Soil Improvement — State-of-the-Art Report

Amélioration des Sols

J.K. MITCHELL Professor and Chairman of Civil Engineering, University of California, Berkeley, CA, USA

SYNOPSIS Because of the increasing need to utilize marginal sites and because many soils can be made into useful construction materials if properly treated, soil improvement has become a part of many present day civil engineering projects. In this state-of-the-art report the principles, applications, and design procedures for soil improvement using different methods are presented. Soil improvement methods reviewed include in-situ deep compaction of cohesionless soils, precompression with and without vertical drains, injection and grouting, admixture stabilization, thermal methods, and soil reinforcement. Comprehensive recent references on each topic are listed.

INTRODUCTION

Most of man's construction is done on, in, or with soil. As the availability of suitable construction sites decreases, the need to utilize poor soils for foundation support and earthwork construction increases. In addition, it is becoming increasingly necessary to strengthen the ground under existing structures to insure stability against adjacent excavation or tunneling, or to improve resistance to seismic or other special loadings. Furthermore, many hundreds of recent successful projects have shown that through the use of suitable reinforcement materials and systems, the uses of nature's most abundant construction material--soil--can be greatly extended. It is not surprising, therefore, that the general area of soil improvement and reinforcement has been of great interest and rapid development in the past several years. The inclusion of this topic for the first time as a main session in an International Conference of our Society is clear recognition of this fact.

The basic concepts of soil improvement; namely, drainage, densification, cementation, reinforcement, drying, and heating, were developed hundreds or thousands of years ago and remain valid today. The coming of machines in the 19th century resulted in vast increases in both the quantity and quality of work that could be done. Among the most significant developments of the past 50 years are the introduction of vibratory methods for densification of cohesionless soils, new injection and grouting materials and procedures, and new concepts of soil reinforcement.

The purpose of the present report is to synthesize the present state-of-the-art of soil improvement into a form suitable for direct application by practicing engineers. At the same time the author has attempted to identify sufficient references that interested readers will be able to locate more detailed information, case histories, etc. Emphasis in this report is on practical aspects of soil improvement, to include such considerations as soil types best suited for treatment, effective treatment depths, properties of treated soils, major applications, and relative costs. Because of the extensive scope of the subject the inclusion of detailed case histories is not possible.

METHODS AND SCOPE

The Conference Organizing Committee has identified the following topics for discussion in Session 12, so they have been chosen for coverage in this state-of-the-art report.

1. Compaction, especially heavy tamping and blasting. Emphasis is on in-situ deep densification of cohesionless soils, and compaction in thin layers is excluded.

2. Consolidation by preloading and/or vertical drains, electro-osmosis.

3. Grouting, excluding control of groundwater flow and seepage.

4. Soil stabilization using admixtures and by ion exchange (chemical stabilization). Emphasis is on new developments and applications other than subgrades and base courses for roads and airfields.

5. Thermal stabilization.

6. Reinforcement of soil. Emphasis is on the inclusion during construction or installation in-situ of both tensile and compression reinforcement elements. Geotextiles used as reinforcement are included.

The report concludes with a tabular summary of the methods discussed and consideration of factors governing the choice of a method for any given case.

DEEP COMPACTION OF COHESIONLESS SOILS

Introduction

Thick deposits of loose cohesionless soils may require improvements in order to eliminate the subsequent development of excessive total and differential settlements and to minimize the possibility for liquefaction under dynamic loading. Suitable improvement can be achieved in many cases by densification; however, the needed densification cannot ordinarily be achieved using preload surcharge fills or compaction at the surface. In-situ densification of loose, cohesionless soil layers is usually done by dynamic methods. In many methods dynamic loading is accompanied by displacement in the form of the insertion of a probe and/or construction of a sand or gravel column in-situ.

Methods that are used for the in-situ deep densification of cohesionless soils include blasting, vibrocompaction, and heavy tamping. Vibrocompaction is used herein to refer collectively to all those methods involving the insertion of vibrating probes into the ground with or without the addition of a backfill material. Compaction piles are also considered in this category. The ability of any of these methods to accomplish the needed improvement in properties depends on several factors, including:

1. Soil type, especially its gradation and fines content
2. Degree of saturation and water table location
3. Initial relative density
4. Initial in-situ stresses
5. Initial soil structure, including the effects of age, cementation, etc.
6. Special characteristics of the method used.

Mechanism of Densification

Densification of cohesionless soil layers with accompanying improvement in mechanical properties requires first that the initial soil structure be broken down so that particles can be moved to new packing arrangements. In saturated cohesionless materials this is most readily accomplished by inducing liquefaction by means of dynamic and cyclic loadings. In the case of methods such as blasting and heavy tamping the compression wave generated by the sudden large energy release can give an immediate build-up in pore water pressure which greatly reduces the shear strength. This wave is followed by a shear wave which is responsible for failure of the mass. After passage of these waves the soil particles settle into new and, ultimately, more stable positions. Vibrocompaction methods are effective in much the same way, except that the energy per event is many times smaller, the vibrations continue over a much longer period, and the effects are felt to distances from the energy source of one to two meters instead of up to 10 m or more as is the case with blasting and heavy tamping.

For partly saturated soils, including some containing fines and many waste fills, densification is mainly by collapse of the soil structure and escape of gas from the voids. The process is much the same as densification by impact compaction as commonly done in the laboratory.

Densification accompanying ground treatment by these methods occurs rapidly. Settlement of the ground surface is essentially complete by the end of treatment. Improvement in properties, as measured, for example, by penetration tests or pressuremeter tests, may continue over extended time periods. This latter point may be of considerable practical importance in the evaluation of ground treatment.

Experience has indicated that it is often easier to densify to a specified high relative density from a loose initial condition than from an intermediate relative density. This is because the initial structure of the loose material is easier to break down.

Soil Type Considerations

Vibrocompaction methods are best suited for densification of clean, cohesionless soils. Experience has shown that they are generally ineffective when the percentage by weight of fines (particles finer than 200 mesh sieve or 0.074 mm diameter) exceeds 20 to 25. This is because the permeability of materials containing greater percentages of fines is too low to allow the rapid drainage of pore water that is required for densification following liquefaction under the action of the vibratory forces, and because the structure may be more difficult to disrupt owing to cohesion contributed by the fines.

Some soils containing greater amounts of fines; e.g., some silty sands and loess, can be densified by blasting and/or heavy tamping, both of which impart large amounts of energy all at once and cause large ground displacements. More specific considerations concerning the influences of soil type are presented in the discussion of particular methods. For preliminary planning, however, it may be considered that the range of particle size distributions shown by Fig. 1 will be best suited for densification by deep in-situ methods.

Evaluation of Treated Ground

Measurement of the effectiveness of deep compaction can be made using one or more of several methods. Techniques that have been used include:

1. Surface settlement markers
2. Volume of soil added to fill craters, to form compaction piles, or to carry out a vibrocompaction process
3. Standard Penetration Tests (SPT)
4. Cone Penetration Tests (CPT)
5. Pressuremeter Tests (PMT)
6. Seismic shear wave velocity determinations
7. Pile driving resistances
8. Plate load tests

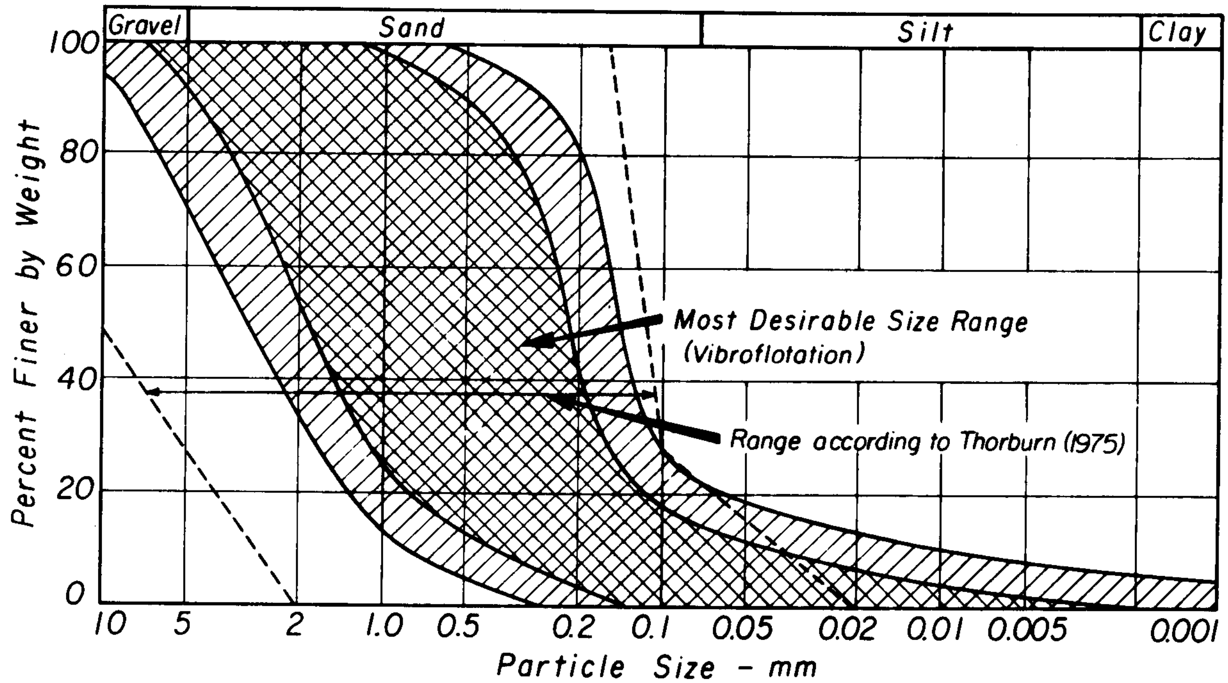


Fig. 1 Range of Particle Size Distributions Suitable for Densification by Vibrocompaction

9. Down-hole density meters.

Settlement measurements and SPT, CPT, or PMT are the most commonly used methods. The CPT is particularly useful because it provides a continuous record of penetration resistance with depth, it is fast, and it is well-suited for use in sands. Penetration tests for evaluation of improved ground are usually done at locations intermediate between probe points in order to provide the most conservative estimate of improvement.

Penetration resistance values, both before and after ground improvement, are often converted to relative densities using one or more of several correlations that have been developed for particular conditions. Design criteria and specifications are many times developed in terms of relative density. A direct conversion of a penetration resistance to relative density is uncertain, however, because penetration resistance depends on factors other than density. The correlations are not independent of soil type. Increased lateral pressure, increased time under pressure, increased stability of structure, and prior seismic strains lead to increased penetration resistance (Seed, 1979). Fortunately, these latter factors also lead to corresponding increases in resistance to settlement and liquefaction, and it is the penetration resistance values themselves that are important, not the actual relative density. It has been found convenient for some applications, however, to work with an "equivalent relative density" which is the true relative density a sand deposit would have to possess to exhibit the measured penetration resistance if it were freshly deposited and normally consolidated.

Approximate correlations between equivalent penetration resistance, sand density and properties pertinent to the assessment of foundation stability are given in Table I.

In a few cases instrumentation has been used to monitor conditions during densification. Total pressure cells and piezometers have provided data useful for developing improved understanding of the densification and property improvement process.

Blasting

Deep compaction by detonation of buried explosives can provide a rapid, low cost means for soil improvement in some cases. The general procedure consists of:

1. Installation of pipe by jetting, vibration, or other means to desired depth of charge placement
2. Placement of charge in pipe
3. Backfilling the hole
4. Detonation of charges according to a pre-established pattern.

In some cases the pipe is withdrawn prior to detonation of the charges. In others it is reclaimed after the blast, a new section is welded to the bottom, and it can be used again. The explosives used include dynamite, TNT, and ammonite. Detailed descriptions of blasting are given by Prugh (1963), Ivanov (1967), Mitchell (1970), Litvinov (1973, 1976), Damitio (1970-72), Donchev (1980), and others.

TABLE I

Penetration Resistance and Sand Properties

	Very Loose	Loose	Medium Dense	Dense	Very Dense
SPT N-value (blows/0.3 m)*	< 4	4-10	10-30	30-35	> 50
CPT cone resistance (kg/cm^2)*	<50	50-100	100-150	150-200	>200
Equivalent Relative Density (%)**	<15	15-35	35-65	65-85	85-100
Dry Unit Weight (kN/m^3)	<14	14-16	16-18	18-20	> 20
Friction Angle (°)	<30	30-32	32-35	35-38	> 38
Cyclic Stress Ratio Causing Liquefaction (τ/σ_o')***	<0.04	0.04-0.10	0.10-0.35	>0.35	

*At an effective vertical overburden pressure of 100 kPa.

**Freshly deposited, normally consolidated sand.

***From Seed (1979), Fig. 6(a).

Saturated, clean sands are well-suited for densification by blasting. Success in any case depends on the ability of the shock wave generated by the blast to break down the initial structure, and create a liquefaction condition for a sufficient period to enable particles to rearrange themselves in a denser packing. It follows, therefore, that the stronger the sand initially, the larger the charges that will be required for effective densification. Thus, the greater the depth to which densification is needed and the higher the initial equivalent relative density, the greater the explosive energy required.

There appear to be no generally accepted theoretical design procedures for densification by blasting, and field trials are usually used prior to production blasting. A number of field cases have been summarized by Ivanov (1967) involving treatment depths up to 20 m. From these experiences the following general guidelines emerge:

1. Charge size: <1 to 12 kg
2. Depth of burial: >1/4 depth to bottom of layer to be treated; 1/2 to 3/4 of depth common
3. Charge spacing in plan: 5-15 m
4. Number of coverages: 1-5 with 2-3 usual. Each coverage consists of a number of individual charges. Successive coverages are usually separated by hours or days.
5. Total explosive use: 8-150 gm/m^3, 10-30 gm/m^3 typical
6. Surface settlement: 2 to 10% of layer thickness.

