İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

# DEEP SOIL IMPROVEMENT METHODS

**M.Sc. Thesis by**
**Gökçe CEYLAN**

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|---|---|
| **Department :** | **Civil Engineering** |
| **Programme :** | **Soil Mechanics & Geotechnical Engineering** |

**JUNE 2011**

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**Gökçe CEYLAN**
**(501091337)**

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**Supervisor (Chairman) :** **Prof. Dr. Ahmet SAĞLAMER (ITU)**
**Members of the Examining Committee :** **Prof. Dr. Derin URAL (ITU)**
**Assoc. Prof. Dr. Mehmet BERİLGEN (YTU)**

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**<u>İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ</u>**

# DERİN ZEMİN İYİLEŞTİRME METOTLARI

**YÜKSEK LİSANS TEZİ**

**Gökçe CEYLAN**
**(501091337)**

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**Tez Danışmanı : Prof. Dr. Ahmet SAĞLAMER (İTÜ)**
**Diğer Jüri Üyeleri : Prof. Dr. Derin URAL (İTÜ)**
**Doç. Dr. Mehmet BERİLGEN (YTÜ)**

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## FOREWORD

I would like to express my deep appreciation and thanks to Prof. Dr. Ahmet SAĞLAMER for all of his help, understanding and support. I also would like to thank to Enar Müh. Mim. Dan. Ltd. Şti. employees for their help, support and kindness during this period. Finally, I would like to thank to my family for their support and understanding throughout my life.

June 2011 Gökçe Ceylan

Civil Engineer

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## ABBREVIATIONS

| | |
|---|---|
| **ASCE** | **:** American Society of Civil Engineers |
| **ASTM** | **:** American Society of Testing Materials |
| **DSM** | **:** Deep Soil Mixing |
| **Es** | **:** Elasticity Modulus |
| **EM** | **:** Engineering Manual |
| **ETL** | **:** Engineering Technical Letter |
| **FHWA** | **:** Federal Highway Administration |
| **NP** | **:** Non-plastic |
| **RQD** | **:** Rock Quality Index |
| **SM** | **:** Soil Mixing |
| **SMW** | **:** Soil Mixing Wall |
| **SPT** | **:** Standard Penetration Test |
| **TCR** | **:** Total Core Percent |
| **TS** | **:** Turkish Standard |
| **USCS** | **:** Unified Soil Classification System |

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**DEEP SOIL IMPROVEMENT METHODS**

**SUMMARY**

In foundation design, soil behaviour that is designed under structural loads is the most important factor. Foundation design is made according to engineering properties of soils and its behaviour. If soil cannot bear structural loads or settlement occurs under these loads by changing the foundation system these problems can be removed. Furthermore, liquefaction which occurs by earthquake impact is an important problem for structure. Liquefaction problem can also be removed by deep foundation application. During the recent years, solution of encountered soil problems as an alternative of foundation system modification it has begun preffering soil improvement methods. The aim of soil improvement methods are; increasing bearing capacity of soil, preventing settlement and liquefaction, obtaining strength for soil and decreasing permeability. One of the reasons for increasing applications of soil improvement and preferring them to deep foundation system is that they are economic.

Soil improvement methods may be divided two categorizes: Shallow Improvement Methods and Deep Improvement Methods. In this thesis, predominantly deep soil improvement methods are investigated. Therefore, deep soil improvement case studies are given. As a case study jet grouting and dynamic compaction methods which are one of the most popular methods are investigated and considered. An Iron and Steel Plant Cold Mill – Galvanized Coating - Dyehouse Building Project in South Part of the Turkey is given as a jet grout case study. Furthermore as an another example, an Iron and Steel Plant Slitting Cutting Package Service and Cold Mill Project is given as a dynamic compaction case study.

## DERİN ZEMİN İYİLEŞTİRME YÖNTEMLERİ

## ÖZET

Temel sistemi tasarımında, zeminin inşa edilmesi planlanan yapı yükleri altında göstereceği davranış tasarımdaki en önemli etkendir. Temel tasarımı zeminin mühendislik özelliklerine ve davranışına göre yapılmaktadır. Eğer zemin yapıdan gelen yükleri taşıyamıyor ya da bu yükler altında zeminde oturma meydana geliyorsa temel sisteminde yapılan değişiklikle bu sorun ortadan kaldırılabilir. Ayrıca deprem etkisi ile zeminde meydana gelen sıvılaşma da üzerindeki yapı için oldukça önemli bir sorundur. Sıvılaşma problemi de gene derin temel uygulamasıyla ortadan kaldırılabilir. Son yıllarda zeminle ilgili karşılaşılan sorunların çözümünde temel sistemindeki değişikliğe alternatif olarak zemin iyileştirme yöntemleri de tercih edilmeye başladı. Zemin iyileştirme işleminin amacı; zeminin taşıma kapasitesini arttırmak, oturmayı ve sıvılaşmayı engellemek, zemine mukavemet kazandırmak ve permeabilitesini azaltmaktır. Zemin iyileştirme uygulamalarının yaygınlaşması ve derin temel sistemine göre tercih edilme sebeplerinden biri de daha ekonomik olmasıdır.

Zemin iyileştirme yöntemlerini genel olarak ikiye ayırabiliriz: Yüzeysel Zemin İyileştirme Yöntemleri ve Derin Zemin İyileştirme Yöntemleri. Bu tez çalışmasında ağırlıklı olarak derin zemin iyileştirme yöntemleri incelenmiştir. Ayrıca, derin zemin iyileştirme yöntemlerine uygulamalardan da örnekler verilmiştir. Uygulama örneği olarak Türkiye'de en çok uygulanan zemin iyileştirme yöntemlerinden 'Jet Grouting' ve 'Dinamik Kompaksiyon' yöntemleri incelenerek değerlendirme yapılmıştır. Jet Grout uygulama örneği olarak; Türkiye'nin güneyindeki bir Demir Çelik Tesisi kapsamında yapılan Soğuk Haddehane - Galvaniz Kaplama - Boyahane Binası, Dinamik Kompaksiyon uygulama örneği olarak da; bir Demir Çelik Tesisi kapsamında inşa edilen Sac Servis Merkezi ve Soğuk Haddehane Tesisi inşaatları anlatılmış ve değerlendirilmiştir.

# 1. INTRODUCTION

If the foundation soil has sufficient bearing capacity there isn't any problem to build proper foundation for designed structure. On the contrary, if the foundation soil is loose or soft which cannot carry the load from structure or superstructure the load must be transferred to deeper soil strata. In this situation pile foundation design can be an alternative but nevertheless it is an expensive solution.

There is also another method that is called as soil improvement. Soil improvement is an alteration of any property of a soil to improve its engineering performance.

All ground improvement methods try to improve soil's characteristic properties. There are various improvement methods in literature such as an increase in density and shear strength to treat problems of stability, the reduction of soil compressibility is influencing permeability to reduce and control ground water flow or to increase the rate of consolidation that relates settlement problems, or to improve soil homogeneity.

Soil improvement:

1. Increases shear strength,

2. Reduces permeability,

3. Reduces compressibility.

Modern ground improvement methods advance with technology developments and increasing awareness of the environmental consideration and also in the view of economical advantages. Because of this development standardizations of various improvement methods and technical recommendations are increasing day by day.

The selection of the optimum ground improvement technique is not only first but also the most important step. However, it has a significant effect on foundation choice and the budget. There are many factors that affect on the selection of improvement method. Firstly, the building's properties and the foundation system type should be examined. Then other factors such as field conditions, the buildings and their foundations which are around the construction site, ground water conditions, risk of earthquake and allowable settlement values are considered.

During the construction process deformation measurement system should be establish. Deformation measurement should be done very often and regularly.

Soil improvement methods can be classified as shallow improvement methods and deep improvement methods. Within the scope of this thesis, predominantly deep soil improvement methods are explained. After description of deep soil improvement methods case histories are given. Firstly, a case history of a jet grouting method in south part of Turkey would be given. Then, a case history that is about dynamic compaction would be explained and at the end of this thesis conclusions of these two methods are given briefly.

# 2. SOIL IMPROVEMENT METHODS

## 2.1 Shallow Soil Improvement Methods

### 2.1.1 Soil replacement

Soil replacement is one of the oldest and simplest methods. Soils which include organic soil or contaminated soil are suitable for this method.

Soil replacement process consists of excavating the soil which needs to be improved and replacing it. Replaced soil can be treated with various admixtures or it can be altered with a new soil which has adequate properties for the planning application. Practically, it is preferred if the soil is above the ground water table (U.S Army Corps of Engineers Washington, ETL 1110-1-185, 1999).

### 2.1.2 Admixture stabilization

Admixture stabilization means injecting and mixing admixtures into a soil to improve its properties. Used admixtures can be cement, lime or bentonite. These admixtures are used to improve soil's properties such as increase strength and decrease permeability. Admixtures fill voids of soil layer. This method is effective for soils in shallow foundations or road, airfield pavements.

**Soil lime stabilization:**

Lime stabilization method is suitable for fine grained soils. This treatment method improves the strength, stiffness and stability. Adding lime to soil produces maximum density under optimum water content. Therefore, lime stabilization decreases swelling potential and swelling potential in clays. The strength of wet clay is also improved by adding lime. When compared with untreated soils lime-treated soils have greater strength and a higher modulus of elasticity. 2 to 10 percentages of lime for soil stabilization is recommended. For coarse soils like clayey gravels and sandy soils the percent of lime varies from 2 to 5 and for fine soils the percent of lime is varies from 5 to 10. Also, lime can be used with fly ash. (Hausmann, Engineering Principles of Soil Modification, 1990).

