ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

Survey of Ground Improvement Techniques for Bearing Capacity

By

Andinet Abebe Gezu

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A paper submitted to School of Graduate Studies of Addis Ababa University
In Partial Fulfillment of the Requirements for
The Masters of Engineering Degree of Civil Engineering in Geotechnical Engineering

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Advisor

Dr. Samuel Tadesse

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Institute of Technology

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Chairman	Signature	Date

DECLARATION

I, the undersigned, declare that this Individual Project paper is my original work performed under the supervision of my Project advisor Dr. Samuel Tadese and has not been presented as a thesis for a degree in any other university. All sources of materials used for this paper have also been duly acknowledged.

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TABLE OF CONTENT

SYMBOL AND VIBRATION

VC- Vibro Compaction,
CPT- Cone Penetration Test,
DPT- Direct penetration test,
SPT- Standard Penetration Test,
PVD- Prefabricate Vertical Drainage,
DPT- Dynamic Penetrometre Test,
DSM - Deep Soil Mixing,
VCC- Vibro Concrete Column,
PSI - pound per square Inch,
RAPs -Rammed Aggregate Piers

Abstract

Many soil improvement methods have been developed due to the ongoing increase in urban and industrial growth and the need for greater access to lands. This has made the use of areas with weak soils inevitable.

The very low bearing capacity of the foundation bed causes shear failure and excessive settlements. Further, the high water table and limited depth of the top sandy layer in these areas restrict the depth of foundation thereby further reducing the safe bearing capacity.

Due to the lack of bearing resistance in these soils, different methods of soil improvement techniques, categorized under Compaction techniques/densification/, Reinforcement techniques, Grouting/mixing techniques, and Drainage techniques including some other innovation of soil improvement techniques as a method of strengthening the weak soil are used.

This paper is intended to give the reader a general understanding of each of the techniques, how each improves the soil performance regarding the shear strength, and an overview of how each is analyzed. The purpose is neither to present all the nuances of each technique nor to be a detail design manual. Indeed, entire papers have been tried to addresses different types of method of soil improvement techniques, the applicable soil type, the material and equipment used in the process, procedure and the quality control method which are of practical in the world. In addition, this paper does not address all the safety issues associated with each technique. Many of these techniques have inherent dangers associated with them and should only be performed by trained and experienced specialty contractors with documented safety records.

1 Introduction

1.1 Background

Soils have modified to improve their engineering properties for hundreds of years. In the past 75 years, however, improved knowledge of soil behavior and geotechnical hazards has led to the development and verification of many innovative soli improvement techniques. Increased recognition of seismic hazards and improved understanding of the factors that control them have led these techniques to be applied to the mitigation of seismic hazards in the past 30 years.

At present, a wide variety of soil improvement techniques are available for mitigation of seismic hazards. The costs of these methods vary widely, and the conditions under which they can be used are influenced by the nature and proximity of structures and constructed facilities. On the basis of the mechanisms by which they improve the engineering properties of the soil, the most common of these can be divided into four categories: densification-techniques, reinforcement techniques, *grouting/mixing techniques, and drainage techniques.* However, not all soil improvement techniques fall neatly into category [5].

1.2 Objective of the study

The purpose of this document is to perform a literature study on the different methods of ground improvement techniques that result in compacting, mixing, reinforcing and grouting agent in to the weak soil to improvement of bearing capacity of Soils by increasing shear strength and make denser the soil. And try to study the different techniques of ground improvement which are of Practical in the world. And to show the procedure, the type of soil applicable for each technique, the material and equipment used for each techniques, the quality control and assurance methods.

1.3 Methodology

The paper prepared by studying thoroughly the different reference books written in the geotechnical areas, journals, research papers and different literatures from internet downloaded books.

The different improvement techniques catagorised into four including detail descriptions of each improvement techniques and verification besides this the summary of the techniques listed in detail in tabulated form.

Based on above, identify possible techniques, improvements and research lines for further studies.

REVIEW ON GROUND IMROVMENT FOR BEARING CAPACITY OF SOIL

Long-term performance of structures is significantly impacted by the stability of the underlying soils. In situ foundations often do not provide the support required to achieve acceptable performance under different loading and environmental demands. Although stabilization is an effective alternative for improving soil properties, the engineering properties derived from stabilization vary widely due to heterogeneity in soil composition, differences in micro and macro structure among soils, heterogeneity of geologic deposits, and due to differences in physical and chemical interactions between the soil and candidate stabilizers. These variations necessitate the consideration of site-specific treatment options which must be validated through testing of soil-stabilizer mixtures.

At present, a wide variety of soil improvement techniques are available for mitigation of seismic hazards. The costs of these methods vary widely, and the conditions under which they can be used are influenced by the nature and proximity of structures and constructed facilities. On the basis of the mechanisms by which they improve the engineering properties of the soil, the most common of these can be divided into four categories: densification-techniques, reinforcement techniques, *grouting/mixing techniques, and drainage techniques.* However, not all soil improvement techniques fall neatly into category [5].

The paper presents a straightforward methodology to determine which stabilizers should be considered as candidates for stabilization for a specific soil, pavement, and environment. The report then presents a protocol for each stabilizer through which the selection of the stabilizer is validated based on mixture testing and mixture design. The mixture design process defines an acceptable amount of stabilizer for the soil in question based on consistency testing, strength testing.

These techniques have proven practically to be effective in improving the shearing strength of the weak soils and this also result in increasing the bearing capacity of the treated soil. So that it’s necessary to discuss about the *shear strength* of soil.

SHEARING STRENGTH

Shear strength is the principal engineering property which controls the stability of a soil mass under loads. It governs the *bearing capacity of soils*, the *stability of slopes in soils*, the *earth pressure against retaining structures* and many other problems. All the problems of soil engineering are related in one way or the other with the shear strength of the soil. Unfortunately, the shear strength is one of the most complex engineering properties of the soil.

The shear strength of a soil is its maximum resistance to shear stresses just before the failure. Soils are seldom subjected to direct shear. However, the shear stress develops when the soil is subjected to direct compression. Although shear stress may also develop when the soil is subjected to direct tension, but these shear stress are not relevant, as the soil in this cases fails in tension and does not fail. In field, soils are seldom subjected to tension, as it causes opening of the cracks and fissures. These cracks are not only undesirable, but are also detrimental to the stability of the soil masses. Thus, the shear failure of a soil mass occurs when the shear stresses induced due to the applied compressive loads exceed the shear strength of the soil. It may be noted that the failure in soil occurs by relative movements of the particles and not by breaking of the particles [9].

'Shearing Strength' of a soil is perhaps the most important of its engineering properties. This is because all stability analyses in the field of geotechnical engineering, whether they relate to foundation, slopes of cuts or earth dams, involve a basic knowledge of this engineering property of the soil. 'Shearing strength' or merely 'Shear strength' may be defined as the resistance to shearing stresses and a consequent tendency for shear deformation. Shearing strength of a soil is the most difficult to comprehend in view of the multitude of factors known to affect it. A lot of maturity and skill may be required on the part of the engineer in interpreting the results of the laboratory tests for application to the conditions in the field.[10]

Basically speaking, a soil derives its shearing strength from the following:
(1) *Resistance due to the interlocking of particles*.
(2) *Frictional resistance between the individual soil grains*, which may be sliding friction, rolling friction, or both.
(3) *Adhesion between soil particles or 'cohesion'*.

Granular soils of sands may derive their shear strength from the first two sources, while cohesive soils or clays may derive their shear strength from the second and third sources. Highly plastic clays, however, may exhibit the third source alone for their shearing strength. Most natural soil deposits are partly cohesive and partly granular and as such, may fall into the second of the three categories just mentioned, from the point of view of shearing strength. The shear strength of a soil cannot be tabulated in codes of practice since a soil can significantly exhibit different shear strengths under different field and engineering conditions.

Taking in to account the above discussions, in this paper, the soil improvement techniques stated in detail for the improvement of bearing capacity of the soil. Hence, all the techniques discussed here are helping to improve the shear strength the soil which has a direct effect on the improvement of the bearing capacity of the weak soil.

Long term performance of structures built on the ground often depends on the strength of the underlying soils. Engineering design of these constructed facilities relies on the assumption that each layer in the pavement has the minimum specified structural quality to support and distribute the super imposed loads. These layers must resist excessive permanent deformation, resist shear and avoid excessive deflection that may result in fatigue cracking in overlying layers. Available earth materials do not always meet these requirements and may require improvements to their engineering properties in order to get better bearing capacity for the soil material. This is often accomplished by physical or chemical stabilization or modification of these problematic soils. Although the solution appears simple and straight forward, engineering properties of individual soils may vary widely due to heterogeneity in soil composition, difference in micro and macro structure among soils, variability and heterogeneity of geologic deposits and due to differences in physical and chemical interactions of air/water with soil particles. These differences necessitate the use of site-specific treatment options for the stabilization.

In both seismically active and in active areas, soil improvement techniques are commonly used at sites where the existing soil conditions are expected to lead to unsatisfactory performance. Unsatisfactory performance can take many forms, but usually involves unacceptably large soil movements. The movements may include horizontal or vertical (or both) components and may take place during and/or after earthquake shaking, in the absence of earthquake shaking. Unacceptable movements usually result from insufficient soil strength and/or stiffness. Consequently, most soil improvement techniques were developed to increase the strength and stiffness of soil deposits. These techniques are described in detail in a number of usefully references.

Over the years engineers have tried different techniques to improve soils that are subject to fluctuations in strength and stiffness properties as a function of fluctuation in moisture content. Soil improvement can be derived from different stabilization methods, such as, thermal, electrical, mechanical or chemical means. The first two options are rarely used. Mechanical stabilization, or compaction, is the densification of soil by application of mechanical energy. Densification occurs as air is expelled from soil voids without much change in water content. This method is particularly effective for cohesion less soils where compaction energy can cause particle rearrangement and particle interlocking. But, the technique may not be effective if these soils are subjected to significant moisture fluctuations. The efficiency of compaction may also diminish with an increase of the fine content, fraction smaller than about 75 μm, of the soil. This is because cohesion and inter particle bonding interferes with particle rearrangement during compaction. Altering the physio-chemical properties of fine-grained soils by means of chemical stabilizers/modifiers is a more effective form of durable stabilization than densification in these fine-grained soils. Chemical stabilization of non-cohesive, coarse grained soils, soils with greater than 50 percent by weight coarser than 75 μm is also beneficial if a substantial stabilization reaction can be achieved in these soils. In this case the strength improvement can be much higher, greater than tenfold, when compared to the strength of the untreated material. This report discusses key factors associated with improvement methods of soils for bearing capacity using physical, chemical and mechanical modifiers [6].

2 Ground Improvement techniques

In the dredging industry soil improvement is typically implemented:

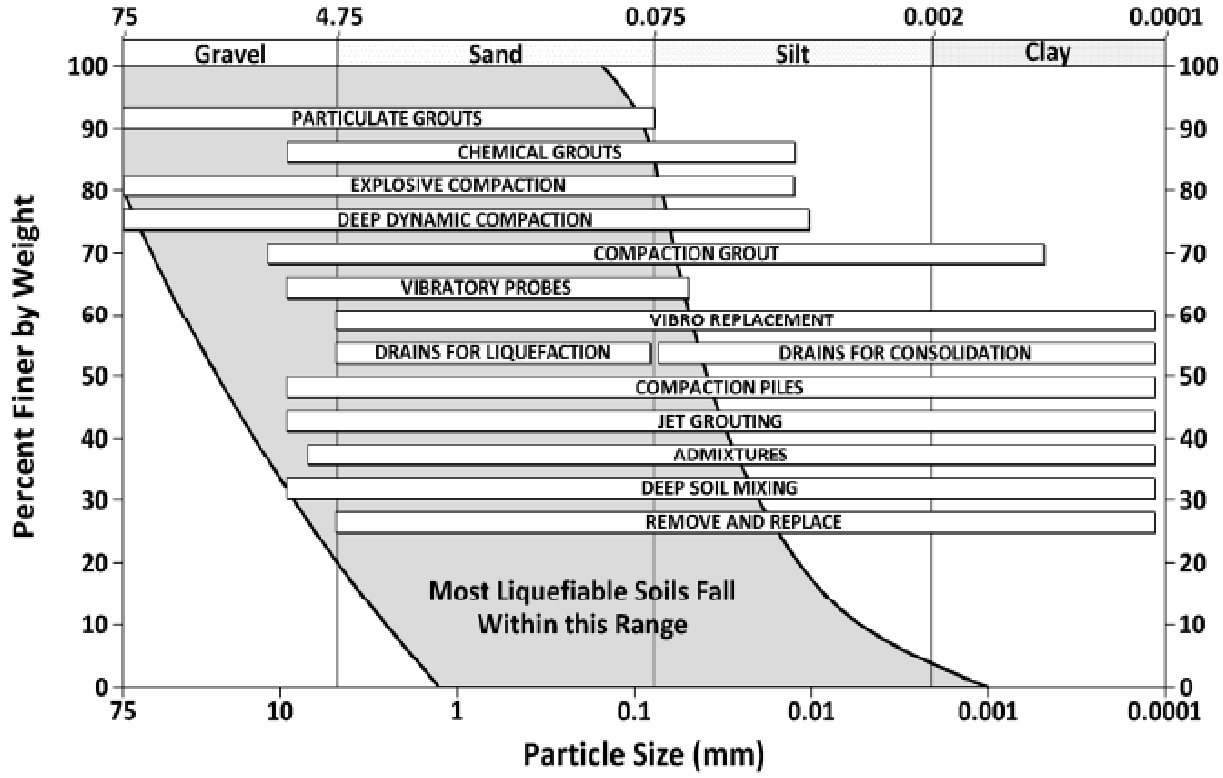
- To prevent excessive settlement of reclaimed land when it is being utilized for construction purposes (roads, airports, bridge and other foundations);
- To enhance the soil stiffness in order to prevent liquefaction and subsequent damage to structures in seismic-sensitive regions;
- To enhance the shear strength of the soil to prevent slip failure;
- To enhance the bearing capacity of the soil; and/or
- To immobilize or stabilize contaminants in dredged soil in order to eliminate environmental impacts.

Ground improvement techniques vary depending on the characteristics of the soil. Some techniques are applied to consolidate existing loose subsoil and some are specifically for compaction of newly reclaimed soil [2].

Ground improvement, is the modification of existing site foundation soils to provide better performance under design and/or operational loading conditions. Ground improvement techniques are used increasingly for new projects to allow utilization of sites with poor subsurface conditions. Previously, these poor soils were considered as economically unjustifiable or technically not feasible and are often replaced with an engineered fill or location of the project is changed. In short, Ground improvement is executed to increase the bearing capacity, reduce the magnitude of settlements and the time in which it occurs, retard seepage, accelerate the rate at which drainage occurs, increase the stability of slopes, mitigation of liquefaction potential, etc.

Based on the soil conditions, a suitable method of ground improvement should be considered keeping in view of the economic feasibility as well as the time frame. In practice, ground improvement is widely used in a broad construction spectrum from industrial, commercial and housing projects to infrastructure construction for dams, tunnels, ports, roadways and embankments. This paper presents different ground improvement techniques.

Ground Improvement techniques based on soil characterization ***[8]***



Jim Mitchell 2008

2.1 Compaction Techniques

The particles that comprise a particular soil can be arranged in many different ways. However, the strength and stiffness of the soil is higher when the particles are parked in a dense configuration than when they are packed loosely. Also the tendency to generate positive excess pore water pressure due to cyclic loading is lower when the soil is dense than when it is loose. As a result, densification is one of the most effective and commonly used means of improving soil characteristics for mitigation of seismic hazards. At the same time, it should be recognized that the increased stiffness of a densified soil deposit will cause it to respond differently to earthquake motion: displacement amplitudes are likely to decrease, but accelerations may be somewhat greater than they would have been had the soil not been improved.

Densification produces Permanent volume changes that often result in settlement of the ground surface.

The most common approaches to densification include vibro-techniques, dynamics compaction, blasting, and compaction grouting. Of these techniques, the first three make use of the tendency of granular soils to densify when subjected to vibrations. As such, their effectiveness is greater for cohesionless soils such as clean sands and gravels, just as a fine tends to inhibit liquefaction during earthquakes; they tend to inhibit densification by vibration [5].

2.1.1 Vibro compaction Techniques

Vibro techniques use probes that are vibrated through a soil deposit in a grid pattern to densify the soil over the entire thickness of the deposit .Vibro techniques can be divided into those based on horizontal vibration (vibroflotation) and those based on vertical vibration (vibro rod systems). Vibro techniques are among the most commonly used techn- iques for mitigation of seismic hazards [5].

Particles of granular soil can be rearranged by vibration in such a way that they obtain a higher density. In non cohesive oils (granular soils), the effective depth of surface compactor and vibratory roller is limited to a few meters below ground level and the larger depths can be reached by deep compaction methods using depth vibrators. The method is referred as Vibro compaction.

The depth vibrator is lowered into the ground under its own weight assisted by water flushing from jets positioned near the tip of the vibrator (i.e. bottom jets). Experience has shown that penetration is most effective if a high water flow rate is used, as opposed to high water pressure. On reaching the designated final depth, the bottom jets are closed and flushing continued by water from jets positioned near the top of the vibrator. These jets direct water radially outward, assisting the surrounding sand to fall into the space around the vibrator [1].

a. Vibroflotation

Vibro compaction (VC), also known as VibroflotationTM was developed in the 1930s in Europe [7]. The process involves the use of a down-hole vibrator (vibroflot), which is lowered into the ground to compact soils at a depth (fig 1.a). The method is used to increase bearing capacity, reduce foundation settlements, reduce seismic subsidence and liquefaction from seismic tremors, and permit construction on loose granular fills.

Applicable soil types: The VC process is most effective in free draining granular soils especially effective in soils with a silt content of up to 20 percent. The expected improvement achieved in specific soil types is shown in (table 1.1a). The typical spacing is based on a 165-horsepower (HP) (124 kW) vibrator. Although most effective below the ground water table, VC is also effective above. Depths down to 65 m have been improved so far by using Vibro compaction technique.

Applicability of the Vibro Compaction:

Vibro compaction is used to increase the bearing capacity of foundations and to reduce their settlements. Another application is the densification of sand to mitigate the liquefaction potential in earthquake prone zones. Vibro compaction may be used as a ground improvement technique to support all type of structures from embankments to chemical plants. The use of vibro compaction mainly depends on the type of granular soil to be compacted.

In vibroflotation, a torpedo like probe (the vibroflot) suspended by a crane is used to densify a soil deposit. Vibroflots, usually 12 to 18 in.(30 to 46 cm|) in diameter and about 10 to16 ft(3 to 4.9 m) long, contain weights mounted eccentrically on a central shaft driven by electric or hydraulic power (fig 1) [5].Another application is the densification of sand to mitigate the liquefaction potential in earthquake prone zones.

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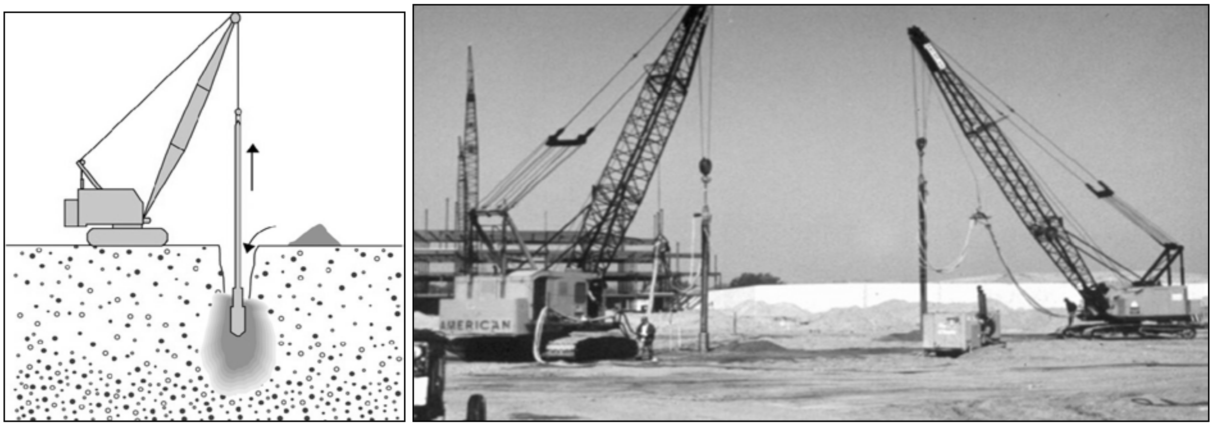


FIGURE 1.a
Vibroflotation: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

TABLE 12.2
Expected Improvement and Typical Probe Spacing with Vibro Compaction [7]

Soil Description	Expected Improvement	Typical Probe Spacing (ft)[a]
Well-graded sand <5% silt, no clay	excellent	9–11
Uniform fine to medium sand with <5% silt and no clay	Good	7.5–9
Silty sand with 5–15% silt, no clay	Moderate	6–7.5
Sand/silts, >15% silt	Not applicable	—
Clays and garbage	Not applicable	—

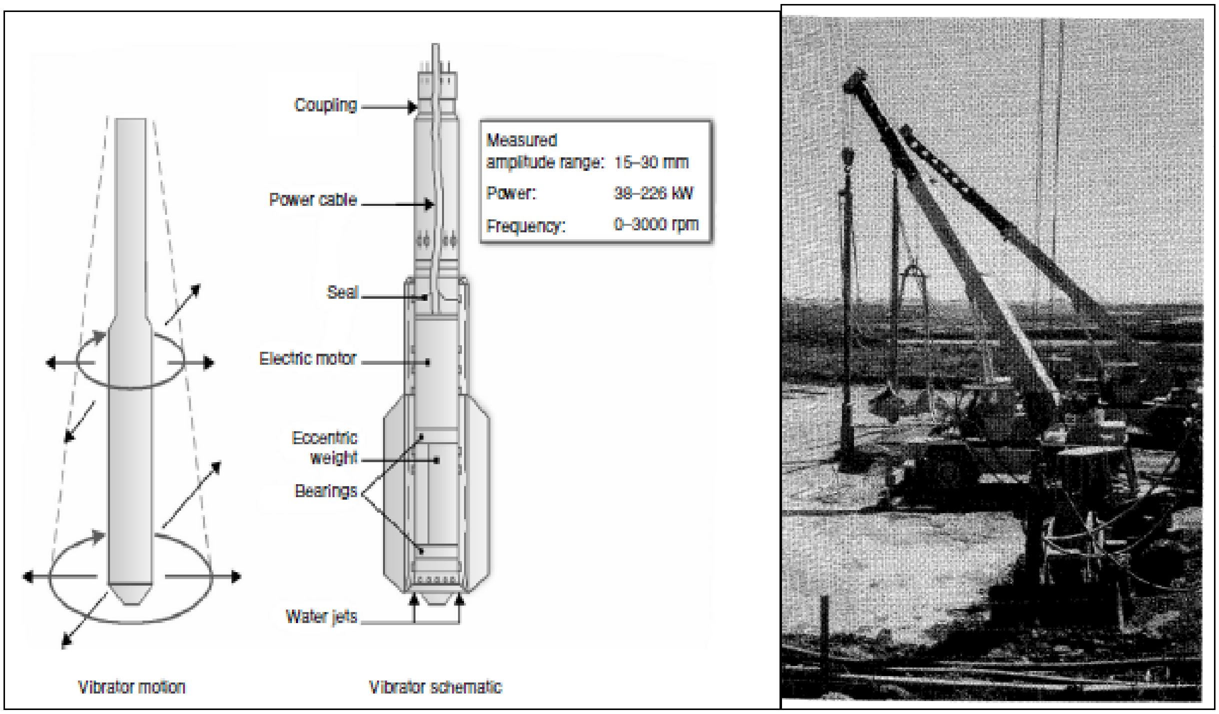
[a] Probe spacing to achieve 70% relative density with 165HP vibroflot, higher densities require closer spacing (1 ft ¼ 0.308 m).
[b] Limited improvement in silts can be achieved with large displacements and stone backfill.

The vibroflot is initially lowered to the bottom of the deposit by combination of vibration and water or air jetting through ports in its pointed nose cone. The vibroflots is then incrementally withdrawn in 2 to 3 ft (60 to 90cm) intervals at an overall rate of about 1ft/min (30cm/min) while still vibrating. Water may be jetted though ports in the upper part of the vibroflot to loosen the soil above the vibroflot temporarily and aid in its withdrawal. The vibrations produce a localized zone of temporary liquefaction that surrounding the vibroflot to densify a conical depression usually forms at the ground surface above the probe. This depression can be filled with granular material (such as clean sand or gravel) as the vibroflot is withdrawn. Alternatively, vibroflots with bottom-feed systems can introduce granular material through the tip of the vibroflot. As the vibroflot is removed, it leaves behind a densified column of soil. When gravel or crushed stone is introduced into the soil, the resulting *stone column* provides benefits of reinforcement and drainage in addition to densification. The use of bottom-feed systems has increased rapidly in recent years. Air delivery systems have become quite common and tend to be preferred over water delivery systems at congested sites and in environmentally sensitive areas.

Vibroflotation is most effective in clean granular soils with fines contents less than 20% and clay contents below 3%.in such soils it typically produces high densities (relative densities of about 100%) within 12 to 18 in.(30 to 46cm) of the vibroflot and lower densities at greater radial distances. To densify an entire site, vibrofloatation is performed in a grid pattern with a spacing that depends on the soil conditions and the power of the vibroflot: spacing's of 6 to 10 ft (2 to 3m) are common. Vibroflotation has been used successfully to densify soils to depths of up to 115ft (35cm).case histories of vibrofloatation have been reported by harder et al. (1984) and Dobson (1987).[5]

Equipment: The vibroflot consists of a cylindrical steel shell with and an interior electric or hydraulic motor which spins an eccentric weight (fig 1.a2) Common vibrator dimensions are approximately 10 ft (3.1 m) in length and 1.5 ft (0.5 m) in diameter. The vibration is in the horizontal direction and the source is located near the bottom of the probe, maximizing the effect on the surrounding soils. Vibrators vary in power from about 50 to over 300HP (37.7 to 226 kW). Typically, the vibroflot is hung from a standard crane, although purpose built machines do exist. Extension tubes are bolted to the top of the vibrator so that the vibrator can be lowered to the necessary treatment depth.

Electric vibrators typically have a remote ammeter, which displays the amperage being drawn by the electric motor. The amperage will typically increase as the surrounding soils densify.



b)

a)

FIGURE 1a.2 vibro compaction; a) schematic diagram b) field implementation

Procedure: The vibrator is lowered into the ground, assisted by its weight, vibration, and typically water jets in its tip. If difficult penetration is encountered, pre-drilling through the firm soils may also be performed. The compaction starts at the bottom of the treatment depth. The vibrator is then either raised at a certain rate or repeatedly raised and lowered as it is extracted. The surrounding granular soils rearranged into a denser configuration, achieving relative densities of 70 to 85%. Treatment as deep as 120 ft (37 m) has been performed.

As a result of Vibro compaction, the occurred settlement may range between 5% to 15% of the compaction depth depending on the original density and the desired density. A schematic showing the stepwise installation process of Vibro compaction is shown in Figure 1a.3. When Vibro compaction is used for large areas, it is typically performed using either a triangular or rectangular grid pattern with probe spacing in the range of 2m to 4m c/c. The spacing depends on several factors, including the soil type, backfill type, probe type and energy, and the level of improvement required. [1]

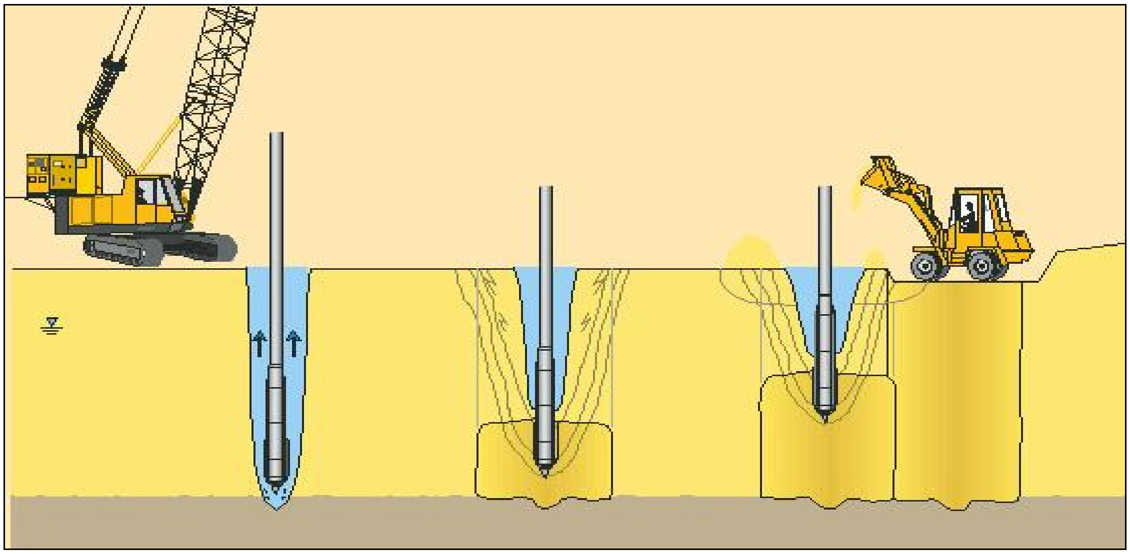


FIGURE (1a.3) Vibro compaction process. (From Hayward Baker Inc. With Permission.)