The maximum depth to which blasting can be used successfully for soil compaction is not known. The author is currently associated with a project in which charges of up to 30 kg have been detonated at depths more than 40 m below ground surface. Significant surface settlements and improvement in the equivalent relative density of loose zones at depths greater than 30 m have been achieved. An interesting feature of this work is the observation that although surface settlement is immediate, which means that densification is also essentially immediate, results of cone penetration tests do not indicate an increase in equivalent relative density to the required value for several weeks, reflecting an aging or healing effect following disruption of the initial structure and formation of a new one.

It would be expected that the maximum depth for effective treatment will be limited by the practical difficulty of placing concentrated charges of sufficient magnitude to create a shock wave great enough to liquefy the initial soil structure. As the depth increases, so do the effective stresses and strength. Accordingly, the required disruptive stress will increase, and the effective radius of influence will decrease. The magnitudes of the shock wave pressure, P_{max} in kg/cm^2, and the impulse per unit area, I, in $kg\text{-}sec/cm^2$, are given by Ivanov (1967) as follows

$$P_{max} = k_1 \left(\frac{\sqrt[3]{C}}{R} \right)^{\mu_1} \qquad (1)$$

$$I = k_2 \left(\sqrt[3]{C}\right) \left(\sqrt[3]{\frac{C}{R}}\right)^{\mu_2} \quad (2)$$

in which C = size of charge (kg of TNT)

R = distance from center of charge (m)

μ_1, μ_2 = empirical coefficients from Table II.

These relationships can be used for comparative studies of the probable influences of charge size, charge spacing, and sand saturation. It may be seen that the presence of even small amounts of gas leads to significant damping of the P-wave pressure.

It has been possible by blasting to densify sands to equivalent relative densities of 75 to 80 percent. In some cases, however, the results may be erratic, initially dense zones may be loosened, and the method is not likely to be effective in the upper one or two meters below the ground surface. Typical behavior may be summarized as follows.

1. Almost immediate settlement of the ground surface, with little further settlement with time.

2. Initially loose zones show little immediate change in penetration resistance. Penetration resistance increases slowly with time until after several weeks the material indicates a marked improvement in properties compared to its initial condition.

3. Zones which are initially very dense may be permanently loosened or weakened by the blast; however, the resultant condition is still likely to be satisfactory.

4. Ultimately, an effective blasting program results in a deposit in which all the initially loose zones have been suitably improved.

When a series of coverages is used, the surface settlement accompanying each coverage is usually less than the one preceding.

Attempts have been made to compact using surface explosions, because of simplicity, low cost, and speed. Because of energy loss above ground, lack of confinement, and the formation of surface depressions, however, this method has been of limited effectiveness.

A hydro-blasting technique has been used very successfully and economically for compaction of collapsible loess deposits (Litvinov, 1973, 1976; Donchev, 1980). Although collapse of the loess can often be accomplished by flooding alone, it has been found that more uniform results can be achieved more quickly and economically by this method. The procedure, which is illustrated in Fig. 2, consists of first cutting a contour trench 0.2 m to 0.4 m wide and several meters deep around the perimeter of the area to be densified. Boreholes spaced a few meters apart in a grid pattern are then used to pump water into the loess, over a period of several days, ideally until the water content is increased to above the liquid limit. Slurry walls or plastic membranes can be installed to prevent lateral migration of the water and softening of adjacent ground.

Explosive charges of about 5 kg each are then inserted down tubes installed at spacings of three to six meters in grid patterns and detonated. Surface settlements of up to 10 percent of the layer thickness and reduction in porosity of several percentage points are not uncommon. Areas of 1000 m^2 to 10,000 m^2 involving 10,000-100,000 m^3 of loess can be treated at one time.

Successful compaction of saturated sand and collapsible loess has been accomplished in the USSR using high energy, high voltage electrical discharges from probes inserted in the ground (Lomize et al., 1963, 1973). Each discharge, which may release 50 to 100 kJ of energy, has an effect similar to that of an explosion of comparable magnitude. A number of discharges are released spaced at intervals of several seconds at each level as the probe is moved upwards from the bottom. It appears that use of this method has not yet been widely adopted.

TABLE II

Parameters for Estimating Blast Pressures and Impulse Values
(from Ivanov, 1967)

	Gas Content (%)	k_1	μ_1	k_2	μ_2
Sand below water table	0	600	1.05	0.080	1.05
	0.05	450	1.5	0.075	1.10
	1	250	2.0	0.045	1.25
	4	45	2.5	0.040	1.40
Moist sand (8-10% water)	-	7.5	3.0	0.035	1.50
(2-4% water)	-	3.5	3.3	0.032	1.50

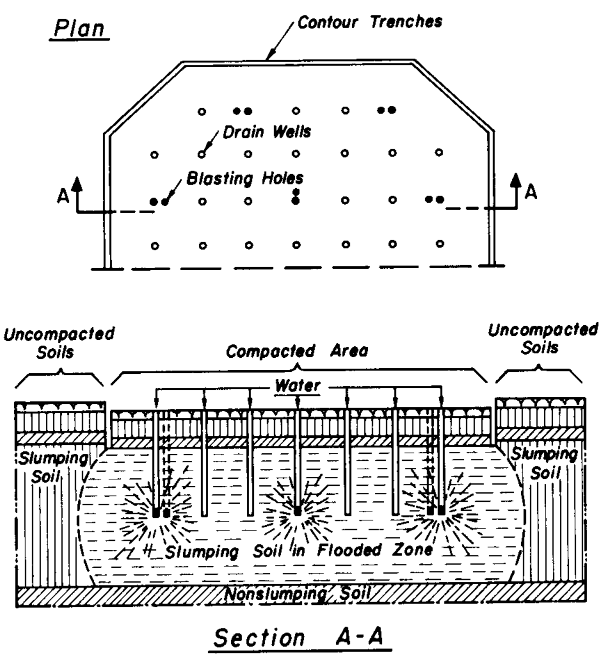


Fig. 2 Loess Compaction by Hydro-Blasting

Vibrocompaction and Compaction Piles

These methods for deep compaction of cohesionless soils are characterized by the insertion of a cylindrical or torpedo-shaped probe into the ground followed by compaction by vibration during withdrawal. In a number of the methods a granular backfill is added so that a compacted sand or gravel column is left behind within a volume of sand compacted by vibration. Sinking of the probe to the desired treatment depth is usually accomplished using vibratory methods, often supplemented by water jets at the tip. Injection of air at the same time has been found to facilitate penetration to large depths. Upward directed water jets along the sides has also been found helpful in some cases. Soil gradations suitable for densification by vibrocompaction are indicated in Fig. 1. Compaction piles of sand and gravel formed by these methods are also used in soft cohesive soils, in which case they function as compression and shear reinforcement, as discussed in a later section of this report. Ground treatment depths of 20 m can be achieved routinely by these methods. Depths in excess of 30 m can be attained in some cases.

A brief description of some of the more extensively used vibro-compaction methods is given below.

1. Vibrating Probes

The Terraprobe method, developed in the U.S.A. (Anderson, 1974), uses a Foster Vibrodriver pile hammer on top of a 0.76 m dia. open tubular probe (pipe pile) that is 3 to 5 m longer than the desired penetration depth. The unit operates at a frequency of 15 Hz and a vertical amplitude of 10-25 mm. About 15 probes per hour can be done at spacings of 1 to 3 m. It is of marginal effectiveness in the upper 3 to 4 m of the zone to be densified.

Vibro-rods developed by Saito (1977), Fig. 3, are also driven using a vibratory pile driving hammer. Several cycles of insertion and withdrawal are used in the densification process.

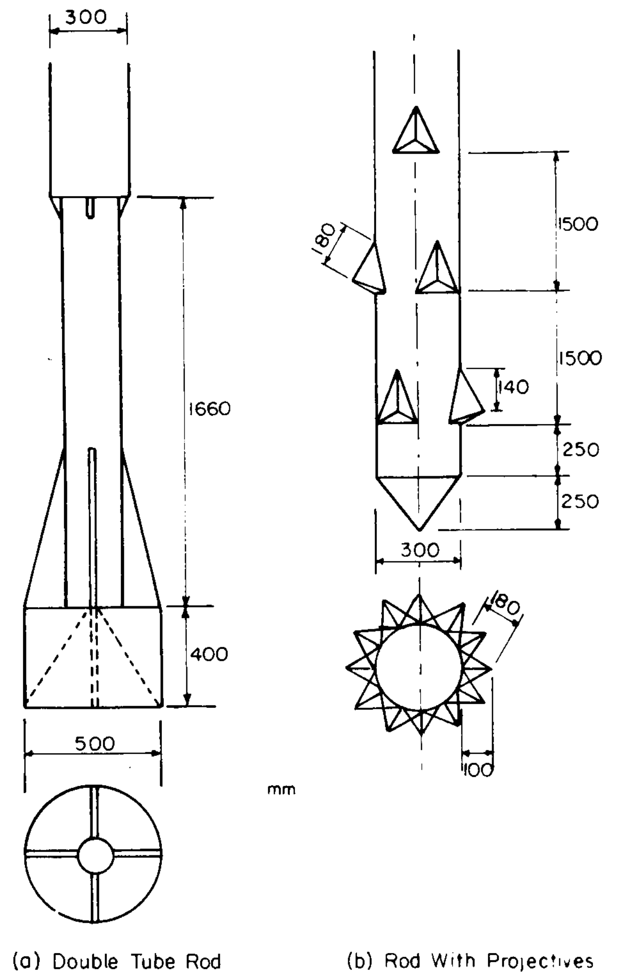


Fig. 3 Vibro-Rods Used for Sand Densification (from Saito, 19--

2. Vibroflotation

This method was developed in Germany almost 50 years ago, and its development has continued there and in the U.S.A. where it was introduced in the 1940's. The equipment consists of three main parts: the Vibrator, extension tubes, and a supporting crane. A schematic diagram of the equipment and process is given in Fig. 4. The vibrator is a hollow steel tube containing an eccentric weight mounted on a vertical axis in the

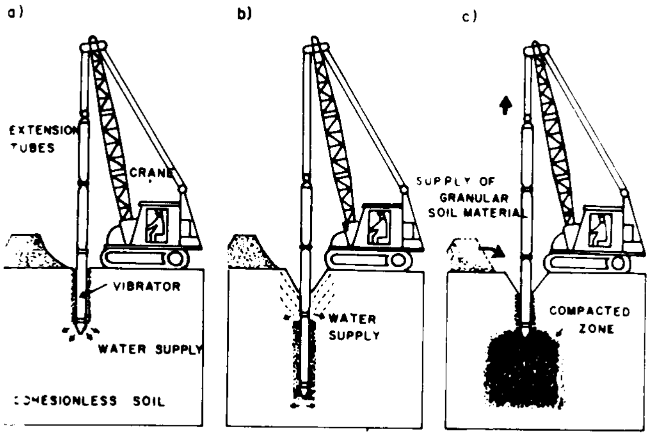


Fig. 4 Vibroflotation Equipment and Process

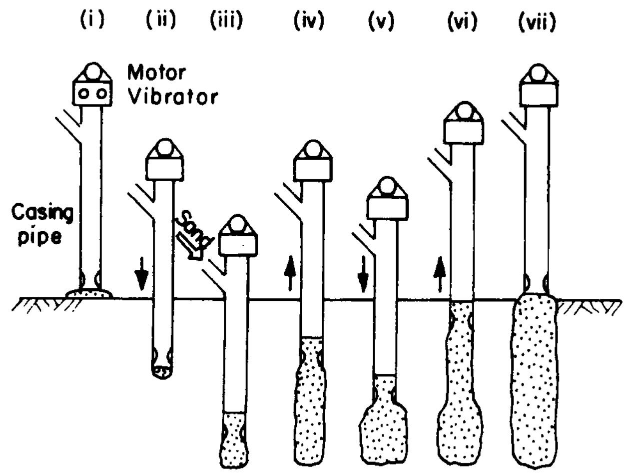


Fig. 5 Construction of Compaction Piles by the Compozer System

lower part so as to give a horizontal vibration. Vibrator diameters are in the range of 350 to 450 mm and the length is about 5 m including a special flexible coupling. One vibrator weighs about 20 kN. Units developing centrifugal forces up to 160 kN and variable vibration amplitudes of up to 25 mm are available. Most usual operating frequencies are 30 Hz and 50 Hz. The extension tubes have a slightly smaller diameter than the vibrator and a length dependent on the depth of penetration required.

Vibroflot sinking rates of 1 to 2 m/min and withdrawal/compaction rates of about 0.3 m/min are typical. Water pressures of up to 0.8 MPa and flow rates up to 3,000 l/min may be used to facilitate penetration. Sand backfill is consumed at a rate of up to 1.5 m^3/m during the compaction process. The zone of improved soil extends from 1.5 m to 4 m from the vibrator, depending upon soil type and vibroflot power. Additional details are presented by Baumann and Bauer (1974), Bell (1975), and Brown (1977), among others.

3. Vibro-Compozer Method

This sand compaction pile method was developed by Murayama in Japan in 1958 (Murayama, 1958). The apparatus and procedure used in the compozer system are shown schematically in Fig. 5. A casing pipe is driven to the desired depth by a vibrator at the top. A sand charge is then introduced into the pipe, the pipe is withdrawn part way while compressed air is blown down inside the casing to hold the sand in place. The pipe is vibrated down to compact the sand pile and enlarge its diameter. The process is repeated until the pipe reaches the ground surface. The resulting pile is usually 600 to 800 mm in diameter. The actual diameter can be estimated from the sand volume discharged into the ground.

4. Soil Vibratory Stabilizing Method

This method, termed both the SVS method and the Toyomenka method, combines both the vertical vibration of a vibratory driving hammer and the horizontal vibration of a Vilot depth compactor. The Vilot is a special probe of about the same size as vibroflot units. Sand backfill is used, but water is not used in either the sinking or compaction process.

The gradation of both the in-situ soil and the backfill, which may or may not be the same material, influence the level of improvement that may be obtained. Coarse sands give greater densification than fine sands, evidently because the coarser material is better able to transmit vibrations. Brown (1977) has defined a suitability number for vibroflotation backfills that is given by

$$\text{Suitability Number} = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}} \qquad (3)$$

in which D_{50}, D_{20}, and D_{10} are the 50, 20, and 10 percent size grain diameters in mm. Corresponding suitability numbers and backfill ratings are

0 - 10	Excellent
10 - 20	Good
20 - 30	Fair
30 - 50	Poor
>50	Unsuitable

The lower the suitability number the faster the vibroflot can be withdrawn while still achieving acceptable compaction.