**Soil cement stabilization:**

Soil cement stabilization is implemented by a mixture that consists of Portland cement, water and pulverized soil. The mixture becomes hard when cement hydrates. If the stabilization is made correctly the mixture is not affected by wetting, drying, freezing and thawing processes.

Soil cement stabilization is used under concrete pavements for highways and airfields. Soil cement mixture is also used to supply wave protection of earth dams.

There are three categories of soil-cement mixture (Mitchell and Freitag, 1959). They are: Normally, 5 to 14 percent cement by weight and is used for stabilizing low plasticity soils and sandy soils. For water proof canal linings and erosion protection; plastic soil-cement has enough water to produce a wet consistency similar to mortar is made. Cement modified soil mix consists of less than 5 percent cement by volume and it decreases swelling potential of soils and also it improves the properties of soils.

**Bituminous stabilization:**

Bituminous materials are asphalts, tars and pitches. These materials mix with cohesive soils to increase bearing capacity and soil strength. The main purpose of mixing bituminious materials with soil is getting imperviousness. Therefore, these materials acting as an agent when they are mixed with sandy soils so, they make soil stronger. For cohesive soils 4 to 7 percent, for sandy soils 4 to 10 percent of bitumen is sufficient.  Generally, this method is used for road construction. (V.N.S. Murthy, Principles and Practices of Soil Mechanics and Foundation Engineering).

## 2.2 Deep Soil Improvement Methods

### 2.2.1 Dynamic compaction

In the past, weak soil areas have been avoided or structures with deep foundations have been constructed over the top of the loose soil deposits.  Today, there are many types of soil improvement methods such as dynamic compaction to solve these kinds of problems. Dynamic compaction technique allows embankments and interchanges to be constructed directly on densified ground.

Dynamic compaction consists of dropping heavy weight onto the ground surface repeatedly to densify soil in a particular depth.

For loose, fully saturated, cohesionless soils; soil particles rearranged in a denser position. For unsaturated soil its execution is closely to proctor test. This application makes the soil more stable. At developed sites, a buffer zone around structures of about 30 to 40 meters is provided (U.S Army Corps of Engineers Washington, ETL 1110-1-185, 1999). Dynamic compaction is most suitable for sands and silty sands. The effective depth of DC is about 10 meters. Typically, 10 to 30 tons of weights are dropped onto the ground surface from heights of 15 to 30 meters. (FHWA-SA-95-037).

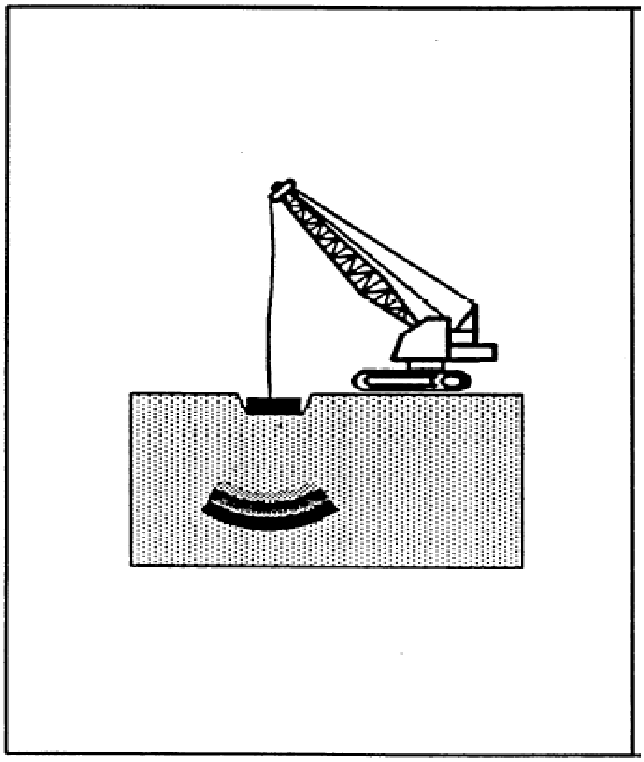


**Figure 2.1 :** Application of a dynamic compaction (from Hayward Baker,1996).

The relationship between the effective depth, the weight and the height of the drop can be expressed as :

$$D = n\sqrt{WH} \tag{2.1}$$

Where:

D = depth of improvement in meters

W = mass of hammer

H = drop height in meters

n = empirical coefficient that is less than 1.0

and, n has been found at project sites to range from 0.3 to 0.8.

The variation of  n is depends on:

**a.** Efficiency of the drop mechanism

**b.** Total amount of energy whic is applied.

**c.** Type of soil deposit which is desired being densified.

**d.** Presence of energy in absorbing layers.

**e.** Presence of a hard layer

**f.** Contact pressure of the tamper.

### 2.2.2 Vibro-compaction

Vibrating probes are used to improve soil in this method. The vibrating probe is driven into the desired depth to densify soil.  Soil starts to densify as the probe is inserted and withdrawn repeatedly. This process occurs a cavity in ground and it is backfilled with sand or gravel to create a densified column in this area. This method is suitable for sands and gravels. Probe spacings depend on several factors such as soil type, backfill type, probe type, energy and rate of improvement which is required for project. Approximate variation of relative density with effective area per compaction probe for a sand backfill is shown in Figure 2.2 (FHWA,1983).

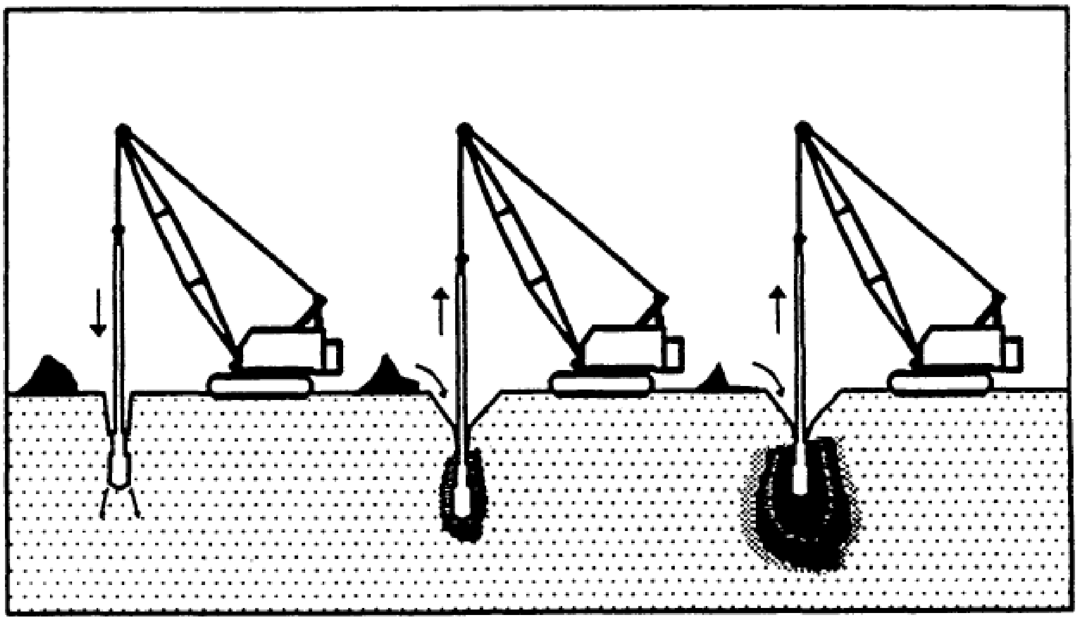


**Figure 2.2 :** The vibrocompaction process (Hayward Baker, 1996).

**Table 2.1:** Effectiveness of vibro-compaction according to ground type.

| Ground Type | Relative Effectiveness |
|---|---|
| Sand | Excellent |
| Silty Sand | Marginal to Good |
| Silt | Poor |
| Clay | Not Applicable |
| Mine Spoils | Good (If Granular) |
| Dumped Fill | Depends Upon Nature or Fill |
| Garbage | Not Applicable |

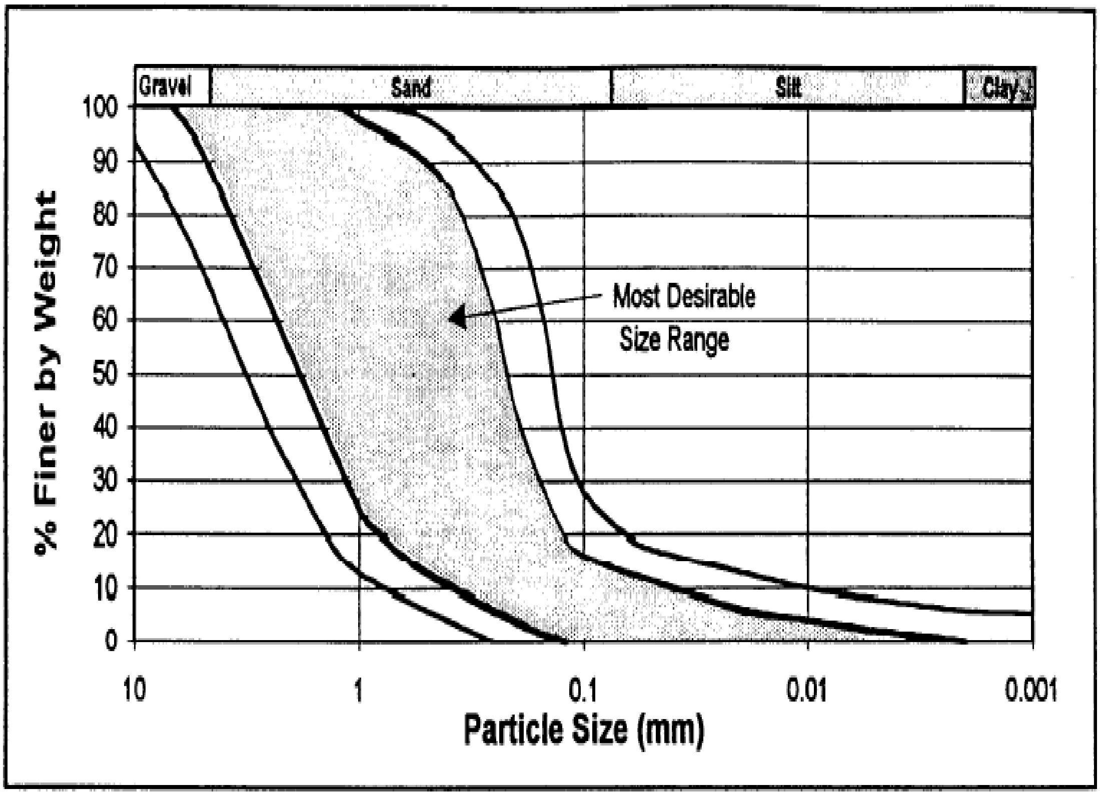



**Figure 2.3 :** Range of particle size distributions suitable for densification by vibrocompaction (U.S. Army Corps of Engineers, ETL 1110, 1999).