Quality Control & Quality Assurance:

In general, quality management of Vibro compaction works are divided into two categories, namely monitoring of compaction parameters and post-compaction testing. The compaction parameters (depth and power consumption) are monitored using real-time computerized system throughout the construction process. The recorded data also printed simultaneously in real-time along with the probe reference number, date and time of compaction. This ensures proper documentation of the work done in order to verify desired end product is achieved.

Post- compaction testing is performed to ensure that the specifications are met. Typically, sounding methods are used to assess the effectiveness of the Vibro compaction. Dynamic penetrometer tests (DPT), standard penetration tests (SPT) and cone penetration tests (CPT) can be used. At present, CPT is the most popular post-compaction test. It is suggested to perform post-compaction testing at least one week after the compaction work such that excess pore water reduced to the initial level before compaction.

Limitations:

Vibro compaction methods are most effective for sands and gravels with less than about 15 to 20 percent fines as shown in Figure 2. Vibro compaction works performed for an oil storage terminal is presented below for better understanding of the technique and its performance.

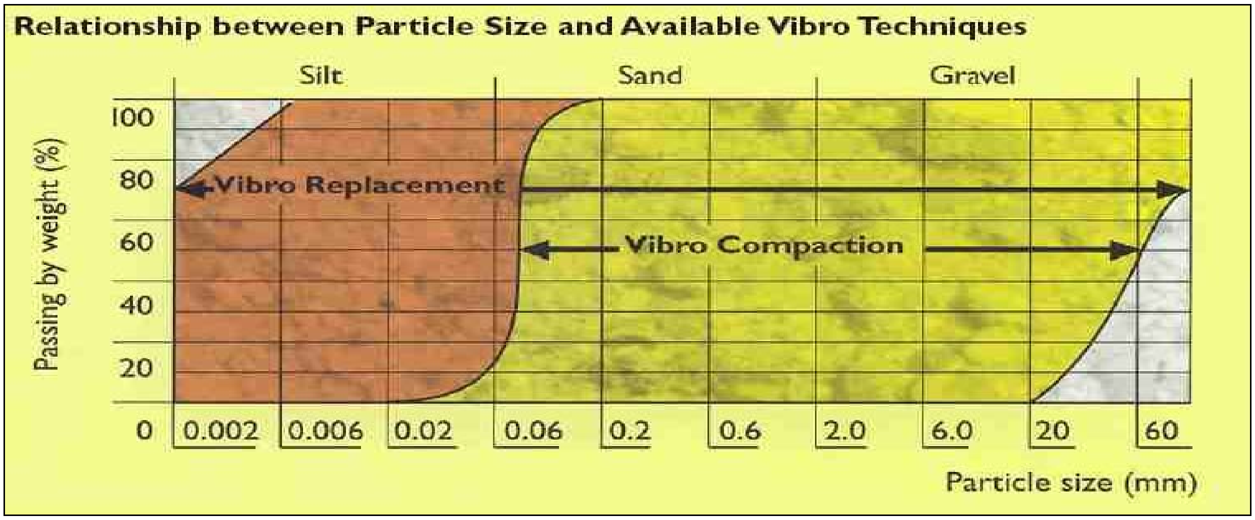


Figure 2. Applicable soil types for Vibro Compaction and Vibro Replacement

Vibro Compaction for Universal Terminal at Jurong Island, Singapore[1]

Project Description:

Jurong Island in Singapore is a 3,200 ha man-made island formed by joining several small islands through extensive land reclamation works. Universal Terminals, Singapore is building an oil storage terminal called as Universal Terminal, at Banyan Logispark on Jurong Island. The Universal Terminal, one of the world's largest and Asia's biggest independent oil storage facility, consists of 73 tanks, ranging in diameter from 13.6m to 78.6m and having heights between 15m and 22.3m (refer to Figure 3). The 73 tanks are divided into 12 tank farms. It also includes valve manifolds, pump stations and other ancillary facilities.

Figure 3. Artist impression of the Universal Terminal upon completion

Soil Conditions:

The soil on site is made up of loose to medium dense reclaimed sand fill having thickness between 20m and 35m underlain by stiff Jurong Formation.

Proposed Ground Improvement Scheme:

Vibro Compaction was proposed to compact loose granular soils and to achieve uniform level of densification. The compaction works were designed and built by M/S Keller to ensure that post construction settlements are within allowable limits with adequate bearing capacity for stability. Vibro Compaction was executed using both single and twin vibro set-up as shown in Figure 4. Vibro Compaction works of approximately 200,000m2 areas which includes areas to be used for construction of tanks, manifolds and pump stations, was completed within a short duration of 6 months between December 2005 and May 2006 using three twin vibro rigs and two single vibro rigs.

Figure 4. Twin vibro compaction rigs during the compaction works at the Universal Terminal

b. Vibro Rod

Vibro rod systems use a vibratory pile driving hammer to vibrate a long probe into the soil. The probe is then withdrawn while still being vibrated to densify the soil. To minimize densification-induced settlement, additional soil may be introduced at the ground surface or at depth. Several types of probes have been used in vibro compaction. In the *terraprobe system,* a 30-in.(76cm)open-ended steel pipes vibrated into the ground: the vibrations densify the soil both inside and outside the pipe. The vibro-wing consists of a central rod with diametrically opposed 31-in.(80-cm) “wings” spaced 19in.(50cm)apart along the length of the rod(fig 1.1.2).

Vibro rod systems are most effective in soils similar to those for which vibro floatation is most effective. Because vibro rods use vertical vibrations, their radius of influence is usually smaller than that observed for vibroflotation. As a result, the grid spacing for soil improvement by vibro rods is generally smaller than for vibroflotation. The effectiveness of vibro rods also appears to vary with depth (janes, 1973).case histories of vibro rod systems have been reported by massarch (1991),neely and Leroy(1991),and Senneset and Nestvold(1992).[5]

C .Vibro replacement

Vibro Compaction method reaches its technical limits where the fines content is high (i.e. in excess of 15 to 20%) as the fine particles cannot respond to the vibration and necessitates the need for externally introduced reinforcement material such as gravels or stones. To overcome the limitations of the Vibro Compaction method, Vibro Replacement method was first developed in 1956. In this method, a hole is created in the ground and is filled with coarse aggregate such as stones, section by section. The coarse aggregate is then densified along with the surrounding soil by repetitive use of the depth vibrator. This process produces vibro stone column that is integral to the surrounding soil.

In simple terminology, the stabilization of soils by displacing / replacing the soil with the help of a depth vibrator, refilling the resulting space with coarse aggregates (gravels or stones) and compacting the same with the vibrator is referred to as Vibro Re-placement. The resulting matrix of compacted soil and stone columns has improved load bearing and settlement characteristics. A schematic showing the basic concept of the Vibro Replacement technique is shown in Figure 5.

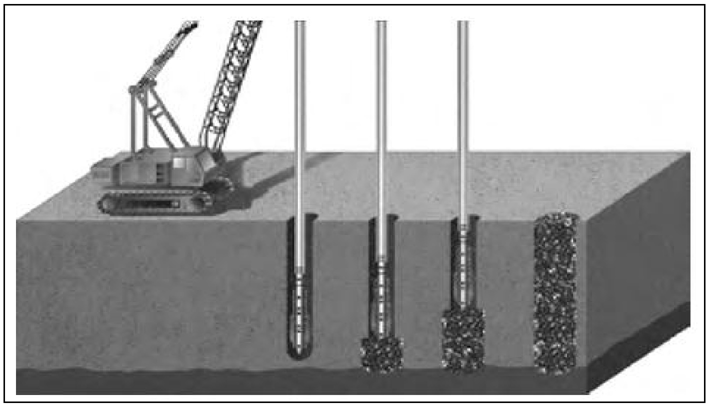


Fig.5 Construction of a vibro-replacement stone column[8]

In addition to long term reinforcement effect, stone columns can also decrease the length of the drainage path in clayey soils by means of radial drainage and there by accelerating the consolidation process. Column diameters typically range between 0.7m and 1.1m and spacing range between 1.5m and 2.5m. Column lengths depend on soils encountered on site but typically range between 6m and 20m. In exceptional circumstances where deep weak soil deposits are found, columns have been installed to depths up to 30m.

The design of the Vibro replacement is based on the loading conations, soil parameters, properties of granular material and intended performance criteria (Preibe, 1995). There are different types of installation methods available which are broadly classified as:
- Wet top feed method
- Dry bottom feed method with purpose-built base machine
- Dry bottom feed crane-hung method
- Offshore bottom feed method

This paper highlights only the wet top feed method. The name arises from the fact that water jetting is used to assist penetration and stone is fed from the top of the vibrator. In this method, a crawler crane of sufficient capacity is used to support the assembly and penetration to the required depth is assisted by the combined action of vibrations and high pressure water jets placed at the tip of the vibrator. After the vibrator reaches the re-quired depth, the stones (typically 35mm to 75mm) are fed to the compaction point from the ground surface with the help of a loader. This method is a partial replacement process where some of the soil is replaced and the rest is laterally displaced and compressed. This method has been successfully used to treat weak soils to depths of 30m. A schematic of the installation procedure is shown in Figure 6.[1]

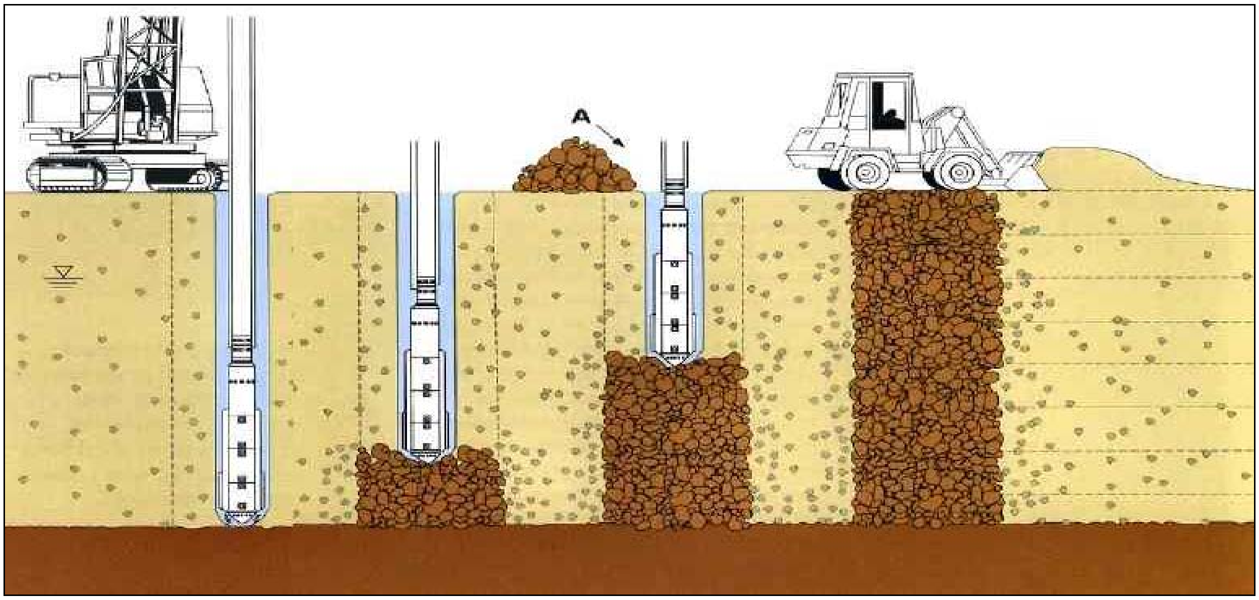


Figure 6. Installation sequence of wet top feed method

Quality Control & Quality Assurance:

In Vibro Replacement method, quality control monitoring consists of monitoring of column construction parameters and post construction testing. During construction, the essential parameters of the construction process (depth, vibrator energy during penetration and compaction processes and stone consumption) are recorded continuously as a function of time, thus ensuring the production of a continuous well compacted column.

The performance of vibro stone columns is monitored using plate load tests which should be carried out by loading a rigid steel plate or cast in-situ concrete pad big enough to span one or more columns and the intervening ground. In contrast to the more familiar load tests on piles, both the column and the tributary area of soil around the column are loaded. In addition, the performance of the treated ground to support the intended structure is evaluated based on the results of instrumentation such as rod settlement gauges, inclinometers, etc.

Applicability of the Vibro Replacement:

The unique characteristic of the Vibro Replacement technique is that it is able to treat a wide range of weak soils ranging from loose silty sands, soft marine clays, ultra soft silts and clays from mine tailings, garbage fills to peaty clays. The structures supported by Vibro Replacement technique have ranged from simple earth embankments, modern expressway embankments, bridge approaches, high speed railway embankments, marine and offshore structures, seaport / airport facilities, power plant structures, chemical plants, sewage treatment plants to large storage tanks. The treatment purpose has been to ensure stability and bearing capacity and to limit settlements. In earthquake prone regions, it has also been used to mitigate liquefaction potential.

Limitations

Vibro Replacement is applicable to a very wide range of soils (refer to Figure 2). Vibro stone columns are not suitable in liquid soils with very low undrained shear strength, because the lateral support may be too small. However, vibro stone columns have been installed successfully in ultra soft soils with undrained shear strengths between 5 and 15kPa.

In general, the technique is applied to structures with high order of settlement tolerance. However, surcharging / preloading can be considered, if a stringent settlement criterion is required. In case of very hard and/or cemented layers or well compacted surface layers, pre boring may be necessary to assist the penetration of the vibrator. A recent case history of a project where Vibro replacement works were carried out for a tank farm is presented below.

Vibro Replacement Works for a Crude Oil Tank Farm at Paradeep, India[1]

Project Description:

Indian Oil Corporation Limited has developed the Paradeep Haldia Crude Oil Pipeline Project for which a tank farm was proposed to be built at Paradeep, Orissa. The tank farm consists of 15 N^{o}s of floating roof crude oil storage tanks with 60,000m3 capacity, having 79m internal diameter and 13.5m height.

Soil Conditions:

The subsoil consists of loose to medium dense fine sands to a depth of 10m. A silty clay layer of 1.5m thickness is sandwiched between sand layers at 3m depth. Below 10m, dense sand layers (SPT N30 >25) are found. The top 3m consist of recently reclaimed fill material.

Proposed Ground Improvement Scheme:

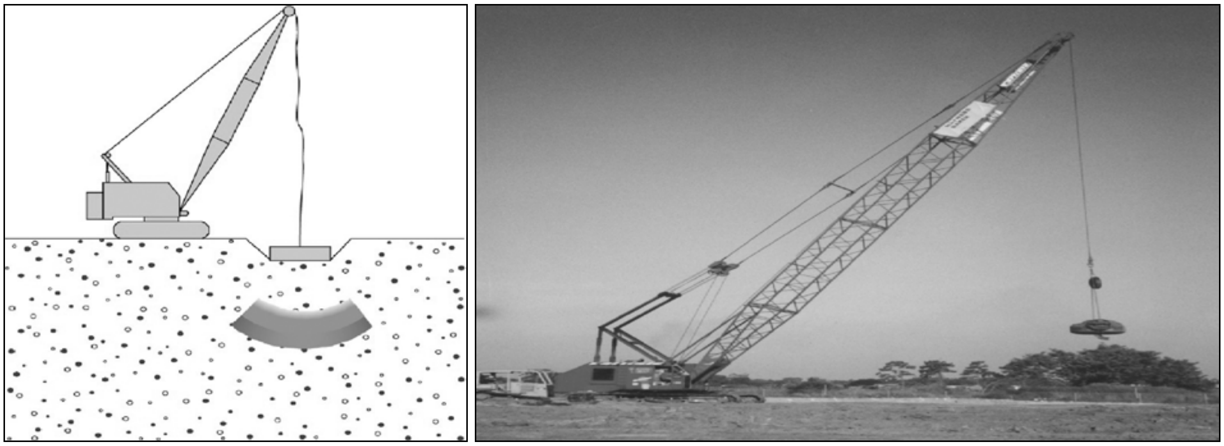
The purpose of the ground improvement was to limit the settlement of the tanks to 200mm at the end of the hydrostatic tests. In addition, it was required to achieve a bearing capacity of 16t/m2. Vibro stone columns of 800mm diameter were installed at a triangular grid spacing of 2m c/c to a depth of about 10m (refer to Figure 7) in order to fulfill the technical requirements. The treatment area included a zone extending 6m beyond the tank footprint to ensure the adequate edge stability of the tank. The success of the ground improvement by vibro replacement was underlined by the monitored final settlement at the end of the hydrostatic test, which did not exceed 100mm.[1]

Figure 7. Installation works for the tank farm at Paradeep, India

Advantages [8]	**Disadvantages**
•Densifies soil •Increases lateral stress •Reinforces soil mass •Provide drainage of excess pore •Most effective technique for liquefiable soils that fall within the typical grain size range •Widely used •Economical	•Ineffective for soils with high fines content (>20%) •Inefficient for liquefiable soils with a limited thickness at a significant depth •Difficult to penetrate stiff strata (cemented, cobbles) •Settlement of surrounding ground •Vertical conduit for environmental contaminants

2.1.2 Dynamic Compaction

Dynamic compaction (DC), also known as dynamic deep compaction, was advanced in the mid-1960s by Luis Menard, although there are reports of the procedure being performed over 1000 years ago. The process involves dropping a heavy weight on the surface of the ground to compact soils to depths as great as 40 ft or 12.5m (Figure 12.2).[7]



a) b)

FIGURE 2.1

Deep dynamic compaction: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

The method is used to reduce foundation settlements, reduce seismic subsidence and liquefaction potential, permit construction on fills, densify garbage dumps, improve mine spoils, and reduce settlements in collapsible soils.

Applicable soil types:

Dynamic compaction is most effective in permeable, granular soils. Cohesive soils tend to absorb the energy and limit the technique's effectiveness. The expected improvement achieved in specific soil types is shown in Table 12.1. The ground water table should be at least 6 ft below the working surface for the process to be effective.

In organic soils, dynamic compaction has been used to construct sand or stone columns by repeatedly filling the crater with sand or stone and driving the column through the organic layer.

Equipment: Typically a cycle duty crane is used to drop the weight, although specially built rigs have been constructed. Since standard cranes are typically not designed for the high cycle, dynamic loading, the cranes must be in good condition and carefully maintained and inspected during performance of the work to maintain a safe working environment. The crane is typically rigged with sufficient boom to drop the weight from heights of 50 to 100 ft (15.4 to 30.8 m), with a single line to allow the weight to nearly "free fall," maximizing the energy of the weight striking the ground. The weight to be dropped must be below the safe single line capacity of the crane and cable. Typically weights range from 10 to 30 tons (90 to 270 kN) and are constructed of steel to withstand the repetitive dynamic forces.

Procedure: The procedure involves repetitively lifting and dropping a weight on the ground surface. The layout of the primary drop locations is typically on a 10 to 20 ft (3.1 to 6.2 m) grid with a secondary pass located at the midpoints of the primary pass. Once the crater depth has reached about 3 to 4 ft (about 1 m), the crater is filled with granular material before additional drops are performed at that location.

The process produces large vibrations in the soil which can have adverse effects on nearby existing structures. It is important to review the nearby adjacent facilities for vibration sensitivity and to document their pre-existing condition, especially structures within 500 ft (154 m) of planned drop locations. Vibration monitoring during DC is also prudent. Extreme care and careful monitoring should be used if treatment is planned within 200 ft (61.5 m) of an existing structure.[7]

The Dynamic compaction is by repeatedly dropping a heavy weight in a grid pattern on the ground surface(figure 2.3).The weights, usually constructed of steel plates and/or reinforced concrete, generally range from 6 to 30 tons (53 to 267KN).although weights of up to 170 tons(1500kN) have been used. Drop heights usually range from about 35 to 100ft (10 to 30m), although weights have been dropped from upto 130ft (40m).the weights are usually dropped three to eight times before moving to the next point on the grid. A detailed description of dynamic compaction was prepared by Lukas(1986).[5]

At a particular site, dynamic compaction is generally performed in several stages, or passes. Empirical evidence suggests that the effective depth of influence (the depth to which significant improvement can be detected) increase with impact energy and that the greatest degree of improvement is usually observed at about half the effective depth of influence (Mayne et al., 1984).To avoid developing a shallow zone of dense soil is densified first with a series of high-energy (heavy weight and/or high drop height) drops on a widely spaced grid. After the craters produced by the first pass have been filled (preferably with well-graded granular soil.), sills at intermediate depth are then compacted using a greater number of drops from a smaller height at closer spacing (often half the spacing of the original grid).finally, the near-surface soils are compacted by dropping relatively light weight on a virtually continuous pattern to smooth or “iron” the ground surface. Additional smoothing by conventional surface grading and compaction equipment is usually required.

The kinetic energy of the weight at impact produces stress waves that travel through the soil. The total energy delivered to the soil is a function of the weight, drop height, grid spacing, and number of drops per grid point. When the ground water table is near the surface, placement of gravel or sand blanket may be required prior to compaction, although dynamic compaction has been used successfully for cohesive soils, its most common use for mitigation of seismic hazards is for potentially liquefiable soils. At each grid point, a series of drops cause the pore water pressure to increase so that the soil particles can more easily move into a denser configuration. Dissipation of the excess porewater pressure results in further densification within a short period (1 to 2 days for sand and gravels: 1 to 2 weeks for sandy silts) after treatment.

Dynamic Compaction is generally effective to depths of 30 to 40 ft (9 to 12m), although extremely high impact energies may produce densification at greater depths. Because the process is rather intrusive it can produce considerable noise, dust, flying debris, and vibration- it is rarely used near occupied or vibration-sensitive structures. Case histories of dynamic compaction of potentially liquefiable soil have been described by Hussin and Ali(1987),Keller et al.(1987), Koutsoftas and Kiefer(1990),Mitchell and wentz(1991),and others.[5]

Materials: The craters resulting from the procedure are typically filled with a clean, free draining granular soil. A sand backfill can be used when treating sandy soils. A crushed stone backfill is typically used when treating finer-grained soils or landfills.

TABLE 12.1

Expected Improvement and Required Energy with Dynamic Compaction [7]

Soil Description	Expected Improvement	Typical Energy Required (tons ft/cf)a
Gravel and sand <10% silt, no clay	Excellent	2–2.5
Sand with 10–80% silt and<20% clay, pI < 8	moderate if dry; minimal If moist	2.5–3.5
Finer-grained soil With pI > 8	Not applicable	—
Landfill	Excellent	6–11

aEnergy ¼ (drop height _ weight _ number of drops)/soil volume to be compacted; 1 ton ft/ft3 ¼ 94.1 kJ/m3

Design: The design will begin with an analysis of the planned construction with the existing subsurface conditions (bearing capacity, settlement, liquefaction, etc.). Then the same analysis is performed with the improved soil parameters (i.e., SPT N value, etc.) to determine the minimum values necessary to provide the required performance. Finally, the vertical and lateral extent of improved soil necessary to provide the required performance is determined. The depth of influence is related to the square root of the energy from a single drop (weight times the height of the drop) applied to the ground surface. The following correlation was developed by Dr Robert Lucas based on field data:

$$D = k\,(W * H)^{1/2} \qquad (2:1)$$

Where D is the maximum influence depth in meters beneath the ground surface, W is the weight in metric tons (9 kN) of the object being dropped, and H is the drop height in meters above the ground surface. The constant k varies with soil type and is between 0.3 and 0.7, with lower values for finer-grained soils. Although this formula predicts the maximum depth of improvement, the majority of the improvement occurs in the upper two-thirds of this depth with the improvement tapering off to zero in the bottom third. Repeated blows at the same location increases the degree of improvement achieved within this zone. However, the amount of improvement achieved decreases with each drop eventually resulting in a point of diminishing returns. The expected range of unit energy required to achieve this point is presented in *table 12.1*

Treatment of landfills is effective in reducing voids; however, it has little effect on future decomposition of biodegradable components. Therefore treatment of landfills is typically restricted to planned roadway and pavement areas, and not for structures. After completion of dynamic compaction, the soils within 3 to 4 ft (1 m) of the surface are loose.
The surface soils are compacted with a low energy ''ironing pass,'' which typically consists of dropping the same weight a couple of times from a height of 10 to 15 ft (3.0 to 4.5 m) over the entire surface area.

Quality control and quality assurance:
In most applications, penetration testing is performed to measure the improvement achieved. In landfills or construction debris, penetration testing is difficult and shear wave velocity tests or large scale load tests with fill mounds can be performed. A test area can be treated at the beginning of the program to measure the improvement achieved and to make adjustments if required. The depth of the craters can also be measured to detect ''soft'' areas of the site requiring additional treatment. The decrease in penetration with additional drops gives an indication when sufficient improvement is achieved.

Crawler cranes can drop tamper masses weighing up to 33 tons from heights of 30 meters [8]

Advantages	Disadvantages
•Densifies soil •Increases lateral stress •Economical – large areas •Simple; no insertion required	•Effective for only upper 10 meters •Difficult to densify soil surrounding large cobbles/boulders •Decreasing effectiveness with decreasing permeability (>20% fines) – damping effect of generated dynamic shear stresses •Disturbing to local structures + occupants

2.1.3 Compaction Grouting

Compaction grouting, one of the few US born ground improvement techniques, was developed by Ed Graf and Jim Warner in California in the 1950s. This technique densifies soils by the injection of a low mobility, low slump mortar grout. The grout bulb expands as additional grout is injected, compacting the surrounding soils through compression.[7]

Besides the improvement in the surrounding soils, the soil mass is reinforced by the resulting grout column, further reducing settlement and increasing shear strength. The method is used to reduce foundation settlements, reduce seismic subsidence and liquefaction potential, permit construction on loose granular fills, reduce settlements in collapsible soils, and reduce sinkhole potential or stabilize existing sinkholes in karst regions.

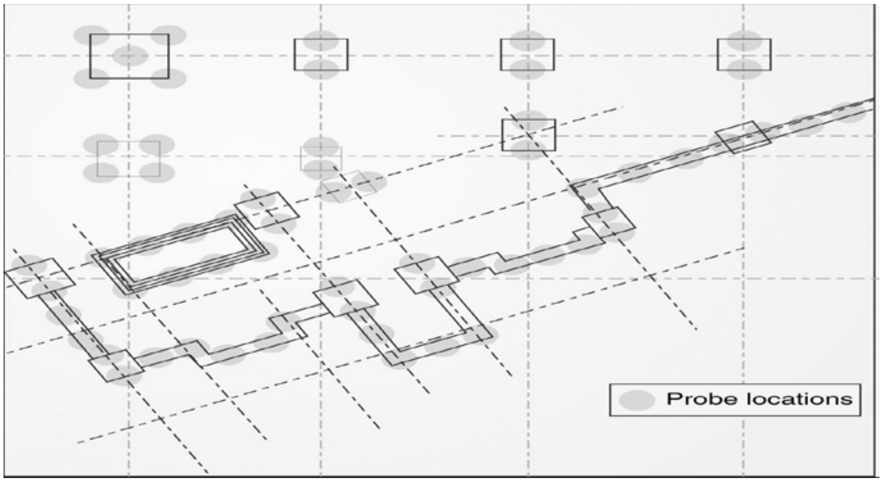


FIGURE 12.6
Typical vibro compaction layout for non seismic treatment beneath foundations. (From Hayward Baker Inc. With permission.)[7]

Soft or weak soils can be densified by injecting a very low slump [generally less than 1 in.(2.5cm)] grout into the soil under high pressure, a process known as *compaction grouting.* Because the grout is highly viscous, it forms an intact bulb or column that densifies the surrounding soil by displacement (figure 12.5).compaction grouting may be performed at a series of points in a grid or along a line.grouit point spacing ranging from 3 to 15ft (1 to 4.6m) have been used. Because higher overburden pressures allow the use of higher grout pressures, larger spacing are generally used when treating deeper soils. At shallow depths, compaction grouting may be used to lift settled slabs or structures: indeed, remediation of foundation settlement is probably the most common application of compaction grouting.

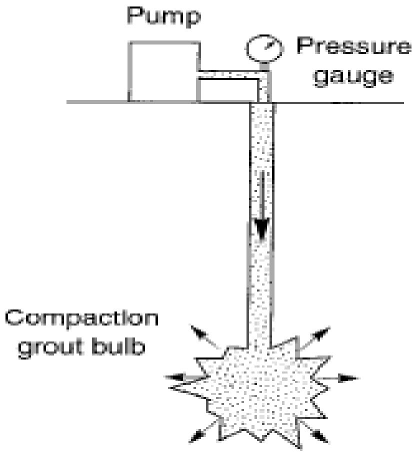


Figure 12.5 Compaction grouting. Low-slump grout is pumped under high pressure to form a bulb that displaces and densifies the surrounding soils. By raising the grout tube while pumping, a column of grout can be created in the soil. (After Hausmann, 1990.)

Compaction grouting may be performed from the top down (downstage grouting) or from the bottom up (upstage grouting).upstage grouting is less expensive and more commonly used than downstage grouting. However, the downstage procedure is preferred (stilley, 1982: Bell, 1993) for underpinning of structures or for sites where loose soils extend to the ground surface. By working from the top down, placement of an upper grout bulb reduces the possibility of subsequent grout escaping at the surface and grout heave, and also provides additional strength and confinement that allows the use of higher grouting pressures at greater depths.[5]

Applicable soil types: Because it is doesn't rely on vibration, compaction grouting can be used in all soil types. It is most commonly used in sands and non plastic silts. compaction grouting can be used to virtually any depth and can easily be used within a given range of depths .The size and shape of the grout bulb or column is influenced by the stiffness and strength of the soil and also by the rate and pressure at which the grout is injected. An important feature of compaction grout masses with diameters greater than 3ft (1m) are not uncommon (warner, 1982). Compaction grouting has been used to depths of 100ft (30m). Case histories of compaction grouting have been described by Salley et al. (1987), Warner (1982), Graf (1992), and Baez and Henry (1993).