The influence of fines on the level of improvement that can be obtained by vibrocompaction is shown clearly by the data in Fig. 6 (Saito, 1977). The data provide excellent support for the rule of thumb that vibrocompaction is ineffective in soils containing more than 20 percent fines. Bhandari (1977) describes a case in which compaction piles led to soil densification only in zones where the fines content was less than 20 percent.

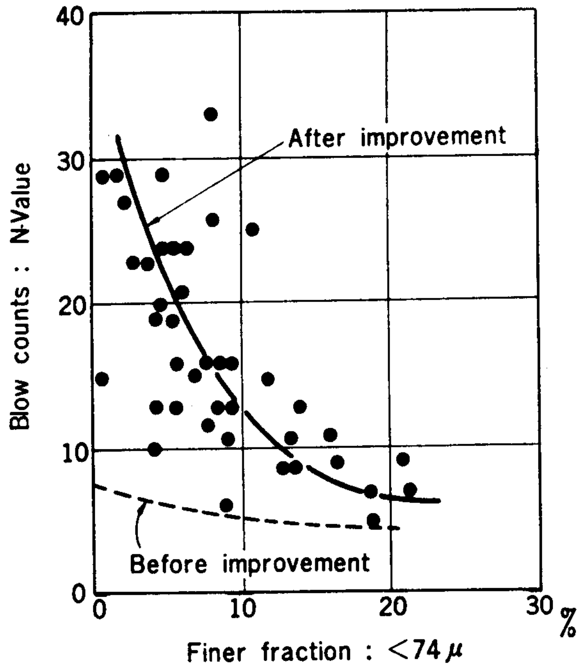


Fig. 6 Effect of Fines Content on Increase in Penetration Resistance by Vibro-compaction
(from Saito, 1977)

In some instances penetration resistances are so high following densification by vibrocompaction that relative densities greater than 100 percent are indicated according to conventional correlations. In reality, however, this result is in most instances caused by the increased lateral pressure induced by the vibration process. In-situ lateral pressure measurements reported by Saito (1977) yielded values of K_o as high as 6 in some zones after densification.

Vibrocompaction of large areas is done in a grid pattern, either triangular or rectangular, with probe spacings usually in the range of 1.5 m to 3 m on centers. The actual spacings depend on the soil and backfill types, probe type and energy, and level of improvement required. Although field tests are usually done to finalize designs, there are some guidelines that can be useful in preliminary studies.

If it is desired to increase the average density of loose sand from an initial void ratio e_o to a void ratio e, and if it is assumed that installation of a sand pile causes compaction only in a lateral direction, the pile spacings may be determined using

$$S = \left(\frac{\pi(1 + e_o)}{e_o - e}\right)^{1/2} d \tag{4}$$

for sand piles in a square pattern, Fig. 7(a), and

$$S = 1.08\left(\frac{\pi(1 + e_o)}{e_o - e}\right)^{1/2} d \tag{5}$$

for piles in a triangular pattern, Fig. 7(b), in which d is the sand pile diameter (up to 800 mm). An influence curve method that can be used to estimate vibroflotation probe spacings to give a specified minimum relative density is given by Brown (1977). Brown's curves are based on the procedure developed by D'Appolonia (1953).

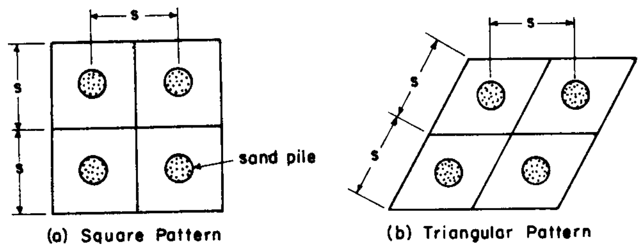


Fig. 7 Usual Vibrocompaction and Compaction Pile Patterns

Empirical design curves for vibrocompaction have been developed by Thorburn (1975). Fig. 8 is a relationship between the relative density of a

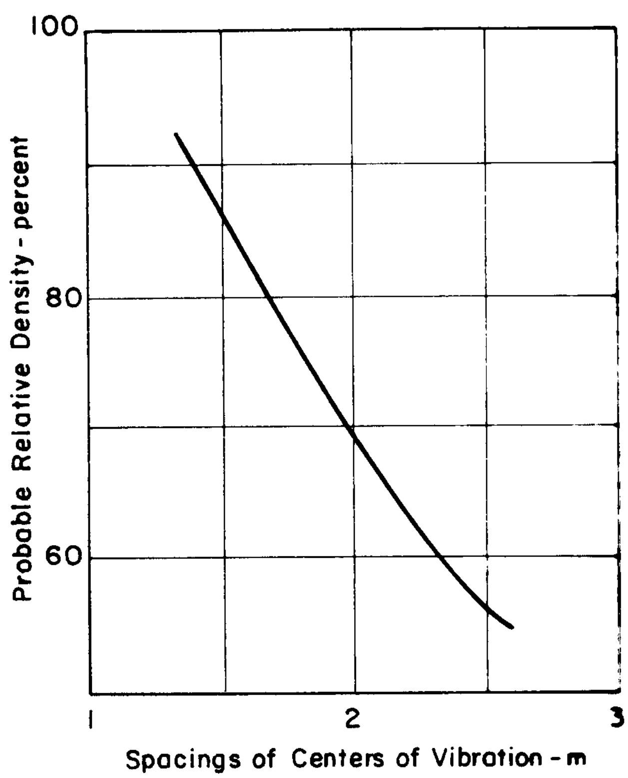


Fig. 8 Relative Density of Clean Sand at Points Midway Between Centers of Vibration as a Function of Probe Spacing (Thorburn, 1975)

clean sand at points midway between the centers of probe points and probe spacing. Any relationship such as this will depend to some extent on soil characteristics, especially particle size, water table conditions, and the amplitude and frequency of the vibrator. A relationship between allowable bearing pressure to limit settlements to 25 mm and probe spacing is given in Fig. 9. Here again it must be realized that for the same reasons such a relationship is suitable only for preliminary design.

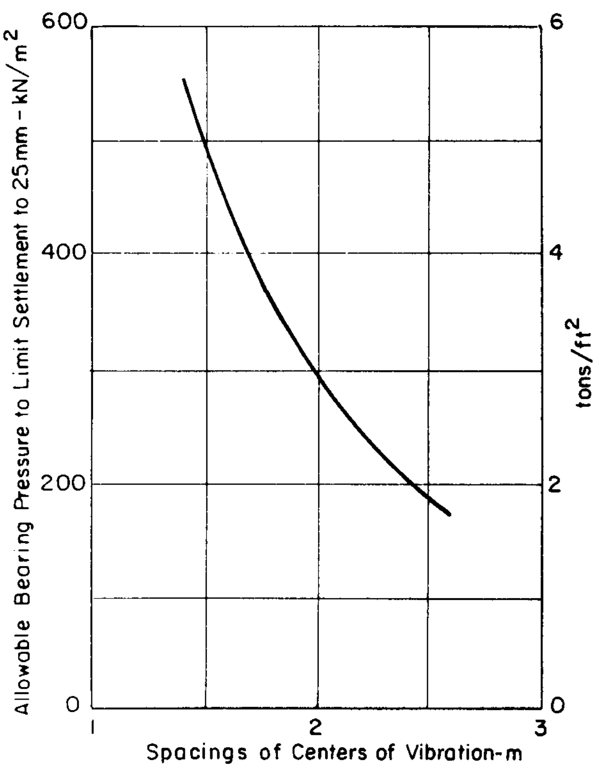


Fig. 9 Allowable Bearing Pressure as a Function of Probe Spacing for Footings Having Widths Varying from One to Three Meters (Thorburn, 1975)

Like blasting, vibrocompaction can loosen very dense zones and break up weakly cemented layers. Although the strength of such zones will therefore be decreased, they will usually still be sufficiently dense and strong for the project at hand. Also, as observed for sands densified by blasting and by heavy tamping, the penetration resistance may increase with time after treatment, even though densification and surface settlement are essentially complete at the end of treatment. Experience at the Kwinana Terminal in Western Australia (Contracting and Construction Engineer, 1974) is a good case in point. At this site 12,500 vibroflotation probes were made full depth into a sand deposit about 25 m thick. Cone penetration resistance values increased by 10 to 15 percent over the three weeks following treatment.

When coarse sands or gravels are used as backfill materials for vibrocompaction, the resulting piles can act as drains because of their high hydraulic conductivity relative to the surrounding sand. As such, they may serve to dissipate pore pressures from potentially liquefiable deposits and thereby prevent liquefaction. A method for designing gravel drain systems for this purpose is given by Seed and Booker (1977). It is important to note in such systems that the use of drains to prevent liquefaction under seismic loading does not eliminate the potential for settlement.

Heavy Tamping

Soil compaction by heavy tamping involves repeated dropping of heavy weights onto the ground surface. The method is also termed dynamic compaction, dynamic consolidation, or pounding. The technique as developed in its present form for improvement of large areas to depths of up to 30 m was pioneered by Ménard (Ménard, 1974; Ménard and Broise, 1975). When applied to partly saturated soils, the densification process is essentially the same as that for impact (Proctor) compaction in the laboratory. In the case of saturated cohesionless soils liquefaction can be induced, and the densification process is similar to that accompanying blasting and vibrocompaction. The effectiveness of the method in saturated, fine-grained soils is uncertain; both successes and failures have been reported. It would appear that in such materials a breakdown in the soil structure, the generation of excess pore water pressures, and the formation of drainage channels by fissuring may be required. Heavy tamping has been especially effective for compaction of waste and rubble fills. Among recent references which provide details of the method and case histories are Bhandari (1977), Božko et al. (1976), Charles (1981), Hansbo (1977, 1978), Krutov et al. (1978), Leonards et al. (1980), Lukas (1980), Ménard (1974), Ménard and Broise (1975), Minkov et al. (1980), Zheng et al. (1980).

The pounders used for heavy tamping may be concrete blocks, steel plates, or thick steel shells filled with concrete or sand and may range from one or two up to 200 tons in weight. Drop heights up to 40 m have been used. The pounders are usually square or circular in plan and have dimensions of up to a few meters depending on weight required, material, and the dynamic bearing capacity at the surface of the ground to be treated. More streamlined shapes have been used for underwater tamping.

For large area compaction several repetitions at points spaced several meters apart in a grid pattern are applied. A typical treatment will result in an average of 2 to 3 blows/m². An illustration of a typical grid pattern and representative equipment is shown in Fig. 10. Two or three coverages of an area may be required, separated by time intervals dependent on the rate of dissipation of excess pore water pressure and strength regain. The general response of the ground as a function of time after a coverage is shown in Fig. 11. The time interval required between coverages may range

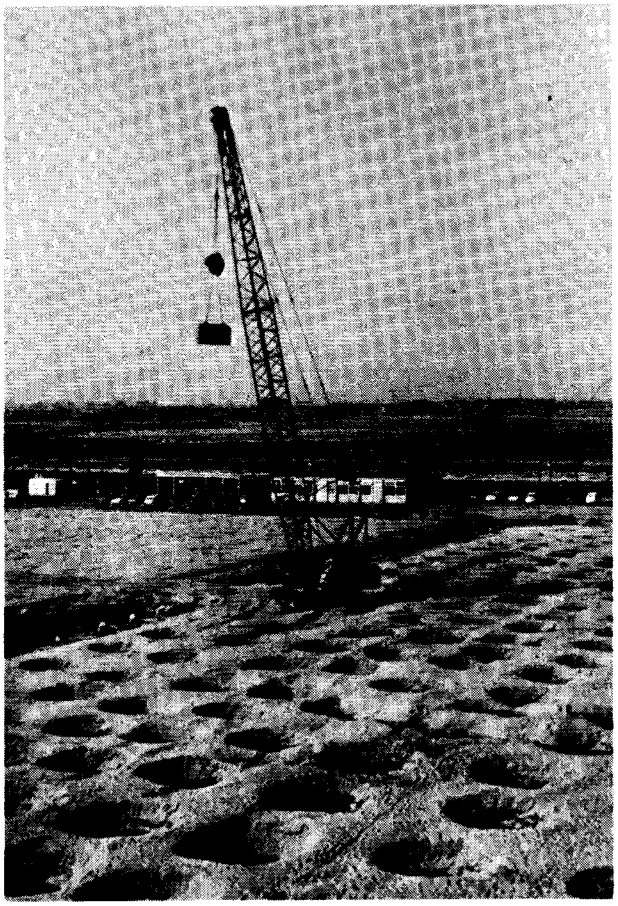
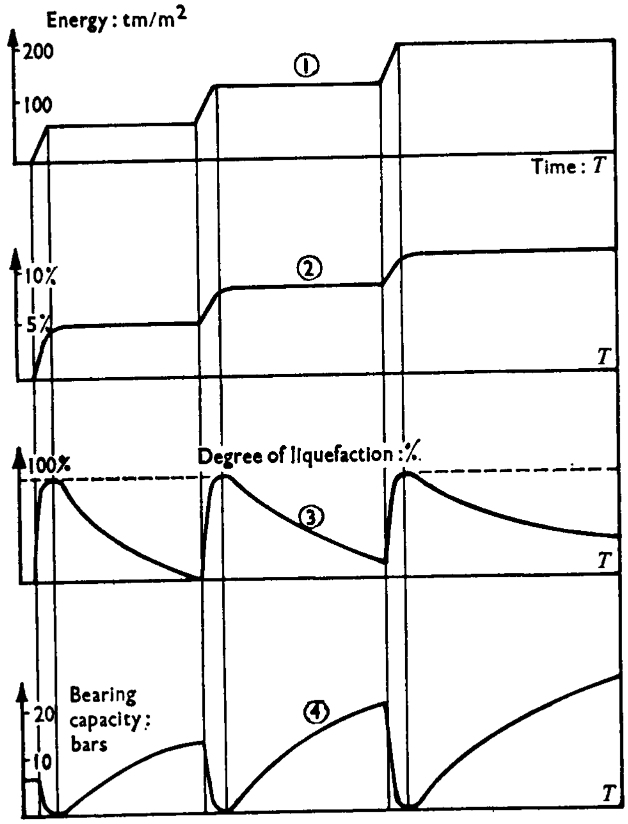


Fig. 10 Illustration of Heavy Tamping



① Applied energy in tm/m^2
② Volume variation with time
③ Ratio of pore-pressure to initial effective stress
④ Variation of bearing capacity

Time between passes varies from one to four weeks according to the soil type.