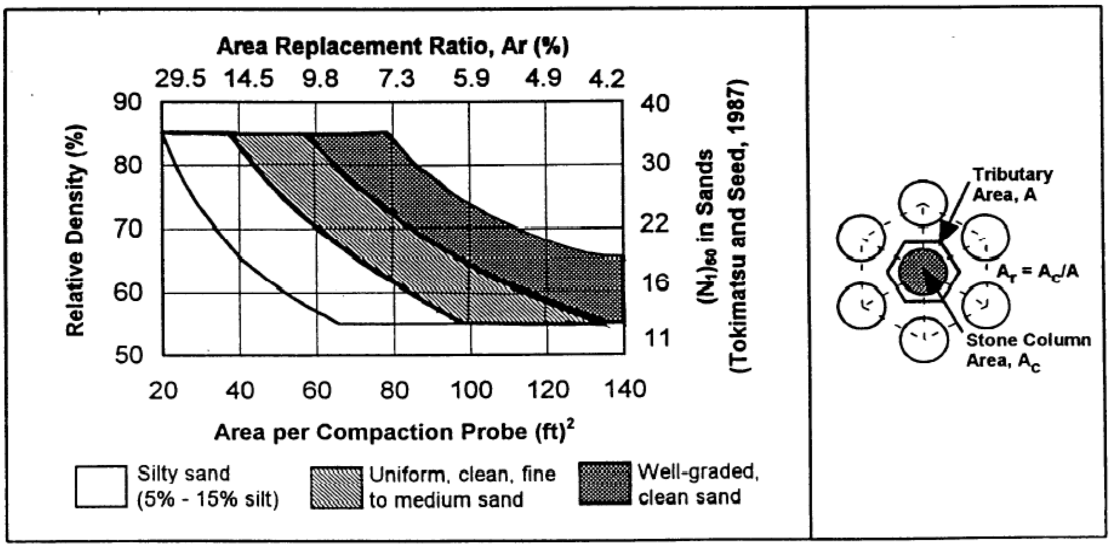



**Figure 2.4 :** Approximate variation of relative density with tributary area or area replacement ratio (after FHW~ 1983).

### 2.2.3 Stone columns

Stone colums are most suitable for soft silts, clays and loose silty sands. The purpose of this method is that increasing bearing capacity and slope stability, decreasing settlement, decreasing the time rate of consolidation and also decreasing liquefaction potential. Stone columns also may be used to support reinforced earth structures. The stone column treatment method is used for economical stabilization of soft soils. There are many factors to applicate stone columns for improving soft soils. These factors can be summarized as follows:

- An effective method for stabilizing large area loads such as embankments and tank fills.
- Between 20 to 50 tons design loads per column
- One of the most effective improvement for compressible silts and clays

If the soil contains organics and peat layers large vertical defflections may result. Thus, when using stone columns in sensetive soils or organic soils special care must be taken.

When compared stone columns with other alternatives stone columns may be more economical if the soil conditions are suitable. Another important benefit is that time dependent settlement is essentially decreased by using stone columns. In Europe, length of the stone columns are commonly chosen 4-10 meters. Less than 6 m is an economical choice however more than 10 m is not an economic alternative for stone column application. Furthermore, to install ver deep stone columns are followed by many construction problems such as insuring stabilization of the hole. Both European and American contractors have experienced many problems in design and construction of very deep stone columns (21m). (FHWA-RD-83-026).

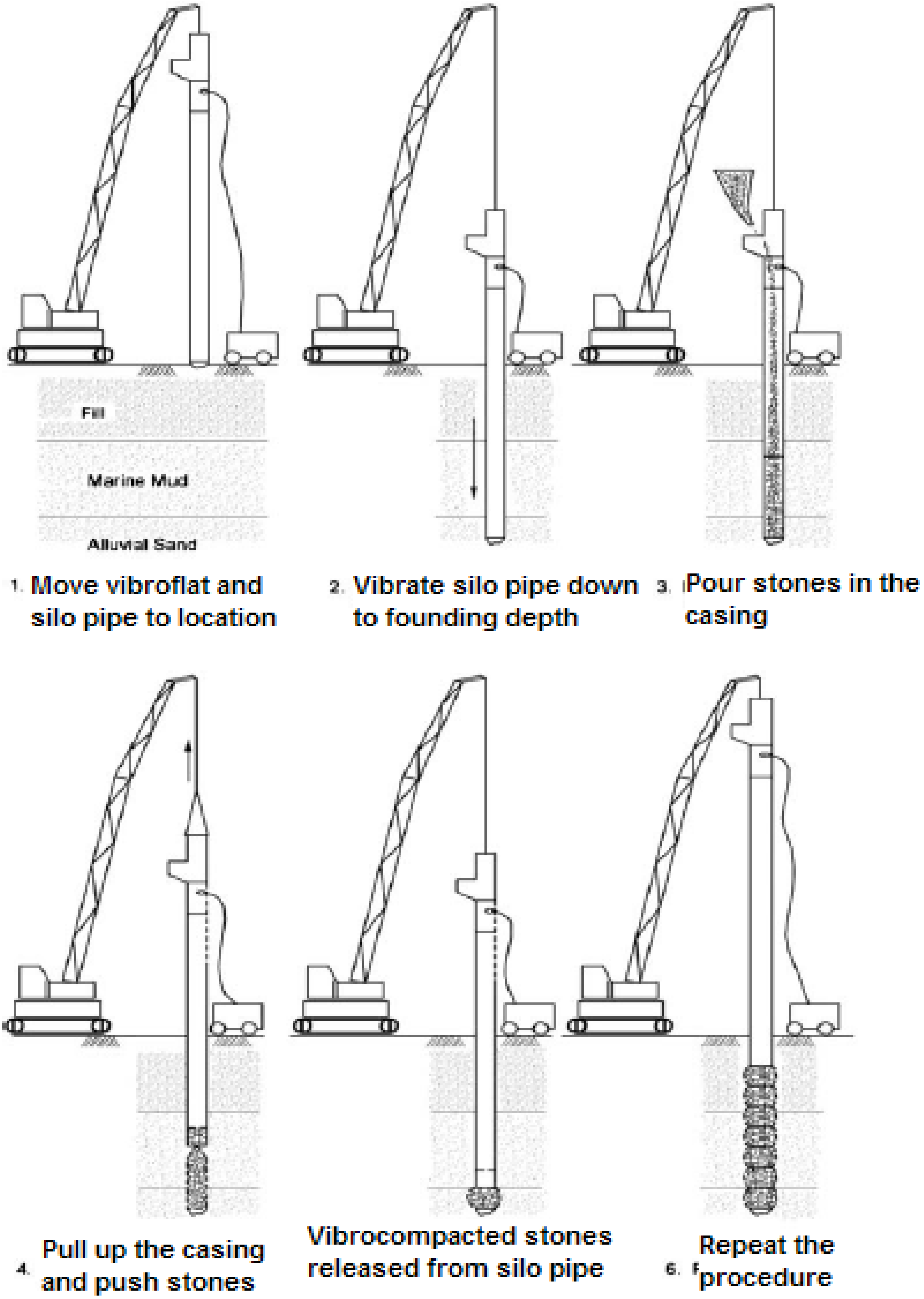



**Figure 2.5 :** Stone column installation process (De Silva, Maunsell Geotechnical Services, Hong Kong).

**Construction of stone columns**

Vibro replacement (The wet process):

A hole is formed into gorund by jetting to the desirable depth. Then, stone is added into the hole and it is densified by means of a vibrator. Vibrator is placed bottom of the probe. The wet process is generally prefered when the stability of hole is questionable. This application method is suitable for very soft soils under high ground water table conditions.

Vibro displacement (The dry process):

In vibro-displacement process, jetting water is not used while the hole is forming. It is the main difference between the wet and the dry process. Furthermore, to be able to chose this method the soils’ undrained shear strengths in excess of 40-60 kN/m2 and low ground table condition is required. Also, the hole must be stable during the application.

Rammed stone columns:

Rammed stone column is formed by means of an end pipe which is driven into ground and boring a hole then, a mixture of sand and stone is placed in the hole. A heavy falling weight is used to ramming the mixture.

Sand compaction piles:

Sand compaction piles are used extensively in Japan. A steel casing is driven into desired depth of soil by means of a heavy, vibratory hammer. Steel casing is removed from soil when the sand compaction pile is completely occured. Sand piles are suitable to use for soft clay soils under high ground water table conditions.