Compaction grouting is most effective in free draining granular soils and low sensitivity soils. The expected improvement achieved in specific soil types is shown in Table 12.3. The depth of the groundwater table is not important as long as the soils are free draining.

Equipment: Three primary pieces of equipment are required to perform compaction grouting, one to batch the grout, one to pump the grout, and one to install the injection pipe. In some applications, ready-mix grout is used eliminating the need for on-site batching. The injection pipe is typically installed with a drill rig or is driven into the ground. It is important that the injection pipe is in tight contact with the surrounding soils. Otherwise the grout might either flow around the pipe to the ground surface or the grout pressure might jack the pipe out of the ground. Auguring or excessive flushing could result in a loose fit. The pump must be capable of injecting a low slump mortar grout under high pressure. A piston pump capable of achieving a pumping pressure of up to 1000 psi (6.9 MPa) is often required (Figure 12.7).

Procedure: Compaction grouting is typically started at the bottom of the zone to be treated and precedes upward (Figure 12.6). The treatment does not have to be continued to the ground surface and can be terminated at any depth. The technique is very effective in targeting isolated zones at depth. It is generally difficult to achieve significant improvement within about 8 ft (2.5m) of the ground surface. Some shallow improvement can be accomplished using the slower and more costly top down procedure. In this procedure, grout is first pumped at the top of the treatment zone.

TABLE 12.3
Expected Improvement with Compaction Grouting

Soil Description	Densification	Reinforcement
Gravel and sand <10% silt, no clay	Excellent	Very good
Sand with between 10 and 20% silt and <2% clay	Moderate	Very good
Finer-grained soil, non plastic	Minimal	Excellent
Plastic soil	Not applicable	Excellent

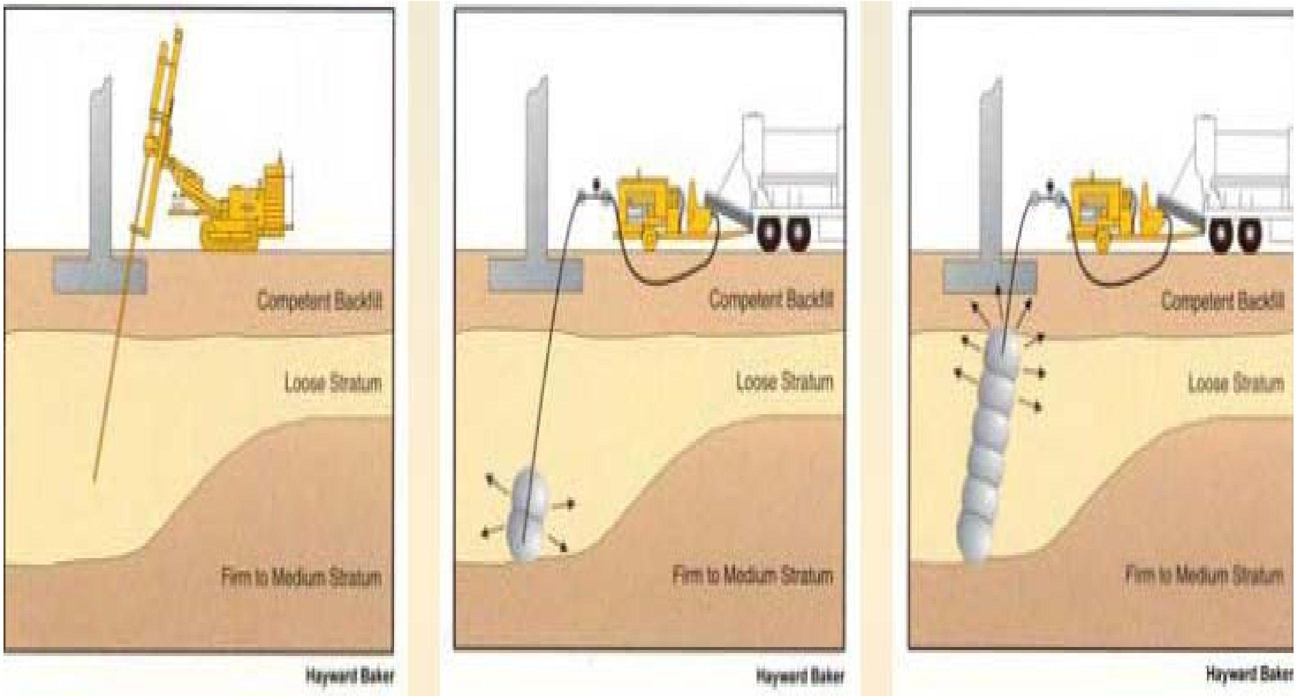


Fig 12.6 Grout rods incrementally pulled up in stages – bottom-up approach [8]

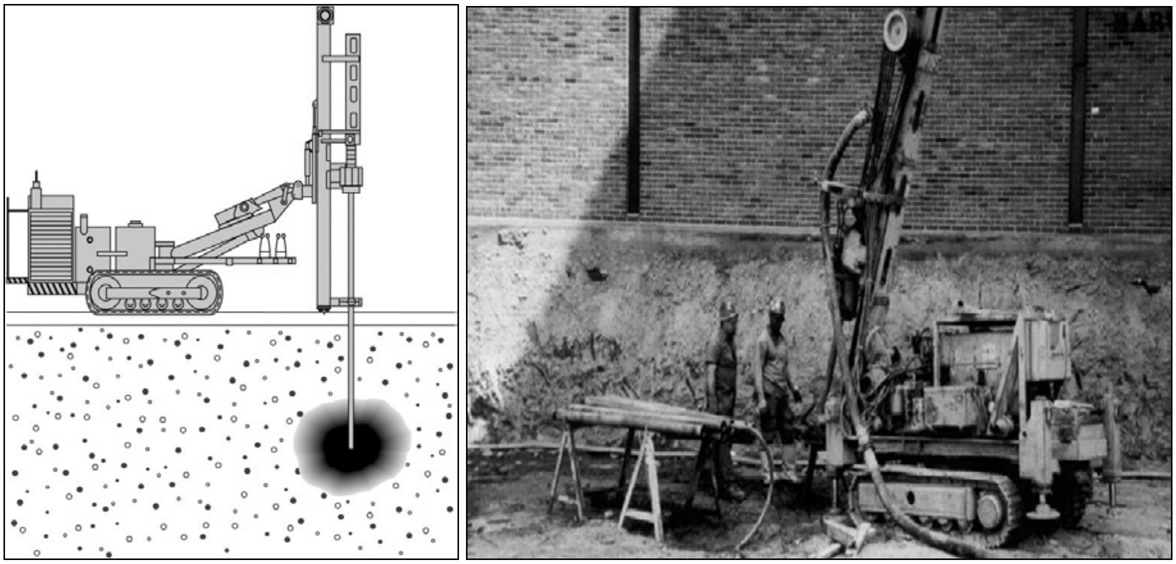


FIGURE 12.7
Compaction grout process: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

After the grout sets up, the pipe is drilled to the underside of the grout and additional grout is injected. This procedure is repeated until the bottom of the treatment zone is grouted. The grout injection rate is generally in the range of 3 to 6 ft3/min (0.087 to 0.175m3/min), depending on the soils being treated. If the injection rate is too fast, excess pore pressures or fracturing of the soil can occur, reducing the effectiveness of the process.

Materials: Generally, the compaction grout consists of Portland cement, sand, and water. Additional fine-grained materials can be added to the mix, such as natural fine-grained soils, fly ash, or bentonite (in small quantities). The grout strength is generally not critical for soil improvement, and if this is the case, cement has been omitted and the sand replaced with naturally occurring silty sand. A minimum strength may be required if the grout columns or mass are designed to carry a load.

Design: The design will begin with an analysis of the planned construction with the existing subsurface conditions (bearing capacity, settlement, liquefaction, etc.). Then the same analysis is performed with the improved soil parameters (i.e., SPT N value, etc.) to determine the minimum parameters necessary to provide the required performance. Finally, the vertical and lateral extent of improved soil necessary to provide the required performance is determined. In the case of settlement improvement for spread footings, it is common to improve the sands beneath the planned footings to a depth of twice the footing width for isolated column footings and four times the footing width for wall footings. A conservative analysis of the post-treatment performance only considers the improved soil and does not take into account the grout elements. The grout elements are typically columns. A simplified method of accounting for the grout columns is to take a weighted average of the parameters of the improved soil and grout. The grout columns can also be designed using a standard displacement pile methodology.

The degree of improvement achievable depends on the soil (soil gradation, percent fines, percent clay fines, and moisture content) as well as the spacing and percent displacement (the volume of grout injected divided by volume of soil being treated).

Quality control and quality assurance:

Depending on the grout requirements, grout slump and strength is often specified. Slump testing and sampling for unconfined compressive strength testing is performed during production. The production parameters should also be monitored and documented, such as pumping rate, quantities, pressures, ground heave, and injection depths. Post grouting penetration testing can be performed between injection locations to verify the improvement of granular soil.[7]

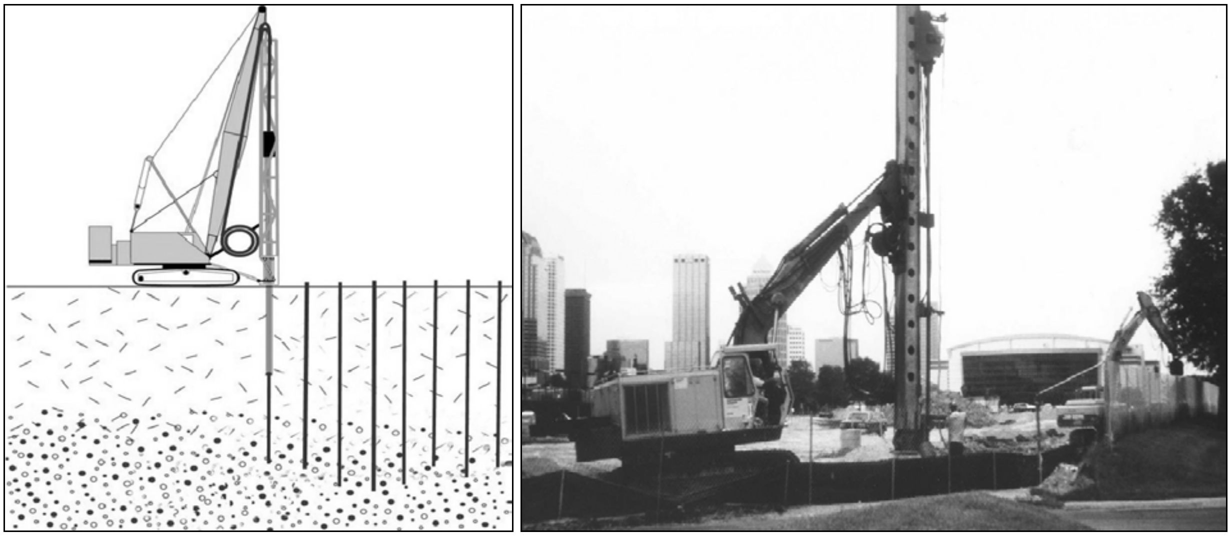
Advantages [8]	**Disadvantages**
•Densifies soil •Increases lateral stress •Reinforces soil mass •Works well in low-overhead + constricted spaces •Can target specific depth interval •Effective with high fines soil (>20%) + large particles (cobbles, boulders)	•Ineffective for small depths (<6 meters) – grout pressures can heave ground surface •Grout bulbs are brittle = tendency to crack with earthquake shaking

2.1.4 Surcharging with Prefabricated Vertical Drains

Surcharging consists of placing a temporary load (generally soil fill) on sites to pre-consolidate the soil prior to constructing the planned structure (Figure 12.9). The process improves the soil by compressing the soil, increasing its stiffness and shear strength. In partially or fully saturated soils, prefabricated vertical drains (PVDs) can be placed prior to surcharge placement to accelerate the drainage, reducing the required surcharge time. Applicable soil types: Preloading is best suited for soft, fine-grained soils. Soft soils are generally easy to penetrate with PVDs and layers of stiff soil may require pre-drilling.

Equipment: Generally, a surcharge consists of a soil embankment and is placed with standard earthmoving equipment (trucks, dozers, etc). Often the site surface is soft and wet, requiring low ground pressure equipment. The PVDs are installed with a mast mounted on a backhoe or crane, often with low ground pressure tracks. A pre-drilling rig may be required if stiff layers must be penetrated.

Procedure: Fill soil is typically delivered to the area to be surcharged with dump trucks. Dozers are then used to push the soil into a mound. The height of the mound depends on the required pressure to achieve the required improvement. The PVDs typically are in 1000 ft (308 m) rolls and are fed into a steel rectangular tube (mandrel) from the top. The mandrel is pushed, vibrated, driven or jetted vertically into the ground with a mast mounted on a backhoe or crane. An anchor plate or bar attached to the bottom of the PVD holds it in place in the soil as the mandrel is extracted. The PVD is then cut off slightly above the ground surface and another anchor is attached. The mandrel is moved to the next location and the process is repeated. If obstructions are encountered during installation, the wick drain location can be slightly offset. In very soft sites, piezometers and inclinometers, as well as staged loading, may be required to avoid the fill being placed too quickly, causing a bearing capacity or slope stability failure. If stiff layers must be penetrated, pre-drilling may be required.



I. a) (b)

FIGURE 12.9

Surcharging with prefabricated vertical drains: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)[7]

Settlement plates are placed in the surcharge. The elevation of these plates is measured to determine when the design settlement has occurred.

Materials: The first layer of surcharge generally consists of a drainage material to drain the water displaced from the ground during compression. Since surcharge soils are generally temporary in nature, their composition and degree of compaction are generally not critical. If the site settlement will result in some of the surcharge soil settling below finish grade, this height of fill is initially placed as compacted structural fill, to avoid having to excavate and replace it at the end of the surcharge program.

The PVD is composed of a 4-in. (10 cm) wide strip of corrugated or knobbed plastic wrapped in a woven filter fabric. The fabric is designed to remain permeable to allow the ground water to flow through it but not the soil.

Design: Generally, a surcharge program is considered when the site is underlain by soft fine-grained soils which will experience excessive settlement under the load of the planned structure. Using consolidation test data, a surcharge load and duration is selected to pre consolidate the soils sufficiently such that when the surcharge load is removed and the planned structure is constructed, the remaining settlement is acceptable.

PVDs are selected if the required surcharge time is excessive for the project. The time required for the surcharge settlement to occur depends on the time it takes for the excess pore water pressure to dissipate. This is dictated by the soils permeability and the square of the distance the water has to travel to get to a permeable layer. The PVDs accelerate the drainage by shortening the drainage distance. The spacing of the PVDs is designed to reduce the consolidation time to an acceptable duration. The closer the drains are installed (typically 3 to 6 ft on center) the shorter the surcharge program is in duration.

Quality control and quality assurance:
The height and unit weight of the surcharge should be documented to assure that the design pressure is being applied. The PVD manufacturer's specifications should be reviewed to confirm that the selected PVD is suitable for the application. During installation, the location, depth, and verticality are important to monitor and record. The settlement monitoring program is critical so that the completion of the surcharge program can be determined.[7]

2.1.5 Infrequently-Used Compaction Techniques

Blast-Densification and Vacuum-Induced Consolidation

Blast-densification densifies sands with underground explosives. The technique was first used in the 1930s in the former Soviet Union and in New Hampshire. The below grade explosion causes volumetric strains and shearing which rearranges of soil particles into a denser configuration. The soils are liquefied and then become denser as the pore pressures dissipate. Soils as deep as 130 ft (40 m) have been treated. A limited number of projects have been performed and generally only for remote location where the blast induced vibrations are not a concern.[7]

Loose granular soils have also been compacted by blasting. Blasting densification involves the detonation of multiple explosive charges vertically spaced 10 to 20ft (3 to 6m) apart in drilled or jetted boreholes. The boreholes are usually spaced between 15 to 50ft (5 to 15m) apart and backfilled prior to detonation. To increase the efficiency of the densification process, the charges at different elevations may be detonated at time delays. Immediately after detonation, the ground surface rises and gas and water are expelled from fractures (fig12.4).the ground surface then settles as the excess gas and water pressure dissipates. Although the efficiency of the densification decreases with each round of blasting, two or three rounds (with later rounds detonated at locations between those of the earlier rounds) are often used to achieve the desired degree of densification.

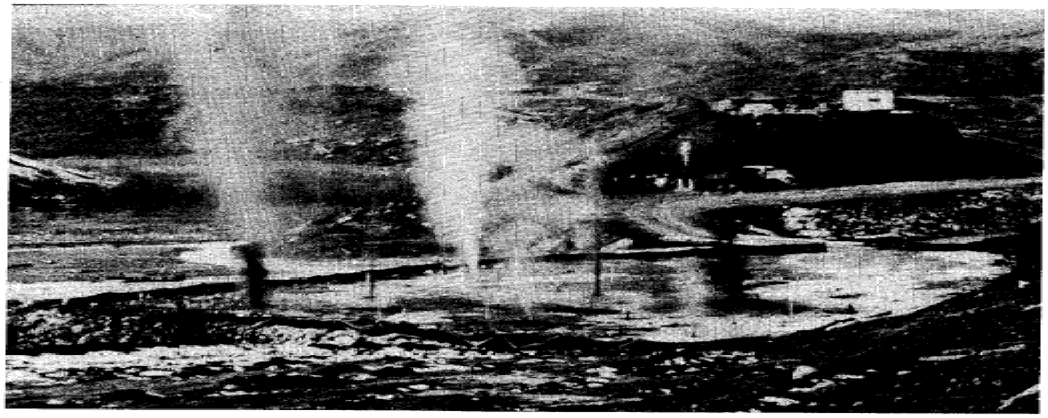


Figure 12.4 Ground surface shortly after detonation of explosives during blast densification of loose soil beneath an abutment prior to construction of Coldwater Creek bridge near Mt. St. Helens in Washington state. (Photo by A. P. Kilian; used with permission).

Applicable soil type
Blasting is most effective in loose sands that contain less than20% silt and less than 5% clay. Even small amounts of clay, or small clay seams, can substantially reduce the effectiveness of blasting. Blasting can be effective in dry soils, but the effects of capillary tension and gas bubbles in partially saturated soils virtually negates its effectiveness. As a result, blasting is most commonly used to densify completely saturated soils. In such soils the shock wave produced by the charges produces localized, temporary liquefaction which allows the soil grains to move into a denser configuration.

Applicability of Blast densification
Although blasting is quite economical, its use is limited by several practical considerations. It produces strong vibrations that may damage nearby structures or produce significant ground movements. It requires the use of potentially hazardous explosives for which strict regulations on handling and storage usually apply. Finally, its effectiveness is difficult to predict in advance.

Case histories of the use of blasting to mitigate seismic hazards have been described by Klohn et al.(1981),solymar and Reed(1986),LaFosse and von Rosenvinge(1992),and hachey et al.(1994).[5]

Vacuum-induced consolidation (VIC) uses atmospheric pressure to apply a temporary surcharge load. The concept of VIC was introduced in the 1950s; however, the first practical project was performed in 1980 in China. Following that, a number of small projects have been performed, but few outside China. A porous layer of sand or gravel is placed over the site and it is covered with an air tight membrane, sealed into the clay below the ground surface. The air is then pumped out of the porous layer, producing a pressure difference of 0.6 to 0.7 atm, equivalent to about 15 ft (4.6 m) of fill. The process can be accelerated by the use of PVDs. The process eliminates the need for surcharge fill and avoids shear failure in the soft soil; however, any sand seams within the compressible layer can make it difficult to maintain the vacuum. [7]

2.2 Reinforcement techniques

2.2.1 Stone Column

Stone columns refer to columns of compacted, gravel size stone particles constructed vertically in the ground to improve the performance of soft or loose soils. The stone can be compacted with impact methods, such as with a falling weight or an impact compactor or with a vibroflot, the more common method. The method is used to increase bearing capacity (up to 5 to 10 ksf or 240 to 480 kPa), reduce foundation settlements, improve slope stability, reduce seismic subsidence, reduce lateral spreading and liquefaction potential, permit construction on loose/soft fills, and pre-collapse sinkholes prior to construction in karst regions.

Applicable soil types: Stone columns improve the performance of soils in two ways, densification of surrounding granular soil and reinforcement of the soil with a stiffer, higher shear strength column. The expected improvement achieved in specific soil types is shown in Table 12.4. The depth of the ground water is generally not critical.

Procedure: The column construction starts at the bottom of the treatment depth and proceeds to the surface. The vibrator penetrates into the ground, assisted by its weight, vibration, and typically water jets in its tip, the wet top feed method (Figure 12.10 and Figure 12.11a). If difficult penetration is encountered, pre-drilling through the firm soils may also be performed. A front end loader places stone around the vibroflot at the ground surface and the stone falls to the tip of the vibroflot through the flushing water around the exterior of the vibroflot. The vibrator is then raised a couple of feet and the stone falls around the vibroflot to the tip, filling the cavity formed as the vibroflot is raised. The vibroflot is then repeatedly raised and lowered as it is extracted; compacting and displacing the stone in 2 to 3 ft (0.75 to 0.9 m) lifts. The flushing water is usually directed to a settlement pond where the suspended soil fines are allowed to settle.

If the dry bottom feed procedure is selected, the vibroflot penetrates into the ground, assisted by its weight and vibrations alone (Figure 12.11b). Again, pre-drilling may be used if necessary or desired. The remaining procedure is then similar except that the stone is feed to the tip of the vibroflot though the tremie pipe. Treatment depth as deep as 100 ft (30 m) has been achieved.

TABLE 12.4

Expected Densification and Reinforcement Achieved with Stone Columns

Soil Description	Densification	Reinforcement
Gravel and sand <10% silt, no clay	Excellent	Very good
Sand with between 10 and 20% silt and <2% clay	Very good	Very good
Sand with >20% silt and non plastic silt	Marginal (with large displacements)	Excellent
Clays	Not applicable	Excellent

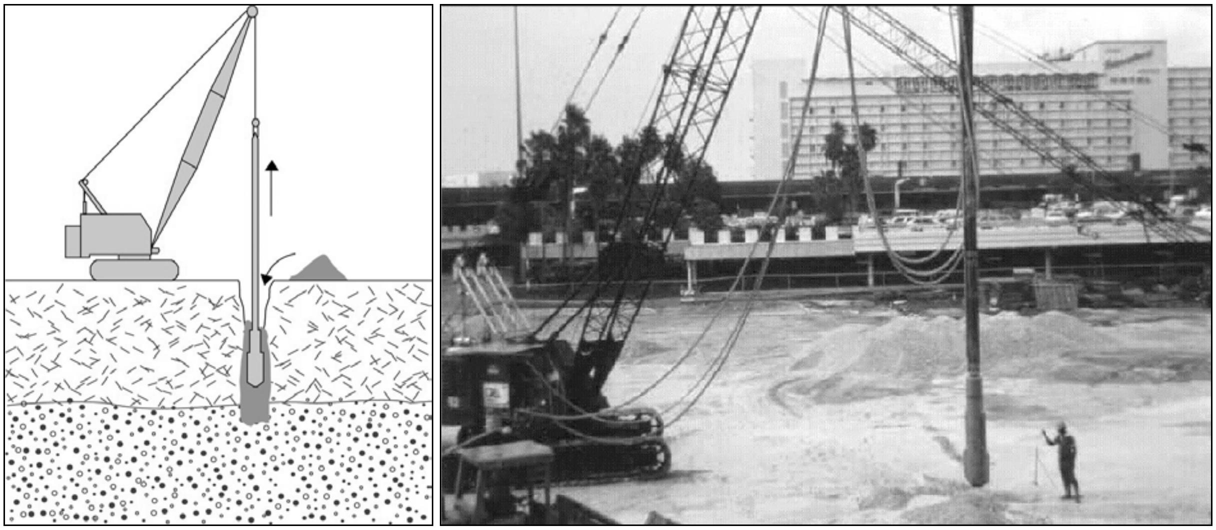
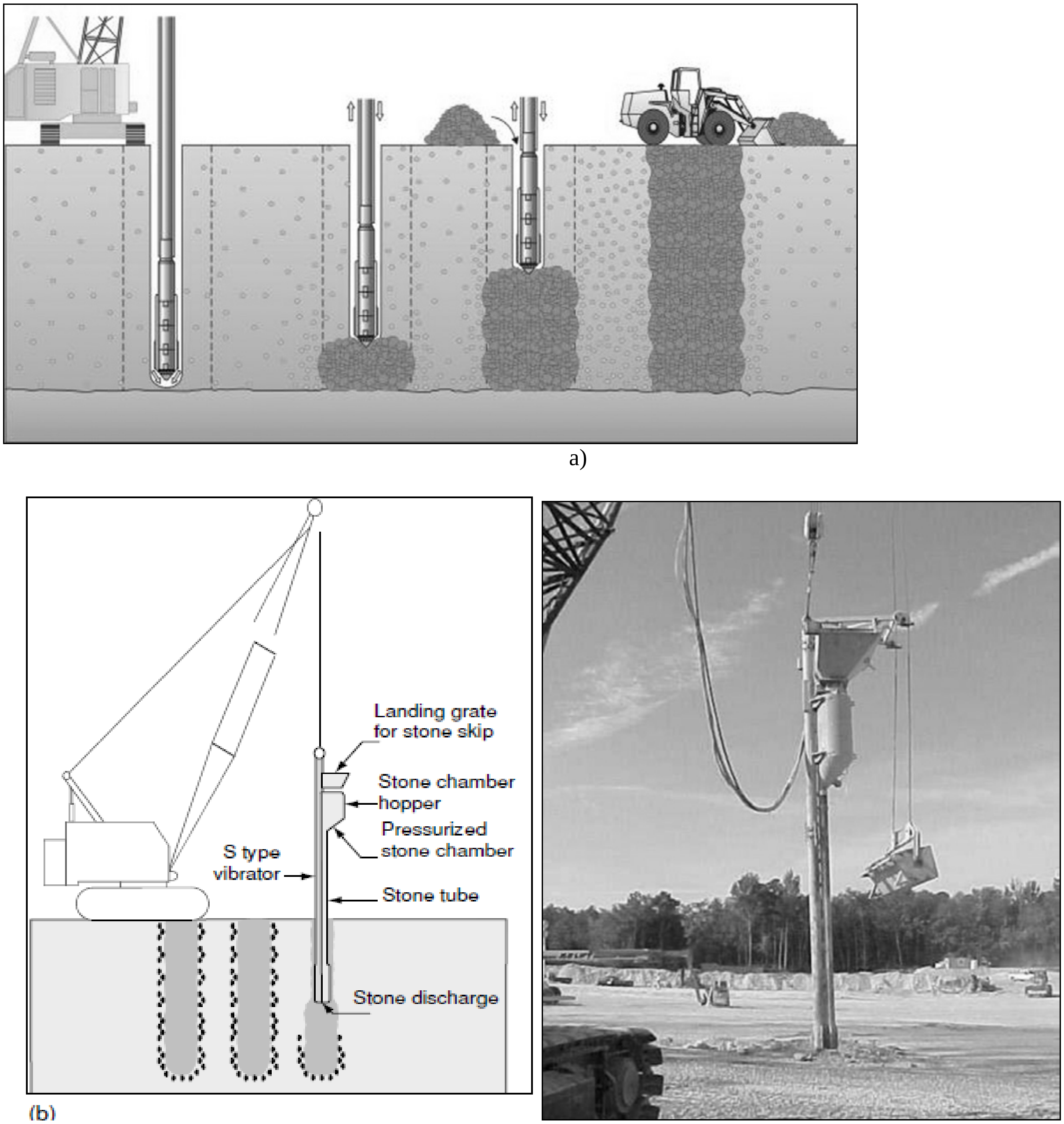


FIGURE 12.10
Installation of stone columns: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

Equipment: When jetting water is used to advance the vibroflot, the equipment and setup is similar to VC. If jetting water is not desired for a particular project, the dry bottom feed process can be used (Figure 12.11b). A tremie pipe, through which stone is fed to the tip of the vibroflot, is fastened to the side of the vibroflot. A stone skip is filled with stone on the ground with a front end loader and a separate cable raises the skip to a chamber at the top of the tremie pipe.

A specific application is referred to as vibro piers. The process refers to short, closely spaced stone columns designed to create a stiff block to increase bearing capacity and reduce settlement to acceptable values. Vibro piers are typically constructed in cohesive soils in which a full depth predrilled hole will stay open. The stone is compacted in 1 to 2 ft (0.4 to 0.8 m) lifts, each of which is rammed and compacted with the vibroflot.[7]



a)

(c)

FIGURE 12.11 Stone column construction: (a) wet top feed method, (b) schematic, and (c) field implementation of dry bottom feed method. (From Hayward Baker Inc. With permission.)

Materials: The stone is typically a graded crushed hard rock, although natural gravels and pebbles have been used. The greater the frictions angle of the stone, the greater the modulus and shear strength of the column.