Fig. 11 Ground Response with Time After Successive Coverages of Dynamic Consolidation

(Ménard and Broise, 1975)

from days for freely draining coarse sands to weeks for finer-grained soils. The ground surface is usually levelled between coverages. To insure uniformity and high density in the near surface zone, surface "ironing" is used. Small impacts by the pounder are made over the entire surface. Surface settlements may be from two to five percent of the thickness of the zone being densified per coverage.

Zheng et al. (1980) determined the "effective deformation," defined as the volume of crater less the volume of adjacent heave from displaced soil, for successful dynamic consolidation of a soft to medium stiff clay and a uniform fine sand. In the clay the deformation was 30 percent effective, and in the sand it was 62 percent effective.

When heavy tamping is used to prepare ground for support of relatively light (low rise) structures on shallow foundations, treatment is sometimes made only at footing locations. This can be an economical and effective means for minimizing total and differential settlements.

Of particular interest when this method is being considered are the depth of influence and the level of property improvement that may be achieved. Ménard and Broise (1975) proposed that

$$D = \sqrt{WH} \tag{6}$$

where D = maximum depth of influence, m
W = falling weight, metric tons
H = height of drop, m.

Leonards et al. (1980) analyzed seven cases and concluded

$$D = 1/2\ \sqrt{WH} \tag{7}$$

was more appropriate, and Lukas (1980) concluded that

$$D = (0.65 \text{ to } 0.80)\ \sqrt{WH} \tag{8}$$

was best suited for the eight cases studied by him.

Clearly, the depth of influence should depend on factors in addition to the impact energy. Soil type might be expected to be the most important. A crane drop is less efficient than a free drop. The presence of soft layers has a damping influence on the dynamic forces. Definition of depth of influence is itself subjective and depends both on the method of measurement and the engineer's definition of what constitutes a measurable ground improvement. Within a homogeneous soil layer the amount of ground improvement decreases with depth as shown, for example, by Fig. 12.

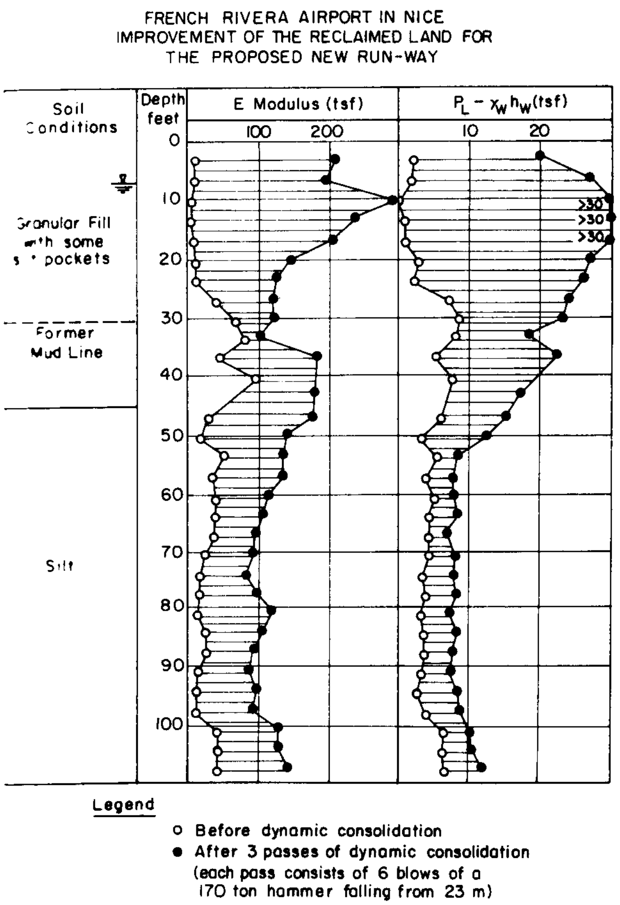


Fig. 12 Variation in Pressuremeter Modulus and Limit Pressure with Depth at Nice Airport

(Courtesy M. Gambin, Techniques Louis Menard)

number of field experiences have been summarized in Fig. 13. Because considerable judgment is required in assessing the depth of ground improvement in a number of cases and because different methods for evaluating improvement were used on different projects, the data in Fig. 13 are not precise. Nonetheless, the trend is clear. The plotted points represent the results of heavy tamping on soils ranging from silts to clean sands to rubble fills. The point referenced to Guyot and Varaksin (1980) is for a strip mined waste pile of clay, silty clay, sandy clay, and boulders of limestone and shale. From these results it appears that the use of equation (7) would provide a conservative estimate of the effective depth of dynamic compaction achieved, in most cases.

The amount of soil improvement that develops in any case depends on soil type, water conditions, and input energy per unit area. Finer-grained soils cannot be strengthened to the same level as can coarser materials. Soft layers of clay and peat inhibit high compaction of adjacent cohesionless material because of their flexibility. A review of available cases suggests that there may be a definable maximum level of improvement. Leonards et al. (1980) suggest that this level may be a cone penetration resistance of about 150 kg/cm^2. A study of the data associated with the cases plotted in Fig. 13 shows maximum values of cone penetration resistance of 180 kg/cm^2, standard penetration resistance of 45 blows/0.3 m, pressuremeter limit pressure of 3 MPa, and pressuremeter modulus of 25 MPa for clean sands. Finer-grained more compressible soils may have maximum values that are less than half of these. Values decrease from a maximum near the ground surface to the original in-situ value at depth D.

An additional concern relative to heavy tamping, and blasting as well, is whether damage may occur to facilities located beyond the edges of the area being densified because of the large impact energies. Measurements of vibration frequencies have given values in the range of 2-20 Hz. Particle velocity is related to scaled energy factor in Fig. 14. The lines for wet sand, dry sand, and clay were given by Wiss (1967) for prediction of particle velocities resulting from pile driving operations. The line for building rubble (construction debris) compaction by heavy tamping was developed by Lukas (1980), and the additional points are for heavy tamping of sand. These latter points plot near the line for rubble and suggest that a given scaled energy produces a somewhat lower particle velocity than the same energy input by pile driving. Fig. 14 may be used to estimate the distance from point of impact where damage could occur.

Progressive Liquefaction

In many cases the volume of soil densified by deep compaction lies within a potentially liquefiable deposit of much larger areal extent. The question arises then concerning whether, if in an earthquake the surrounding soil liquefies, there will be the possibility of loss of stability in the densified zone. Conceivably, the development of high pore pressures in the liquefied zone could generate higher pore pressures in the densified zone with consequent loss of strength. To guard against this possibility it should be sufficient to extend the zone of soil improvement laterally outward from the required foundation area a distance equal to the thickness of the layer being densified. More precise evaluations can be made using a computer analysis of pore pressure generation for the given conditions as described by Booker et al. (1976). Alternatively, the method proposed by

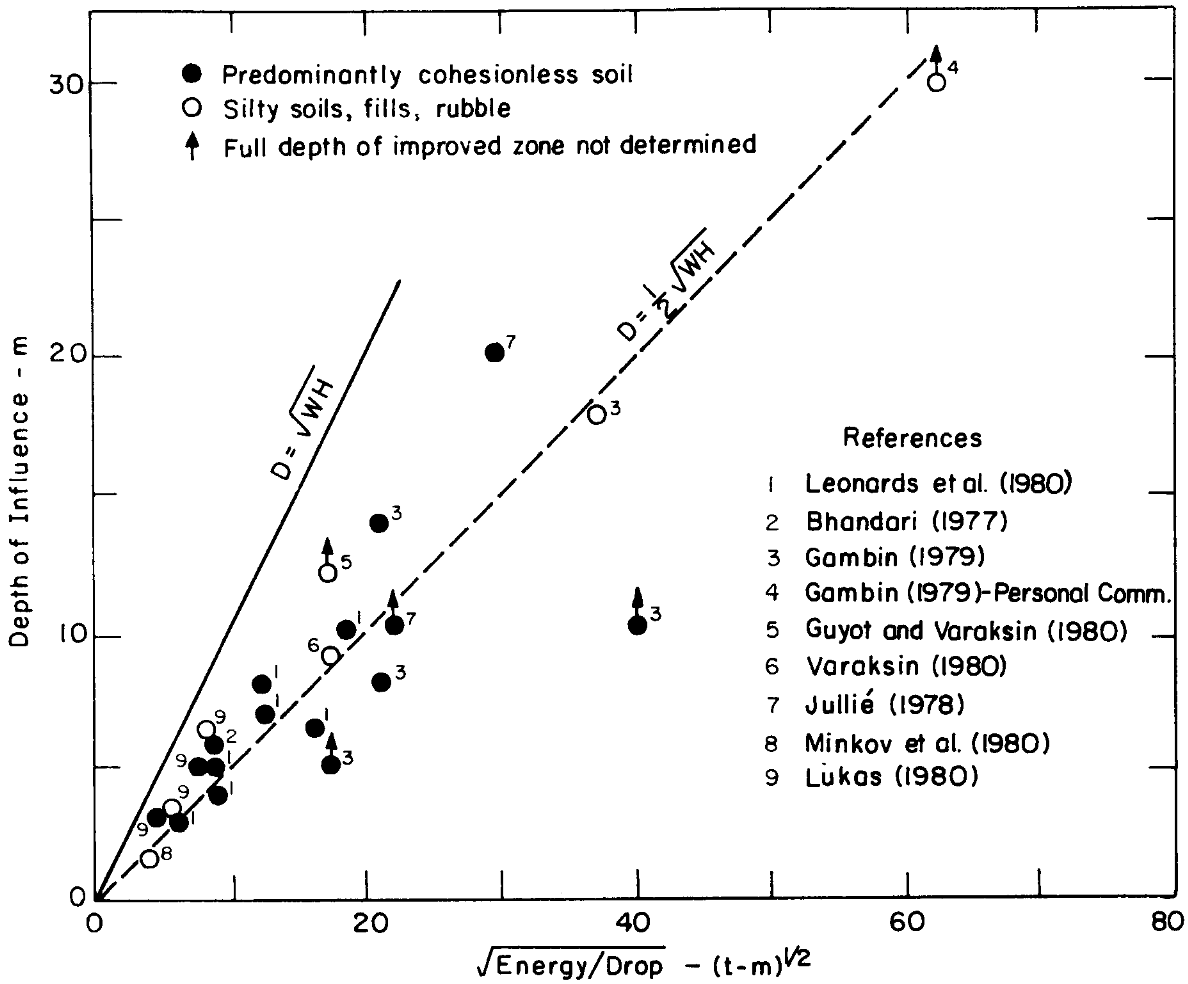


Fig. 13 Depth of Influence as a Function of Impact Energy for Heavy Tamping

Nandakumaran et al. (1977), which appears quite conservative, might be considered.

Time Effects

More and more evidence is becoming available to indicate that in many sands time-dependent increases in strength and decreases in compressibility develop after densification by any of the deep compaction methods. Because these effects continue over periods of many weeks or months, they cannot be explained in terms of pore pressure dissipation, which continues only for periods of several minutes at the most in the case of clean sand. The aging effect has been shown to give substantial increase in the strength of sands under cyclic loading, as may be seen in Fig. 15.

Although a number of hypotheses have been advanced to explain this behavior; e.g., thixotropic hardening, chemical cementation, the effects of dissolved gases, the mechanism is not yet completely clear. From a practical standpoint, however, it would be reasonable to conclude that evaluations of the ground shortly after the completion of deep densification will give conservative results.

SOIL IMPROVEMENT BY PRECOMPRESSION

Introduction

The strengthening and preconsolidation of weak and compressible soils by preloading prior to construction is one of the oldest and most widely used methods for soil improvement. It is particularly well-suited for use with soils that undergo large volume decreases and strength increases under sustained static loads and when there is sufficient time available for the required compressions to occur. Surcharge loads; i.e., loads in excess of those to be applied by a permanent fill or structure can be used to accelerate the process. When the anticipated time for compression is excessive, vertical drains may be used to shorten the time required provided the compression is of the primary consolidation type. The soil types best suited for

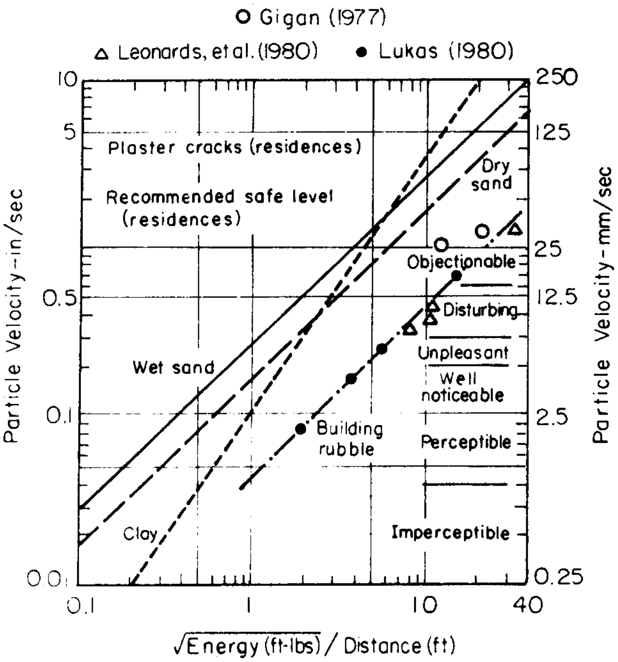


Fig. 14 Particle Velocity as a Function of Scaled Energy Factor

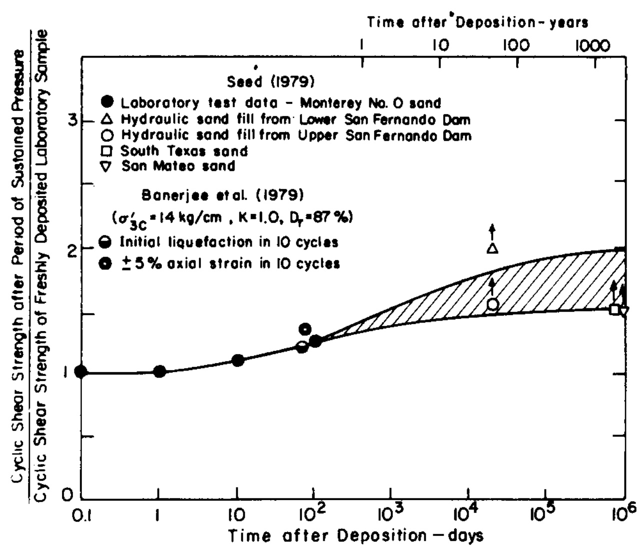


Fig. 15 Influence of Period of Sustained Pressure on Stress Ratio Causing Peak Cyclic Pore Pressure Ratio of 100%

improvement by precompression are saturated soft clays, compressible silts, organic clays, and peats. Vertical drains are of greatest effectiveness in inorganic clays and silts that exhibit little secondary compression. Preloading and precompression have been used successfully to improve the foundation soils for buildings, embankments, highways, runways, tanks, and abutments.