Stone columns are installed in cohesive soils or silty sands. A cylindirical hole is formed by vibrator and it is filled with gravel or crushed rock. Compaction is gotten by vibration during the process. Dimension of stone columns depends on soil condition, equipment which is used and construction procedures. It can also be used to support footings and walls. (FHWA-RD-83-026).

### 2.2.4 Gravel drains

Gravel drains are a type of stone column proposed for use in liquefiable soils to mitigate liquefaction risk by dissipation of excess pore water pressures generated during earthquakes (ASCE, Geotechnical Specification No.69, 1997).

Gravel drains can be used as a treatment method for liquefiable soil zones or as a treatment around cured zones to prevent pore pressure from adjacent untreated soil. A typical layout for gravel drains is shown in Figure 2.6. Gravel drains installation is smilar to stone columns' installation process except, they installed in cohesionless soil.

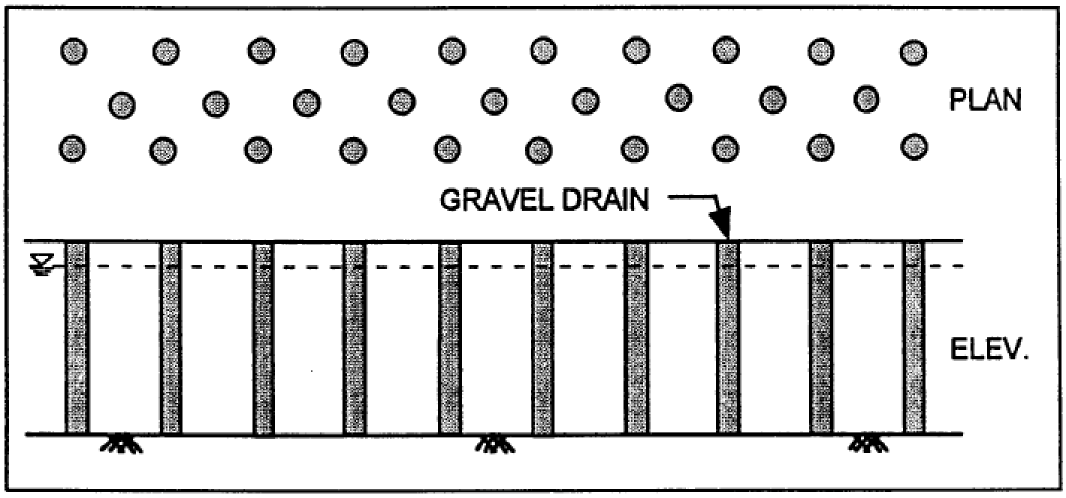



**Figure 2.6:** Arrangement of gravel drains (Seed and Booker, 1977).

Seed and Booker (1977) first proposed design methods for gravel drains to prevent liquefaction of sands. Their purpose is that drainage would develop on radial direction thus, the drain resistance could be neglected. In reality, seepage develops vertically in the drain so the drainage path is much longer than Seed and Booker' assupmtion. As a result, drain resistance would be an important design factor. Onoue (1988) presented design diagrams which consider drainage path length and drain resistance. Boulanger et al. (1998) performed designs using both methods. Boulanger et al. (1998) presented a detailed discussion of design for gravel drains. Boulanger et al. (1998) found that a drain permeability of 200 times the native soil permeability is not adequate to neglect the effects of drain resistance.

Construction mistakes can results low permeability. Intermixing native soil and drains material can cause to occur low permeability which would be less than 100 times of native soil's permeability. Rows of gravel drains around the treated zone would be useful in intercepting excess pore pressure from adjacent liquefiable soil as mentioned at the beginning. (U.S. Army Corps of Engineers Washington, ETL 1110-1-185, 1999).

### 2.2.5 Sand and gravel compaction piles

Compaction piles densify the soil. The installation of compaction piles densify the soil by occuring vibrations while driving them to ground or they cause displacement of a volume of soil that is equal to piles' volume. Generally, piles are spaced 1 to 3 meters from center to center.

In loose sand to increase the density it is assumed that sand pile causes compaction only in a lateral direction. Pile spacings can be determined by using;

For sand piles as a square pattern:

$$S = d\sqrt{\left(\frac{\Pi(1+eo)}{eo-e}\right)} \tag{2.2}$$

And

For piles in a triangular pattern:

$$S = 1.08d\sqrt{\left(\frac{\Pi(1+eo)}{eo-e}\right)} \tag{2.3}$$

Disadvantages of compaction piles are they are relatively expensive and application is slow.

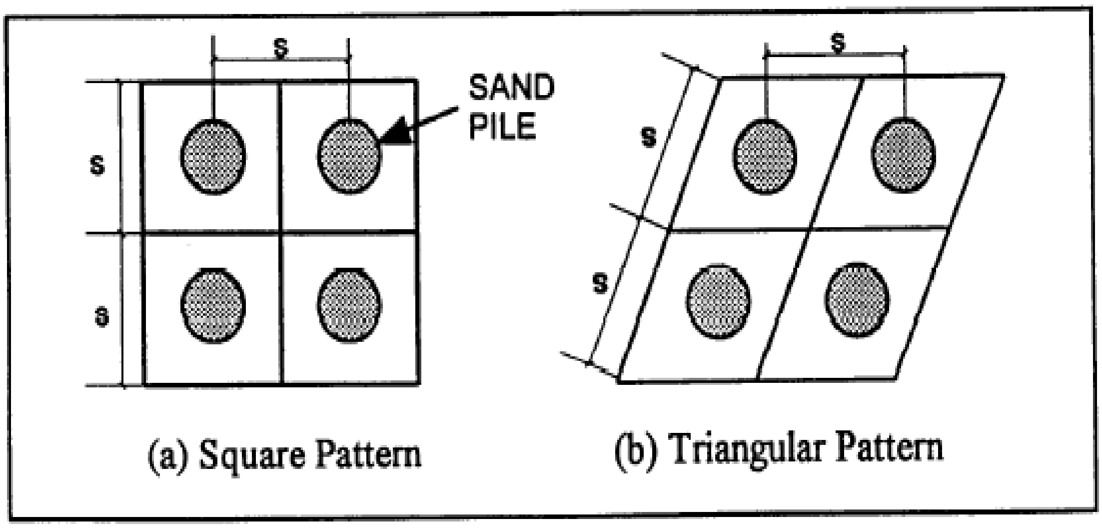



**Figure 2.7 :** Usual compaction pile patterns (U.S. Army Corps of Engineers, ETL-1110, 1999).

### 2.2.6 Explosive compaction

The detonation below ground surface occurs densification in explosive compaction method. The detonation compacts soil to a denser and more stable configuration by blasting and gravity.

Hydroblasting is generally used to treat collapsible soils. Hydroblasting process is that saturated soil is prewetted before detonation. A typical layout for explosive compaction is shown in Figure 2.8.

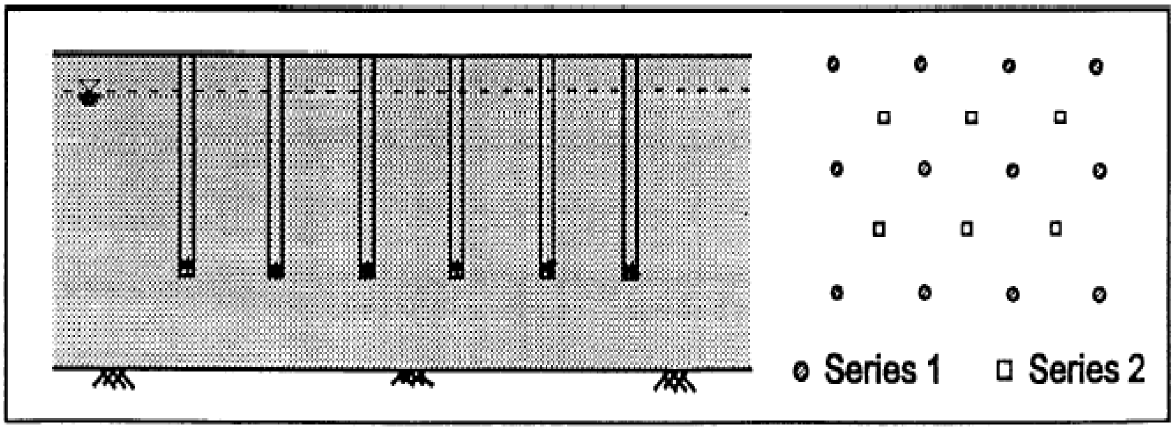



**Figure 2.8 :** Typcial layout for explosive compaction (U.S. Army Corps of Engineers, ETL-1110, 1999).

Explosive compaction is most suitable for clean sands and silty sands. Therefore, this improvement method is time-dependent. It may takes several weeks or months. Typical charge spacings are 3 to 8 meters in developed areas and 8 to 10 meters in remote areas with charge weights between 2 to 15 kilograms (U.S. Army Corps of Engineers Washington, ETL 1110-1-185, 1999).

For soil layers less than 10 m thick the charges are usually placed at a depth between one-half and three-quarters the thickness of the layer to be treated, with a depth of two-thirds the layer thickness common. If a layer is more than 10 m thick it is recommended that it be divided into sublayers, where each sublayer is treated separately with decked charges. In explosive compaction method depending on the amount of explosive is used and the initial conditions of soil surface settlement of 2 to 10 percent are expected (Narin van Court and Mitchell, 1994).

