Mixed face; water bearing sand & mud	Compressed air ruled out be- cause of hazards & noise	2 lines, one each side of tunnel from gnd. surf.	---	2'-6"	40' to 80'	CaCl$_2$ brine	---	270 tons	8 weeks
Ellis & McConnell, 1959	Pump station ad- jacent to old 48" brick sewer	Silty fine sand, below water table	Minimize damage to exist. sewer	35'φ	---	2'-9"	50'	CaCl$_2$ brine	---	---	13 weeks ±
Stewart, et al., 1963	20'φ x 925' deep shaft (top 120' frozen)	Rubble fill over sand, gravel, & boulders, below water table	Control water; compressed air req'd high pres- sures, dewater- ing may cause settlements	30'φ	1 1/2" in 3"	3'-4"	120'	CaCl$_2$ brine	-50°F	125 tons	4 to 8 wks
Const. Methods & Equip, 1960	14 1/2'φ x 318' deep shaft (top 123' frozen)	Rubble fill, silt, clay be- low water table	Dewatering may have caused settlement of adjacent bldgs.	26 1/2'φ	2"in6"	4'	123'	CaCl$_2$ brine	-32°F	---	11-12 wks
Const. Methods & Equip, 1954	16'φ x 720' deep shaft (entire depth frozen)	90' mixed over- burden over lime- stone & sand- stone	Solution fea- tures in lime- stone contain- ing water	32'φ	2"in6"	3 1/2'	720'	CaCl$_2$ brine	-20°F	200 tons	8 weeks
Braun, 1972	Sewer tunnel, 8'φ, 13' to 15' cover, 2 sections; 1 @ 137', 1 @ 199' separated by access shafts	Mixed silts fine sands, (some clay)	Possibility of blowouts with compr. air; difficulty in grouting or dewatering.	11'φ (installed horiz. parallel to tunnel axis)	1 1/2" in 4"	2'-9"	125' max length (inst. from access shafts)	CaCl$_2$ brine	-13°F±	75 tons	1 week

economical to construct frozen walls around the perimeter of the tunnel.
If face stability is a problem, the entire tunnel cross section may be frozen
This scheme is thermally more efficient since only the ground immediately
adjacent to or within the tunnel needs to be frozen, the thickness of the
frozen wall being established to resist water and overburden pressures.
Freeze elements are installed horizontally parallel to the tunnel from
access shafts or possibly from pilot drifts. Braun describes a case where
a tunnel was successfully constructed in this manner using jacked-in-place
freeze elements about 125 ft (38 m) long which were lapped 25 ft (7.6 m)
with freeze elements from an adjacent shaft. Alignment control of the
freeze elements is important to insure closure of the frozen wall, particu-
larly where freeze elements overlap from adjacent shafts.

It is extremely important in tunnel and shaft construction to a-
void damage to the freeze elements during installation or excavation. Many
European countries prohibit blasting in a frozen tunnel or shaft because of
possible damage to freeze elements (Walli) and because heat generated
during drilling can thaw the frozen ground. If freeze elements are
damaged, brine can escape into the ground, necessitating the use of a
coolant with a lower freezing point than the lost brine, a larger
refrigeration plant, and a longer time to achieve freezing. Scott reports
such problems on a shaft sinking project in Canada where calcium chloride
brine was lost into the ground to be frozen. Since the calcium chloride
brine froze at -40°F (-40°C), it was necessary to lower the temperature to
-45°F (-43°C) which required the use of lithium chloride brine at greater
expense. Including delays, 13 months was required to complete freezing.

The original outer freeze pipe was standard NX casting which apparently ruptured during driving; a 2 in. (50.8 mm) extra heavy pipe was then installed in the NX casing with a 1 in. (25.4 mm) inner freeze pipe to complete freezing.

6.7 ILLUSTRATIVE CASE HISTORIES

6.7.1 SEWER TUNNEL, DORTMUND, GERMANY (Braun)

This project is a sewer tunnel of 7 ft-10 in. (2.4 m) excavated diameter driven beneath a railway yard for a distance of about 350 ft (110 m). Depth of cover was only 13 to 15 ft (4 to 4.5 m) in subsoils consisting of alternating silty fine sand and sandy fine silt below the watertable. Dewatering was considered but was ruled out because of the uncertainty of successful dewatering, and because of the possibility of causing damaging settlement of the railroad. Compressed air was ruled out because the small overburden made danger of a blow-out high. Thus, artificial freezing was selected as the stabilization technique.