Several recent comprehensive treatments of precompression and topics related to its application, with and without drains, are available, including those by Johnson (1970a, 1970b), Bjerrum (1972), U.S. Navy (1971), Pilot (1977), Schlosser and Juran (1979), Akagi (1977, 1979), Hansbo (1979). The state-of-the-art by Pilot (1977) is especially complete and well-documented and contains an extensive list of case histories. Because of these references, detailed treatment of all topics important to precompression is not given here. Emphasis is on recent developments and practical considerations that influence design and performance of precompression systems.

Types of Preloads

Although earth fills are the most commonly used type of preload, any system that leads to drainage of pore water and compression of the soil may be suitable. Water in tanks has been used to preload small areas, and water in lined ponds can be used to preload larger areas. Vacuum preloading by pumping from beneath an impervious membrane placed over the ground surface can produce surcharge loads up to 60 to 80 kPa (Holtz and Wager, 1975; Pilot, 1977). Anchor and jack systems can be devised for special cases. Groundwater lowering provides an increase in consolidation pressure equal to the unit weight of water times the drawdown distance. Consolidation by electro-osmosis is the same in many respects as consolidation under externally applied stresses, except that the driving force for drainage is induced internally by an electrical field.

Preloading by vacuum, water table lowering, and electro-osmosis offer the advantages that there are no stability problems, and large volumes of surcharge fill are not required. On the other hand, they are more complex in execution than the other methods.

Theoretical Basis for Design

The usual objective of design of surcharge loads and duration of their application is to reduce the magnitude of settlement after construction. Settlement at any time may be expressed as

$$s_t = s_i + \bar{U}s_{cons} + s_s \qquad (9)$$

in which s_t = settlement at time t

s_i = immediate settlement (from gas compression, shear distortion)

$\bar{U}$ = average degree of consolidation

s_{cons} = final consolidation settlement

s_s = secondary compression settlement.

The objective may be either: (1) to determine the magnitude of surcharge pressure (p_s) required to insure that the total settlement anticipated under the final pressure (p_f) will be complete in a given length of time, or (2) to determine the length of time required to achieve a given amount of settlement under a given surcharge load.

It is convenient to consider primary consolidation and secondary compression as separate phases, even though both types of volume change may be in progress concurrently. If the time of loading is short relative to the total consolidation period, then the assumption of instantaneous application of the total load at the end of one half the loading period will not lead to serious error. If not, then more exact analyses may be made.

Consolidation settlement estimates for permanent fill and structure loads are made in the usual way. In those cases where soil properties and/or stress conditions vary with depth, it may be necessary to analyze the profile as a series of sublayers.

The time rate of settlement for primary consolidation in one dimension may be determined using the classical Terzaghi theory. In those cases where the strain profile with depth is not essentially constant, as assumed in the Terzaghi solution, a non-linear stress-strain method, such as that presented by Duncan and Buchignani (1976) based on Janbu (1965) may give better results. The effect of decreasing vertical strains with depth is to increase the rate of settlement compared to the Terzaghi solution. The stress vs. time and settlement vs. time relationships for a soil layer under stress increase p_f would be as shown in Fig. 16 in the absence of secondary compression.

If a surcharge load that applied an additional stress p_s were also used and left in place

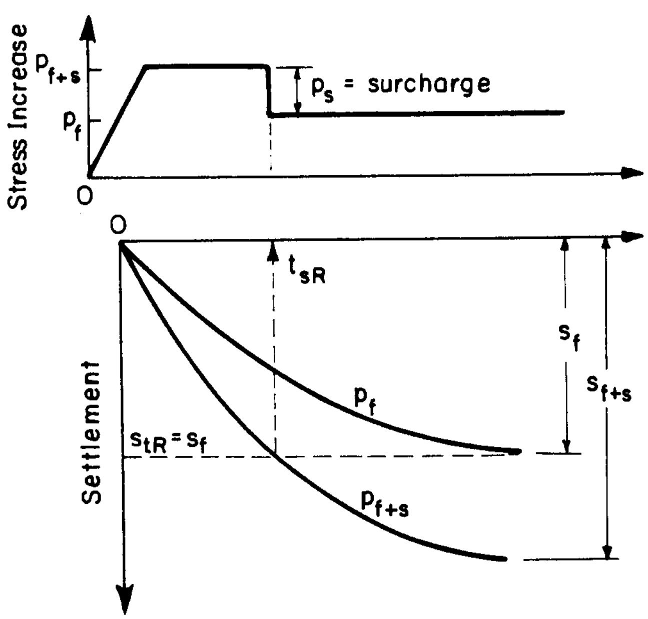


Fig. 16 Compensation for Primary Compression Using Surcharge Loading

indefinitely, then a time-settlement curve could be obtained in the same way and plotted as shown by the lower curve in Fig. 16. If the surcharge were left in place until time t_{SR}, then the layer will have settled an amount equal to that to be expected under the permanent fill alone; i.e., $s_{tR} = s_f$. At this time the layer will have reached an average degree of consolidation $\bar{U}_{SR}$ given by

$$\bar{U}_{SR} = \frac{s_f}{s_{f+s}} \tag{10}$$

As pointed out by Aldrich (1965) and extended by Johnson (1970a) the distribution of effective and excess pore water pressures before and after surcharge removal will be as shown in Fig. 17 for a clay layer drained at both boundaries. Thus, a substantial portion of the layer may undergo further consolidation after removal of the surcharge load, whereas the remainder will be unloaded. Although the unloading of the zones near drainage boundaries will generally not lead to significant heave, the additional consolidation in the central portion may be important.

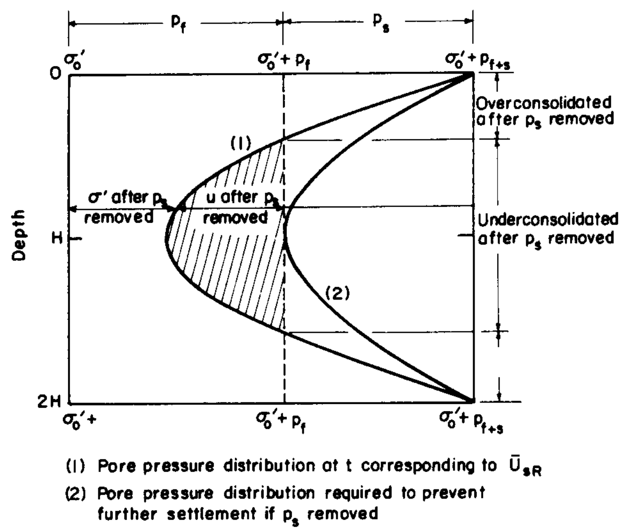


Fig. 17 Pore Pressure Distribution after Removal of Surcharge Load at Time t_{SR}

To eliminate further primary consolidation following the removal of the surcharge, the surcharge should be left in place until the pore pressure at the most critical point; i.e., the point that is last to consolidate, has itself reached a consolidation ratio U_z given by

$$(U_z)_{f+s} = \frac{p_f}{p_f + p_s} \tag{11}$$

Times corresponding to $\bar{U}_{SR}$ and U_z are found using the Terzaghi theory and the coefficient of consolidation c_v according to

$$t = \frac{TH^2}{c_v} \tag{12}$$

where T is the appropriate value of dimensionless time factor.

Because this approach requires that the surcharge be left in place until all points are fully consolidated under p_f, overconsolidation will develop in most of the soil layer. Thus it is conservative, and the actual precompression settlement will be significantly greater than s_f. A further complicating factor arises in that equation (11) is stress-based; whereas c_v is usually determined using a strain-based procedure. As the stress-strain behavior of the soil is non-linear, there is some uncertainty in the predicted times and surcharge loads required. This problem is presently under study at the University of California, Berkeley, and preliminary indications are that in its present form the method gives conservative results; i.e., higher values of p_s or time of surcharging are predicted than actually required.

Secondary compression may represent a very significant portion or the total compression of some soils, especially organic clays and peats. Precompression using surcharge loadings may be effective for minimizing the effects of subsequent secondary compression under permanent loads. The concept is to estimate the total settlement under p_f as the sum of that due to primary consolidation and that due to secondary compression, s_{sec}, anticipated to occur during the life of the structure. s_{sec} is determined according to

$$s_{sec} = C_\alpha H_p \log(t/t_p) \qquad (t > t_p) \tag{13}$$

in which C_α is the vertical strain per log cycle increase in time subsequent to the end of primary consolidation at t_p, and H_p is the layer thickness at time t_p. The analogous equation to (11) for the critical point for this case is

$$(U_z)_{f+s} = \frac{s_f + C_\alpha H_p \log(t/t_p)}{s_{f+s}} \tag{14}$$

The nature of secondary compression is such that some time after surcharge p_s is removed, secondary compression will reappear under p_f. This effect is small, however, and can usually be neglected (Johnson, 1970a).

The increase in undrained shear strength which accompanies preloading may be the most important effect in many cases. The undrained strength after a given duration of preloading may be estimated using the principles of the SHANSEP procedure (Ladd and Foott, 1974) together with an analysis to estimate the increase in effective consolidation pressures within the preloaded layers.

Vertical Drains

In many cases the time required for surcharging is excessive, the surcharge required for the time available is too great, or the rate of strength gain is too slow to permit rapid fill placement. For soils whose compression is dominated by primary consolidation, vertical drains may be used to accelerate the rate of settlement, because consolidation times vary as the square of the drainage path length, and because most deposits have greater permeability in the horizontal than in the vertical direction. A schematic diagram of a typical vertical drain installation is shown in Fig. 18. Vertical drains are ineffective in peats, organic clays, and other soils whose settlement behavior is dominated by secondary compression.

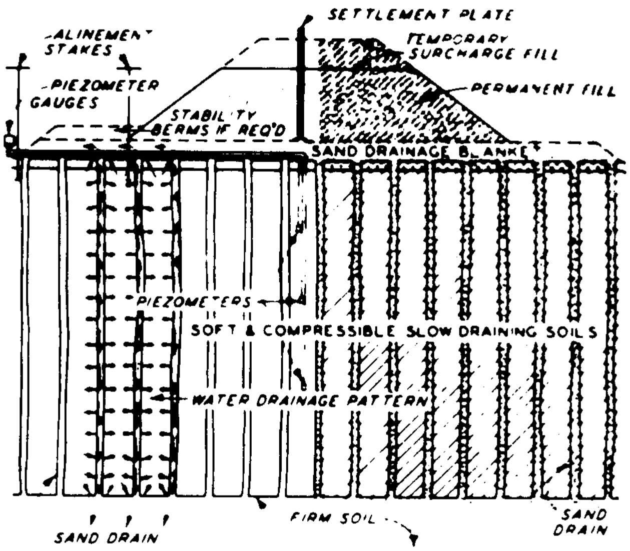
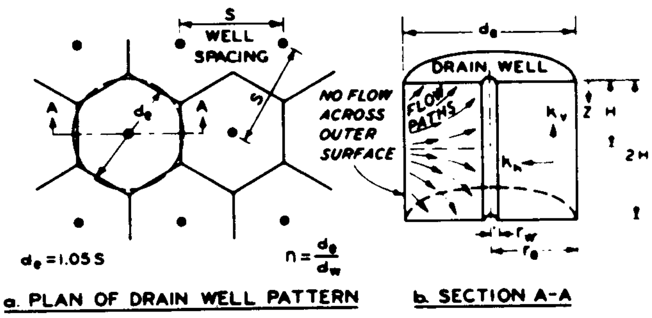


Fig. 18 Typical Vertical Drain Installation

The theory for consolidation by radial drainage and by combined radial and vertical drainage is well developed (Barron, 1948; Carillo, 1942). The results are summarized in Figs. 19 and 20. It is important to note that drain spacing is a far more significant parameter in the determination of consolidation time than is drain diameter. Olson et al. (1974) have developed a finite difference computer program that can be used to investigate the effects of time-dependent loading, variable consolidation coefficients, non-linear soil stress-strain behavior, the influences of large strains, and the installation of drains after one-dimensional settlement has begun.