### 2.2.7 Permeation grouting

Permeation grouting is a filling process with grout in pore spaces of soil or joints of rock. Maximum injection grouting pressure is 20 kPa per meter of depth. It is limitted to prevent some changes in native formation. Injection grouting is limited to coarse grained soils. However, grout must be able to flow through the spaces and joints easyly. There are particular grouts such as cement and bentonite which are suitable for medium to coarse sands. For fine-grained soils micro-fine cement can be used for treatment it can be able to penetrate formation. For smaller pore spaces chemical grouts such as silicates can be used. There are many kinds of chemical grouting may be used. The most common types are sodium silicate, acrylate, lignin, urethane, and resin grouts that their properties are illustrated in Figure 2.9 below. (U.S. Army Corps of Engineers Washington, ETL 1110-1-185, 1999).

| | Property | | | | | |
|---|---|---|---|---|---|---|
| Type | Penetration in Grouted Units | Durability | Ease of Application | Potential Toxicity | Flammability of Materials | Relative Costs |
| Portland-cement-based grouts | L[1] | H | M | L | N | L |
| Silicates | H | M | H | L | N | L |
| Acrylates | H | M | H | M | L | H |
| Lignins | H | M | H | H | L | H |
| Urethanes | M | H | M | H | H | H |
| Resins | L | H | M | H | M | H |

[1] N = non-flammable; L = low; M = moderate; H = high.

**Figure 2.9 :** Ranking of major grout properties ( US Army Corps of Engineers, EM 1110-1-3500, 31 Jan 1995).

Rows of grout holes are used to develop a seepage barrier for water cut-off applications. Penetration grouting can also be used for ground strengthening which is possible with incomplete replacement of the pore water and preventing liquefaction process which is required complete replacement of the pore water. ( US Army Corps of Engineers, EM 1110-1-3500, 31 Jan 1995).

**Advantages & disadvantages**

- Chemical grouts don't involve solid particles. Thus, chemical grouts can be injected into foundation soil which contains very small voids. Other types of grouts such as cementious grouts are not suitable for this kind of too small voids.

- Chemical grouts are also used to control water movement into voids and they can increase the strength.

- Generally chemical grouts are used for stabilizing and increasing bearing capacity of fine grained soils.

- Chemical grouts are more expensive than other grout types. If there are large voids in the soil, usually, cementious grout type is prefered.

- Unfortunately, chemical grouts have toxic effects for the enviroment. While selecting grouting type it is also important to consider groundwater pollution.

**Grouting equipments**

Firstly, mixing and blending tanks should not react with chemical grout. These tanks can be made of aluminum, stainless steel, plastic, or plastic-coated.

There are different grout mixing methods. One of them is called batch system which is the simplest. Conventional portlant cement grouting is used for this system. In this system all of the components are mixed in a tank at the same time. Besides it's simplicity there can be clogging in the flow channels. Because in this method pumping time is limited.

Another method is two tank system. In this system, catalysis components are put in a tank and other components are put into other tank. These materials are sent into a common pump then the grout is injected to the desirable point.

There is also another method which is called as equal volume method. In this method, exact pumps are attached to each tank and they are controlled from a mutual order. The components in each tank is mixed in desing concentration twice and it supplies an advantage that is two grout components are mixed appropriately by manufacturer.

**Pumping systems**

**a. Variable volume pump system**

In this system one man can control gel times, pumping rates, and pumping pressures by mechanical means. Two variable motors offers without changing the concentration of the mixture the gel time can be changed and also the pumping volume can be changed without changing the gel time.

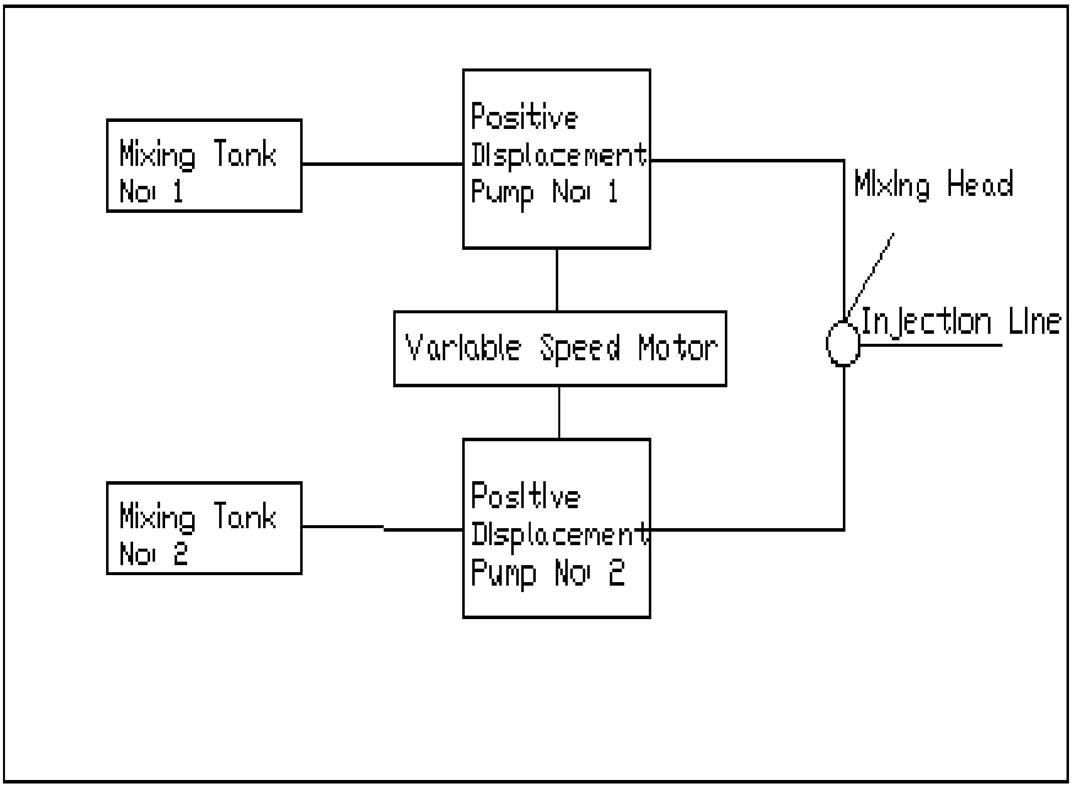



**Figure 2.10 :** Variable-volume pump system or proportioning system (U.S. Army Corps of Engineers EM 1110-1-3500, 1995).

**b. Two tank gravity feed system**

This system allows one predetermined gel time. In this method mixing tanks should be at the same size and volume. Also, the solution’s surface level shoul be the same, either. The solutions from mixing tanks are placed into bleding tank. After mixing process they are fed to the pump.

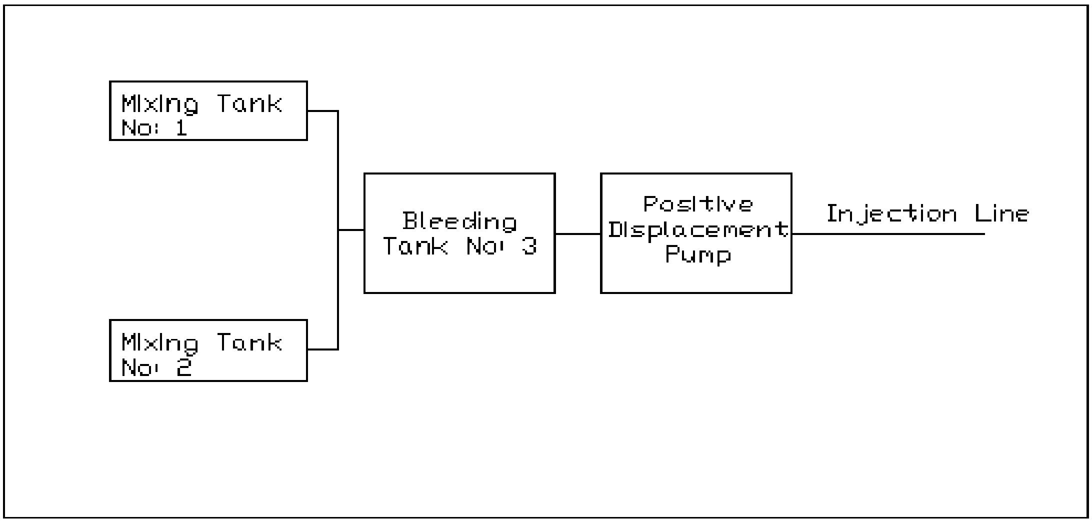



**Figure 2.11 :** Two tank gravity-feed system (U.S. Army Corps of Engineers EM 1110-1-3500, 1995).

**c. Batch system**

In batch system all materials are mixed in a tank. The whole batch must be placed during the gel time. Very short gel times cannot be used for this sytem. If only small batches are used short gel time would be useable.

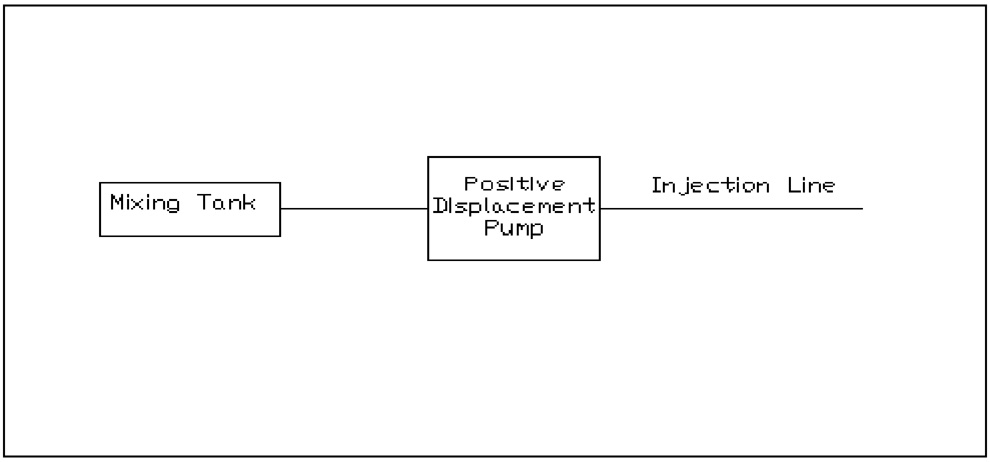



**Figure 2.12 :** Batch system (U.S. Army Corps of Engineers EM 1110-1- 3500, 1995).

**d. Gravity feed system**

In some circumstances it is allowed that the grout flows through it's location by pumping or pouring. This can be making by means of mixing units. (U.S. Army Corps of Engineers Washington, EM 1110-1-3500, 1995).