Design: Several methods of analysis are available. For static analysis, one method consists of calculating weighted averages of the stone column and soil properties (cohesion, friction angle, etc.). The weighted averages are then used in standard geotechnical methods of analysis (bearing capacity, settlement, etc.). Another method developed by Dr Hans Priebe, involves calculating the post-treatment settlement by dividing the untreated settlement by an improvement factor (Figure 12.12). In static applications, the treatment limits are typically equal to the foundation limits.

For liquefaction analysis, stone column benefits include densification of surrounding granular soils, reduction in the cyclic stress in the soil because of the inclusion of the stiffer stone columns, and drainage of the excess pore pressure. A method of evaluation for all three of these benefits was presented by Dr Juan Baez. Dr Priebe has also presented a variation of his static method for this application. In liquefaction applications, the treatment generally covers the structure footprint and extends laterally outside the areas to be protected, a distance equal to two-thirds of the thickness of the liquefiable zone. This is necessary to avoid surrounding untreated soils from adversely affecting the treated area beneath the foundation.[7]

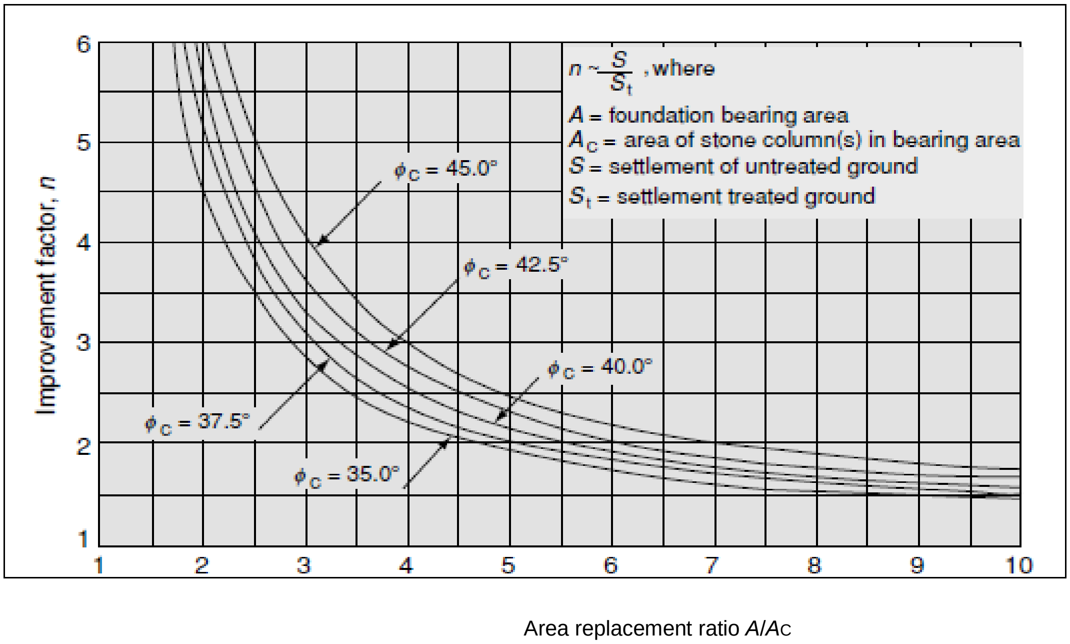


FIGURE 12.12
Chart to estimate improvement factor with stone columns.

Loads through soft cohesive and organic soils. The method is used to reduce foundation settlements, to increase bearing capacity, to increase slope stability, and as an alternative to piling.

Quality control and quality assurance: During production, important parameters to monitor and document include location, depth, ammeter increases (see Section 12.2.2), and quan tity of stone backfill used. Post-treatment penetration testing can be performed to measure the improvement achieved in granular soils. Full-scale load tests are becoming common with test footings measuring as large as 10 ft square (3.1 m) and loaded to 150% of the design load (Figure 12.13).[7]

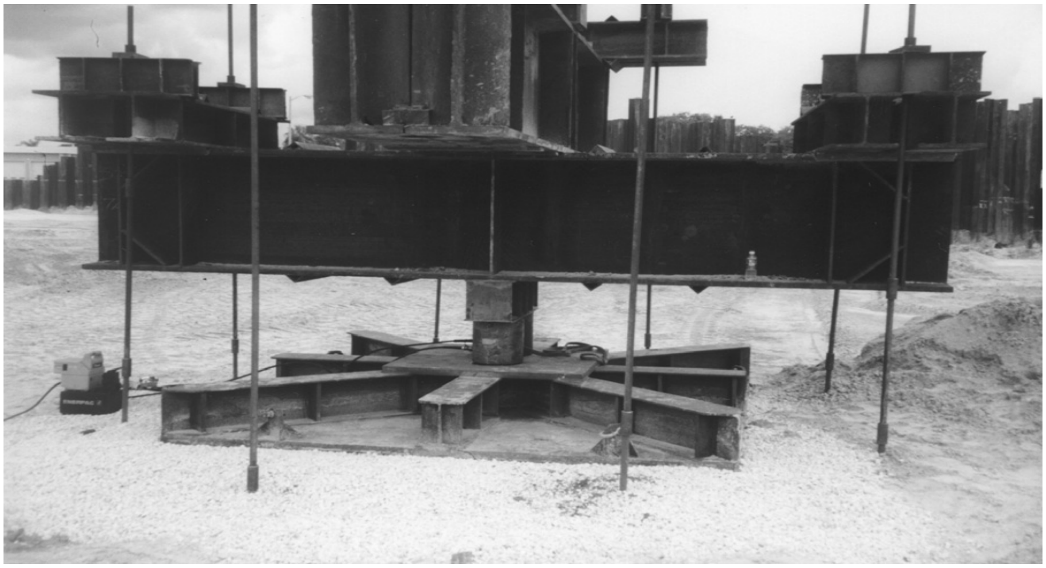


FIGURE 12.13Full-scale load test (10 ft or 3.1m2, loaded to 15 ksf or 719 kPa). (From Hayward Baker Inc. With permition

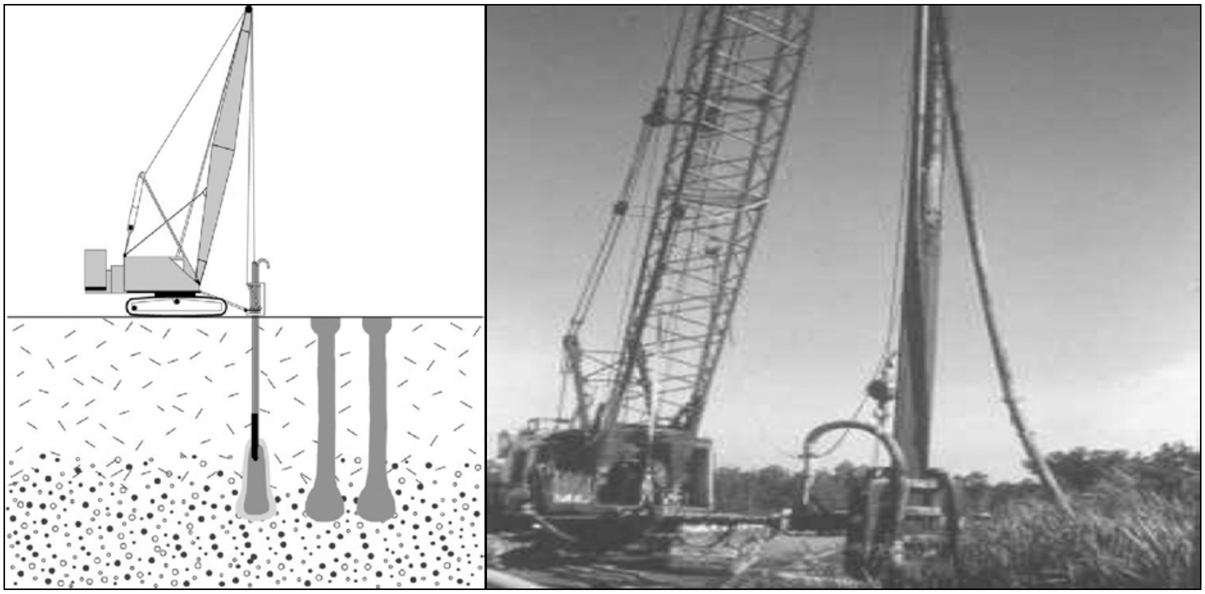
2.2.2 Vibro Concrete Columns

Vibro concrete columns (VCCs) involve constructing concrete columns in situ using a bottom feed vibroflot (Figure 12.14). The method will densify granular soils and transfer loads through soft cohesive and organic soils. The method is used to reduce foundation settlements, to increase bearing capacity, to increase slope stability, and as an alternative to piling.

Applicable soil types: VCCs are best suited to transfer area loads, such as embankments and tanks, through soft and/or organic layers to an underlying granular layer. The depth of the groundwater table is not critical.

Equipment: The equipment is similar to the bottom feed stone column setup. A concrete hose connects a concrete pump to the top of the tremie pipe. Since verticality is important, the vibroflot is often mounted in a set of leads or a spotter.

Procedure: The vibroflot is lowered or pushed through the soft soil until it penetrates into the bearing stratum. Concrete is then pumped as the vibroflot is repeatedly raised and lowered about 2 ft (0.75 m) to create an expanded base and densifying surrounding granular soils. The concrete is pumped as the vibroflot is raised to the surface. At the ground surface, the vibroflot is again raised and lowered several times to form an expanded top. Most VCC applications are less than 40 ft (12.3 m) in depth.[7]



(a) (b)

FIGURE 12.14
Installation of vibro concrete columns: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

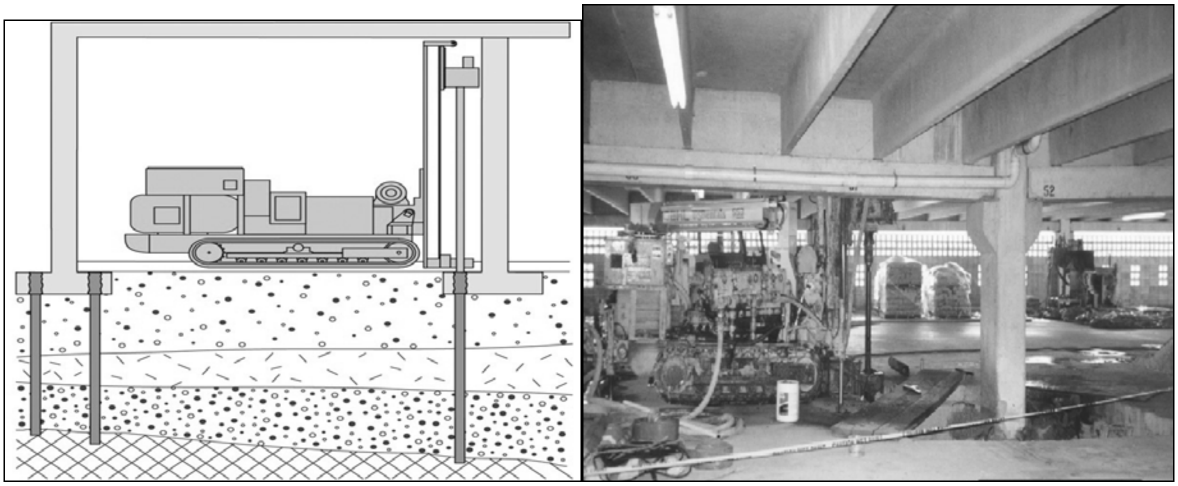
Materials: Concrete or cement mortar grout is typically used. The mix design depends on the requirements of the application.

Design: The analysis and design of VCCs are essentially the same as would be performed for an expanded base pile except that the improved soil parameters are used.

Quality control and quality assurance: During production, important parameters to monitor and document include location, depth, verticality, injection pressure and quantity, and concrete quality. It is very important to monitor the pumping and extraction rates to verify that the grout pumping rate matches or slightly exceeds the rate at which the void is created as the vibroflot is extracted. VCCs can be load tested in accordance with ASTM D 1143.[7]

2.2.3 Micropiles

Micropiles, also known as minipiles and pin piles, are used in almost any type of ground to transfer structural load to competent bearing strata (Figure 12.17). Micro piles were originally small diameter (2 to 4 in., or 5 to 10 cm), low-capacity piles. However, advances in drilling equipment have resulted in design load capacities in excess of 300 tons (2.7MN) and diameters in excess of 10 in. (25 cm). Micropiles are often installed in restricted access and limited headroom situations. Micropiles can be used for a wide range of applications; however, the most common applications are underpinning existing foundations or new foundations in limited headroom and tight access locations.[7]



(a) (b)

FIGURE 12.17
Micropiling: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

Applicable soil types: Since micropiles can be installed with drilling equipment and can be combined with different grouting techniques to create the bearing element, they can be used in nearly any subsurface soil or rock. Their capacity will depend on the bearing soil or rock.

Equipment: The micropile shaft is usually driven or drilled into place. Therefore, a drill rig or small pile driving hammer on a base unit is required. The pipe is filled with a cement grout so the appropriate grout mixing and pumping equipment is required. If the bearing element is to be created with compaction grout or jet grout, the appropriate grouting equipment is also required.

Procedure: The micropile shaft is usually either driven or drilled into place. Unless the desired pile capacity can be achieved in end bearing and side friction along the pipe, some type of bearing element must be created (Figure 12.18). If the tip is underlain by rock, this could consist of drilling a rock socket, filling the socket with grout and placing a full length, high-strength threaded bar. If the lower portion of the pipe is surrounded or underlain by soil, compaction grouting or jet grouting can be performed below the bottom of the pipe. Also, the pipe can be filled with grout which is pressurized as the pipe is partially extracted to create a bond zone. The connection of the pipe to the existing or planned foundation must then be constructed.

Materials: The micropile typically consists of a steel rod or pipe. Portland cement grout is often used to create the bond zone and fill the pipe. A full length steel threaded bar is also common, composed of grade 40 to 150 ksi steel. In some instances, the micropile only consists of a reinforced, grout column.

Design: The design of the micropile is divided into three components: the connection with the existing or planned structure, the pile shaft which transfers the load to the bearing zone, and the bearing element which transfers the load to the soil or rock bearing layer.

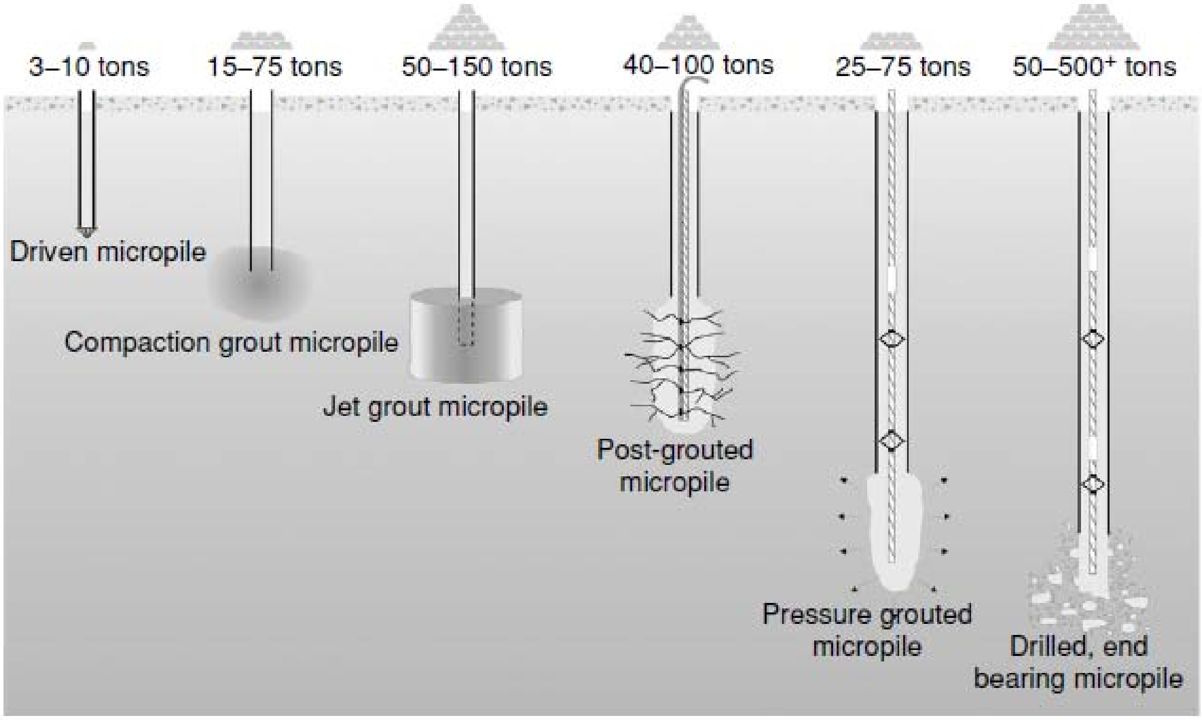


FIGURE 12.18
Sample of micropile bearing elements. (From Hayward Baker Inc. With permission.)

A standard structural analysis is used to design the pile section. If a grouted friction socket is planned, Table 12.5 can be used to estimate the sockets diameter and length. Bond lengths in excess of 30 ft (9.2 m) do not increase the piles capacity.

Quality control and quality assurance: During the construction of the micropile, the drilling penetration rate can be monitored as an indication of the stratum being drilled. Grout should be sampled for subsequent compressive strength testing. The piles verticality and length should also be monitored and documented. A test pile is constructed at the beginning of the work and load tested to 200% of the design load in accordance with the standard specification ASTM D 1143 Fig 12.19[7]

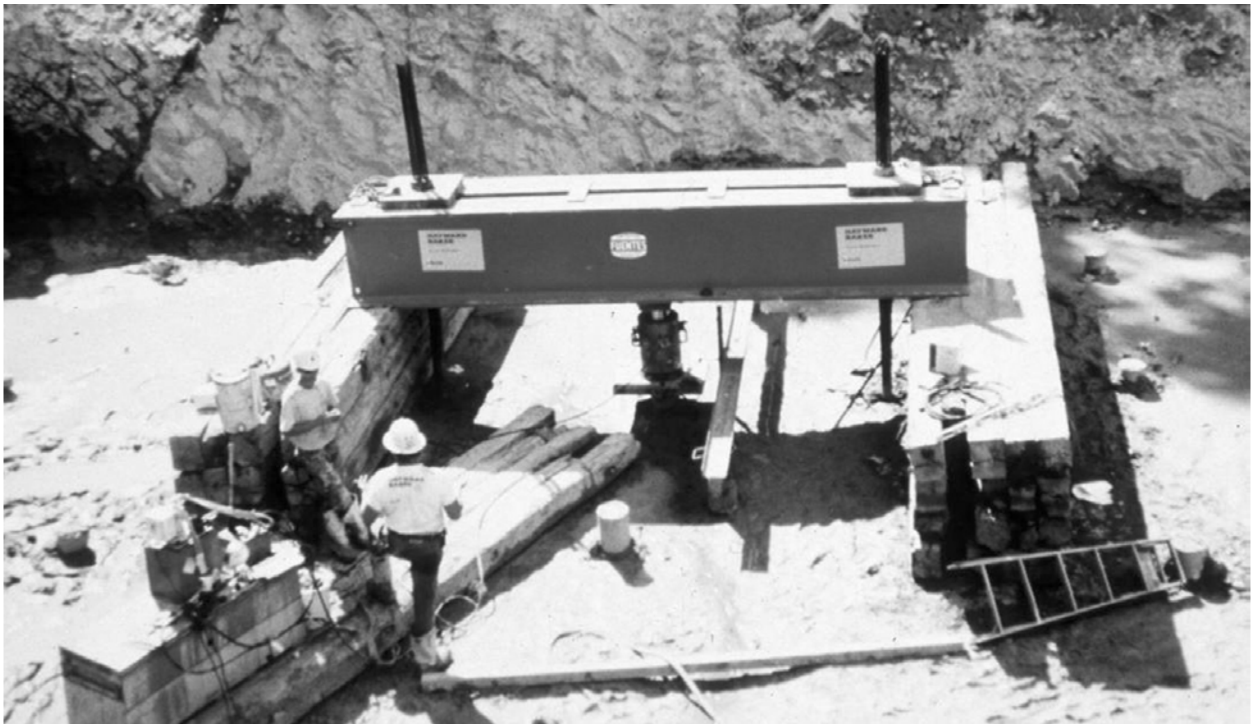


FIGURE 12.19
Micropile load test. (From Hayward Baker Inc. With permission.)

2.2.4 Fracture Grouting

Fracture grouting, also known as compensation grouting, is the use of grout slurry to hydro-fracture and inject the soil between the foundation to be controlled and the process causing the settlement (Figure 12.20). Grout slurry is forced into soil fractures, thereby causing an expansion to take place counteracting the settlement that occurs or producing a controlled heave of the foundation. Multiple, discrete injections at multiple elevations can create a reinforced zone. The process is used to reduce or eliminate previous settlements, or to prevent the settlement of structures as underlying tunneling is performed.

A variation of fracture grouting is injection systems for expansive soils. The technique reduces the post-treatment expansive tendencies of the soil by either raising the soils' moisture content or filling the desiccation patterns in the clay or chemically treating the clay to reduce its affinity to water.

Applicable soil types: Since the soil is fractured, the technique can be performed in any soil type.
Equipment: For fracture grouting, the equipment consists of a drill rig to install the sleeve port pipes, grout injection tubing with packers, grout mixer, and a high-pressure grout pump.

TABLE 12.5

Estimated Soil and Rock Bond Values for Micropiles

Soil/Rock Description	SPT *N* value (blows/ft)	Grout Bond with Soil/Rock (ksf)[a]
Non pressure grouted		
Silty clay	3–6	0.5–1.0
Sandy clay	3–6	0.7–1.0
Medium clay	4–8	0.75–1.25
Firm clay or stiffer	>8	1.0–1.5
Sands	10–30	2–4
Soft shales		5–15
Slate and hard shales		15–28
Sandstones		15–35
Soft limestone		15–33
Hard limestone		20–35
Pressure grouted		
Medium dense sand		3.5–6.5
Dense sand		5.5–8.5
Very dense sand		8–12

[a] Design values, 1 ft ¼ 0.308 m, 1 ksf ¼ 47.9 kPa.

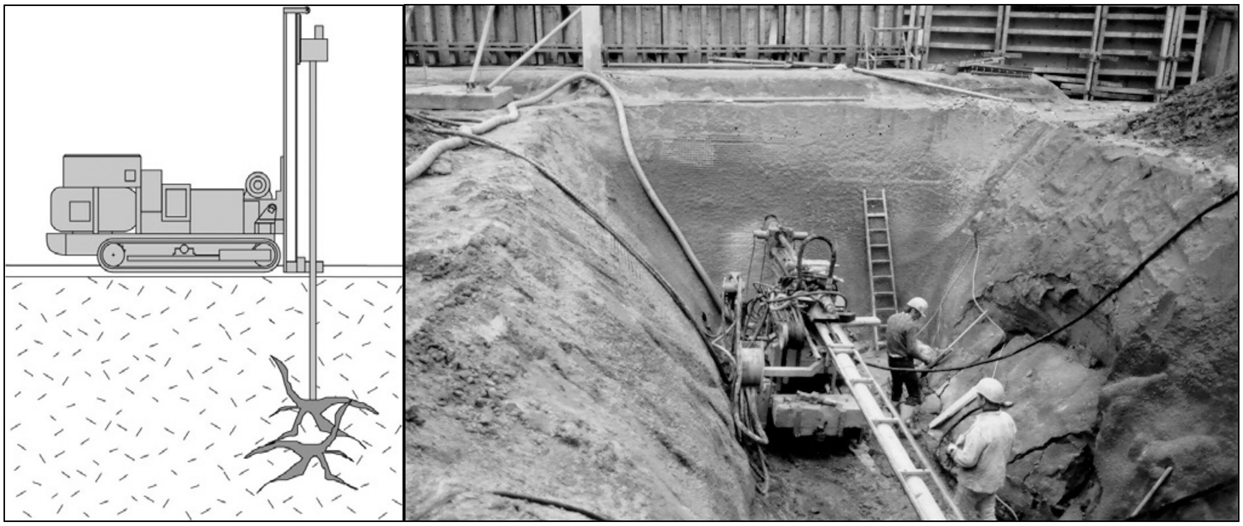

(a) (b)

FIGURE 12.20 Fracture grouting: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

A sleeve port pipe is a steel or PVC pipe with openings at regular intervals along its length to permit grout injection at multiple locations along the pipes length. Also a precise real-time level surveying system is often required to measure the movements of the structure or the ground surface.

For injection of expansive soils, the equipment generally consists of a track mounted rig that pushes multiple injection pipes into the ground at the same time (Figure 12.21). A mixing plant, storage tank and pump prepare, store, and deliver the solution to be injected.

Procedure: For fracture grouting beneath existing structures, large diameter shafts (10 to 15 ft, or 3 to 4.6 m, in diameter) or pits are constructed adjacent to the exterior of the structure to be controlled. From these shafts, a drill rig installs the sleeve port pipes horizontally beneath the structure. Then a grout injection tube is inserted into the sleeve port pipe. Packers on the injection tube are inflated on either side of an individual port and grout is injected. The packers are then deflated, the injection tube moved to another port, and the process repeated as necessary to achieve either the desired heave or prevent settlement. A level surveying system provides information on the response of the ground and overlying structure which is used to determine the location and quantity of the grout to be injected.

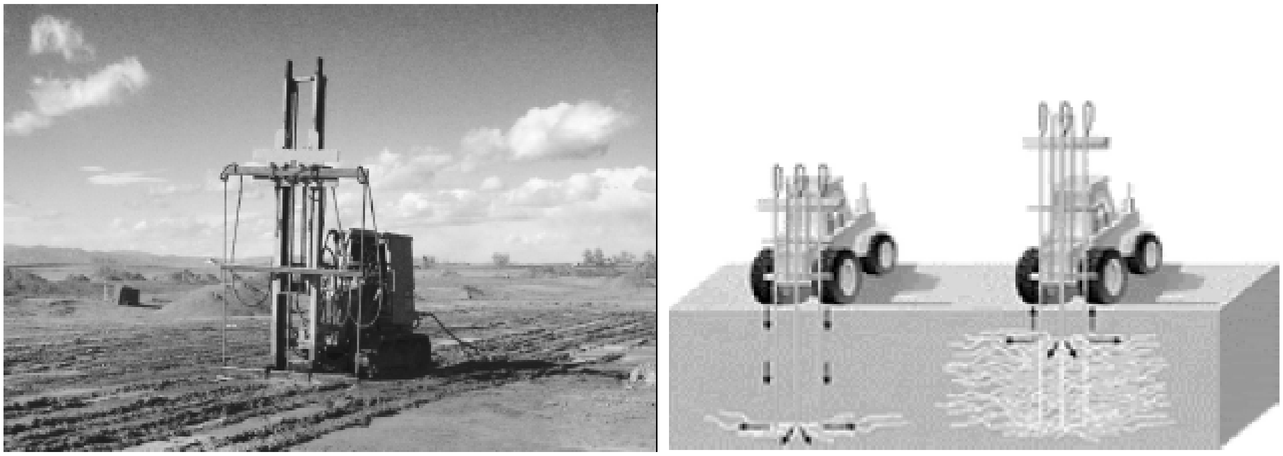


FIGURE 12.21
Injection rig for treatment of expansive soils. (From Hayward Baker Inc. With permission.)

For injection of expansive soils, multiple injection rods are typically pushed into the ground to the desired treatment depth (typically 7 to 12 ft, or 2.2 to 3.7 m) and then an aqueous solution is injected as the rods are extracted.

Materials: For fracture grouting beneath structures, the grout typically consists of Portland cement and water.

For injection of expansive soils, the following solutions have been used:

- Water — used to swell expansive clays as much as possible prior to construction.
- Lime and fly ash — used to fill the desiccation pattern of cracks, reducing the avenues of moisture change.
- Potassium chloride and ammonium lignosulfonate — used to chemically treat the clay and reduce its affinity for water.[7]

Design: For fracture grouting beneath a structure, the design involves identifying the strata which has or will result in settlement, and placing the injection pipes between the shallowest stratum and the structure. For injection of expansive soils, the design includes identifying the lateral and vertical extents of the soils requiring treatment.

Quality control and quality assurance: For fracture grouting beneath existing structures, it is critical to know where all the injection ports are located, both horizontally and vertically. The monitoring of the overlying structure is then critical so that the affected portion of the structure is accurately identified and the injection is performed in the correct ports.

For injection of expansive soil, acceptance is typically based on increasing the in situ moisture content to the plastic limit plus 2 to 3 moisture points, reducing pocket penetrometer readings to 3 tsf (288 kPa) or less, and reducing the average swell to 1% or less within the treatment zone.[7]

2.2.5 Infrequently-Used Reinforcement Techniques

Fibers and Biotechnical

Fiber reinforcement consists of mixing discrete, randomly oriented fibers in soil to assist the soil in tension. The use of fibers in soil dates back to ancient time but renewed interest was generated in the 1960s. Laboratory testing and computer modeling have been performed; however, field testing and evaluation lag behind. There are currently no standard guidelines on field mixing, placement and compaction of fiber-reinforced soil composites.