Until a few years ago vertical drains of sand, typically 200 to 500 mm in diameter and spaced anywhere from 1.5 to 6.0 m on centers, were widely used. Installation was accomplished using a variety of techniques of both the displacement and non-displacement type. Displacement drains, while generally less expensive and faster to



VERTICAL CONSOLIDATION:

$$C_{v-v} = \frac{k_v(1+e_0)}{a_{v-v}\gamma_w} = \frac{H^2 T_v}{t} \quad \text{OR} \quad t = \frac{T_v H^2}{C_{v-v}}$$

RADIAL CONSOLIDATION:

$$C_{v-h} = \frac{k_h(1+e_0)}{a_{v-v}\gamma_w} = \frac{d_e^2 T_h}{t}, \quad \text{OR} \quad t = \frac{T_h d_e^2}{C_{v-h}}$$

COMBINED RADIAL AND VERTICAL FLOW:

AT ANY TIME:

EXCESS PORE WATER PRESSURE RATIOS

$$\left(\frac{u}{u_0}\right)_{v-h} = \left(\frac{u}{u_0}\right)_v \cdot \left(\frac{u}{u_0}\right)_h \quad \text{AT A POINT}$$

$$\left(\frac{\bar{u}}{u_0}\right)_{v-h} = \left(\frac{\bar{u}}{u_0}\right)_v \cdot \left(\frac{\bar{u}}{u_0}\right)_r \quad \text{AVERAGE VALUES}$$

DEGREE OF CONSOLIDATION

$$U = 1 - \frac{u}{u_0} \quad \text{AT A POINT}$$

$$\bar{U} = 1 - \frac{\bar{u}}{u_0} \quad \text{AVERAGE VALUE}$$

Fig. 19 Summary of Consolidation Theory for Vertical Drain Design

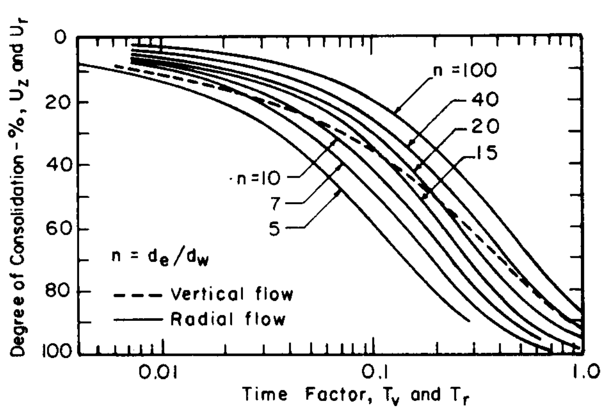


Fig. 20 Consolidation Solution for Radial Flow and Equal Vertical Strain at Ground Surface (Barron, 1948)

install, can disturb the surrounding soils. The resulting "smear" zone can impede drainage, and the disturbed soil may be weakened. These effects may not be as detrimental as earlier believed, however, owing to the possibilities of (1) reconsolidation to a higher strength than the original, and (2) the opening of cracks and fissures that fill with sand during installation and thereby increase the effective drainage area. Akagi (1977, 1979) concluded that reliable data are lacking to establish whether non-displacement drains are indeed more effective than the displacement type.

Present indications are that conventional sand drains installed for the acceleration of consolidation may soon be things of the past, as a variety of prefabricated drains are coming into wide use. Band-shaped drains of the order of 100 mm wide by 1 to 7 mm thick are produced by several manufacturers. These drains can be rapidly installed to depths up to 50 m by machines with special mandrels. Drain spacings of the order of 1 m are typical. Both dynamic and static methods of installation are used. Circular prefabricated drains or wicks composed of sand within cylindrical fabric containers are also used. A detailed discussion of these new drains, their specific features, and methods of installation is beyond the scope of this report. Hansbo (1979) has presented a comprehensive overview of consolidation using prefabricated band-shaped drains. Only a few general characteristics are summarized here.

Kjellman's cardboard wick, introduced in 1937, was the first of the prefabricated drains. Presently there are several types on the market under such names as Alidrain, Geodrain, Castle Board, Colbond, Mebradrain, and PVC Drain. The drains usually consist of a core of plastic and a filter sleeve of paper, fibrous material, or porous plastic. The cross section design provides for a system of vertical channels for water flow. The Colbond Drain differs from the others in that it is a non-woven fabric throughout, and it is significantly wider (30 cm) than the others. The mandrels used for drain installation are of cross section considerably larger than the drain itself. Fig. 21 is an example. Thus installation of the drain necessarily causes some disturbance of the surrounding ground.

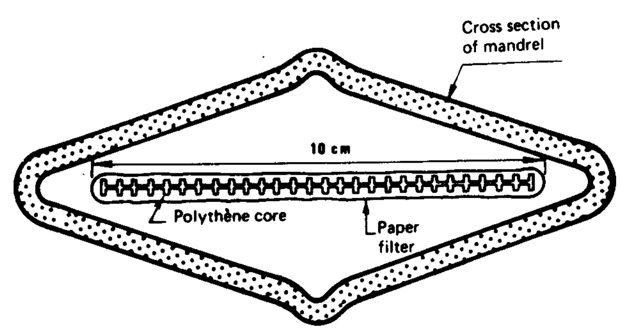


Fig. 21 Cross Section of a Plastic Drain and Mandrel

The filter sleeve surrounding the plastic core must satisfy several criteria:

1. Its permeability should not be significantly less than that of the surrounding soil.
2. Fine soil particles should be retained to prevent clogging of flow channels in the core.
3. The filter should be stiff enough so as not to be pressed into core channels by high lateral soil pressures and strong enough not to be damaged during installation.
4. The filter should not undergo physical, chemical, or biological deterioration during the intended life of the drain.

Theoretical determination of required drain spacing can be made in the same way as for sand drains; i.e., with the aid of Figs. 19 and 20. Alternatively, if consolidation due to vertical flow is negligible compared to that due to radial flow, which is often the case, then the time of consolidation t can be determined according to

$$t = \frac{d_e^{\,2}\mu}{8c_{v-h}} \ln \frac{1}{1 - \bar{U}_h} \qquad (15)$$

where $\bar{U}_h$ is the average degree of consolidation. The parameter μ is given, to a good approximation, in terms of $n(=d_e/d_w)$ by

$$\mu \simeq \ln(n) - 0.75 \qquad (16)$$

for the values of n (>12, except for Colbond, 8) and drain spacing (>0.8 m) used in practice. The equivalent diameter d_w of a band-shaped drain of width b and thickness t is taken as

$$d_w = \frac{2(b + t)}{\pi} \qquad (17)$$

Both well resistance and disturbance during installation may cause the actual times for consolidation to be greater than predicted by Fig. 20 or Eq. (15). Both of these considerations are discussed by Hansbo (1979).

Prefabricated band-shaped drains can be installed at orientations other than vertical, which enables their use for special applications such as under-drainage and on slopes. Because they can tolerate significant displacements without rupture, prefabricated drains are not as susceptible to loss of effectiveness due to shear displacements as can occur in the case of sand drains.

Some Practical Considerations

For precompression without vertical drains the rate of consolidation is controlled by the coefficient of consolidation for vertical flow, c_{v-v}. Both laboratory and field tests may be used for its determination, and there is extensive literature on the methods, their accuracy and limitations.

Settlement predictions for two- and three-dimensional deformations and drainage conditions are more complex than the one-dimensional procedure described herein. Computer analyses can be developed for special cases. Some solutions are available for determination of approximate or limiting cases.

If the compressible layer thickness H is large relative to the width of the loaded area B, then there will be an immediate settlement s_i in Equation (9). Its magnitude can be estimated using elastic theory. The consolidation settlement, s_{cons} in these cases may be reduced depending on the magnitude of the pore pressure coefficient $\bar{A}$. Estimation of consolidation settlements according to the procedure of Skempton and Bjerrum (1957) may be appropriate. Lateral drainage becomes important in accelerating the rate of consolidation when the value of H/B exceeds 0.25 to 1.0, depending on the shape of loaded area and whether the compressible layer is singly or doubly drained. The rate of settlement will be even greater if the horizontal permeability, and therefore c_{v-h}, is greater than the vertical permeability and c_{v-v}. A quantitative estimate of both of these effects can be made using the solutions developed by Davis and Poulos (1972).

When vertical drains are used, the coefficient of vertical consolidation due to horizontal flow, c_{v-h}, controls the rate of consolidation. Both special laboratory tests on large samples, e.g., Hansbo (1960), Rowe (1964), Berry and Wilkinson (1969), and Paute (1973), and in-situ tests, summarized by Mitchell and Gardner (1975), have been used to determine c_{v-h}. A major difficulty in laboratory testing for determination of c_{v-h} is that very large samples are required in many cases if results representative of the true in-situ stratification are to be obtained. An approach that has yielded reasonable results is to measure k_h in the field and compute c_{v-h} using a compressibility value determined by conventional laboratory tests. A typical condition is that c_{v-h}/c_{v-v} = 2 to 10.

Precompression by Electro-osmosis

Because water can be made to flow through fine-grained soils from anode to cathode in a direct current electrical field; i.e., by electro-osmosis, consolidation will result if water is removed at the cathode but not replaced at the anode. For certain soil conditions and limited soil volumes, electro-osmosis may be an economical and effective means for consolidation. If, at the same time as water is being removed at the cathode, stabilizing chemicals are injected at the anode, soil improvement by electro-kinetic injection can be achieved. Electro-kinetic injection is discussed in a later section. The mechanism of electro-osmosis has been elaborated by Gray and Mitchell (1967). The theory for consolidation by electro-osmosis has been developed by Esrig (1968), Wan and Mitchell (1976), and Mitchell and Wan (1977). Recent case histories are summarized by Pilot (1977).

The water flow rate, q_h, in a one-dimensional direct current field is initially

$$q_h = k_e \, i_e \, A \qquad (m^3/sec) \qquad (18)$$

where $k_e \simeq 1 \times 10^{-9}$ to 7×10^{-9} m/sec per volt/m

i_e = electrical potential gradient (volt/m)

A = cross-sectional area (m^2)

Alternatively, the flow rate can be expressed by

$$q_h = k_i \; I \quad (m^3/sec) \tag{19}$$

where k_i = water flow per unit time per ampere (m^3/sec/amp)

I = current (amps)

The coefficients k_e and k_i are related by the specific electrical conductivity, σ,

$$k_i = k_e/\sigma \tag{20}$$

Values of σ range from about 0.02 mho/m for low salt content soils to 0.30 mho/m for high salt content soils. Values of k_i for water contents in the range of 50 to 100 percent are given in Table III. The power consumption P is given by

$$P = q_h \; \Delta V/k_i \times 10^{-3} \quad (kwh) \tag{21}$$

where ΔV is the voltage drop.

TABLE III

Values of Electro-Osmotic Water Transport Coefficient (Water content range 50-100%)

Soil Type	Pore Water Salt Concentration (N)	k_i m^3/sec/amp
Silty clay, kaolinite	10^{-3}	$1x10^{-5}-5x10^{-7}$
Silty clay, kaolinite	10^{-2}	$5x10^{-8}-1x10^{-7}$
Clay (illitic)	10^{-3}	$3x10^{-8}-6x10^{-8}$
Clay (illitic)	10^{-2}	$2x10^{-8}-3x10^{-8}$

During consolidation, the water flow rate decreases with time. It ceases when a hydraulic gradient, caused by a decrease in pore water pressure at the anode relative to that at the cathode, causing flow from cathode towards anode, exactly balances the electrically induced hydraulic gradient causing flow from anode towards cathode. At this condition the increase in effective stress, $\Delta\sigma'$, from that at the start of treatment is

$$\Delta\sigma' = (k_e/k_h) \; \gamma_w V \tag{22}$$

where k_h = the hydraulic conductivity

γ_w = the unit weight of water

V = voltage (a function of position)

The amount of consolidation associated with this effective stress increase is obtained from a void ratio vs. effective pressure relationship for the soil determined in the usual manner. Strength increases, in the absence of electrochemical hardening effects, can be estimated in the same way.

The rate of consolidation is governed by the same relationships that apply to consolidation under directly applied loading. The time t for a given degree of consolidation is

$$t = \frac{TL^2}{c_v} \tag{23}$$

where T = time factor for the specified degree of consolidation and boundary conditions

L = electrode spacing

c_v = coefficient of consolidation.

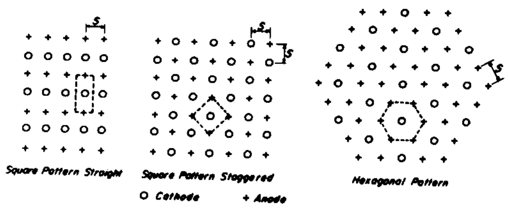
Values of T for different degrees of consolidation for the case of parallel plate electrodes with a linear variation in voltage between them are given in Table IV. Measurements by Johnston and Butterfield (1977) indicate that rather than a linear variation in voltage between electrodes, an instantaneously infinite electrical gradient develops at the anode initially which decreases in a consistent way, to a uniform gradient at the completion of consolidation. Values of T for these conditions are also listed in Table IV. It may be seen that consolidation occurs more rapidly for the latter case.

TABLE IV

Time Factor for Various Degrees of Consolidation by Electro-osmosis Between Parallel Plate Electrodes

Degree of Consolidation	Time Factor, T	
$\bar{U}$ (%)	Linear V Variation	Infinite Initial Gradient
0	0	0
10	0.05	0.001
20	0.10	0.007
30	0.16	0.017
40	0.22	0.020
50	0.29	0.05
60	0.38	0.07
70	0.50	0.10
80	0.66	0.14
90	0.95	0.20

Actual electrode arrangements are an array of rods or pipes, spaced typically at 2 to 4 meters in patterns, rather than parallel flat plates. In addition, variations in properties, especially in the ratio k_e/k_h, that develop during consolidation lead to deviations from the theory (Mitchell and Wan, 1977). Thus the values in Table IV can be used only as an approximation for prediction of consolidation time. Analysis of the electrical flows for different electrode arrangements, Fig. 22, shows that a hexagonal arrangement is efficient in terms of (1) power consumption, (2) average voltage (the higher the average voltage the greater the average amount of consolidation that can be obtained for a given applied voltage), and (3) anode to cathode ratio.