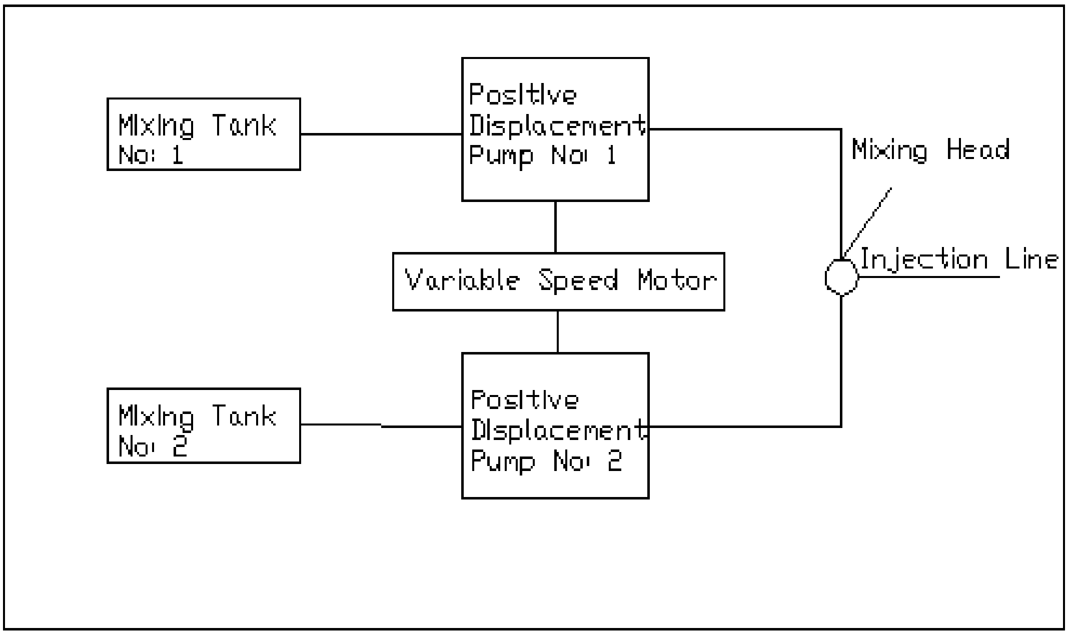



**Figure 2.13 :** Two-tank gravity-feed system (U.S. Army Corps of Engineers EM 1110-1- 3500, 1995).

**Injection methods**

The aim of the grouting is that to place specified amount of grout into desired location. The common and also the simplest grout placement application is doing by pumping and pouring grout into hole or fracture. Before injecting the grout packers or tubes can be placed into the opened hole then grout enjection is made. Additionally, it is possible to drive pipes to desired elevation and then grouting. The pipe keeps open the drilled hole during the injection temporarly. (Hayward Baker, 1995).

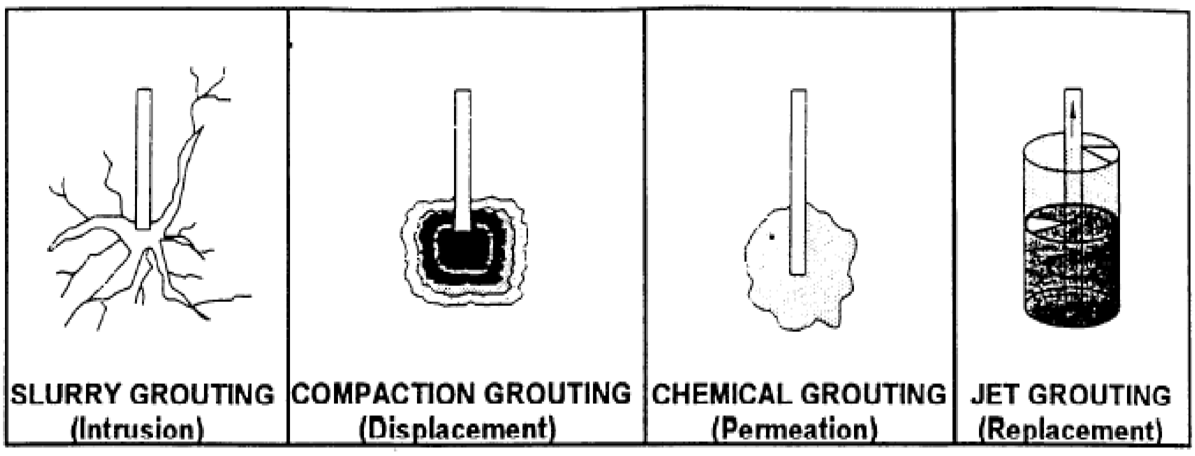



**Figure 2.14 :** Types of grouting (Hayward Baker, 1995).

### 2.2.8 Compaction grouting

Compaction grouting is suitable for loose soils, collapsible soils, unsaturated fine grained soils and cavities. The purpose of compaction grouting method is that causing densification by injecting a very-low slump grout into soil. The grout compresses the surrounding soil and it is a mix of sandy soil, cement and water. The grout forms like a bulb and it hardens in time. Accordingly, the void ratio of the soil formation decreases. The pumping rate depends on the soil type and 5 to 7 feet spacing is commonly used. (Warner et al., Recent Advances in Compaction Grouting Technology, 1992).

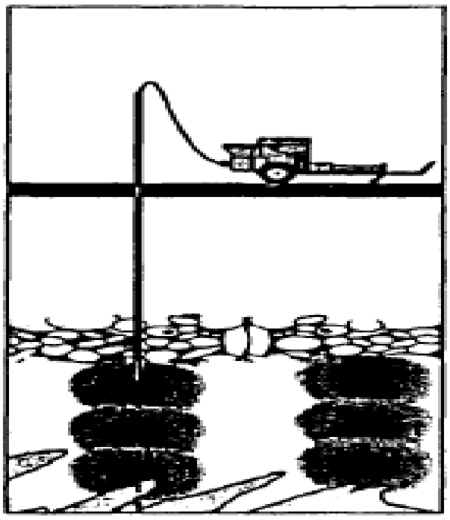


**Figure 2.15 :** Compaction grouting bulb construction (ASCE, Geotechnical Specification Publication No.69, 1997).

### 2.2.9 Jet grouting

Jet grouting is suitable for most soil types. The aim of the jet grouting method is that mixing the native soil with stabilizer such as cement or bentonite to create columns. These columns can be used to get low permeability or high strength depending of their function. Commonly, jet colums are up to 1 m but to form larger columns are also possible. This method is best fits on cohesionless soils. To get qualified jet columns can be difficult for cohesionless soils especially, for high plasticity clays.

This treatment method can be used for most soil types, it can be applied for a specific layer which is needed to improve and the injection rods can be inclined. Despite it's advantages jet grouting is an expensive method. (ASCE, Geotechnical Specification Publication No.69, 1997).

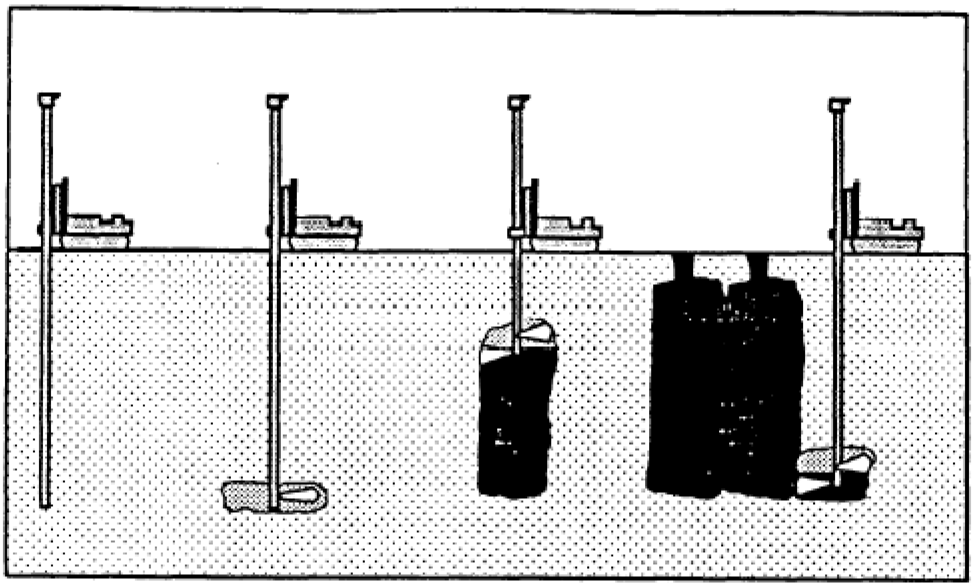


**Figure 2.16 :** Jet grouting process (Hayward Baker, 1996).

### 2.2.10 Deep soil mixing

Deep mixing method is an in-situ soil improvement method that the soil is mixed with admixtures. Admixtures are injected to soil at a specific depth which is desired to improve and then they are mixed with native soil to form columns. These columns diameter can be up to 1 m or more. The purpose of the method is that to increase strength, to decrease permeability and compressibility. Generally, cement and lime are used as admixtures. The formed columns can be used solely or in groups. Therefore, deep mixing method can be used to lessen the liquefaction risk of soils. (ASCE, Geotechnical Specification Publication No.69, 1997).