An access shaft was sunk on each side of the railroad yard and a third shaft was sunk in the yard dividing the tunnel into two sections of 199 ft (60 m) and 137 ft (42 m). Four inch (101.6 mm) diameter freeze pipes were jacked in from each access shaft parallel to the tunnel axis on an 11 ft (3.4 m) diameter circle, resulting in a freeze pipe spacing of 2 ft-9 in. (0.8 m) around the outline of the tunnel. Pipes were jacked one-half the distance to the adjacent shaft plus 25 ft (7.6 m) for overlap. Relief pipes were also installed horizontally within excavation limits to permit dewatering of the unfrozen core.

The design of the frozen soil wall was apparently by a pseudo-elastic analysis from which the maximum theoretical compressive stress in the frozen soil could be obtained. Frozen triaxial and unconfined compression tests were then run to determine compressive strength of the samples at various temperatures. These tests are not described in detail but apparently were at conventional loading rates. The minimum factor of safety (ratio of laboratory compressive strength to calculated maximum compressive stress) in frozen walls determined from these tests and for a 2 ft-6 in. (0.8 m) thick frozen wall was 5.

Calcium chloride brine was used as the refrigerant and the refrigeration plant was capable of cooling the brine to -13°F (-25°C). Closure of the ice wall occurred in about 6 days as determined from water level rise in relief holes and by temperature measurements from thermometers installed in the area to be frozen.

Excavation was intended to be totally within the unfrozen core using a boom cutter-loader; however, in the areas where freeze elements from adjacent shafts overlapped, the entire cross section was frozen. In this area, special cutters were required because of the wear on the standard cutters which were designed for excavating plastic clays. Excavation rates averaged about 26 ft (8 m) per 12 hour shift where the core was unfrozen to 9 ft to 13 ft (3 m to 4 m), depending on soil type, where the tunnel cross section was entirely frozen.

6.7.2 RAILWAY TUNNEL CONSTRUCTION (Low)

This project involved the reconstruction of a double track railway

tunnel which had originally been constructed with a double arch and center
dividing wall. The reconstruction involved removal of the double arch and
dividing wall for a distance of 175 ft (53.3 m). The reconstruction had to be
done while maintaining traffic. The affected portion of the tunnel was beneath
a city street between a modern 9 story reinforced concrete building and an
older 3 story wood and brick building; numerous utilities were buried in
the street overlying the tunnel. The new tunnel had to be advanced through
a layer of water bearing silt just below sub-base of the street. This silt was
the bearing layer for the foundations of the older building. Total depth
of cover above the new arch was 15 to 20 ft (4.6 to 6.1 m).

Grouting was rejected as a possible stabilization technique because
of the low permeability of the silt. Dewatering was rejected because of the
high vacuum which would be required and because of the suspicion that leaking
water pipes were responsible for the water in the silt and would replenish
the water even if drainage were possible. Compressed air couldn't be used
because the equipment would have interfered with the train service which had
to be maintained. As a result, it was decided to freeze the silt. Freezing
eliminated the need for underpinning the smaller building, and freeze elements
could be located to avoid interference with the overlying utilities.

Methanol was used as the coolant and was circulated through 1 in.
(25.4 mm) interior and 2-1/2 in. (63.5 mm) exterior freeze pipes extending
from the surface to the old tunnel crown. First a frozen curtain was formed
around the excavation limits and then the interior freeze elements were
opened to form a frozen mass ahead of the excavation. As the advancing exca-
vation encountered freeze pipes, they were disconnected from the freezing

6-25

circuit and used as supports for scaffolding. The coolant temperature was -10°F (-23°C), requiring 3 to 4 weeks to completely freeze the silt.

Drilling and blasting was used to excavate the frozen silt. Excavation to the new arch outline proceeded above the old arches without interfering with train traffic. Tunneling progress averaged about 1-1/2 ft (0.5 m) per day. This low rate of progress was probably due primarily to the limited work space which required hand loading of muck, etc. However, problems were also encountered in drilling the frozen silt since heat at the bit thawed the silt and clogged the holes with mud. To overcome this, it was necessary to drill in short bursts only.

The amount of heave of the street due to frost expansion of the silt was estimated at about 5 in. (127 mm) due to a calculated 10 percent volume increase in the silt upon freezing (no further details on these calculations are given). Heave of about this amount was observed and badly cracked the pavement. This damage was repaired when full thawing had occurred.

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