	Square Pattern Straight	Square Pattern Staggered	Hexagonal Pattern
Area/Basic Unit	$2S^2$	$2S^2$	$2.6S^2$
Anodes/Basic Unit	1	1	2
Cathodes/Unit Area	$0.5/S^2$	$0.5/S^2$	$0.38/S^2$
Anodes/Unit Area	$0.5/S^2$	$0.5/S^2$	$0.77/S^2$
Average Voltage, $\bar{V}$	$0.5V$	$0.5V$	$0.74V$
Power/Unit Area (for same V)	$0.53V^2/\rho S^2$	$0.60V^2/\rho S^2$	$0.60V^2/\rho S^2$
Power/Unit Area (for same $\bar{V}$)	$2.10\bar{V}^2/\rho S^2$	$2.40\bar{V}^2/\rho S^2$	$1.10\bar{V}^2/\rho S^2$

V = Applied Voltage ρ = Resistivity

Fig. 22 Characteristics of Different Electrode Patterns in Electro-osmosis

Increasing the number of anodes relative to cathodes is generally beneficial for two reasons. Well points are often used for cathodes, whereas, reinforcing bars or aluminum rods are used as anodes. Hence, anodes cost less. Iron and aluminum anodes decompose during treatment and participate in electro-chemical hardening reactions that give strength increases beyond those attributable solely to consolidation.

In application it is known that electro-osmosis may be effective and economical under the following conditions:

1. Saturated silts or silty clay soils
2. Normally consolidated conditions
3. Low pore water electrolyte concentration.

Gas generation and drying and fissuring at the electrodes can impair the efficiency of the method and limit the magnitude of consolidation pressures that develop. Treatment results in non-uniform changes in properties between electrodes, because the induced consolidation depends on the voltage, and the voltage varies between anode and cathode. Thus reversal of electrode polarity may be desirable to achieve a more uniform stress condition. Electro-osmosis may also be used to accelerate the consolidation under a preload or surcharge fill. Analysis methods for these conditions are presented by Wan and Mitchell (1976).

Recent Developments

Several recent studies have been directed at the development of new approaches to and applications for precompression.

Some studies have been made (Leong, 1977) of consolidation induced by inflatable cylindrical membranes in vertical boreholes. One can imagine, for example, a hexagonal pattern of expandable membranes in lieu of the anodes in Fig. 22 which induce an excess pressure in the surrounding soil and, hence, water flow towards drain wells at positions shown by cathodes in the figure. The magnitude of consolidation that can be produced is dependent on the excess pore pressure that can be induced, which, in turn, depends on the inflatable membrane diameter, hole spacing and configuration, soil strength, and soil stress-strain properties. Recent studies of consolidation using both vertical and radial cyclic loads have been summarized by Schlosser and Juran (1979).

Densely packed quicklime piles in soft, saturated clay have been used. Four effects are beneficial:

1. Water is taken from the soil to hydrate the lime.
2. Hydration of quicklime is accompanied by a specific gravity decrease from 3.3 to 2.2. This leads to expansion against the borehole and development of pore pressure in the native soil. Recent experiments by Kuroda et al. (1980) showed that expansion pressures in excess of 2 MPa could be developed for conditions of low porosity and restrained expansion.
3. The heat of hydration helps to further reduce the water content.
4. Slow diffusion of lime into the surrounding soil leads to cementation and ground strengthening.

The need for densification of fine-grained slurries, sludges, and slimes has become a major concern in the containment and storage of disposal materials and the subsequent utilization of disposal sites. Dewatering and densification of these materials by special drainage and consolidation systems can be among the most effective techniques available (Johnson et al., 1977). Seepage consolidation; i.e., surface ponding without surface membranes but with underdrainage, can be especially useful because of the greater effective stresses that can be developed. Prefabricated drains can be utilized effectively both within and beneath the materials to be densified. Because of the large volume changes and strains involved, new consolidation theories have been developed for analysis of these materials. The work of Imai (1978) and Somogyi et al. (1981) is useful in this regard.

Finally, precompression with vertical drains is now being used in conjunction with other methods of ground improvement and reinforcement. Two recent examples may be cited. At the Changi Airport in Singapore a loose sand fill was placed over a thick layer of soft clay.

Prefabricated drains were installed to a depth of 43 m to accelerate consolidation of the clay. Heavy tamping was then used to densify the sand fill (Hansbo, 1978).

At the site of the east approach embankment to the Dumbarton Bridge currently under construction across the southern end of the San Francisco Bay, the soft mud foundation soil has a shear strength less than 5 kPa. Consolidation under the proposed embankment would have taken 30 to 40 years. Special measures were therefore required to provide embankment support and to accelerate the anticipated settlement of 2 m. The solution, shown in Fig. 23, consisted of (1) a geotextile reinforcement to distribute embankment loading and reduce differential settlement, (2) lightweight fill (sawdust) to reduce loading, (3) prefabricated vertical drains to reduce the consolidation time to less than one year, and (4) a geotextile filter to prevent contamination of a drainage blanket.

INJECTION AND GROUTING

Introduction

In 1802 French engineer Charles Bériguy repaired a scouring sluice at Dieppe by injecting a grout of clay and hydraulic lime beneath it. Since then injection of materials into the ground has developed into a widely used method for soil stabilization and ground improvement. Because of its high cost, grouting is usually limited to zones of relatively small volume and to special problems that cannot be solved by other methods. Most of the early applications of grouting were for groundwater control or shut off, and these continue to be very important applications today. More recently injections have been used for ground strengthening and ground movement control, and it is these applications that are of primary concern in this report.

Three modes of injection are possible, as shown in Fig. 24:

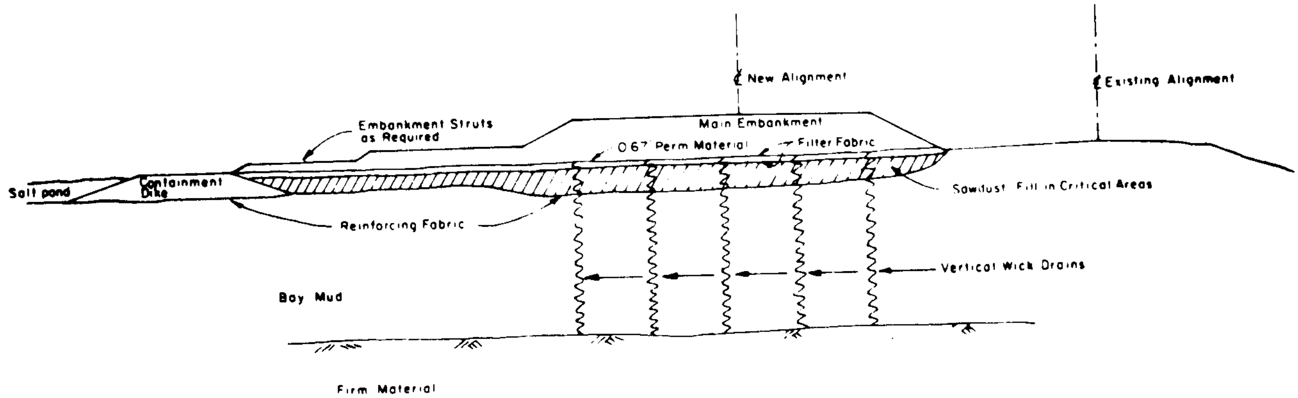


Fig. 23 Generalized Cross Section Showing Stabilization Measures at the Site of the Dumbarton Bridge Approach Fill (Hannon, 1980)

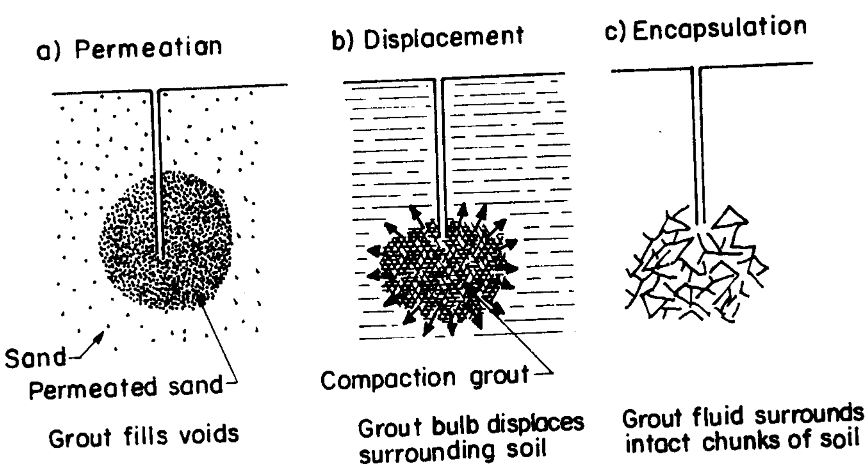


Fig. 24 Types of Grouting

1. Permeation grouting in which the grout fills soil pores. There is essentially no change in the volume or structure of the original ground. This type of grouting can generally only be accomplished in soils coarser than fine sands and in fissured rocks.
2. Displacement grouting in which a stiff mixture fills voids and compresses the surrounding soil.
3. Encapsulation in which naturally fragmented ground or ground fractured hydraulically under high grout fluid pressures is injected by grout which coats but does not permeate the individual chunks of soil. A lens structure in the form of a cardhouse is formed.

Recent references that present comprehensive reviews of injection and grouting include ASCE (1980), Bowen (1975), Cambefort (1973), Caron et al. (1975), Herndon and Lenahan (1976), Kirsch and Samol (1978), and Lenzini and Bruss (1975). Although successful grouting may be complicated, it is more than an art. Some underlying concepts have been defined, and principles guiding its success have been established. It is these that are the focus of this section.

Applications

A number of applications of grouting for soil improvement apart from seepage control may be noted, including:

1. Void filling to prevent excessive settlement.
2. Ground strengthening under existing structures to prevent movements during adjacent excavation, pile driving, etc.
3. Ground movement control during tunneling operations. Tan and Clough (1980) present a design method for determining the required size and strength of stabilized zones around tunnels for effective ground movement control.
4. Soil strengthening to reduce lateral support requirements.
5. Soil strengthening to increase the lateral load resistance of piles.
6. Stabilization of loose sands against liquefaction.
7. Foundation underpinning.
8. Slope stabilization.
9. Volume change control of expansive soils through pressure injection of lime slurry. This technique is controversial and likely to be effective only under special conditions (Ingles and Neil, 1970; Wright, 1973; Blacklock, 1975; Thompson and Robnett, 1976).

Grout Types

Permeation grouts are of two types. Particulate grouts are made up of cement, soil, or clay and mixtures of these. Chemical grouts are composed of various materials in solution. Displacement or compaction grouts are stiff, low slump (0 to 50 mm) mixtures of cement, soil, and/or clay and water. Lime slurries are the most commonly used encapsulation-type grouts; however, there is no inherent reason, except perhaps for economics, why other chemicals could not be used.

Particulate Grouts

Neat cement and soil-cement grouts are the most commonly used particulate grouts, although soil-water grouts have been used in some cases. In water-cement grouts water:cement ratios of 0.5:1 to 6:1 have been used. With low water:cement ratios there is less segregation and filtering, and higher strengths are obtained, but they are harder to inject than grouts with a higher water content. Chemical additives are sometimes used to facilitate penetration, to prevent cement flocculation, and to control set times. Set times can be as short as 30 seconds or very long.

In soil-cement grouts a soil volume of four to six times the loose volume of cement is common. Water volumes from one third to twice the soil volume per bag of cement are used. The low water content mixes are typical of high viscosity displacement grouts. Zero slump compaction grouts with 30 to 60 second gel times can be made using cement, clay, and flyash mixes with an alkaline accelerator. If bentonite is used, expanded particles may collapse if the groundwater has a high salt content. Care should be taken in the use of cement in the presence of sulfate-bearing soils or groundwater.

Particulate grouts cannot be injected as permeation grouts into soils finer than medium to coarse sands. Some "groutability ratios" that have proven useful are

For soils: $$N = \frac{(D_{15})_{soil}}{(D_{85})_{grout}}$$

$N > 24$: grouting consistently possible

$N < 11$: grouting not possible

$$N_c = \frac{(D_{10})_{soil}}{(D_{95})_{grout}}$$

$N_c > 11$: grouting consistently possible

$N_c < 6$: grouting not possible

For rock: $$N_R = \frac{\text{Width of fissure}}{(D_{95})_{grout}}$$

$N_R > 5$: grouting consistently possible

$N_R < 2$: grouting not possible

Additional guidelines relating to particulate grout types and particle size are:

Types I and II Portland cement are suitable for soils coarser than 0.60 mm.

Type III Portland cement is suitable for soils coarser than 0.42 mm.

Bentonite is suitable for soils coarser than 0.25 mm.

Chemical Grouts

Chemical grouts offer the advantages over particulate grouts that they can penetrate smaller pores, as may be seen in Fig. 25, they have a lower viscosity, and there is a better control of set time. On the other hand their technology is more complex and costs are high. Soils containing less than 10 percent fines (<74 μm particles) can usually be permeation grouted with chemicals. If the fines content is greater than 15 percent effective chemical grouting may be difficult. For fines content greater than 20 percent permeation grouting will not be possible, but chemical grouts can be distributed along and through hydraulic fractures.

The most common chemical grout classes are silicates, lignins, resins, acrylamides, and urethanes. Hundreds of different formulations have been developed within these classes. Of them, however, the silicates account for more than 90 percent of present chemical grout use, the others being limited for reasons of cost and toxicity.

"Two-shot" silicate systems, in which a first injection of sodium silicate is followed by a second injection of a material such as $CaCl_2$ to cause formation of insoluble precipitated silica gel, have largely given way to one-shot silicate systems. Premixed combinations of sodium silicate and appropriate catalysts and activators are formulated to give specified design strength and setting times. A two-shot silicate--CO_2 gas system has been developed and is used in the loess soils of the USSR (Sokolovich, 1973). Ammonia gas injection has also been found effective in loess wherein only modest strengthening is required to prevent collapse. The reaction is to replace adsorbed Ca++ by NH_4+. The liberated Ca++ combines with water to form $Ca(OH)_2$, which acts as a cementing stabilizer.

Grouts containing 25 to 30 percent silicate are typical for waterproofing applications. Where high strength is required for structural applications, silicate concentrations of 40 to 60 percent are used. Unique local chemical conditions may influence reactivity. Temperature effects, dilution by groundwater, catalyst adsorption by sand, and other factors may influence gel time and character of the resultant grout. Accordingly, preliminary testing is desirable.