Applications:

There are various deep mixing methods for treatment as shown in Figure 2.17.

. Single elements.

. Rows of overlapping elements (walls or panels).

. Grids or lattices.

. Blocks.

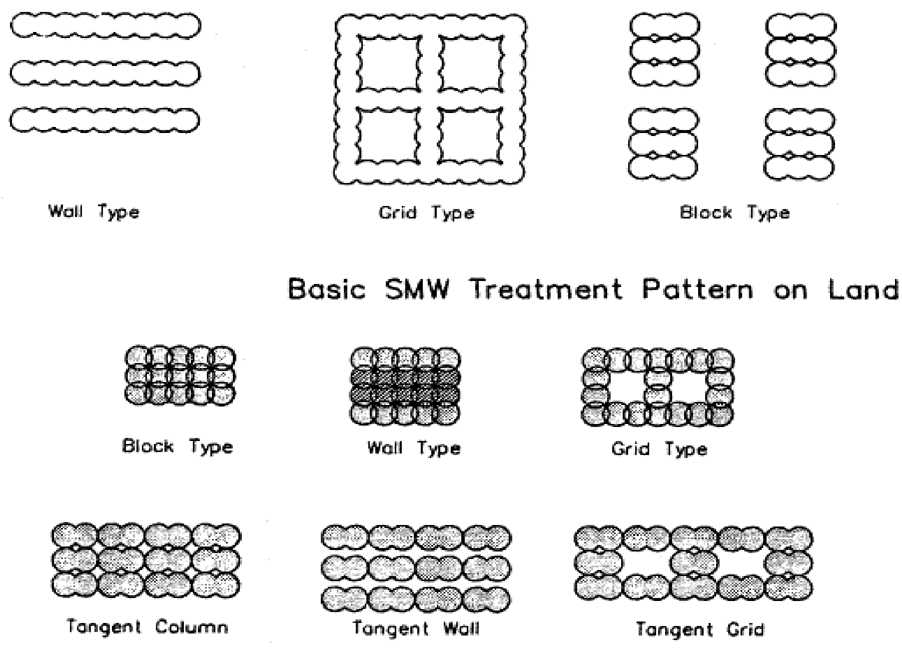



**Figure 2.17 :** Basic Deep Mixing Treatment Patterns (Yang, 1997).

Therefore, the applicability of deep soil mixing method depends on soil type an it's effects are summarized in table.

Mainly deep soil mixing application types are as follows,

1. Hydraulic cut-off walls (Japan, U.S.).

2. Excavation support walls (Japan, China, U.S.).

3. Ground treatment (Japan, China, U.S.).

4. Liquefaction mitigation (Japan, U.S.).

5. In situ reinforcement, piles, and gravity walls (Sweden, Finland, France).

6. Environmental remediation (U.S., U.K.).

**1. Hydraulic cut-off walls (Japan, U.S.)**

Hydraulic cut-offs are formed by installing overlapping columns or panels to prevent the seepage flow path. The columns or panels are installed through the permeable soil layer, usually it is installed the top of the bedrock. Generally, highly permeable coarse soil deposits or interbedded strata of fine and coarse grained soils are suitable for this treatment.

A major seepage control project was conducted in conjunction with the installation of a new radial gated spillway structure for Lake Cushman Dam, near Hoodsport, WA (Yang and Takeshima, 1994). Two embankments were constructed on low permeability bedrock. Deep mixing method walls were installed the cores of embankments to bedrock. The purpose of this application is controlling seepage through embankment and the underlying permeable deposits (Figure 2.18). The maximum depth of the treatment was 43 m, and the cut-offs were 51 to 61 meters long.

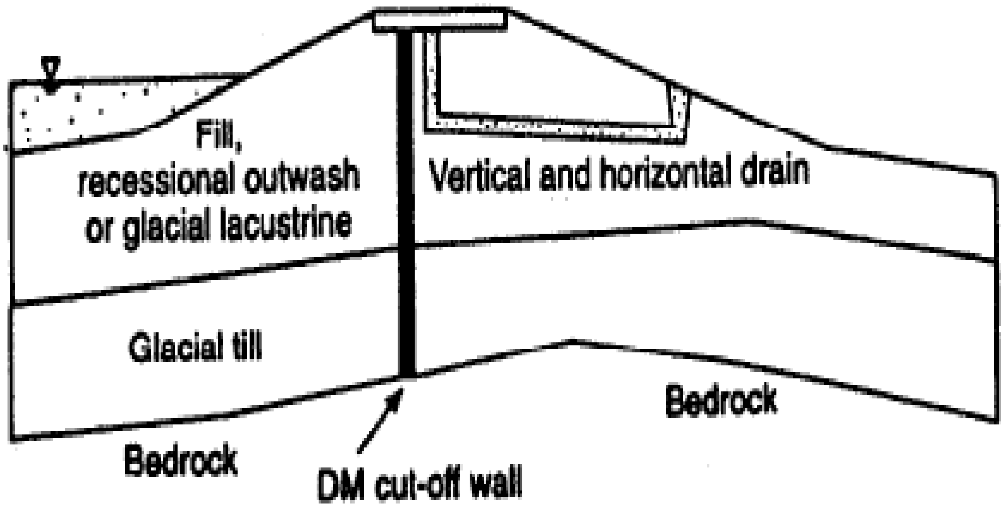



**Figure 2.18 :** Cushman Dam rehabilitation project, WA, U.S.A. (Yang and Takeshima, 1994).

**Excavation support walls (Japan, China, U.S.)**

This creates a structural wall for both excavation support and groundwater control (Taki, 1997). Major recent projects in the United States include structures for the Ted Williams Tunnel approach, Boston, MA; the Cypress Freeway Replacement Project, Oakland, CA; the Islais Creek Sewerage Project, San Francisco, CA; the Marin Tower, Honolulu, HI (Yang and Takeshima, 1994); and the Lake Parkway, Milwaukee, WI (Figure 2.19).

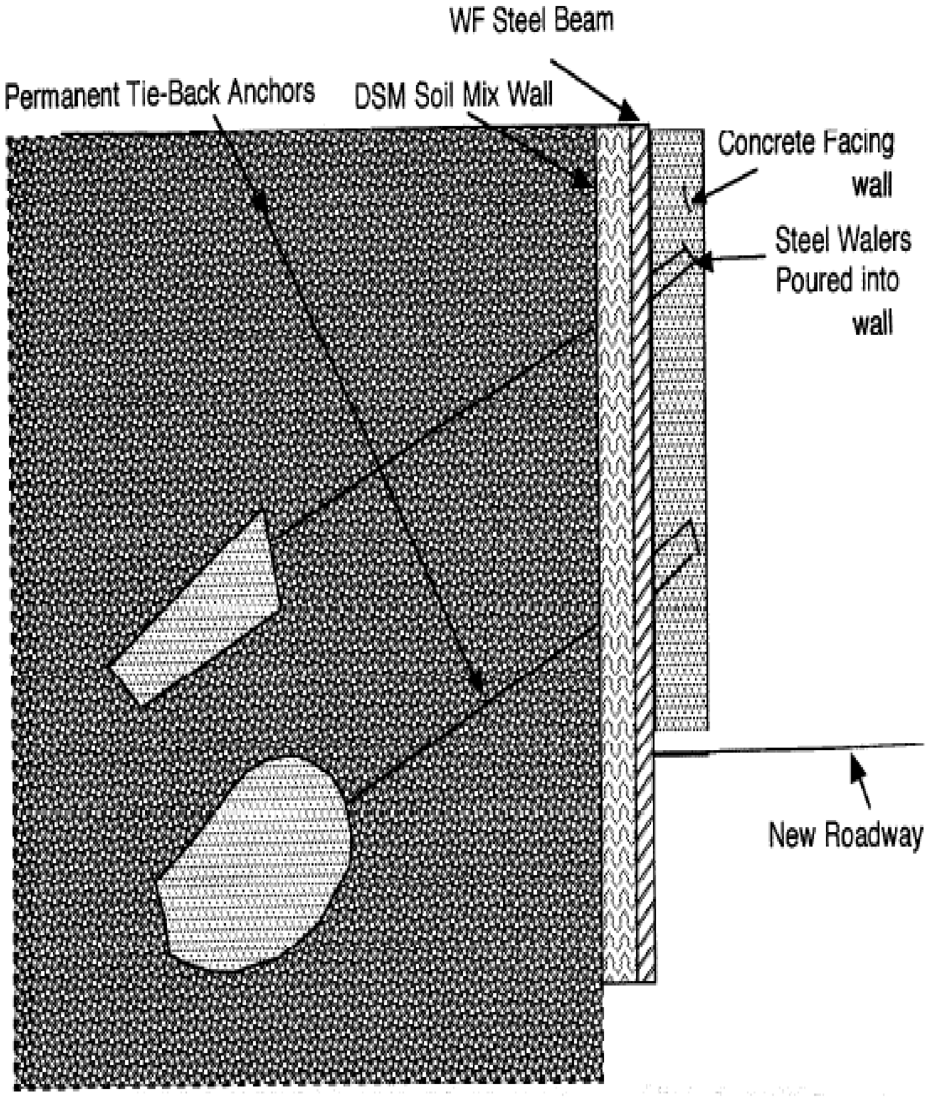



**Figure 2.19 :** Cross section: Lake Parkway, Milwaukee, WI (Geo-Con, Inc., 1998).

**2. Ground treatment (Japan, China, U.S.)**

Deep mixing method increases strength and reduces compressibility of natural and placed soils. Thus, it can provide ground stability and control of ground movements during construction. Application area is variable such as large-scale civil works in marine environments, the construction of manmade islands, tunnels, harbors, seawalls and breakwaters.