Biotechnical reinforcement involves the use of live vegetation to strengthen soils. This technique is typically used to stabilize slopes against erosion and shallow mass movements. The practice has been widely used in the United States since the 1930s. Recent applications have combined inert construction materials with living vegetation for slope protection and erosion control. Research has been sponsored by the National Science Foundation to advance the practice.[7]

2.2.6 Reinforced Soil Foundations

Bearing capacity of foundation soils can be improved using geogrids and geosythetics placed as a continuous single layer, closely spaced continuous multilayer set or mattress consisting of three-dimensional interconnected cells. Although standards on design of footings on reinforced soils are currently unavailable, Koerner (1998) provides some numerical guidelines on the extent of the improvement of bearing capacity and reduction of settlement. Figure 12.34(a) and (b) shows the results of laboratory tests where geotextiles were used to improve the bearing capacity of loose sands and saturated clay, respectively.

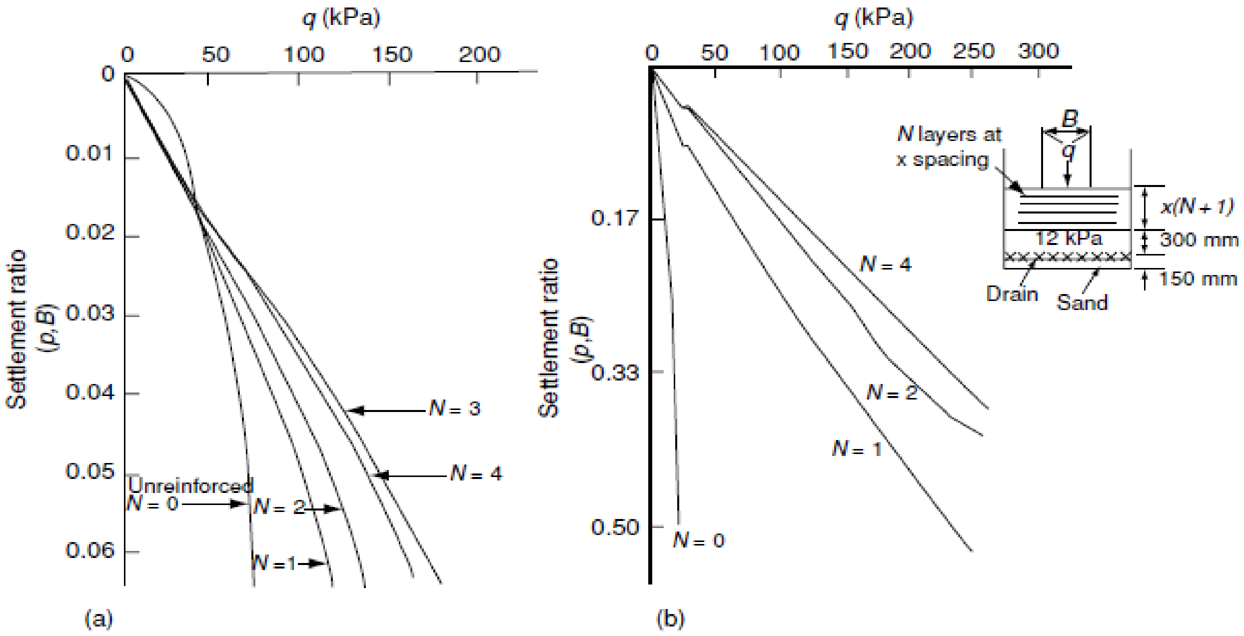


FIGURE 12.34
Improvement of soil bearing capacity with geotextiles: (a) loose sand, (b) saturated clay. (From Koerner, R., 1994.Designing with Geosynthetics, Prentice Hall, Englewood Cliffs, NJ. With permission.)

Figure 12.35 also shows the general approximations that the author has drawn from the results of large laboratory tests (Milligan and Love, 1984), which shows the improvement of settlement properties of saturated clay reinforced with geogrids. A large number of load tests have been conducted in the test pits at the Turner-Fairbank Highway Research Center (TFHRC) in Alaska, USA, to evaluate the effects of single and multiple layer of reinforcement placed below shallow spread footings (FHWA, 2001). In this test program, two different geosynthetics were evaluated; a stiff biaxial geogrid and a geocell. Parameters of the testing program include: number of reinforcement layers; spacing between reinforcement layers; depth to the first reinforcement layer; plan area of the reinforcement; type of reinforcement; and soil density. Test results indicated that the use of geosynthetic reinforced soil foundations may increase the ultimate bearing capacity of shallow spread footings by a factor of 2.5 (FHWA, 2001).[7]

2.2.6.1 Mechanisms of Bearing Capacity Failure in Reinforced Soils

In spite of the known favorable influence of geotextiles and geogrids on soil bearing capacity, the foundation designer needs to be aware of a number of mechanisms of bearing capacity failure even with reinforcements. These are discussed in Koerner (1998) as seen in Figure 12.36(a)–(b). Figure 12.36(a) shows the lack of reinforcement in the foundation influence zone while Figure 12.36(b) illustrates insufficient embedment of geotextiles or geogrids. Bearing capacity failures leads to inadequate tensile strength and excessive creep (long-term deformation) of reinforcements.[7]

These are discussed in Koerner (1998) as situations arising from;

- the lack of reinforcement in the foundation influence zone while Fig. 12.37
- Insufficient embedment of geotextiles or geogrids.
- bearing capacity failures leading to inadequate tensile strength, and
- excessive creep (long-term deformation) of reinforcements

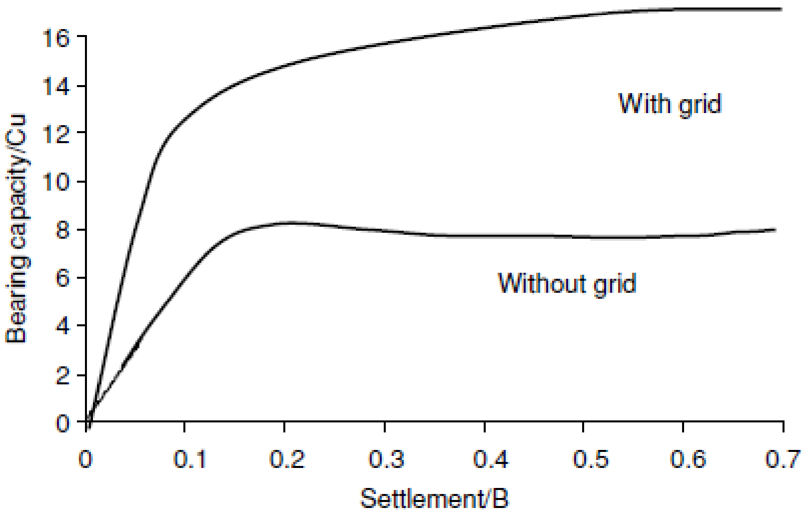
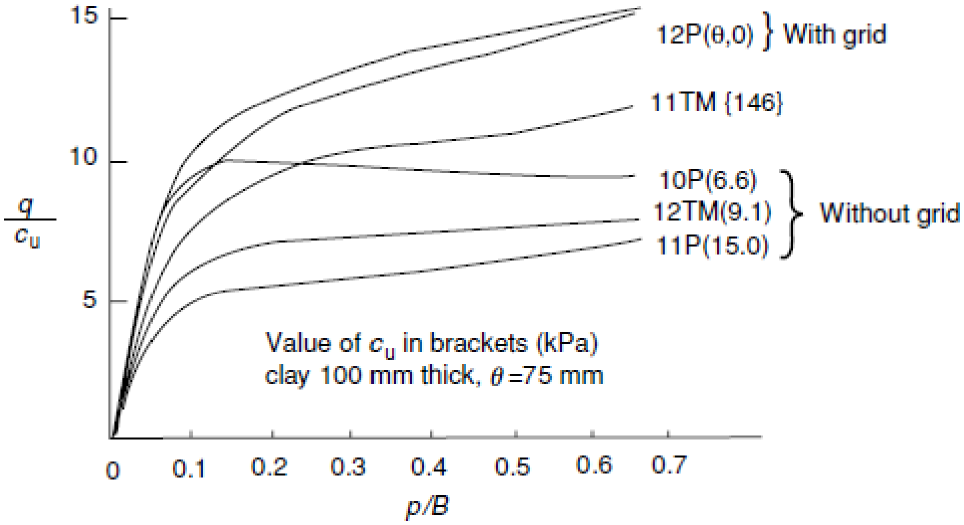


FIGURE 12.35

Improvement of settlement properties of saturated clays.



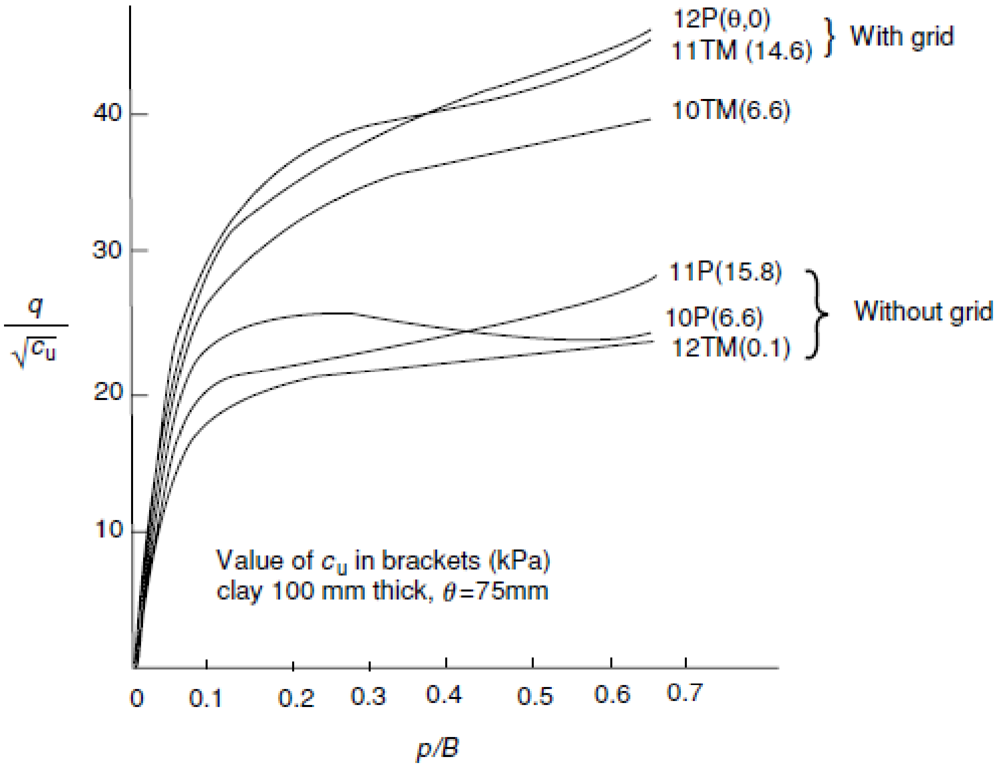


FIGURE 12.36

Improvement of settlement properties in saturated clay with geogrids. (From Koerner, R., 1994. Designing with Geosynthetics, Prentice Hall, Englewood Cliffs, NJ. With permission.)

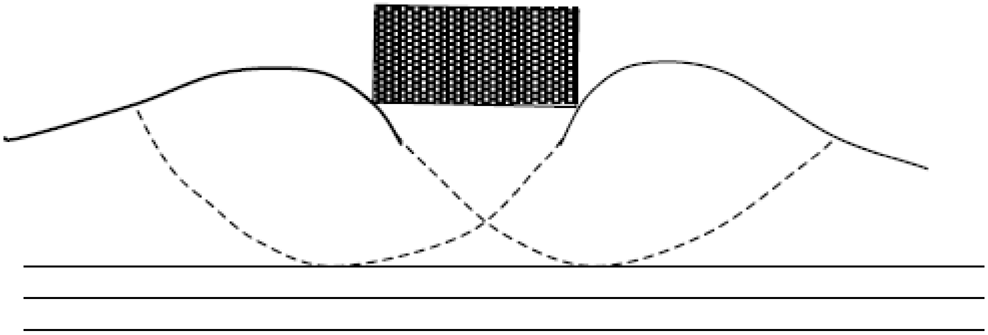


FIGURE 12.37

Lack of reinforcement in the foundation influence zone.

2.3 Grouting/Mixing/ techniques

2.3.1 Permeation Grouting

Permeation grouting is the injection of a fluid grout into granular, fissured or fractured ground to produce a solidified mass to carry increased load and/or fill voids and fissures to control water flow. The primary role of permeation grouting is to improve the strength, imperviousness and stiffness of soil or rock formations. The process is quiet flexible and it can be designed with a minimal disruption at the surface and therefore, it is advantageous for use in urban areas or areas with limited access.

The process is generally used to create a structural, load carrying mass, a stabilized soil zone for tunneling, and water cutoff barrier (Figure 12.22).[1]

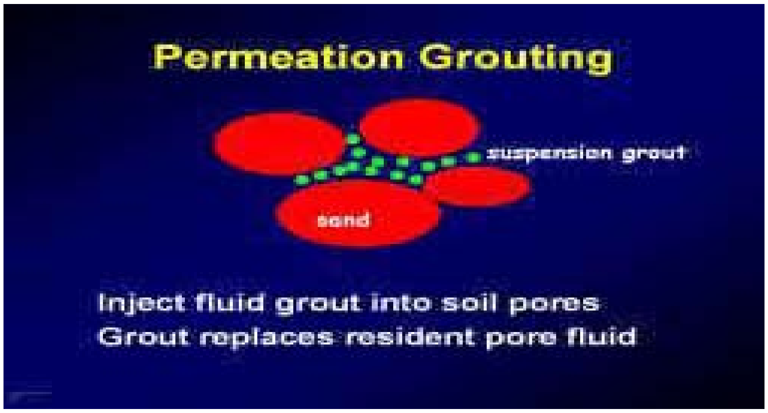


Figure 3.1. Basic concept of Permeation Grouting

Applicable soil types: The permeability requirement restricts the applicable soils to sands and gravels, with less than 18% silt and 2% clay. The depth of the groundwater table is not critical in free-draining soils, since the water will be displaced as the grout is injected. Loose sands will have reduced strengths when grouted compared to sands with SPT N values of 10 or greater.

Applicability of Permeation Grouting
The Permeation Grouting is very effective in sands, gravels, coarser size materials (e.g. boulders and cobbles) and fissured/ jointed/ fractured rock formations (refer to Figure 13). The technique is well suited for a wide variety of applications, such as foundation retrofitting, dam rehabilitation, subsidence and liquefaction mitigation, contaminant containments and barriers, tunneling and mining operations, offshore construction, etc. Applications can be categorized into the following general areas, site improvement, foundation rehabilitation, excavation support, groundwater control, and contaminant/pollution control (Karol ,1990).[1]

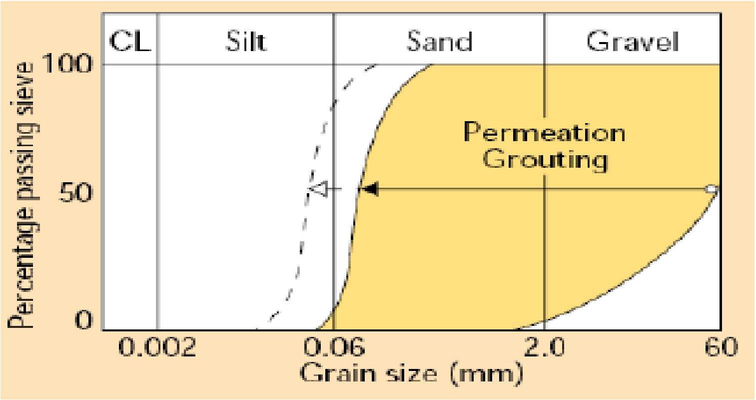


Figure 13. Applicable soil types for Permeation Grouting

Equipment: The mixing plant and grout pump vary depending on the type of grout used. Drill rigs typically install the grout injection pipe. The rigs can vary from very small to very large, depending on the project requirements. When the geometry of the grouted mass is critical, sleeve port pipes will be used.

Procedure: The grout can be mixed in batches (cementacious slurries) or stream mixed (silicates and other chemical grouts). Batch mixing involves mixing a selected volume of grout, possibly 1 yard3 or 0.79m3, and then injecting it before the next batch is mixed. The amount batched depends on the speed of injection and amount of time the specific grout

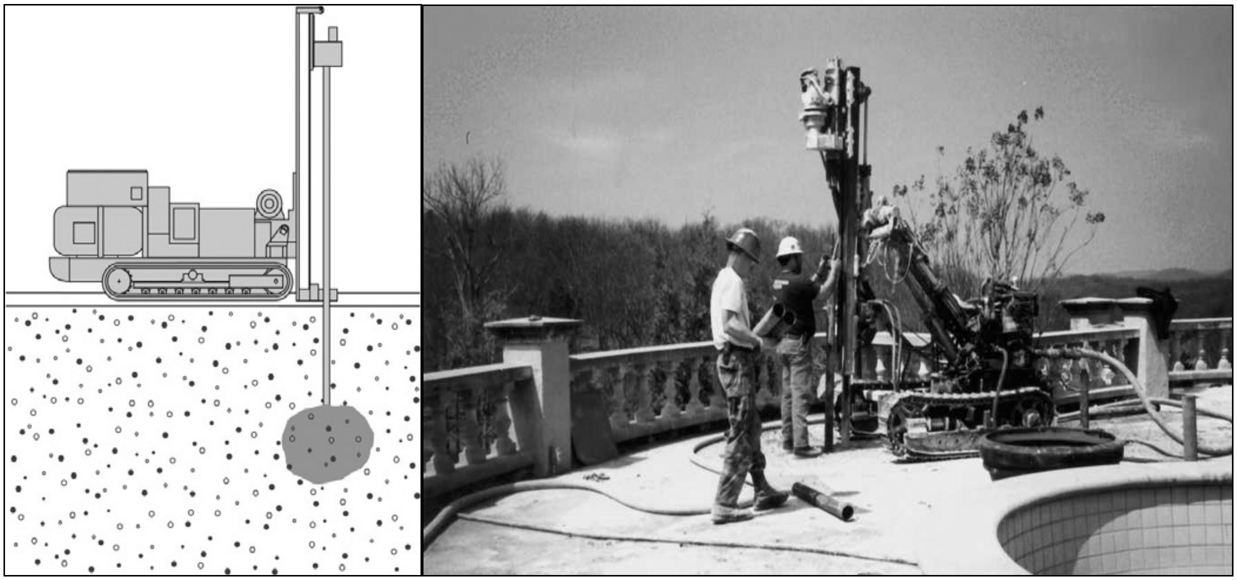

(a) (b)

FIGURE 12.22

Permeation grouting: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

Can be held and still be usable. Steam mixing involves storing the grout components in several tanks and then pumping them through separate hoses that combine before the grout reaches the injection pipe. If the geometry of the grouted mass is not important, the grout can be pumped through and out the bottom of the injection pipe. The pipe is then raised in increments, 1 to 3 ft (0.3 to 0.9 m), as the specified volume is injected at each interval.

A sleeve port pipe is used when the grouted geometry is important, such as excavation support walls. A sleeve port pipe is a steel or PVC pipe with holes, or ports, located at regular intervals, possibly 1 to 3 ft (0.3 to 0.9 m), along its length. A thin rubber membrane is placed over each port. The rig drills a hole in the soil, fills it with a weak, brittle, Portland cement grout, and inserts the sleeve port pipe. After the weak grout has hardened, a grout injection pipe with two packers is inserted into the sleeve port pipe allowing the grout to be injected through one port at a time (Figure 12.24). The injection pipe is then raised or lowered to another port and the process repeated in a sequence that includes primary, secondary, and tertiary injections.[7]

During grouting process, injection pressures are usually limited to prevent fracturing or volume change in the natural soil/rock formation. As a rule of thumb, maximum injection pressure is about 20kPa per meter of depth. Based on the field trials and the soil conditions, the injection pressures and the grout volumes will be justified to meet the intended performance. The process is limited to relatively coarse-grained soils (refer to Figure 12.23), so as to enable the grout to flow through the formation to replace the void spaces or joints.

Particulate grouts (e.g. cement or bentonite) are generally used for medium to coarse grained sands, such that the particles in the grout easily percolate through the formation. Micro fine cement is also used for fine grained sands where Ordinary Portland Cement cannot percolate through the formation. Chemical grouts (e.g. silicates) are used in formations with smaller pore spaces, but are limited to soils coarser than fine grained sands. The typical spacing for permeation grouting holes is between 4 to 8 feet.[1]

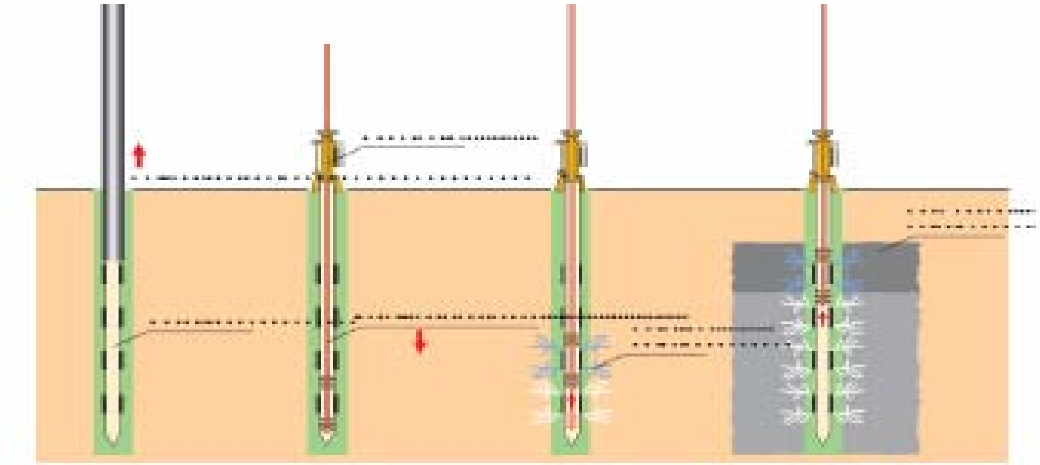
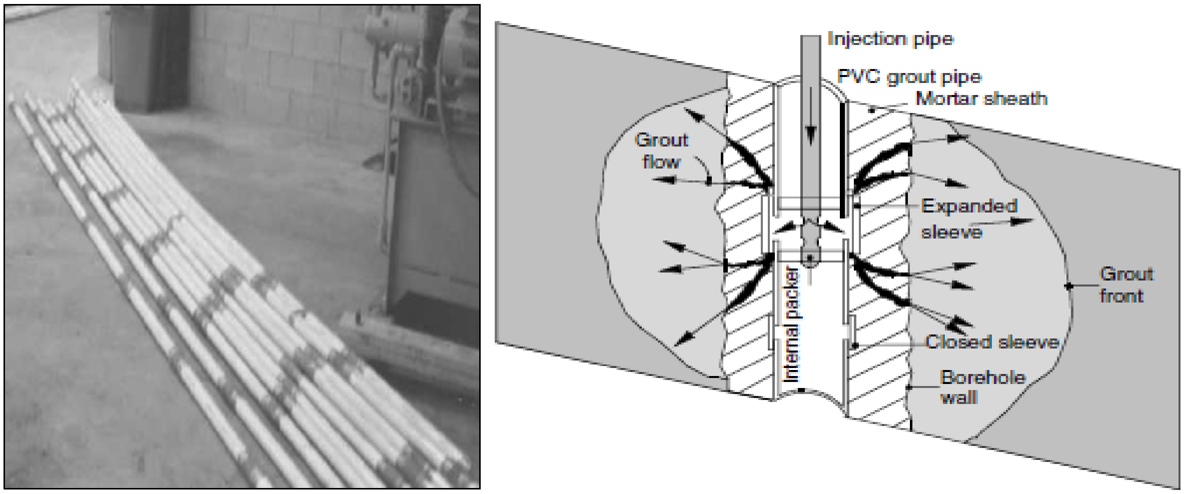


Figure 12.23. Schematic showing process of Permeation Grouting

Materials: The type of grout used depends on the application and soil grain size. For structural applications in gravel, Portland cement and water can be used. However, the particle size of the Portland cement is too large for sands. A finely ground Portland cement is available for use in course to medium sands. In fine, medium, and coarse sand, chemical grout can be used. The most common chemical grout used for structural applications is sodium silicate. Other chemical grouts are acrylates and polyurethanes.



(a) (b)
FIGURE 12.23
(a) Sleeve port pipes and (b) cross section of grout injection through a port. (From Hayward Baker Inc. With permission.)

Design: Generally, unconfined compressive strength and permeability are the design parameters. Sands grouted with sodium silicate can achieve a permeability of 1 * 10-5 cm/sec and an unconfined compressive strength of 50 to 300 psi (0.345 kPa to 2.07MPa), although consistently achieving values in the field greater than 100 psi (0.69 MPa) is difficult. A standard analysis is performed assuming that the grouted soil is a mass with the design parameters. For excavation support walls, the mass is analyzed as a gravity structure, calculating the shear, sliding and overturning of the mass, as well as the global stability of the system.

Quality control and quality assurance: Like any other grouting improvement process, the quality control during permeation grouting is very important to ascertain the effectiveness of the technique. The mix design of the grouted soil can be estimated in the lab by compacting the soil to be grouted in a cylinder or cube molds at about the same density as exists in situ and then saturating the soil with the grout. Laboratory permeability or unconfined compressive strength tests can be performed after a specified cure time, such as 3, 7, 14, and 28 days. During production, the grout volume ,grout pressure and flow rate should be monitored and documented. The grouted soil can also be cored and tested after grouting.[7]

Limitations

Permeation grouting is not suitable in cohesive soils such as silts and clays. The other types of grouting techniques such as fracturing grouting and jet grouting can be considered to improve such soils. A recent case history where Permeation Grouting works were executed for a dam project is presented below.[1]

Soil Conditions:

The Soils of the coffer dam consisted of a typical river bed material with boulders intermixed with silty sand. The permeability of the coffer dam formed with these natural soils was in the range of 10^{-3} to 10^{-4} m/sec and the target permeability was set to be 10^{-6} m/sec.

Proposed Ground Improvement Scheme:

The trial grouting works were carried out to establish a grid pattern suitable to the existing soil conditions in order to achieve the desired imperviousness, before commencement of the actual works. The grout curtain was formed from the coffer dam surface and socketed 1m into the bed rock. For this purpose a total of 1,420 holes were drilled in the coffer dam in three rows. The height of the grout curtain ranged from about 20m at the dyke portion and reduced to 5m towards the up-stream. The variation is due to the non-uniformity in the rock head levels.

The grouting was carried out from bottom to the surface progressively at predetermined steps, pressures and grout volumes which were established after the initial site trial. Basic grout material consisted of cement in the outer rows and Silica gel in the middle row with some additives. The effectiveness of the grout curtain was demonstrated by means of in-situ permeability tests, which recorded the average permeability values to be about 10^{-7} m/sec.[1]

2.3.2 Jet Grouting

Jet grouting (Figure 12.24) was conceived in the mid-1970s and introduced in the United States in the 1980s. The technique hydraulically mixes soil with grout to create in situ geometries of soilcrete. Jet grouting offers an alternative to conventional grouting, chemical grouting, slurry trenching, underpinning, or the use of compressed air or freezing in tunneling. A common application is underpinning and excavation support of an existing structure prior to performing an adjacent excavation for a new, deeper structure.

Super jet grouting is a modification to the system allowing creation of large diameters (11 to 16 ft, or 3.4 to 4.9 m) and is efficient in creating excavation bottom seals and treatment of specific soil strata at depth.[7]

Applicable soil types: Jet grouting is effective across the widest range of soils. Because it is an erosion-based system, soil erodibility plays a major role in predicting geometry, quality, and production. Granular soils are the most erodible and plastic clays the least. Since the soil is a component of the final mix, the soil also affects the soilcrete strength (Figure 12.25). Organic soils are problematic and can be the cause for low strengths unless partially removed by an initial erosion pass before grouting. Flowing water can also be a problem.

Equipment: An on-site batch plant is required to mix the grout as needed. Pumps are also required to pump the grout and sometimes water and air to the drill rig. The drill rig is necessary to flush the jet grout monitor into the ground. Compact drills are capable of low head room and tight access work. Pumps may also be required to remove the soilcrete waste.

Procedure: Jet grout is a bottom-up process (Figure 12.26). The drill flushes the monitor to the bottom of the treatment zone. The erosion and grout jets are then initiated as the monitor is rotated and extracted to form the soilcrete column. Varying geometries can be formed. Rotating the monitor through only a portion of a circle will create a portion of a column. Extracting the monitor without rotating it will create a panel. Treatment depths greater than 60 ft (18.5 m) require special precautions.