The properties of chemically grouted sands cover a wide range. Some typical values of viscosity and compressive strength for different grout

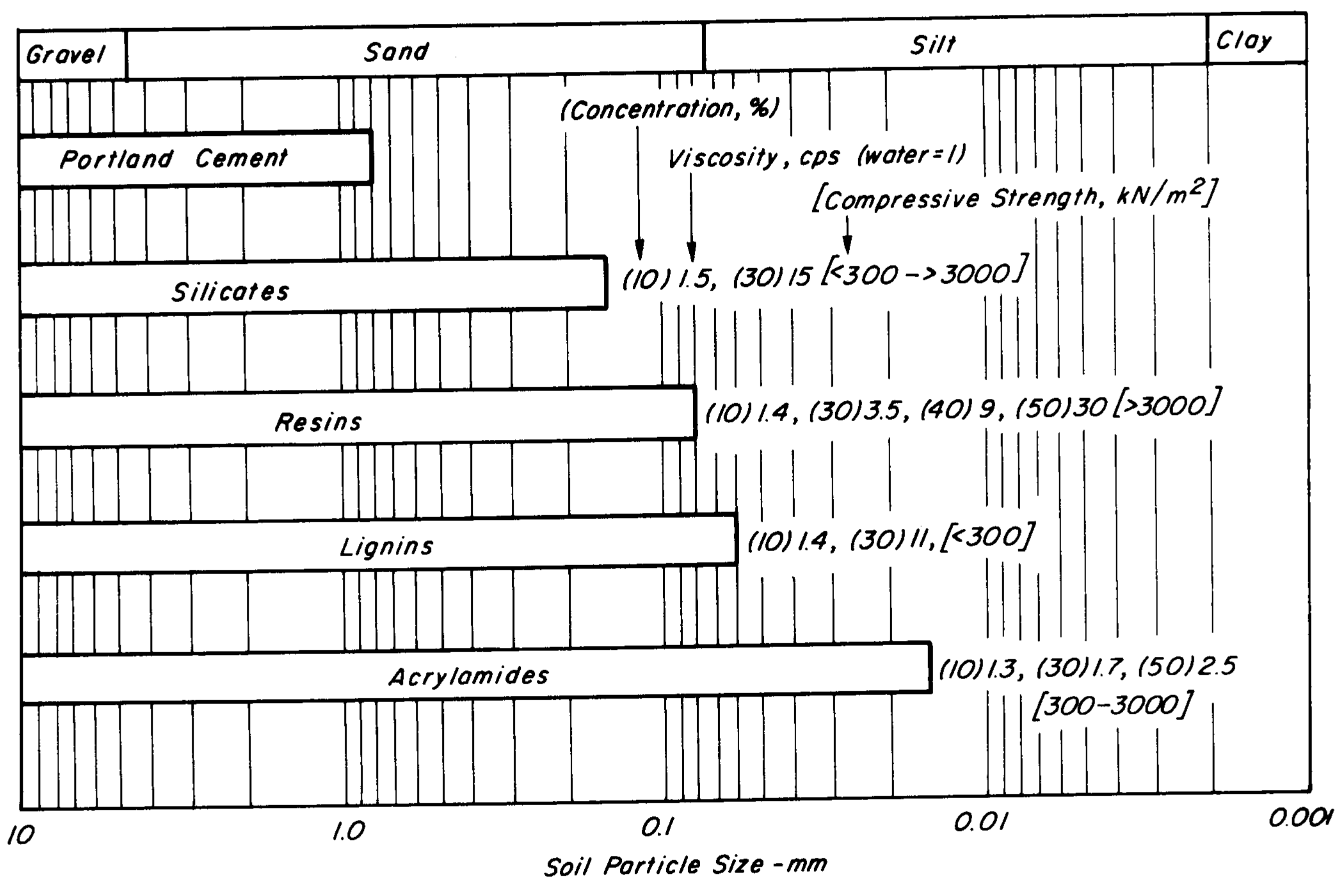


Fig. 25 Soil Particle Sizes for Different Grout Types and Grouted Soil Properties

types and the particle size range for which they are suited are shown in Fig. 25. The unconfined compressive strength of chemically grouted sand may range from 0.3 to more than 10 MPa. The results of triaxial tests, e.g., Perez et al. (1981), indicate that grouting produces an increase in the cohesion intercept but has little effect on the friction angle.

The stiffness of silicate grouted sand increases with confining pressure in about the same way as an ungrouted sand. Stress-strain behavior is non-linear. Strength and stiffness decrease as loading rate decreases, creep occurs under constant load, and creep rupture may develop under high stress levels. These aspects of behavior are important when grouting is used in underpinning, tunneling, and open cut construction. The results of recent studies on these aspects of grouted soil properties are given by Clough et al. (1979), Gartung and Kany (1975), and Perez et al. (1981).

Syneresis, i.e., a time-dependent tightening of the gel structure resulting in fissuring, may occur in silicate-grouted sands, and the seriousness of the resulting problems increases with increasing silicate concentration. In cases where sealing these cracks, as well as others that may have formed by hydraulic fracturing during the injection process, is required, a second injection stage using a cement grout may be needed.

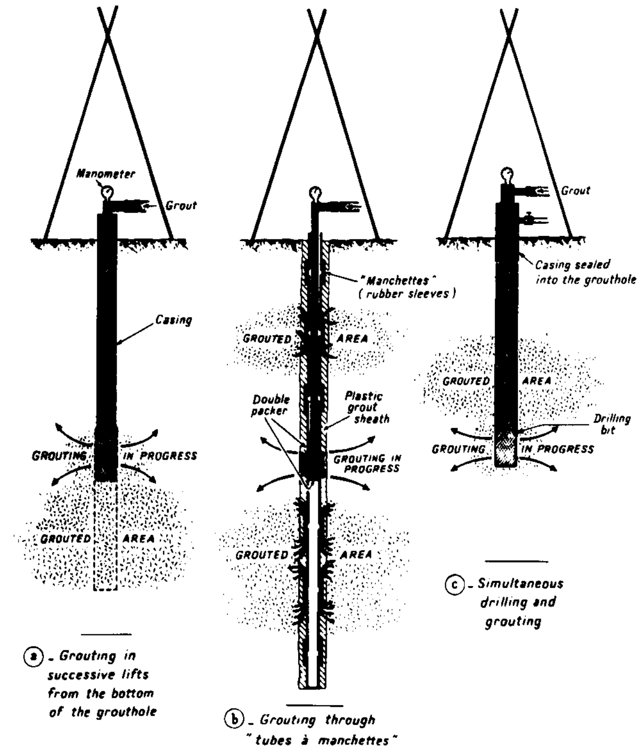
Equipment and Techniques

The design of a successful grouting program requires not only the selection of a suitable grout material, but also the correct drilling equipment, procedures, and grouthole patterns. It is essential that the pipes and injection ports be in the right place. It is more important to ensure that the full design soil volume is permeated with grout when the objective is water cutoff than when the objective is to improve mechanical properties.

Both batch and continuous flow systems can be used effectively. Batch systems give better mixing of components, but require longer gel times than do continuous flow systems. Gel times of 50 to 90 min. are common for batch systems. Continuous flow systems may give better placement control but are more complex.

Hole spacings of about 1.3 m to 2.5 m are typical. Costs become excessive for smaller spacings, and grout placement in desired locations can't be ensured if larger spacings are used. Both open-pipe and sleeve-pipe (tube à manchettes) injection methods are used, as shown in Fig. 26. Although the sleeve-pipe method is more expensive, it is becoming widely used because it gives much better control of grout location. The recent test program at Locks and Dam No. 26 described by Perez et al. (1981) is an excellent comparative study of the two methods. At this site multiple-stage sleeve-pipe grouting was much more effective than open-bottom pipe grouting.

The use of gel times less than the pumping time, termed fast gel times, has the advantages of grout location control in flowing ground water and in stratified soils. It also will limit



(a) - Grouting in successive lifts from the bottom of the grouthole

(b) - Grouting through "tubes à manchettes"

(c) - Simultaneous drilling and grouting

Fig. 26 Injection Methods (Caron et al., 1975)

grout takes in very pervious materials. Pumping time to gel time ratios of 10 are common. The disadvantage of fast gel times, of course, is that human or mechanical failure can lead to grout set up in the equipment.

A common rule of thumb for maximum injection pressure for open pipe grouting is about 20 kPa/m; i.e., a value equal to the over-burden pressure, unless grouting under a heavy structure or in a situation with greater confinement. Values two to three times as great may be used with fast gel time systems. Because of the need to open the rubber sleeve and to break through the plastic grout sheath with the tube à machette method, high pressures may be required initially. High initial pressures may also be useful if it is desired to open up the formation by hydraulic fracturing.

Electrochemical Injection

Just as direct electrical current can be used in lieu of a physical loading to consolidate a compressible soil, it can be used also to move solutions into and through a porous material in lieu of a hydraulic injection pressure. Because the coefficient of electro-osmotic permeability, k_e, is insensitive to particle size and generally falls within a narrow range of about 1×10^{-5} to 7×10^{-5} cm/sec/volt/cm, a unit electrical gradient (1 volt/cm) can be more effective than a unit hydraulic gradient for moving fluids through finer grained soils. Hence, electrokinetic injection might be considered for use in silty soils which cannot be injected using ordinary grouting techniques. Electro-kinetic injection might be useful also where lack of confinement prevents grouting in the usual way.

Chemical stabilizers are introduced at the anode and carried toward the cathode using electrical gradients of the order of 50 to 100 volts per meter. Several experiences with electrochemical injection up to the mid-1970's are summarized by Pilot (1977). O'Bannon et al. (1976) describe the electrochemical hardening of expansive clay. Oncescu and Bally (1977) used an electro-kinetic injection of sodium silicate to strengthen the loess under the foundations of a theater. "Piles" of treated soil 3 to 4 m long and 1.6 to 2.2 m in diameter were formed.

The electro-kinetic stabilization of a potentially liquefiable sand has been investigated by Yamanouchi and Matsuda (1975). The concept was to fill the voids of the loose sand with a gel or colloidal material and thus prevent collapse under cyclic loadings. Silicate solutions, bentonite, and aluminum hydroxide were investigated as injection materials. The bentonite and aluminum hydroxide, as negatively charged colloids, were injected at the cathode and moved into the pores by electro-phoresis. The results demonstrated a marked increase in resistance to liquefaction after treatment.

Segall et al. (1980) report the results of a large number of chemical analyses on water leached both hydraulically and by electro-osmosis from dredged soil. It was found that because of the electrochemical reactions at electrodes, the electro-osmosis water became very alkaline (pH = 13.4). As a result organic materials went into solution. Heavy metals were desorbed and carried out at the cathode, as were also pesticides. Deposits of iron oxide, magnesium hydroxide, and calcium carbonate were formed in the soil near the anode. These results suggest a potentially useful approach to cleaning up hazardous waste-contaminated soil.

Displacement (Compaction) Grouting

Highly viscous soil, cement, and water displacement or compaction grout acts as a radial hydraulic jack which compresses the surrounding soil. The hardened grout mixture is a bulb of strong, relatively incompressible material. Displacement grouting can be used in partly saturated soil masses and loose materials containing void spaces. It is used to correct differential settlements or to provide underpinning and ground strengthening adjacent to open excavation or tunneling activities. Available equipment can develop up to 2.5 to 3.0 MN/m^2 pumping pressure, and zero slump grout can be pumped distances in excess of 30 m. To be effective, compaction grouting should not be undertaken at depths less than 1 to 2 m unless there is an overlying structure to provide confinement.

Compaction grouting materials and procedures are described by Warner and Brown (1974), Graf (1969), and Warner (1978), among others. Recently, during construction of the Baltimore, Maryland, U.S.A. subway near the Bolton Hill Station, the projected cost of underpinning existing structures was so high that compaction grouting was used to prevent surface subsidence (Hayward Baker, 1980). Grout bulbs were inserted via 76 mm diameter pipes placed 3 m apart to provide support between the tunnel crown, after the shield passed, and the ground surface. The scheme is shown schematically in Fig. 27. Pumping pressures averaged about 2 MPa, injection depths were about 12 m below ground surface, and work was carried out about 2 m behind the shield.

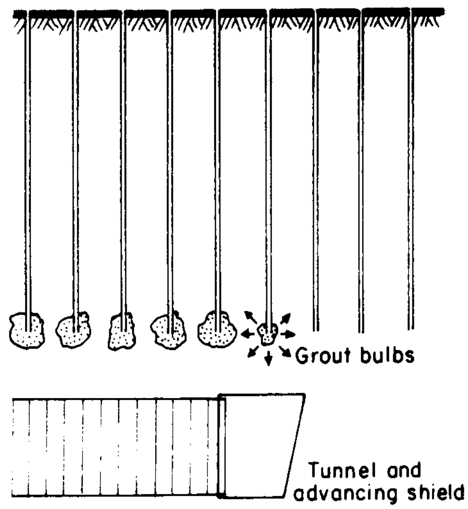


Fig. 27 Compaction Grouting During Tunneling to Prevent Settlements

Jet Grouting

A new grouting technique termed jet grouting was introduced in Japan several years ago (Yahiro and Yoshida, 1973; Miki, 1973; Miki et al., 1980). The basis for jet grouting is a special high speed water jet acting under a nozzle pressure of 15 to 70 MPa. The native soil may be mixed in place with a suitable stabilizer as shown in Fig. 28. Alternatively, poor soils can be removed by in-situ excavation and replaced by a mortar grout to form hard, impervious columns, panels or sheets, as shown schematically in Fig. 29. Jet grouted columns up to 3 m in diameter are possible. The use of air jetting in conjunction with grout jetting can yield diameters up to twice as great, for a given jet pressure, as the grout jet alone.

The method offers the advantages of both close control over the zones being treated and applicability to clays as well as sands. Miki et al. (1980) report an increase in unconfined compressive strength of an originally soft clayey soil to 1.5 to 4 MPa, some 30 times the original strength. The secant modulus at 50 percent of the failure stress was increased by a factor of 200.

Evaluation of Effectiveness

Precise determination of exactly where all the grout went in the ground is usually not possible. Assessment of grouting effectiveness is usually made on the basis of grout take records and the results of in-situ tests and laboratory tests on