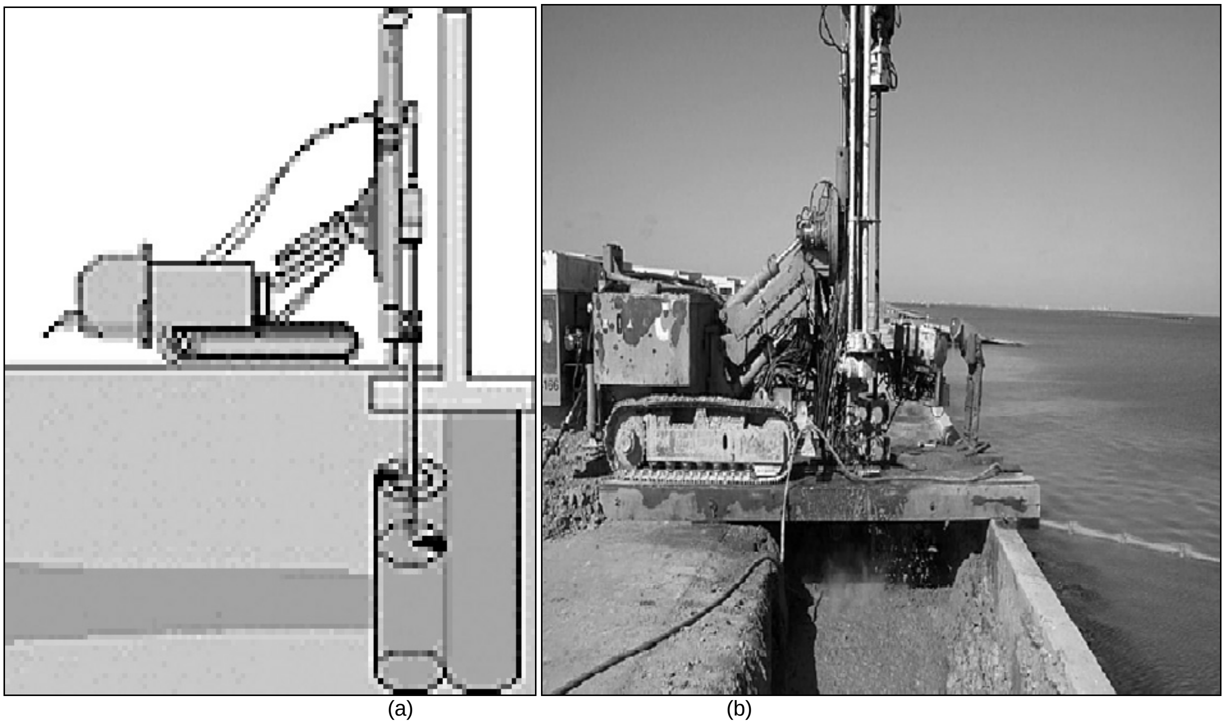

(a) (b)

FIGURE 12.24
Jet grouting: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

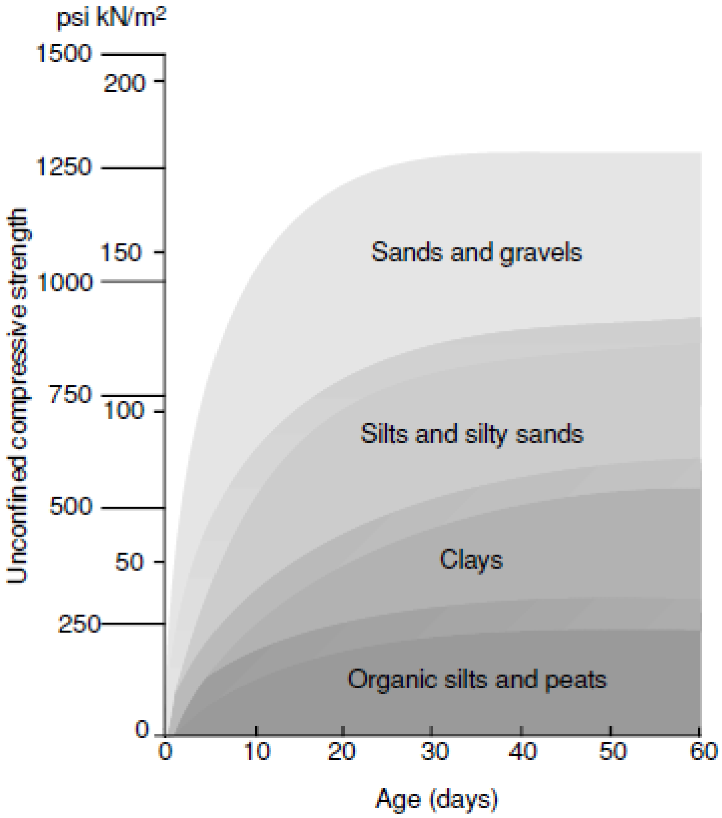


FIGURE 12.25
Range of soilcrete strengths based on soil type. (From Hayward Baker Inc. With permission.)

There are three traditional jet grout systems Selection of the most appropriate system is determined by the in situ soil, the application, and the required strength of the soilcrete. The three systems are single, double, and triple fluid.

The single-fluid system uses only a high-velocity cement slurry grout to erode and mix the soil. This system is most effective in cohesionless soil and is generally not an appropriate underpinning technique because of the risk of pressurizing and heaving the ground.

The double-fluid system surrounds the high-velocity cement slurry jet with an air jet. The shroud of air increases the erosion efficiency. Soilcrete columns with diameters over 3 ft (0.9 m)can be achieved in medium to dense soils, and more than 6 ft (1.8 m)in loose soils. The double-fluid system is more effective in cohesive soils than the single-fluid system.

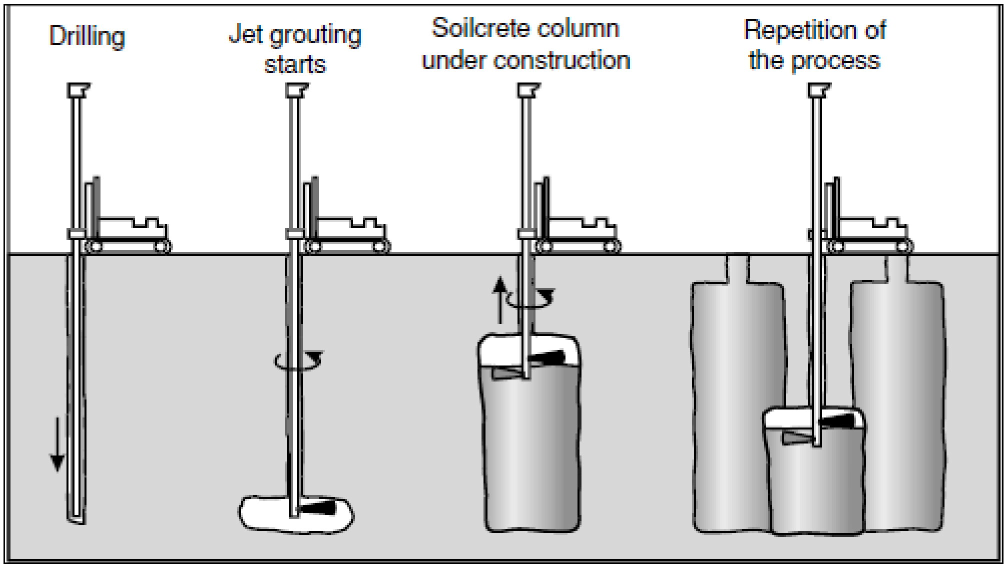


FIGURE 12.26
Jet grout process. (From Hayward Baker Inc. With permission.)

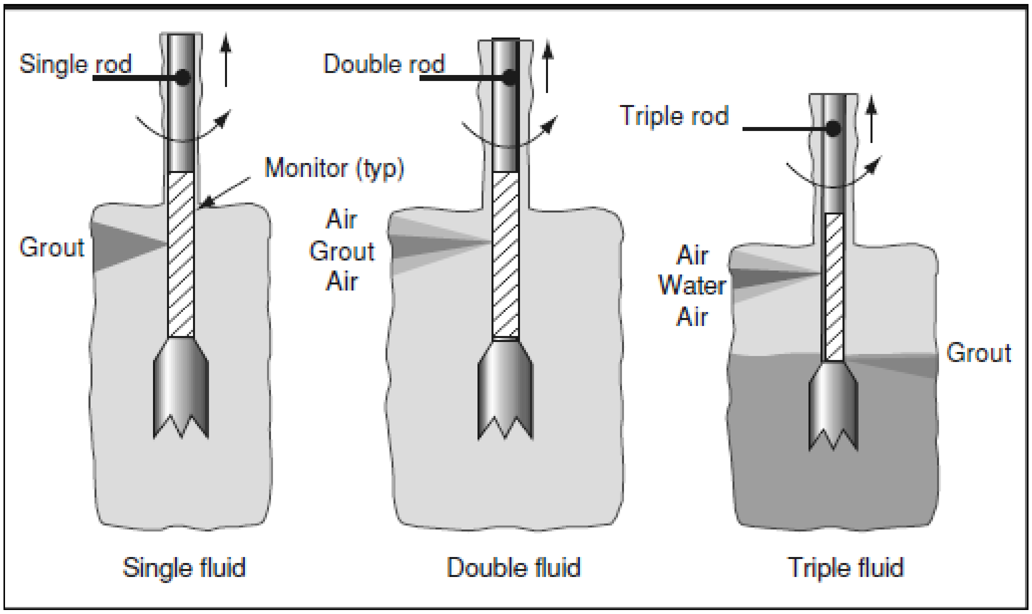


FIGURE 12.27
Single-, double-, and triple-jet grout systems. (From Hayward Baker Inc. With permission.)

The triple-fluid system uses a high-velocity water jet surrounded by an air jet to erode the soil. A lower jet injects the cement slurry at a reduced pressure. Separating the erosion process from the grouting process results in higher quality soilcrete and is the most effective system in cohesive soils.

Since material is pumped into the ground and mixed with the soil, the final mixed product has a larger volume than the original in situ soil. Therefore, as the mixing is performed, the excess soilcrete exits to the ground surface through the annulus around the drill steel. This waste material must be pumped or directed to an onsite retention area or trucked off-site. Since the waste contains cement, the waste sets up overnight and can be handled as a solid the following day. [7]

Materials: Portland cement and water are generally the only two components, although additives can be utilized.

Design: Generally, either unconfined compressive strength or permeability is the design parameter. A standard analysis is performed to determine the required soilcrete geometry necessary based on the parameters achievable in the soil to be mixed. For excavation support walls, the mass must resist the surcharge, soil and water pressure imposed after excavation. This may include analysis of shear, sliding and overturning, as well as the global stability of the system. For underpinning applications, a standard bearing capacity and settlement analysis is performed as would be done for any cast in place pier.

Quality control and quality assurance: Monitoring and documenting the production parameters and procedures is important to assure consistency and quality. Test cylinders or cubes made from the waste material give a conservative assessment of the in situ characteristics. Wet sampling of the soilcrete in situ can also be performed although it is problematic. Coring of the hardened soilcrete is typical.[7]

2.3.3 Soil Mixing

Deep Soil Mixing (DSM) technology was invented almost 30 years ago and is a form of soil improvement involving the introduction and mechanical mixing of in-situ soft and weak soils with a cementitious compound such as lime, cement or a combination of both in different proportions (CDIT, 2002). The mixture is often referred to as the binder. The binder is injected into the soil either in a dry (dry method) or slurry (wet method) forms. In case of dry method, the moisture in the soil is utilized for the binding process. Whereas in the case of wet method, the slurry of grout with appropriate water-binder ratio is mixed thoroughly with in-situ soil. The technique forms columns within the treated zone whilst improving shear and compressibility parameters of the in-situ loose/soft soils.[1]

Mixing with cement slurry was originally developed for environmental applications; however, advancements have reduced the costs to where the process is used for many general civil works, such as in situ walls, excavation support, port development on soft sites, tunneling support, and foundation support. Mixing with dry lime and cement was developed in the Scandinavian countries to treat very wet and soft marine Clays.

In Scandinavia, because of the high in situ water content of sensitive soft soils, dry mixing has been the norm, and in this process the stabilizing agents are fed in powder form with the help of compressed air. In contrast, in Japan and Poland, wet mixing is extensively used, using single or multiple mixing blades. The methods have been large standardized, and in addition to national guidelines, for applications in Europe there is a European design guide (EuroSoilStab 2002) and European standard for Execution of Special Works - Deep Mixing (EN: 14679: 2005).

The process of deep mixing involves three principal phases (Larsson 2005):

1. Penetration of the mixing tool to the required depth, which involves remolding and disaggregation of soil structure. According to Japanese practice, stabilizing agents are introduced already at this stage, if wet mixing is used.
2. Dispersion process, which includes incorporation and spreading of the binder, wetting of the solid particles (in the case of dry mixing), breakdown of agglomerates by kneading action and the distribution, which is the process by which the disaggregated agglomerates are (hopefully) randomly scattered through the mixture. As already mentioned, in wet mixing the stabilizing agents are often introduced already during the penetration, i.e. Phase 1.
3. Molecular diffusion, which continues after the execution, involves the migration of calcium ions into the unstabilized soil, or within the stabilized soil, from parts with high concentration of ions to parts with low concentration. The binder will also migrate into any shrinkage cracks in the surrounding soil, as well as any vertical fractures and shear cracks created by the mixing process. The process of molecular diffusion is most likely the reason why the columns sometimes exhibit some self-healing tendencies. [3]

The technique ensures adequate bearing capacity whilst limiting settlement within serviceability limits. Typically undrained shear strength of the columns ranges between 100 and 2000 kPa whereas load carrying capacity of the columns ranges between 20T and 50T depending on the method of mixing, characteristics of the in-situ soil, binder content and column diameter. Depending on the type of application, following patterns as shown in Figure 3.3 can be implemented.

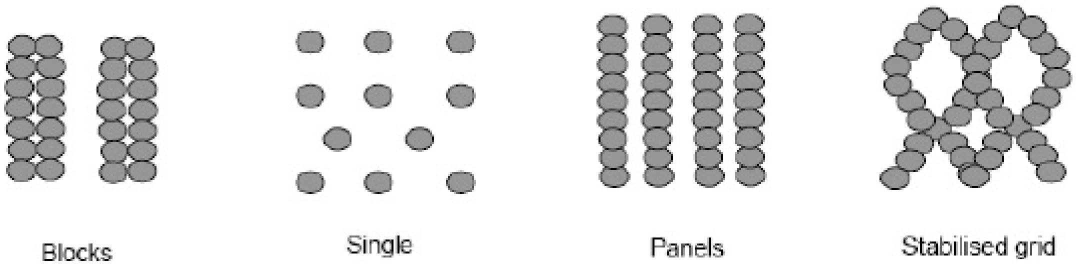


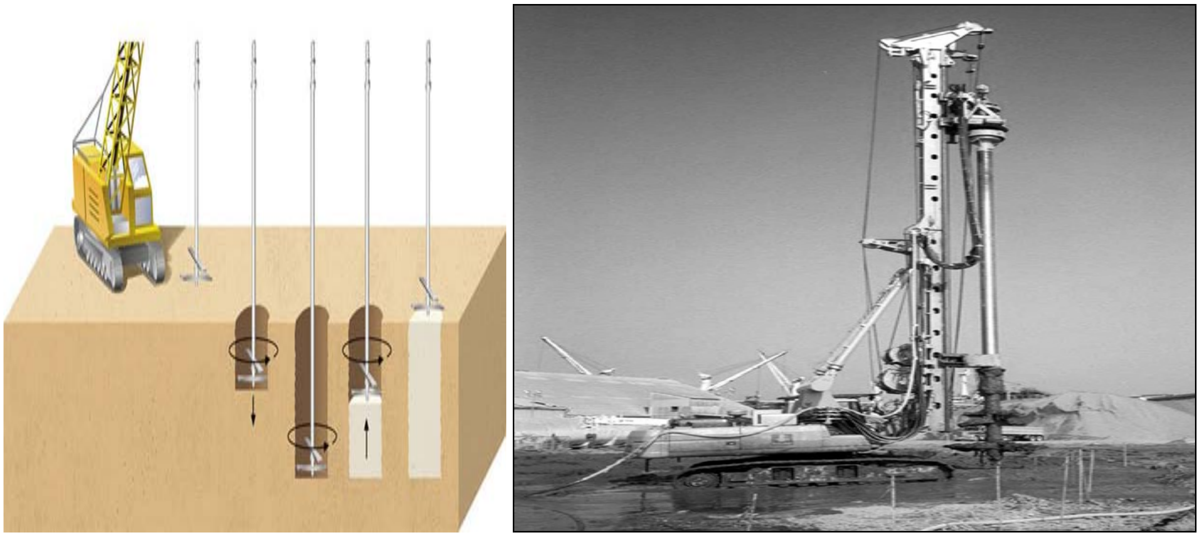
Figure 3.3 Typical patterns of Deep Soil Mixing

In the deep soil mixing process, admixtures/binders are introduced into the in-situ soils throughout the treatment depth and mixed thoroughly using large diameter single or multiple-shaft mixing tools to form columns or panels of improved material (refer to Figure 12.28). The mix-in-place columns can be up to 1m or more in diameter. Typical admixtures are cement and lime, but slag / fly ash and/or other additives can also be used.[1]

Applicable soil types: The system is most applicable in soft soils. Boulders and other obstructions can be a problem. Cohesionless soils are easier to mix than cohesive soils. The ease of mixing cohesive soils varies inversely with plasticity and proportionally with moisture content. The system is most commonly used in soft cohesive soils as other soils can often be treated more economically with other technologies. Organic soils are problematic and generally require much larger cement content. The quality achieved with soil mixing is slightly lesser than that achieved with jet grouting in the same soils, with unconfined compressive strengths between 10 and 500 psi (0.69 to 3.45 MPa), and permeability as low as 1 * 10-7 cm/sec, depending on the soil type and binder content.

Equipment: A high-volume batching system is required to maintain productivity and economics. The components consist of an accurately controlled mixer, temporary storage, and high-volume pumps. A drilling system is required to turn the mixing tool in the ground. The system varies from conventional hydraulic drill heads to dual-motor, crane-mounted turntables with torque requirements ranging from 30,000 to 300,000 ft lb (41 to 411 kJ). Multi axis, electrically powered drill heads are also used, primarily for walling applications. The mixing tool is generally a combination of partial flighting, mix blades, injection ports and nozzles, and shear blades. It can be a single- or multiple-axis tool (Figure 12.28). Tool designs vary with soil types and are often custom-built for specific projects (Figure 12.29). The diameter of the tool can vary from 1.5 to 12 ft (0.46 to 3.7 m).

Procedure: The binder is injected as the tool is advanced down to assist in penetration and to take advantage of this initial mixing. The soil and binder are mixed a second time as the tool is extracted. The rate of penetration and extraction is controlled to achieve adequate mixing. Single columns or integrated walls are created as the augers are worked in overlapping configurations. Treatment depths as great as 100 ft (31 m) have been achieved.[7]



(a) (b)

FIGURE 12.28 Soil Mixing: (a) Schematic, (b) Field implementation.

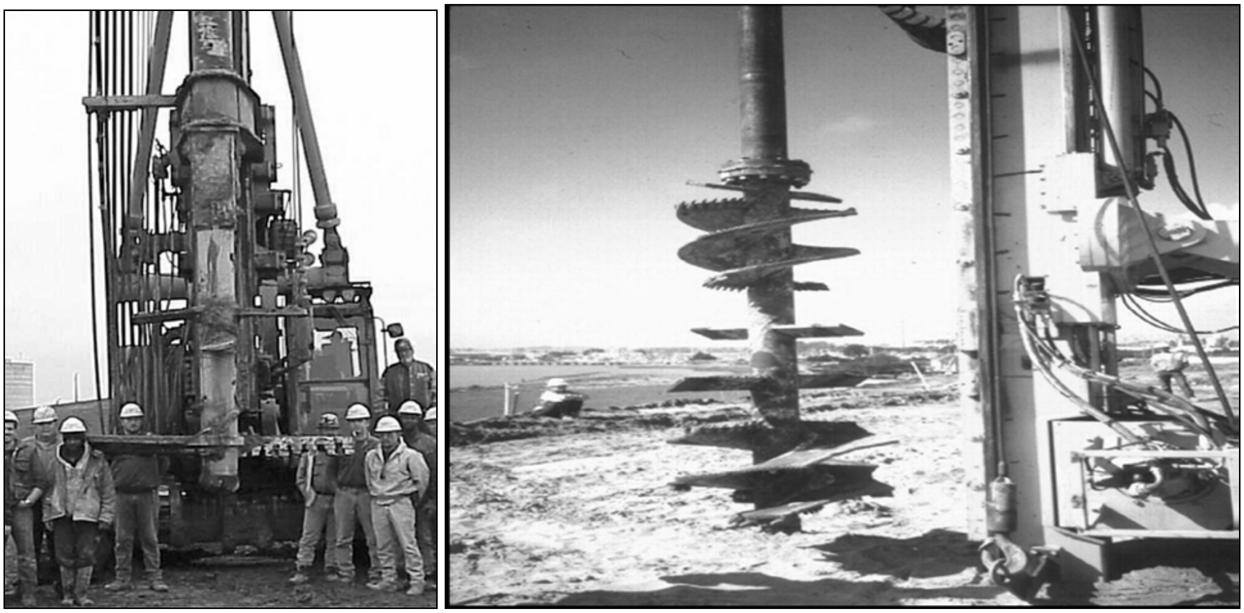


FIGURE 12.29
Example of soil mixing tools. (From Hayward Baker Inc. With permission.)

Materials: For wet soil mixing, the binder is delivered in a slurry form. Slurry volumes range from 20 to 40% of the soul volume being mixed. Common binders are Portland cement, fly ash, ground blast furnace slag, and additives. For dry soil mixing, the same materials (also line) are pumped dry using compressed air. Preproduction laboratory testing is used to determine mix energy and grout proportions.

Design: As with jet grouting, unconfined compressive strength and permeability are generally the design parameters. A standard analysis is performed to determine the required geometry based on the parameters achievable in the soil to be mixed. For excavation support walls, the mass can be designed as a standard excavation wall, or a thicker mass can be created and analyzed as a gravity structure, calculating the mass' shear, sliding and overturning, as well as the global stability of the system. When used as structural load bearing columns, a standard bearing capacity and settlement analysis is performed as would be for any cast in place pier. Anchored retention using steel reinforcement is common for support walls.

Applicability of the Deep Soil Mixing

The technology of deep soil mixing can be implemented on wide range of weak and problematic soil types such as loose sands, soft marine clays, ultra soft slimes, weak silty clays and sandy silts. Typical applications include foundations of embankment fill for roads, highways, railways and runways; slope stabilization, stabilization of cuts and excavations; foundations for structures (Topolnicki, 2004 and Raju et. al., 2005). The DSM technology can also be used for vibration reduction applications and to partially reduce water paths for water tight applications.

The choice of dry or wet Deep Soil Mixing largely depends on many factors such as characteristics of in-situ soil to be treated, type of structure to be founded, type of application, performance criteria, etc.

Limitations: Deep Soil Mixing technology is applicable to broad spectrum of soils, but due considerations shall be given to peaty soils with high organic content in terms of achievable strength and required curing period. Systematic series of trial mixing with varying binder contents and subsequent laboratory tests after allowing for varying curing periods will ensure reliable design parameters such as achieved strength, required binder content and curing period. Another limitation of DSM technique is treatment of soils in ex-landfill areas, where large content of waste dump soils to be improved. [1]

Quality Control & Quality Assurance

For both wet as well as dry Deep Soil Mixing, quality control during execution is important to ensure uniform improvement of the soil and to ascertain the required amount of binder has been mixed uniformly over the entire depth of treatment. For this purpose, the mixing units are equipped with automated computerized recording devices to measure the real-time operating parameters such as depth of mixing tool, volume or weight of binder used, flow rate of grout, rotation speed and rate of penetration and withdrawal. After allowing for sufficient curing period (typically, 3 to 4 weeks), the mixed columns can also be tested using single/group column plate load tests, unconfined compressive strength tests on cored/backflow samples, visual examination of exposed columns, etc.

A recent case history of a project where Deep Soil Mixing works were carried out for a sewage treatment plant is presented below.[1]

Deep Soil Mixing Foundation for Jelutong Sewage Treatment Plant in Penang Island, Malaysia[1]

Project Description: A Sewage Treatment plant is under construction in Penang Island and when completed will cater for an ultimate capacity of 1.2 million population equivalent. The project will serve as a centralized sewage treatment facility and will include 12 nos. of Sequential Batch Reactor (SBR) tanks and associated process tanks. The SBR tanks are major process tanks in the entire plant and were designed as twin tanks made up of reinforced concrete (total 6 nos. of twin tanks separated by very narrow gap) supported on treated ground. The dimension of each twin tank is approximately 90m x 60m x 7m high. At the time of writing this paper, the building works are almost completed, whilst mechanical and process installation works are ongoing.

Soil Conditions: The site was originally reclaimed from the sea and approximately, half of the SBR tanks area was covered by former domestic landfill (3m to 5m thick) waste dumps (where Vibro Concrete Columns were constructed as the rubbish dumps cannot mixed using DSM technology). In the remaining half of the SBR tanks area (where DSM technique is used), the subsoil primarily consists of 3m to 5m thick reclaimed fill followed by 5m to 7m thick soft marine clay. This is followed by stiff to very stiff cohesive deposits to over 50m depth.

Proposed Ground Improvement Scheme:

The original foundation design was piled foundation to over 40m depth; but this was later found to present a few undesirable construction limitations like noise pollution during pile driving; requirement of pre-boring and removal of landfill material; and transportation and storage of pre-cast piles on a congested site; as well as relatively high cost.

As an alternative, ground improvement techniques (Vibro Concrete Columns and Deep Soil Mixing) were utilized to support the SBR tanks. Vibro Concrete Columns (VCC) were constructed for 3 nos. of twin tanks in the former landfill area, forming concrete pile-like elements by displacing the domestic waste dumps rather than requiring removal. Deep Soil Mixing columns were constructed for remaining 3 nos. of twin tanks in the marine clay outside the landfill area which increased strength of marine clay by more than 40 times. The execution of DSM works (typically, 800mm diameter to a maximum depth of 14m) are shown in Figure 3.2.

The alternative foundation system was designed to ensure adequate bearing capacity, limit the total settlement of the structure to be less than 75mm and differential settlement to be less than 1:360. The technical performance of the alternative foundation system was well proven by means of factual monitoring results during hydro-testing stage.

Figure 3.2: Deep soil Mixing works at Jelutong Sewage Treatment Plant

a. **Dry Soil Mixing**

Dry soil mixing (Figure 12.30) is a low-vibration, quiet, clean form of ground treatment technique that is often used in very soft and wet soil conditions and has the advantage of producing very little spoil. The high speed rotating mixing tool is advanced to the maximum depth, ‘‘disturbing’’ the soil on the way down. The dry binder is then pumped with air through the hollow stem as the tool is rotated on extraction. It is very effective in soft clays and peats. Soils with moisture content, greater than 60% are most economically treated.

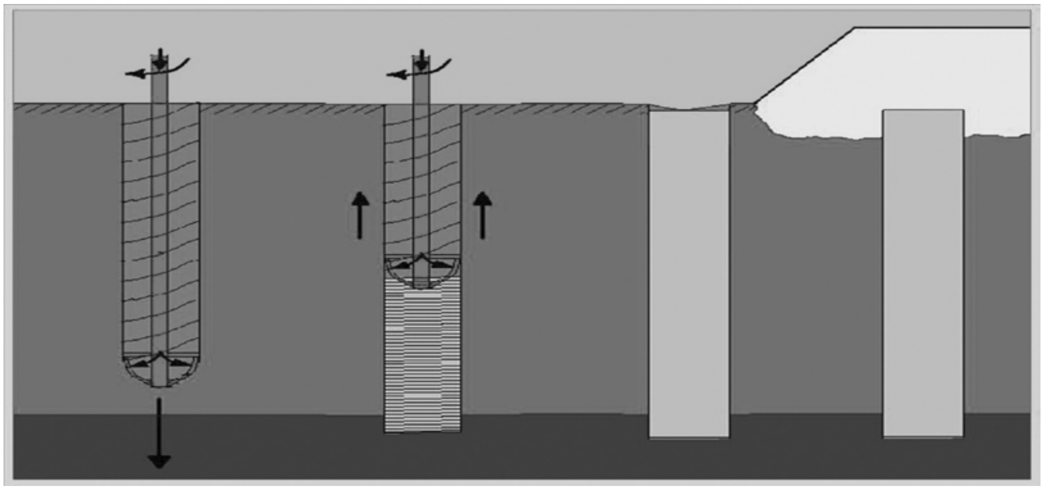


FIGURE 12.30
Illustration of dry soil mixing technique. (From Hayward Baker Inc. With permission.)

This process uses cementacious binders to create bond among soil particles and thus increases the shear strength and reduces the compressibility of weak soils. The most commonly used binding agents are cement, lime, gypsum, or slag. Generally, the improvement in shear strength and compressibility increases with the binder dosage. By using innovative mixtures of different binders engineers usually achieve improved results. It is known that strength gains are optimum for inorganic soils. It is realized that the strength gain would decrease with increasing organic and water content. The binder content varies from about 5 lb/ft3 for soft inorganic clays to about 18 lb/ft3 for peats with a high organic content.

b. Wet Soil Mixing

Wet soil mixing (Figure 12.31) is a similar technique except that a slurry binder is used making it more applicable with dryer soils (moisture contents less than 60%). The grout slurry is pumped through the hollow stem to the trailing edge of the mixing blades both during penetration and extraction. Depending on the in situ soils, the volume of grout slurry necessary varies from 20 to 40% of the soil volume. The technique produces a similar amount of spoil (20 to 40%) which is essentially excess mixed soil which, after setting up, can often be used as structural fill. The grout slurry can be composed of Portland cement, fly ash, and ground granulated blast furnace slag.

Quality control and quality assurance: Preproduction laboratory testing is often performed to prescribe the mixing energy and binder components and proportions. During production, it is necessary to monitor and document parameters such as mixing depth, mixing time, grout mix details, grout injection rates, volumes and pressures, tool rotation, penetration, and withdrawal rates.

Test cylinders or cubes can be cast from wet samples, but are problematic. The hardened columns can also be cored. In weaker mixes, penetration tests can be performed.

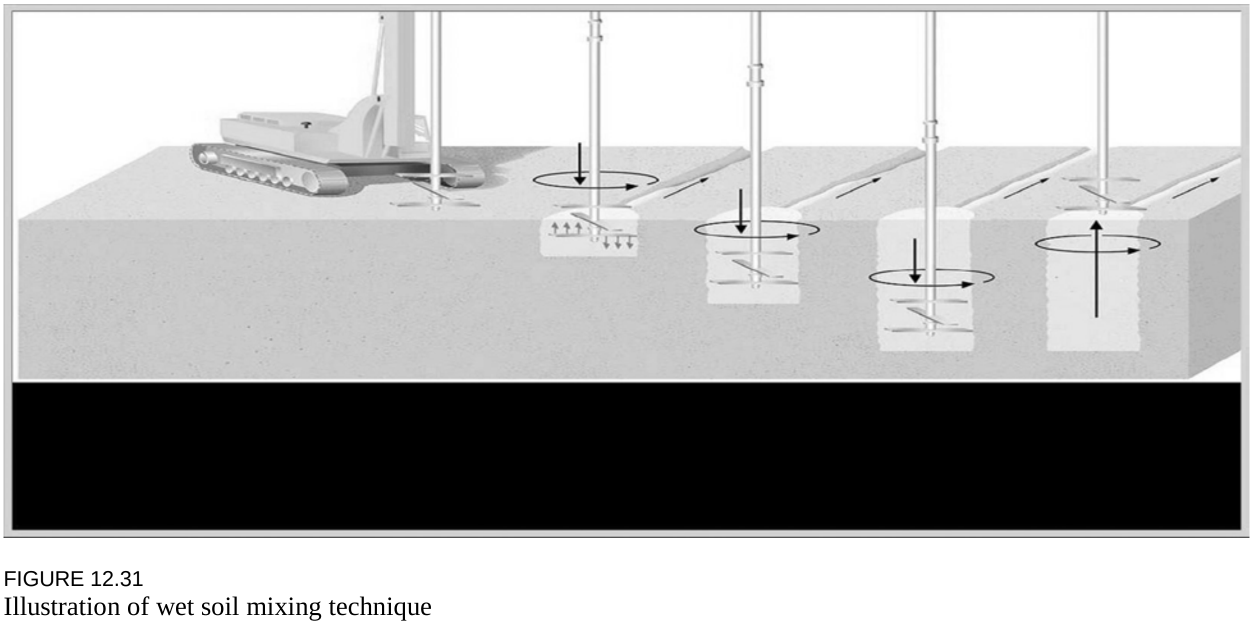


FIGURE 12.31
Illustration of wet soil mixing technique

The mechanism of soil improvement by mixing admixtures with soil is shown in detail below. The stabilization mechanism may vary widely from the formation of new compounds binding the finer soil particles to coating particle surfaces by the additive to limit the moisture sensitivity. Therefore, a basic understanding of the stabilization mechanisms involved with each additive is required before selecting an effective stabilizer suited for a specific application.

2.3.3.1 Guidelines for Stabilizer Selection

Soil characteristics including mineralogy, gradation and physio-chemical properties of fine grained soils influence the soil-additive interaction. Hence stabilizer selection should be based on the effectiveness of a given stabilizer to improve the physio-chemical properties of the selected soil. The preliminary selection of the appropriate additive(s) for soil stabilization should consider:

- Soil consistency and gradation
- Soil mineralogy and composition
- Desired engineering properties
- Purpose of treatment
- Mechanisms of stabilization
- Environmental conditions and engineering economics

Soil index properties (i.e., sieve analysis, Atterberg limit testing, and moisture density testing) should be determined based on laboratory testing of field samples. Soil samples should be prepared following AASHTO T 87. The initial processing of most soils involves thorough air drying or assisted drying at a temperature not to exceed 60oC. Aggregations of soil particles should be broken down into individual grains to the extent possible. A representative soil fraction should be selected for testing following AASTHO T 248. The required quantity of soil smaller than 0.425 mm (No. 40 sieve) should be used to determine the soil index properties. Liquid limit testing should be performed following AASHTO T 89 and plastic limit and plasticity index testing should be measured following AASHTO T 90.[6]

a) Lime Stabilization

Lime has been found to react successfully with medium, moderately fine and fine grained soils causing a decrease in plasticity and swell potential of expansive soils, and an increase in their workability and strength properties. Research has proven that lime may be an effective stabilizer in soils with clay content as low as 7 percent and in soils with plasticity indices below 10.

The National Lime Association recommends a plasticity index of 10 or greater in order for lime to be considered as a potential stabilizer whereas the U.S Army Corps of Engineers recommends a plasticity Index of 12 or greater for successful lime stabilization. Based on AASHTO classification, soil types A-4, A-5, A-6, A-7 and some of A-2-6 and A-2-7 are suitable for stabilization with lime.

b) Cement Stabilization

Cement stabilization is ideally suited for well graded aggregates with a sufficient amount of fines to effectively fill the available voids space and float the coarse aggregate particles. General guidelines for stabilization are that the plasticity index should be less than 30 for sandy materials. For fine-grained soils, soils with more than 50 percent by weight passing 75µm sieve, the general consistency guidelines are that the plasticity index should be less than 20 and the liquid limit (LL) should be less than 40 in order to ensure proper mixing (6). A more specific general guideline based on the fines content is given in the equation below which defines the upper limit of P.I. for selecting soil for cement stabilization.

P.I $\leq 20 + 50-$ (% smaller than 0.075mm)

Cement is appropriate to stabilize gravel soils with not more than 45 percent retained on the no. 4 sieve. The Federal Highway Administration recommends the use of cement in materials with less than 35 percent passing no. 200 sieve and a plasticity index (PI) less than 20 (18). Based on this system, soils with AASHTO classifications A-2 and A-3 are ideal for stabilization with cement, but certainly cement can be successfully used to stabilize A-4 through A-7 soils as well. The Portland cement Association (PCA) established guidelines to for stabilizing a wide range of soils from gravels to clays.

c) Fly Ash Stabilization

The literature lacks a clear direction in selection parameters for the use of fly ash in soil stabilization. However, the literature documents that a wide range of aggregates can be suitably stabilized with fly ash including sands, gravels, crushed stones and several types of slags. Fly ash can be used effectively to stabilize coarse grained particles with little or no fines. In coarser aggregates, fly ash generally acts as a pozzolan and/or filler to reduce the void spaces among larger size aggregate particles to float the coarse aggregate particles. After the appropriate amount of fly ash is added to coarse grained soils to fill the voids, optimize density, an activator is often used to maximize the pozzolanic reaction in the mixture. The activator content is generally in the range of 20 to 30 percent of the fly ash used to fill the voids. The activator is normally either lime or Portland cement, but lime kiln dust or cement kiln dust can also be used. Similarly, consider a clay soil that is stabilized with lime but the clay is not pozzolanically reactive. The addition of fly ash and lime can substantially increase strength in the blend due to the reactive pozzolans provided by the ash. In these fine-grained soils, fly ash is typically used in Conjunction with lime or cement to enhance the reactivity of the fine-grained soil with lime or cement.

Class C fly ash has been used alone to stabilize moderately plastic soils. The basis for stabilization is free lime that becomes available upon hydration of the ash. The large majority of this lime is combined with the silica and alumina, but upon hydration, just as in the hydration of Portland cement, cementitious products are formed which stabilize the soil. However during this hydration process, just as in the hydration of cement, free lime is released, which can react pozzolanically with the clay. This reaction reduces clay particle plasticity and improves strength. Successful application is often achieved with fine grained, plastic soils, by first applying lime or cement to reduce plasticity and improve workability of the soil and then adding the fly ash to boost strength of the soil, lime blend. Again, the impact of a given class F (with activator) or a given class C fly ash without activator may be very different depending on the pozzolan content of each ash, the degree of self cementing property of the class C ash, etc. Hence, the superior filler cannot be determined before hand and without evaluation.

2.3.3.2 Techniques for Stabilizer Selection

A range of options are available for selecting soil stabilizers most of which are based on the soil classification following either the AASHTO or Unified classification system. A simple, but well accepted methodology by which to select the appropriate stabilizer is the Soil Stabilization Index System (SSIS).

The methodology was developed by U.S Air Force, and is based on soil index properties: plasticity index and percent passing the Nº. 200 sieve. These laboratory tests are easy to perform and are necessary inputs for AASHTO and Unified systems. Both these characteristics can be effectively correlated to the engineering properties of the soil and therefore can be used to differentiate engineering applicability. Figures 2 (for soils) use these two index properties, *PI and percent passing the no. 200 sieve* (percent smaller than 75 μm), to identify the appropriate stabilizer. Once the stabilizer is selected, detailed laboratory tests to determine strength and performance characteristics of soils are required.

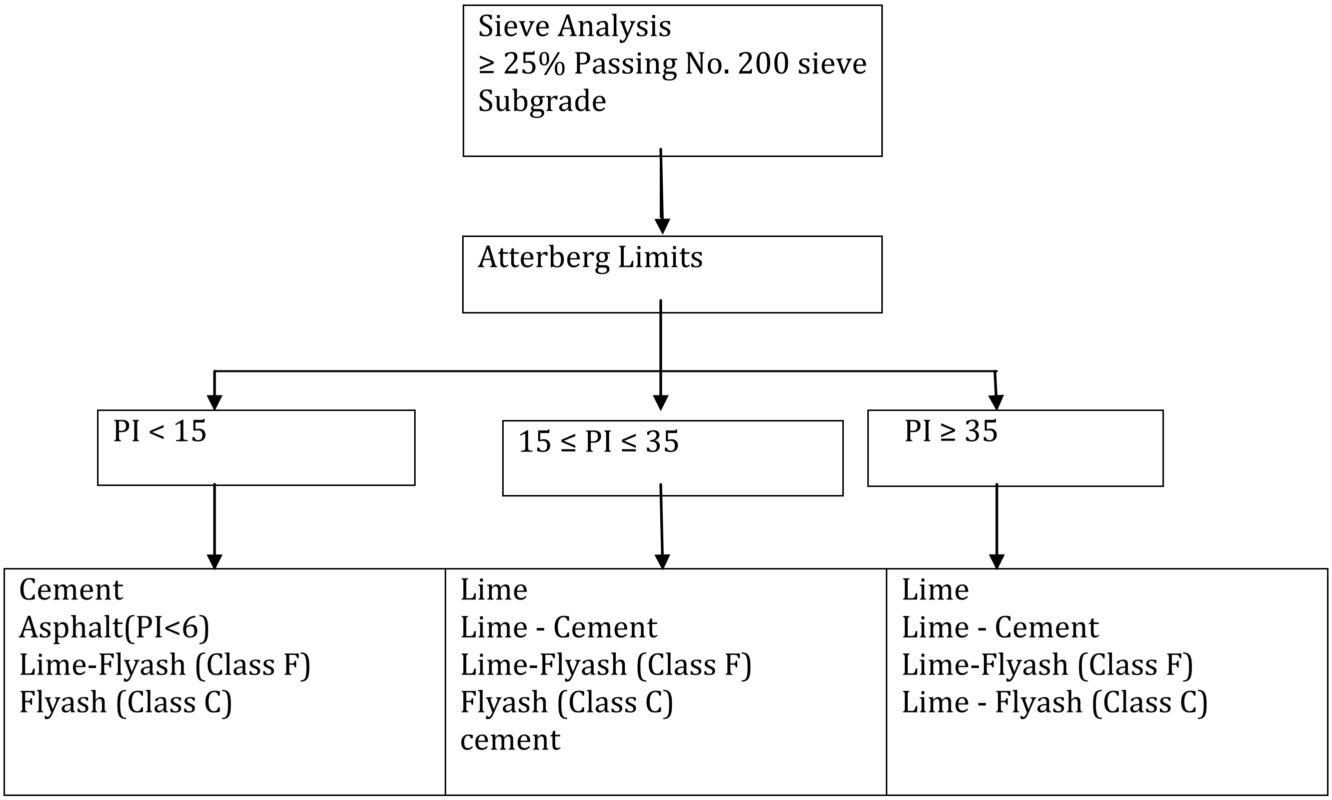


Figure 2. Decision tree for selecting stabilizers for use in sub grade soils .

Figures 2 present a set of general guidelines for selecting candidate stabilizers for soil. Agencies, however, should alter or adjust these guidelines based on their own unique experiences as tempered by local conditions. It is important to remember that Figures 2 is "guidelines" but the final selection should be based on a more specific analysis of the soils. These involve identifying the reactivity of the pozzolans in the clay with the selected stabilizers. For example, lime may be an ideal stabilizer for reactive plastic clay because the lime can immediately reduce plasticity due to cation exchange reactions. Pozzolanic reaction continues over time to further reduce plasticity and increase strength due to the formation of, primarily, calcium-silicate-hydrates. On the other hand, a different clay bearing soil may not be pozzolanically reactive, and, even though the application of lime initially reduces plasticity and improves workability, the desired strength gain does not develop. In this case the stabilizer of choice may have to be Portland cement or a combination of lime and fly ash or lime and cement.

As discussed earlier in this document, modification refers soil improvement that occurs in the short term, during or shortly after mixing (within hours) where as Stabilization is generally a longer term reaction and the degree of strength gain required to achieve stabilization varies based on the expectations of the user. Again, as discussed earlier a strength increase of at least 50 psi greater than that of the untreated soil fabricated and cured under the same conditions as the stabilized material is used in this document to define stabilization. This value was used by Thompson in the Illinois method of mix design for lime treated soils. The researchers on this project recommend that a method of moisture conditioning be included in all strength testing protocols. This research team recommends capillary soak as the form of moisture conditioning before strength testing. In the capillary soak protocol, the sample is placed on a porous stone and wrapped in an absorptive fabric and allowed to absorb water through capillary rise until the moisture front ceases to move or for at least 24 hours.

Additional Tests Involved in Stabilizer Selection Once an additive has been selected based on the index properties of plasticity index and percent of the soil mass smaller than 75μm; the possible impact of deleterious components of the soil must be considered. Organic contents in excess of one percent on a mass basis have been proven to be potentially deleterious. However, some soils with organic contents well over one percent have been successfully treated and stabilized with lime and Portland cement. The second deleterious component is high salt content. A high potassium or sodium content may negatively impact stabilization by competing with calcium cations. However, this can normally be overcome simply by adding the additional calcium-based stabilizer. However, salts containing sulfates have the potential to react with calcium and aluminum released from soil in the high pH environment formed during stabilization to form expansive minerals that can disrupt the stabilized layer.

Soil organic content should be measured following ASTM D 2974. Soils with an organic content of 1-2 percent as determined by ASTM D 2974 may be difficult to stabilize or may require uneconomical quantities of additives in order to stabilize. Stabilized soils, in some cases, may also not be able to meet the recommended strength criteria when excess amounts of organic matter are present. This is because the presence of organic materials in soils inhibits the normal hydration process and reduces the strength gain in stabilized soils. Sulfate contents in soil should be determined following Modified AASHTO test method T 290 or equivalent. Generally, water soluble sulfate levels greater than 0.3 percent (3,000 ppm) suggest the potential for expansive reactions to occur that may result in disruptive volume change in the stabilized layer.

2.3.3.3 VALIDATION OF STABILIZER SELECTION

The procedure outlined below provides a guideline for mixture design for *lime, Portland cement and fly ash*.[6]

1) Lime stabilization

Lime is an appropriate stabilizer for most cohesive soils but the level of reactivity depends on the type and amount of clay minerals in the soil. The steps described in the following paragraphs ensure that the appropriate amount of lime is used to meet design expectations. If design expectations cannot be met with lime, that will become clear by following this protocol described in this section.

a*) Mix Design Considerations*

The mix design protocol presented here follows the National Lime Association protocol. The mix design protocol is designed to optimize the potential for long-term strength gain and durability of lime stabilized soils.

b*) Soil Evaluation*

The first step in the NLA protocol is similar to the approach described in Figure 2 and in fact either the criteria described in Figure 2 or the criteria described in this section can be used. In this step, the soil fraction passing the no. 200 sieve is determined following AASHTO T-27. Liquid limit and plastic limit should be determined following AASHTO T 89 and AASHTO T- 90, respectively. Soils with a plasticity index of 10 or above and a minimum of 25 percent passing the no. 200 sieve are considered desirable for lime stabilization. The NLA protocol requires screening for organic contents above one percent following ASTM D 2974. The NLA protocol does not restrict or eliminate lime stabilization when the organic content of the soil is above one percent, but the protocol recommends that the designer maintain an awareness of this condition throughout the design process and also maintain an awareness of the fact that high organic contents may disrupt the pozzolanic reaction process and may require a greater lime content than normal for the soil in question to reach the desired strength. Water soluble sulfate should be evaluated following AASHTO T 290 (modified). The NLA protocol recommends that if the soluble sulfate content is greater than 3,000 ppm then the user should perform swell tests to verify the expected degree of expansion and take construction steps to mediate the potential expansive reactions. Additional steps to be followed in stabilizing soils with sulfate content above 3,000 ppm are detailed in the AASHTO draft recommended practice for stabilizing sulfate bearing soils.

c) Optimum Lime Content

The first step in assessing the optimum lime content to ensure optimal long term strength gain is to perform the Eades and Grim pH test. For reliable test results, the lime used in the pH test should be the same as that to be used in construction and this lime should be carefully stored to avoid carbonation. The lime used, whether it is in the form of CaO or Ca(OH)2, must meet AASHTO M 216 (ASTM C 977) or equivalent for purity requirements. The standard test method, ASTM D 6276, is used to determine the amount of lime needed to achieve the design pH at 250C (770F), which is about 12.45, depending on specific soil characteristics. The goal of this test is to identify the amount of lime necessary to satisfy immediate lime-soil reactions and also provide a sufficient quantity of calcium to maintain a high residual pH and sustain significant long-term pozzolanic reactions. The pH test is only a first step. The optimum lime content must be validated based on strength testing.

d) Moisture Density Relationship

The addition of lime changes the optimum moisture content (OMC) and maximum dry density (MDD) of soils because the effects of cation exchange and short-term pozzolanic reactions between lime and the soil results in flocculation and agglomeration of clay particles leading to textural changes that are reflected in the moisture-density relationships. For this reason it is necessary to verify the moisture-density relationship of the lime-soil mixture when the amount of lime identified by the Eades and Grimm pH test has been added. The moisture-density relationship of lime-soil mixtures should be determined in accordance with AASHTO T 99.

e) Fabrication and Curing of Samples for Compression Testing

Lime-soil mixtures should be fabricated following ASTM D 3551 for compressive strength testing. The samples should be prepared at the moisture content and density expected in the field. Normally, for compressive strength testing, samples are not allowed to mellow before samples are fabricated. However, if it is difficult to achieve satisfactory homogeneity during laboratory mixing, it is reasonable to consider a mellowing period (between initial mixing and final mixing before compaction) of up to 24 hours to simulate field mellowing. However, with the high efficiency of lab mixing compared to field mixing, it is assumed that lab mellowing will not be necessary in most applications.

Triplicate samples are prepared for compressive strength testing following ASTM D 5102 procedure B with the lime content determined from the pH test. Samples are fabricated at between optimum moisture content (OMC) and OMC ± 1 percent. Additional mixtures with lime contents one and two percent higher than the optimal lime content identified by the Eades and Grim pH test as optimum should also be fabricated and tested following ASTM D 5102 to verify the optimum lime content, which may be greater than that identified by Eades and Grim pH test.

After compaction the test specimens should be wrapped in a plastic wrap and stored in an air tight moisture proof bag with about 10 ml of free water to ensure proper moisture for pozzolanic reactions. The specimens are then cured at 40^{o}C (1040F) for 7 days before compression testing. Since the accelerated cure is not always a good approximation of strength gain by long term normal cure, it is appropriate to subject one set of lime soil samples to normal cure for 28 days before compression testing.

After the curing period, the specimens are removed from the storage bags and plastic wraps are removed. The specimens are then wrapped with a wet absorptive fabric or geotextile and placed on a porous stone for capillary soak. Capillary soaking should continue for as long as it takes for the moisture front to move to the top of the sample or until the moisture front ceases to move. A soaking period of at least 24 hours is recommended. Research work by *Thompson* and *Little* demonstrated that the reduction in compressive strength due to soaking is not substantial (less than about 10 percent) for stabilized soil with a significant level of pozzolanic reaction. But the deleterious effects can be significant (up to 40 percent) if soaking occurs prior to significant pozzolanic strength gain. During capillary soak, the water used in soaking should never come in direct contact with the specimen. The water level should be maintained to the top of the porous stone and kept in contact with the fabric wrap.

f) Unconfined Compression Strength Testing

Following capillary soak, unconfined compression strength testing should be performed in accordance with ASTM D 5102 procedure B. The results of compression tests are compared with the suggested minimum requirements given in Table 2. If more than one lime contents are considered in compression testing, the lowest lime concentration that meets the compression strength requirement is considered as the required lime content for stabilization purposes. *If the specimens do not meet the strength criteria, then the soils can be considered as* ***modified soils*** *and* ***not stabilized soils****. Higher lime content may be used in these soils and the mix design procedure, starting from moisture density relationship, should be repeated.* It should be noted that the compressive strength values given in table below are suggested minimum values and field requirements may vary depending on purpose of Stabilization, exposure conditions, expected freeze thaw cycles and cover material over Stabilized soil.

2 Cement Stabilization

The American Concrete Institute (ACI) defines soil cement as a mixture of soil and a measured amount of cement and water mixed to a high density. Soil cement has been classically defined as a stabilized soil in which the coarse aggregate, sand size and larger (coarser than 75 μm) is surrounded and bonded by a matrix of cement paste and fine soil particles. The goal of mix design for this type of soil is to float the coarse aggregate in the matrix. The durability of this matrix is determined by durability tests such as AASHTO T 135 and T 136 (or by their ASTM equivalents D 559 and D 560) or by compressive strength testing. However, Portland cement has also been successfully used to stabilize fine grained silt and clay soils. In fact cement stabilization of silty soils provides perhaps the most dramatic improvement of any soil type (when the properties of the cement treated silty soil are compared to the properties of untreated soil). However, the amount of cement required to stabilize fine grained soils can be substantially more than that required to stabilize coarse grained soils because of the higher surface area of fine grained soils. The transition from silt to clay means that the particle surface area increases by orders of magnitude. However, in actuality cement does not need to coat all particles for successful stabilization and substantial improvement of moderately plastic clay soils, plasticity indices of below 30, has been achieved with about the same amount of Portland cement as would be required of hydrated lime. This is primarily because the cement forms a stabilized matrix around agglomerates of clay particles. Obviously if the integrity of cement matrix surrounding the agglomerates is compromised, then the durability of the matrix will begin to degrade.

The ability to stabilize soils with plasticity indices above about 20 with cement is based on the ability to intimately mix cement with the soil to a degree that will produce a reasonably homogeneous and continuous, stabilized matrix of the agglomerates. This requires a certain efficacy of mixing, which is in turn associated with the energy imparted to the soil by the mixing equipment and by the time span over which mixing occurs.

The limitation associated with mixing Portland cement with plastic clay soils is the short time of initial set of the cement, usually not more than 2 hours is provided for mixing before compaction. However, this mixing time has been extended under certain circumstances. During the extended mellowing period, the release of free lime during cement hydration alters plasticity and textural properties of the clay soil, which can improve workability. However, mixing following this extended mellowing must be performed with equipment that has the ability to impart sufficient energy to mix the soil and cement after the cement has reached a final set, which normally occurs within 8 hours. It must be understood, when extended mellowing is adopted, that all the strength lost during remixing may not be recovered with additional curing.

Hardened soil cement mixtures must withstand adverse environmental conditions. Other stabilization objectives include reducing plasticity index, increasing shrinkage limit, meeting strength thresholds, and improving resilient modulus. Soil cement can provide a strong and uniform support for pavement layers and provide a firm and stable working platform for construction.

In summary, most soil types, except those with high organic content, highly plastic clays and poorly reacting sandy soils, are amenable to stabilization with Portland cement. General gradation specifications limit the nominal maximum size at 2-inches with at least 55 percent passing the no. 4 sieve. For uniformly graded materials, the addition of non plastic fines like fly ash, aggregate screenings, cement and lime kiln dust may help fill the voids in the soil structure and help reduce the required cement content

a) Mix Design Considerations

As with lime stabilization, soils must be screened for organic content and sulfate content prior to verifying whether Portland cement is an acceptable stabilizer. Soils with higher organic content may require a higher cement content as the organic matter can inhibit normal hardening processes. A pH test, as recommended by the U. S. Army Corps of Engineers, using a mixture of 10 parts soil to one part cement (by weight) is used to verify if organic matter might interfere with the hydration process. If the pH of the paste after 15 minutes of mixing is 12.0 or higher then it is probable that organics will not interfere with the normal hardening process. If not, then higher cement content than that recommended based on AASHTO soil groups (Table 3) may be needed. Again the required cement content must be confirmed based on strength testing. The following procedure outlines the steps to be followed in developing an effective mix design for cement stabilized soils.

b) Preliminary Estimate of Cement Content

The first step in determining the required cement content is to classify the soil, AASHTO M 145. Table 3 defines a starting point to be considered in treatment. These cement contents are based on a data base of empirical evidence of soil cement mixtures that have proven to be able to meet the durability requirements established in AASHTO T 135 and T 136 or their respective ASTM equivalents D 559 and D 560. In Table 3, the cement quantities are proportioned on a weight basis in terms of the percent of oven dry soil.

Table 3. Cement requirement for AASHTO soil Groups.

AASHTO Soil Group	Usual Range in Cement Requirement		Estimated Cement Content, Percent by weight
	Percent by volume	Percent by weight	
A-1-b	7-9	5-8	6
A-2	**7-10**	**5-9**	**7**
A-3	**8-12**	**7-11**	**9**
A-4	**8-12**	**7-12**	**10**
A-5	**8-12**	**8-13**	**10**
A-6	**10-14**	**9-15**	**12**
A-7	10-14	10-16	13

These cement contents are only preliminary estimates and must be verified or modified based on additional test results. Additional cement requirement for soils with higher organic contents should be considered based on pH test of soil cement mixtures . It is important to understand that the requirements in Table 3 are based on durability tests, ASTM D 559 and D 560, and that many soils can be successfully stabilized with considerably lower cement contents.

c) Determine the Moisture Density Relationship

Changes in optimum moisture content and dry density with addition of cement are not always predictable. Flocculation of clay particles by cement can cause an increase in optimum moisture content and decrease in maximum dry density for cement-soil mixes whereas the higher density of cement relative to soil can result in a higher density for mixes. Therefore, it is appropriate to use the median cement content as estimated in Table 3 for determination of moisture density relationships as the maximum dry density varies only slightly with modest changes in percent cement content. However, as previously discussed, if it is expected that acceptable treatment can be achieved with considerably lower cement contents than those in Table 3, then that cement content should be used to determine the moisture-density relationship. After the required amount of cement is added to the soil, the blend should be mixed thoroughly until the color of the mixture is uniform. Fabrication and testing of samples for moisture density relationship should be done in accordance with AASHTO T 134 or its ASTM equivalent D 558.

d) Sample Preparation for Compressive Strength and Durability Testing

Two types of tests are typically used to evaluate the efficacy of a soil cement mixture: strength tests and durability tests. The Portland Cement Association (PCA) considers the ability to withstand adverse environmental conditions as the primary requirement for soil cements. The PCA manual recommends durability tests based on weight loss under wet-dry and freeze thaw conditions for evaluating usability of soil cement mixtures. Both PCA and ACI determine the weight loss in samples subjected durability tests in accordance with ASTM D 559 or ASTM D 560 as appropriate. These methods are highly subjective and carry significant user variability. In addition, these test methods may not reflect field conditions that are applicable to all stabilized pavement layers.

Since the results of freeze thaw testing does not simulate field conditions, many state departments of transportation currently recommend minimum unconfined compressive strength testing based on ASTM 1633 in lieu of durability tests. The research work by Thomson and Dempsey in lime stabilized soils has shown that compressive strength of samples subjected to freeze thaw can be used as a criteria in deciding durability issues in soil cements. Thompson’s data demonstrate that the compressive strength decreases by approximately 8-10 psi for every freeze thaw cycle endured. The U. S. Army Corps of Engineers recommends using 12 freeze-thaw cycles as described by ASTM D 560 (but omitting the wire brushing part) for cement modified soils. This method may also be considered an alternative method by which to assess the durability of cement stabilized soils.

Whether the cement requirements in Table 3 are used or alternative cement requirements are used, cement contents above and below the nominal value of cement should be considered. Therefore, the accepted approach is to prepare mixtures at the nominal stabilizer content and two percent above and below the nominal content. Again, the samples should be prepared following AASHTO T 134.

f) Unconfined Compressive Strength Testing

Compressive strength is indicative of the degree of reaction in the soil-cement-water mixture based on the rate of hardening of the mixture. Since the compressive strength is directly related to density, it is affected by the degree of compaction and water content in soil cement. Similar to lime stabilization, moisture conditioning of cement-soil mixtures is recommended prior to testing as most soil cement structures are either intermittently or permanently saturated during their service life. Preparation and curing of samples compressive strength testing should be performed in accordance with ASTM D 1632 which recommends moist cure for soil cement samples. Testing of cured samples should be done following ASTM D 1633 that requires the cured samples to be immersed in water for 4 hours prior to testing.

Typical ranges of unconfined compressive strength criteria of moisture conditioned soil cement specimens for varying soil classifications are given in Table 4.

Table 4. Range of compressive strength in soil cements.

Soil Type	AASHTO Classification	Soaked Compressive Strength (psi)	
		7 Days	28 Days
Sand and gravelly	A-1, A-2, A-3	300-600	400-1,000
Silty	A-4, A-5	250-500	300-900
Clayey	A-6, A-7	200-400	250-600

Strength requirement for stabilized layers may vary considerably from agency to agency. The required compressive strengths for soil cements shown in the Table 4 are based on ACI and the U. S. Army Corps of Engineers recommendations.

3. Fly Ash Stabilization

Fly ash typically contains at least 70 percent glassy material with particle sizes varying from 1μm to greater than 1 mm. Based on AASHTO M 295 (ASTM C 618), fly-ash can be classified into two groups: class C and class F. Class C refers to as a self cementing or cementitious fly ash that has enough available calcium to react with soil in the presence of water. Most of the calcium in class C fly ash is combined with the silica and/or alumina so that when water is added, a hydration reaction similar to the hydration reaction in Portland cement occurs. Some free lime is produced in the hydration process, as it is in the hydration of Portland cement. This free lime can participate in the pozzolanic reaction process between silica and/or alumina released from clay or silica and/or alumina from the fly ash, which are not combined with calcium.

Class C fly ash is a by-product of burning lignite or sub-bituminous coal in power plants. Class F fly ash on the other hand is more of a pure pozzolan, with a low concentration of available calcium. Therefore stabilization with class F fly-ash requires the use of an activator like lime or cement to initiate hardening processes during stabilization. Low lime ash or class F fly ash is formed during burning of anthracite or bituminous coal. Although these fly ash types are known to induce cementitious reactions in stabilized soils, mix properties cannot be predicted solely from chemical composition of the ash. Due to the complex nature of ash hydration, the utility of fly ash for stabilization applications must be based on physical properties of ash treated materials.

a) Mix Design Considerations

Prior to stabilization, the cementitious properties of fly ash should be characterized following ASTM D 5239-04. But, it should be noted that ASTM D 5239-04 does not evaluate the interaction between fly ash and soil or aggregate which must to be verified separately based on mix design procedures outlined in the following paragraphs.

3.1 Self Cementing Fly Ash

Class C fly ash can be used as a standalone material. At present there are no standard test procedures available for design of materials stabilized with self-cementing fly ash. The American Coal Ash Association recommends using moisture density and moisture strength relationships for developing effective mix designs in soils.

a) Design Considerations

For self cementing fly ash, one of the primary design considerations is the rate at which fly ash hydrates upon exposure to water. Hydration reactions can start immediately on exposure to water and hence the time delay in mixing and compaction of the specimens needs to be accounted for and included in laboratory mix designs. As hydration progresses, soil particles are bonded in a loose state and a portion of the compaction energy used in densification is lost in breaking bonds in the mix. Maximum dry density achieved for a given compaction energy therefore decreases with increase in compaction delay. In addition, compaction delay can cause a significant reduction in compressive strength. This is most likely due to the inability to maximize the impact of cemetitious and/or pozzolanic product development at lower densities. In other words, if a soil mass is under-compacted, the cemetitious/pozzolanic product does not have the same opportunity to develop “bonds” among soil particles (or agglomerates of particles) as they would if the soil mass were compacted to within a reasonable range of target density. This effect is much more likely to be significant in class C fly ash mixtures due to the faster rate of reaction.

An additional design consideration when selecting the optimal fly ash content is to determine the optimum moisture content at which maximum strength gain is achieved. Optimum moisture content for strength gain may typically be 1–8 percent below optimum moisture content needed to attain maximum dry density. This value may vary with soil type and the mineralogy of ash particles.

b) Mix Design

Addition of fly ash alters the compositional characteristics of soils and hence the moisture density relationship must be established for each soil type and fly ash content. These can be measured based on adaptations of ASTM C 593 and ASTM D 1633. Once the optimum moisture content for the mix is determined, the moisture-strength relationship is evaluated. In order to evaluate strength, specimens are prepared by blending soil, fly ash, and water and molded after the specified compaction delay. Test specimens are compacted at different moisture levels below optimum to determine the moisture content that will produce the maximum compressive strength. Test specimens are cured for 7 days at 38^{o}C (100^{o}F) in accordance with ASTM C 593 before compression testing. ASTM C 593 recommends moisture conditioning for 4 hours after curing period where the specimens are allowed to cool down to room temperature and are then immersed in water for 4 hours. However, as with lime mixtures, the authors recommend an alternative moisture conditioning regime of capillary soak until the moisture front ceases to migrate or for a minimum of 24-hours. The strength requirements typically vary based on objectives and requirements specified by the agency and these requirements should be followed in selecting the mix design for field application.

3.2 Non Self Cementing Fly Ash

For stabilization with non self cementing fly-ash, the addition of activators such as lime or cement is required to initiate stabilization reactions. These materials typically continue to gain strength after a curing period due to pozzolanic activity. The slow strength gain in these materials helps reduce shrinkage cracking and improves healing of micro cracks forming in the stabilized layers.

The methodology given below is adapted from coal ash association mix design procedure.
Typical fly ash contents in granular mixes vary from about 10-15 percent with activator contents varying from about 2-8 percent lime by weight of the mixture. These materials are similar to cement stabilized base in production, placement and in appearance. Strength development depends on curing time and temperature and is typically measured after accelerated curing of 7 days at 38^{0}C (100^{0}F).

a) Selection of Optimal Fly Ash Content

The first step in selecting the optimal fly ash content is to determine the utility of the stabilized product and the target level of strength required based on the utility of the product. The purpose of using fly ash in soils can broadly be divided into two categories: to achieve maximum strength for the mix or to achieve a target level of strength for the mix. If an aggregate base course is to be stabilized and the goal is to achieve maximum strength and durability then the strategy is to fill the voids with fly ash to achieve maximum density, then to determine the moisture density relationship for this optimal blend. This is followed by the addition of the amount of activator that will produce the maximum level of strength. If the goal is to achieve a target level of strength for either base courses or soils, then the strategy is different. In this case, experience or a trial and error process is required to identify trial fly ash percentages and activator contents. These estimates are used to establish moisture density relationships and to determine compressive strengths.

Five different samples are prepared with varying fly ash proportions typically starting at about 6 percent and ranging up to as high as about 20 percent by weight of the coarse aggregate fraction. Mixes are molded at estimated optimum moisture content in accordance with ASTM C 593 to determine the dry density of each mix. A two percent fly ash concentration above the proportion that gives the maximum dry density is selected as the optimum content for the mix. Optimum moisture content and maximum dry density are determined for the selected blend.

b) Sample Preparation for Selection of Optimal Activator Content

Determination of the optimal activator content is best achieved on a trial and error basis realizing that the required lime content or Portland cement content to activate the fly ash is typically between one part lime to three parts fly ash (1:3 ratio) to one part lime to four parts fly ash (1:4 ratio). Compressive strengths of the resulting mixture should then be compared to target values in order to judge whether or not the blend produces acceptable strengths for loading and environmental conditions.

If lime kiln dust (LKD) or cement kiln dust (CKD) is used as an activator, then higher activator ratios are required based primarily on the CaO content of the kiln dust. ASTM C 593 requires preparation of three replicate samples for compressive strength testing for each blend of fly ash and activator.

c) Curing of Samples for Compression and Durability Testing

Fly-ash soil mixes are cured for 7-days in sealed containers. Samples prepared with lime and kiln dust activators are cured at 37.8^{o}C (100^{o} F) for seven days. Portland cement activator fly ash mixes are cured at a 100 percent relative humidity environment at 22.8^{o}C (73^{o} F) for seven days. ASTM C 593 recommends moisture conditioning following curing period in which the samples are subjected to 4 hour soak after cooling to room temperature. Then the compressive strength of the samples is measured. However, the authors recommend the NLA capillary soak described under the NLA recommendations for lime mixtures as an alternative moisture conditioning regime.

d) Compression and Durability Testing

The three replicates prepared are tested for compressive strength testing should be subjected to vacuum saturation or strength testing without moisture conditioning as recommended in ASTM C 593. Durability testing in fly-ash soil mixes can also be performed in accordance with AASHTO T 136/ASTM D 560. But the issues discussed earlier regarding the effectiveness of AASHTO T 136/ASTM D 560 are applicable in this case also. In areas where there is no freeze thaw effect, durability testing may be waived in accordance with local practice.

e) Acceptability Criteria

A 7-day compressive strength of 400 psi is considered acceptable for field applications. A mix that attains the required properties with the lowest percentage activator is selected as the design mix for use in field.

4 Lime-Fly Ash

Lime and fly ash may be used to achieve mixtures with targeted strength instead of in an attempt to optimize strength of a mixture. This approach may be applied to any soil (coarse-grained or fine-grained). In this case various ratios of lime and fly ash should be tried until the target strength is achieved. A reasonable guideline is to begin with is to use four percent fly ash and increase the ash content in two percent increments for various trials. The initial trial activator ratio (lime content) added to each should be one part lime to two parts fly ash as a general rule, but this can be varied based on experience.

Approximately six trial ratios of lime and class F fly ash (three ash contents and two activator contents per ash content) should be used. A moisture-density relationship should be developed for each ratio to determine optimum moisture for each blend. Samples should be prepared at the target moisture content following ASTM C 593. Strength testing on each candidate mixture should be used to establish the acceptable mixture design. The authors recommend the same curing regime as described for lime stabilized soils.

On occasion, the goal is for fly ash to provide a strength increase to lime treated fine-grained soils that are not sufficiently pozzolanically reactive. This normally occurs in clay soils where the lime is effective in reducing plasticity and improving workability but not in providing the target strength. In this case an acceptable approach is to determine the lime content required based on the Eades and Grimm pH test. This content should provide sufficient lime to modify the soil and still provide sufficient residual lime to provide pozzolanic reaction. Next trial quantities of fly ash should be added to the blend beginning with four percent fly ash and increasing in two percent increments until acceptable strength is achieved. A separate moisture density relationship is required for each blend.

2.3.3.5 Infrequently-Used Fixation Techniques

Freezing and Verifications

Ground freezing involves lowering the temperature of the ground until the moisture in the pore spaces freezes. The frozen moisture acts to ‘‘cement’’ the soil particles together. The first use of this technique was in 1862 in South Wales. The process typically involves placing double walled pipes in the zone to be frozen. A closed circuit is formed through which a coolant is circulated. A refrigeration plant is used to maintain the coolant’s temperature. Since ice is very strong in compression, the technique has been most commonly used to create cylindrical retaining structures around planned circular excavations. Verification is a process of passing electricity through graphite electrodes to melt soils in situ. Electrical plasma arcs have also been used and are capable of creating temperatures in excess of 40008C. The soil becomes magma, and after several days of cooling it hardens into an artificial igneous rock. Although laboratory testing is ongoing, the electrical usage of the process to date appears to make it uneconomical. It is possible that the process could find application in the field of environmental cleanup.

2.4 Drainage techniques

Unacceptable movements of slopes, embankments, retaining structures, and foundations can frequently be eliminated by lowering table prior to earthquake shaking. A number of dewatering techniques have been developed and proven useful in engineering practice. Procedures for the design of dewatering systems are well established and widely used(eg., Cedergren ,1989;powers,1992).these standard techniques may be used to increase the stiffness and strength of a soil deposit for mitigation of seismic as well as non seismic hazards.

The building of excess pore water pressure during earthquake shaking can be suppressed using drainage techniques, although drainage alone is rarely relied upon for the mitigation of liquefaction hazards. The installation of stone column, for example, introduces columns of freely draining gravel into a liquefiable soil deposit (though mixing of the gravel and native soil during installation may reduce the permeability of the stone column).earthquake-induced excess pore pressure may be rapidly dissipated horizontal flow of pore water into the stone columns. The rate of pore pressure dissipation depends on the diameter and spacing of gravel drains (or stone columns) for mitigation of liquefaction hazards. The use of gravel drains for suppression of excess pore water pressure requires careful attention to drain permeability and filtration behavior of the drains-soil boundary. Even though drainage techniques can mitigate liquefaction hazards by suppressing excess pore water pressure buildup, post earth quake settlement may still occur. Case histories of the use of drainage techniques for mitigation of seismic hazards have been described by Ishihara et al.(1980),Aboshi et al.(1991),and lai et al.(1994)

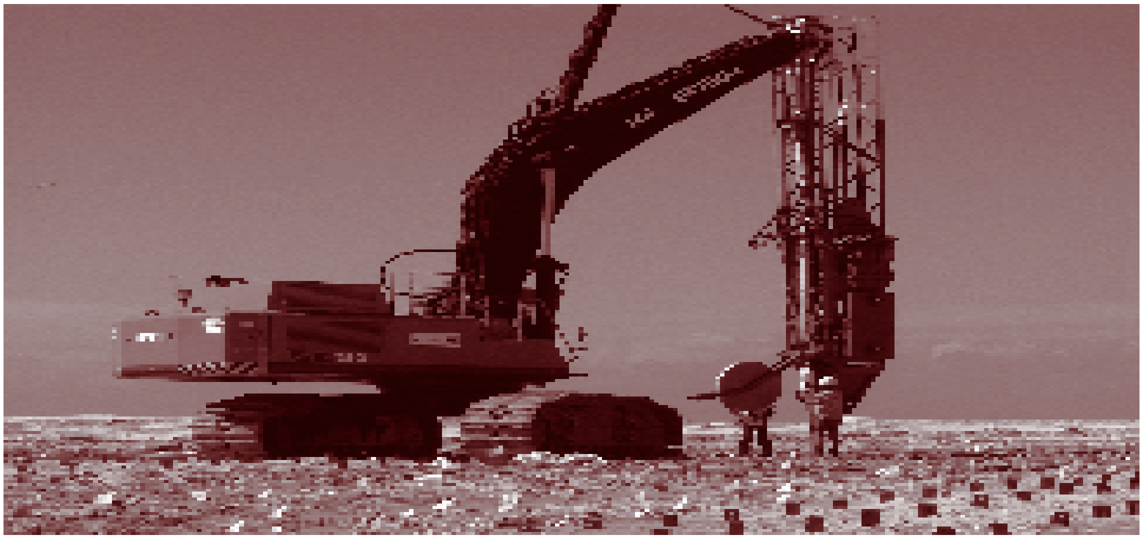
a) Vertical or wick drains

Soil stabilization using prefabricated vertical drains (PVDs) or wick drains are applied in areas with loose, compressible and water-saturated soils such as clay and silty clays. These soils are characterized by a very weak soil skeleton and a large pore space, usually filled with water (pore water). When a load such as a road embankment, a hydraulic fill or a dike, is placed on soft compressible soils, significant settlements may occur. These settlements can create serious problems. Any increase in load can result in an increase of pore water pressure. In impermeable soils, this water dissipates very slowly, gradually flowing from the stressed zone. Increased pore pressure may also cause soil instability and slip plane failures may result.

A vertical drainage system – drains are generally placed in a square or triangular pattern, spaced at about 1 to 3 meters – allows for a faster removal of excess pore water decreasing the risk of slip plane failure. The consolidation of soft cohesive soils using vertical drains can reduce settlement time from years to months ensuring that bearing capacity is adequate and construction can commence rapidly.[2]

b) The Beau Drain, IFCO and PTD systems

Recently several new systems for forced consolidation by pumping off groundwater have been developed. Variations of this technique are called IFCO, PTD and Beau Drain. The Beau Drain-IFCO-PTD concept combines existing, proven methods such as vertical drainage (wick drains), Atmospheric loading (vacuum consolidation), and the possibility to apply additional surcharge to accelerate the consolidation process of soft, compressible soils. The IFCO and PTD have slots made in the sand a short distance from each other at a depth of about 7 meters, with a drain at the bottom. The excess groundwater streams away from the surrounding land through this drain at a faster pace. The Beau Drain system works with closely placed rows of vertical wick drains, all connected to a horizontal collection drain. The horizontal collection drain is installed at a depth of approximately 1 to 2 meters below the top of the compressible strata and is connected to a vacuum pump, which through pressure, removes excess water.[2]



The wick drain system is one method used to hasten the removal of water from soil. Here, wick anchor plates are being used to mark the location of each wick prior to installation.

c) The sand drain techniques

The sand drain system allows the mixing of soil improvement ingredients, such as cement and anti-separating agents, with the sand which is left in a casing. When dry, this forms sand posts or piles which provide greater bearing capacity in the sub-sea soil in order to suit the needs of the project.

A specially developed sand drain vessel equipped with casings is available which uses this system. The vessel can mix the soil improvement ingredients with the sand on board forming sand posts at the desired locations. These casings are driven into the seabed to the required depth. The soil improvement is then carried out within the casings which form high quality sand piles on site.[2]

2.5 Other Innovative Soft-Ground Improvements Techniques

Rammed Aggregate Piers

Rammed aggregate piers (RAPs) are a type of stone column as presented in Section 12.3.1. Aggregate columns installed by compacting successive lifts of aggregate material in a pre augured hold (Figure 12.32). The predrilled holes, which typically have diameters of 24 to 36 in. (0.6 to 1.2 m), can extend up to about 20 ft. As seen in Figure 12.33, aggregate is compacted in lifts with a beveled tamper to create passive soil pressure conditions both at the bottom and the sides of the piers. RAPs are generally restricted to cohesive soils in which a pre drill hole will stay open.

FIGURE 12.32
Installation of rammed aggregate piers, a type of stone column. (From Geopiers Foundation Co. With permission.).

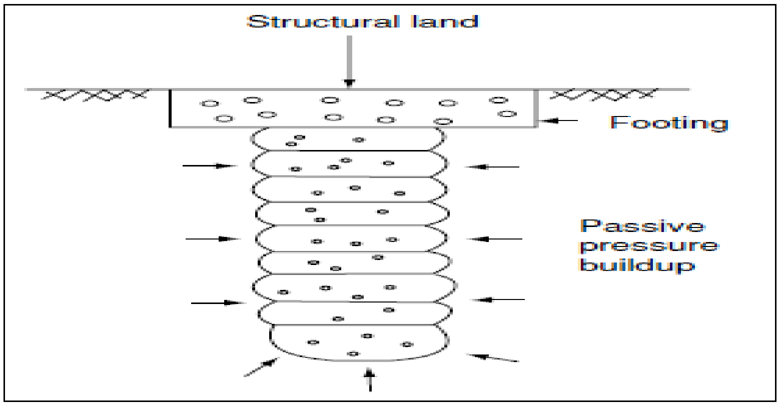


FIGURE 12.33
Schematic diagram of a rammed aggregate pier.

Although constructed differently than store columns or vibro piers (Section 12.3.1) all provide similar improvement to cohesive soils. The vertical tamping used to construct RAPs results in minimal densification in adjacent granular soils compared to vibratory probe construction. RAPs can be used in some of the following stone column applications that are outlined below:

1. Support shallow footings in soft ground.
2. Reinforces soils to reduce earthquake-induced settlements, however, does not densify sands against liquefaction.
3. Increase drainage and consequently expedite long-term settlement in saturated fine-grained soils.
4. Increase global stability and bearing capacity of retaining walls in soft ground.
5. Improve stability of slopes if RAPs can be installed to intersect potential shear failure planes.
6. Reduce the need for steel reinforcements when RAPs are installed below concrete mat or raft foundations.[7]

3. Verification of soil Improvement

All attempts at soil improvement should be checked to confirm that the desired improvement has taken place. The most direct way of verifying the effectiveness of particular soil improvement techniques is to measure the soil characteristic that was considered deficient both before and after improvement. For example, if the improvement was undertaken to increase the strength of the soil, measurement of strength before and after improvement would provide the most direct verification of the effectiveness of the improvement process. However, it is not always feasible to measure the deficient characteristics directly. In such cases, verifications is usually accomplished using related characteristics that more easily measured. Verification may be based on the result of laboratory or field tests. Field testing techniques may be divided into in situ testing and geophysical testing techniques.

Laboratory testing techniques

Laboratory testing techniques have number of advantages over other methods for verification of soil improvement, but they also suffer from drawbacks that can significantly limit their usefulness for certain types of soil improvement. The requirement of obtaining a sample of the improved soil leads directly to many of the advantage of using the laboratory testing techniques and also to many of the disadvantage. Obtaining a sample of improved soil allows visual inspection of the effects of improvement. For many improvement techniques (e.g., permeation grouting, soil mixing, etc),the ability to inspect the treated soil provides direct and available evidence of the effectiveness of the treatment. Laboratory tests allow greater control and more accurate measurement of stress, strain, and environmental conditions than are possible in field tests.

On the other hand, laboratory test only provide verification at discrete points. When soil improvement is used to improve or eliminate localized zones or seams of weakness, verification by methods that require discrete sampling may be ineffective. Laboratory tests may also be influenced by the inevitable effects of sample disturbance, a problem that is particularly significant in the improvement of liquefiable soils. The density changes produced by even thin-walled samplers (Marcuson et al., 1977; seko and Tobe; singh et al., 1979) can lead to considerable uncertainty in evaluation of improvement effectiveness.

In situ testing techniques

Many of the limitations of laboratory testing based approaches to the verification of soil improvement effectiveness may be overcome by the use of in situ tests indeed. The use of in situ test for verification of soil improvement effectiveness has increased dramatically in the past 15 to 20 years. Because many geotechnical seismic hazards are evaluated using in situ tests parameters. Those parameters can provide direct evidence of hazard mitigation.

The SPT, CPT, PMT and DMT can all be used for verifications of soil improvement effectiveness. The SPT and CPT are tests performed relatively quickly and inexpensively compared to sampling and laboratory tests. The CPT particularly is useful because it provides a continuous record with depth. The PMT is more expensive, but it also allows l measurement of lateral stress and direct measurement of strength. For gravely soils, a Becker Hammer penetration test may be used for verification purpose.

Soil improvement techniques that result in lateral stress may produce uncooperative estimate of the density of the improved soil if the post improvement stress state is not carefully considered in the interpretation of penetration test results. Because time-dependent changes in strength, stiffness and penetration resistance are often observed after densification (Mitchell and Solymar, 1984; Mitchell.1986). In situ tests performed immediately after densification may not reflect the actual degree of improvement of the soil. Verification testing is usually performed at least 72 hours after densification taken place. Many soil improvement techniques are applied at grid of treatment points, and the degree of improvement usually decreases with distance from the treatment point. The relationship between the location of in situ tests and the location of treatment points should be considered in the interpretation soil improvement effectiveness from in situ test result. In situ test have limited effectiveness for verification of grouting effectiveness (welsh, 1986).

Geophysical Testing Techniques

Many soil improvement techniques increase the stiffness of the treated soil. The effectiveness of these techniques can verified using seismic geophysical techniques. In most cases it is desirable to perform seismic tests both before and after improvement.

Crose-hole and down hole (including seismic cone) tests are most commonly used for verification of soil improvement. These techniques can measure p or S wave velocities over considerable distances. There by providing spatially averaged stiffness measurements. However, each requires at least one borehole. For sites where soil improvement has been performed over a large area, seismic reflection and seismic refraction tests may be useful for verification purposes. SASW tests provide similar information without the need for boreholes. At site where stiffness changes irregularly in two or three dimensions or sites that contain inclusions, the results of SASW tests may difficult to interpret. Such tests must also be performed when background noise (including, that produced by on-going soil improvement work) will not adversely affect their results. Tests that measure average wave propagation velocities may not accurately reflect the degree of improvement of thin, loose zones unless the distance over which over which velocities are averaged is quite small.

4 conclusion and Summary of paper

Ground Improvement techniques forms technically sound and cost effective solution where the sub soils are weak and needs to be treated to enable the intended construction. Its applicability has been proven in the recent past for a wide range of structures such as roads, runways, ports, power plants, railways, dams, slope stabilization, excavations, tunneling and other infrastructure facilities. These techniques have been used all over the world for a wide range of soils starting from loose sands, silts, marine clays to weak rocks. Based on the soil conditions, loading intensity and intended performance, an appropriate ground improvement technique can be designed to attain the desired performance.

Each technique has its own advantages and disadvantages in relation to time, cost and performance. The best method is always to consider the specific needs of a project and contact specialist contractors to evaluate the needs of the project. Although this evaluation stage may bring additional costs, proper preparation, be it through undertaking trials or field and laboratory testing and intense performance monitoring, will ultimately be recovered in the heightened efficiency with which the ground is secured. A well-managed soil improvement system appropriate to the site will enhance the prospects of on-time and safe project delivery.

The summary of the different improvement techniques for Bearing Capacity is shown below in tabulated form;

No	**Type of ground improvement technique**	**Applicable soil type**	**Material used**	**equipment**	**limitation**	**Quality control**
1	Vibro flotation (horizontal vibration)/ Vibro Rod (vertical vibration)	Free draining granular soils,upto silt content of upto 20%,	Water or Air jet	Cylindrical steel shell, electric or hydraulic motor	Not effective for sands &gravels with <15-20%	Date, time of compaction, DPT,SPT,CPT, most common effective is CPT
2	Vibro Replacement	Wide range of weak soils upto 30m depth (from silty sands, soft marine clays, ultra soft silts and clays from mine tailings, garbage fills to peaty clays.)	Coarse aggregate(such as stones),water jet	Crawler crane, loader, vibrator	Not suitable in liquid soils with very low undrained shear strength	(depth, vibrator energy during penetration and compaction processes and stone consumption) are recorded continuously, plate load test
3	Dynamic Compaction(DC)	Permeable, granular soils	Clean, free draining granular soil or crushed stone backfill	Crane with boom	Cohesive soils decrease the effectiveness ,water table presence at less than 6ft depth	Penetration test, shear wave velocity, large scale load tests
4	Compacting Grouting	Free draining granular soil	Portland cement, sand, and water	Injection pipe, drill-rig, batching ,pumping and injection equipment	Grout pressure can heave ground surface, grout bulbs are brittle, ineffective for small depths (<6meters)	Slump testing and UCS test during production, pumping rate, quantities, pressures, and ground heave and injection depth. penetration test sampling and strength
5	PVD	Soft, fine-grained soils, for layers of stiff soil may require pre-drilling.	Surcharge material	Trucks, Dozers, Ground pressure equipment, PVDs, backhoe or crane, drilling-rig maybe for stiff soil. corrugated or knobbed plastic ,woven		Height& unit weight of surcharge documented, record the location, depth ,and verticality
6	Infrequently-used compaction	Loose granular soils, dry soil, saturated soils	explosives	Drilling hole machine	Its effectiveness is difficult to predict in advance	Consideration of nearby structures
7	Vacuum-induced consolidation	Soft soil	Sand or gravel, PVD		Sand seams make difficult to maintain the vacuum	

8	Stone column	Gravel and Sand <10%silt,sand 20%silt and <2%clay,sand with>20% silt and non plastic silt clays	Graded crushed hard rock, natural gravel	Tremie pipe, loader, vibro-flot, plate load test equipment		Monitor and document the location, depth, ammeter quantity of stone backfill, penetration test, full scale load test
9	Vibro Concrete columns	Soft and/or organic layers ,depth of Ground water not critical	Concrete or cement mortar grout	Concrete pump, hose, treime pipe, vibroflot		Monitoring Pumping and extraction rates, Monitor and document the location depth, verticality, 10injection pressure and quantity
10	micro piles	Wide range, any soil or rock type	Steel rod or pipe, Portland cement grout, threaded bar	Drill rig, pile shaft, grout mixer, pump		Monitor the drilling penetration rate, strength testing, verticality of piles, load test with ASTM D1143
11	Fracture Grouting	Fractured soil, expansive soil	Portland cement, waterlime and flyash, potassium chloride and ammonium lignosufonate,	Drill rig, sleeve port pipe		Identify injection ports location (horizontally &vertically),check increaseing in situ moisture content, reducing pocket penetrometre readings, reducing average swell to 1%
12	Permeation Grouting	Sands and gravels with <18%silt and <2%clay,depth of water table not critical	Portland cement, water, sodium silicate, chemicals(acrylates and polyurethanes)	Mixing plant, grout, drill rigs, sleeve port pipes for critical geometry needed	Not suitable for cohesive soils(silts, clay) soils, maximum injection pressure about 20KPa/meter of depth	Permeability or UCS test after curing. Grout volume, grout pressure, flow rate should be monitored. cored the treated soil and test.
13	Jet Grouting	Wide range(granular)	Portland cement, water, some additives	Batching plant, pump, drill rig,		Test cylinder and cubes from the waste material, coring of the hardened soilcrete
14	Soil mixing(Deep Soil Mixing),DSM/DRY & WET mix/	Wide range: Cohesive Soft soils, weak soils such as; loose sands, soft marine clays, ultra soft slimes, weak silty clays and sandy silts.	Slurry, binders: Portland cement, lime,fly ash, ground blast furnace slag, and additives for wet &dry soil mix,	Batching system, temporary storage, high volume pumps, drilling system	Treatment of soil in ex-landfill areas,(large content of waste dump soils to be improved)Boulders and other obstructions can be problem	Uniformity of mix over the entire depth, recording operating parameters: depth of mixing tool, weight or volume of binder,flow rate of grout,rotationspeed and rate of penetration and withdrawal,;after curinf plate load test,UCS test,visual examination of exposed columns

15	Drainage techniques	Loose, compressible and water-saturated soils; such as clay and silty clays,	For sand Drain techniques; Cement, anti separating agents,	PVD,Vacum pump,wick anchor plates, casings, case driven machine		Consolidation tets
16	Rammed Aggregate Piers(RAPs)	Soft ground, cohesive fine grained soils,	aggregate	Compacting machine with crane, driil rig,	Restricted to cohesive soils	

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