The deep mixing technology is extensive used in the construction of the Trans-Tokyo Bay Tunnel (Figures 2.20 and 2.21).

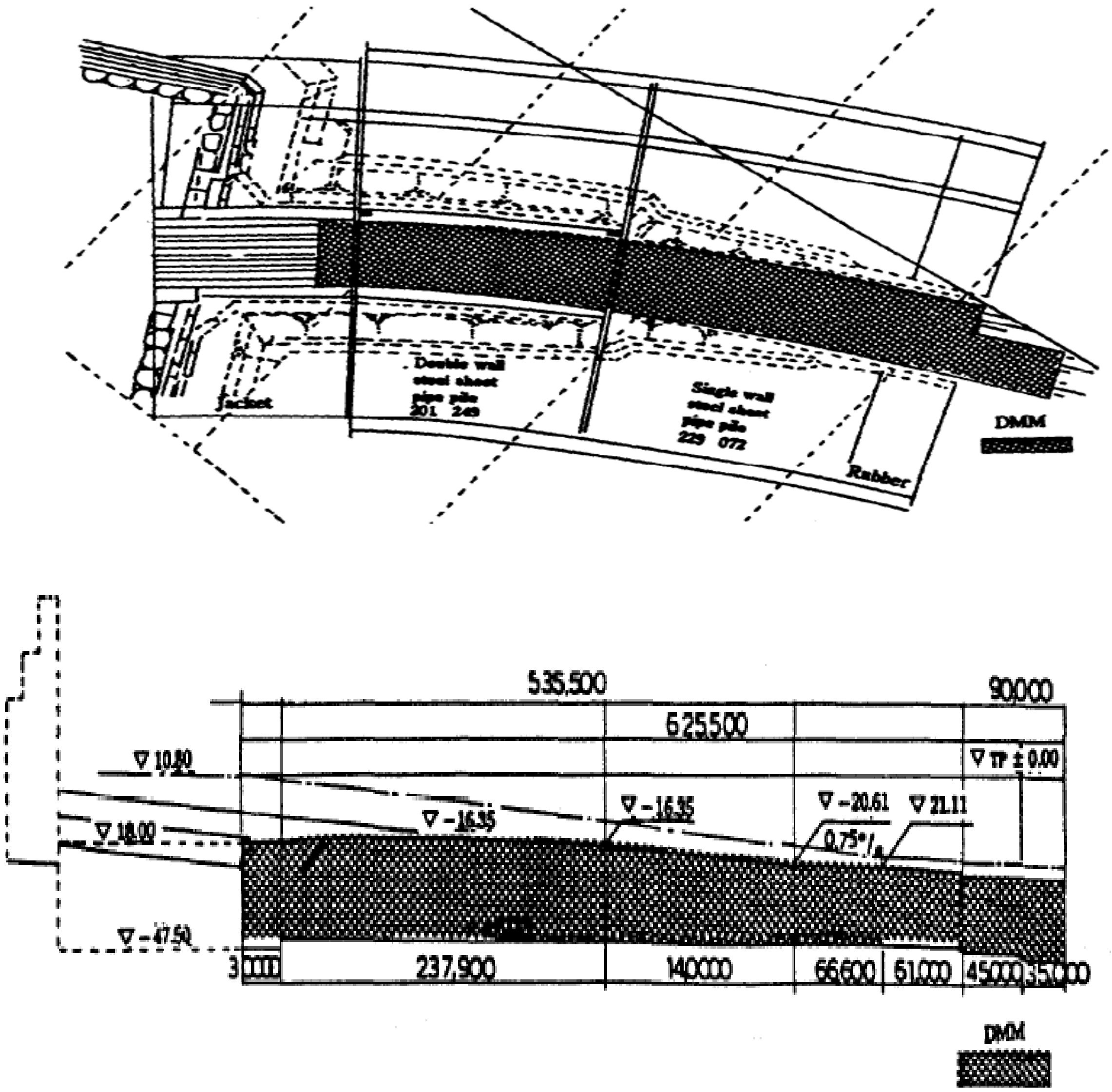



**Figure 2.20 :** Plan of deep soil mixing method Tram Tokyo Bay Tunnel (Unami et d., 1996).

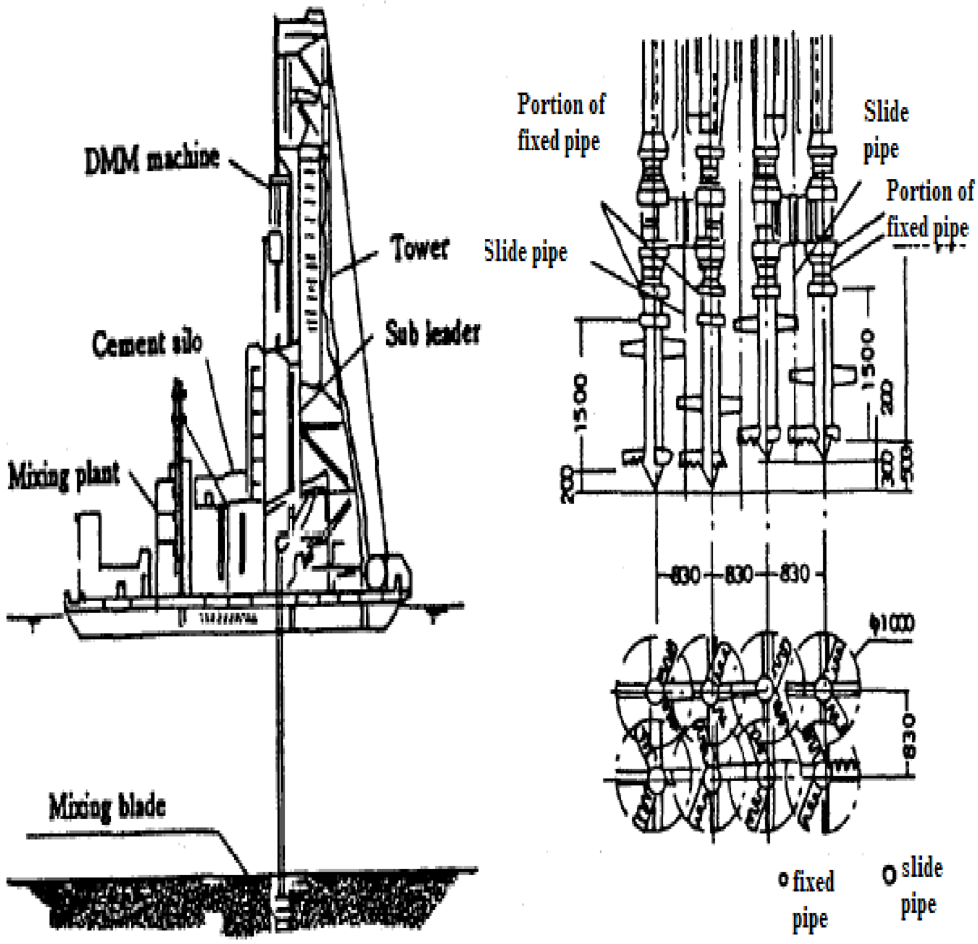



**Figure 2.21 :** Elevation of deep soil mixing work and details of mixing equipment (Unami et al., "Deep Mixing Method at Ukishima Site of the Trans-Tokyo Bay Highway Project", pp. 777-782, 1996).

**3. Liquefaction mitigation (Japan, U.S.).**

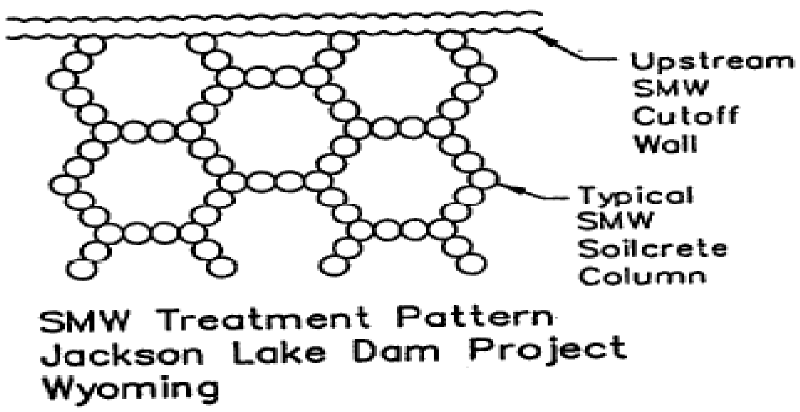
The main purposes of this method are reinforcement of liquefiable soil, reduction of pore pressure and prevention of liquefaction. Firstly, a perimeter wall is installed to get isolation. The groundwater which is interior the perimeter wall is lowered so dry soil zone that may be also called as non-liquefiable zone is occured under the structure.

According to Mitchell (1997) referring to the work at Jackson Lake Dam (Figure 2.22) the mechanism of improvement consists of three parts:

1. The “cells” absorb shear stresses and reduce the amplitude of the lateral granular movement and the development of excess pore pressure.

2. The confinement prevents lateral spreading.

3. The compressive strength of the columns minimizes settlement.



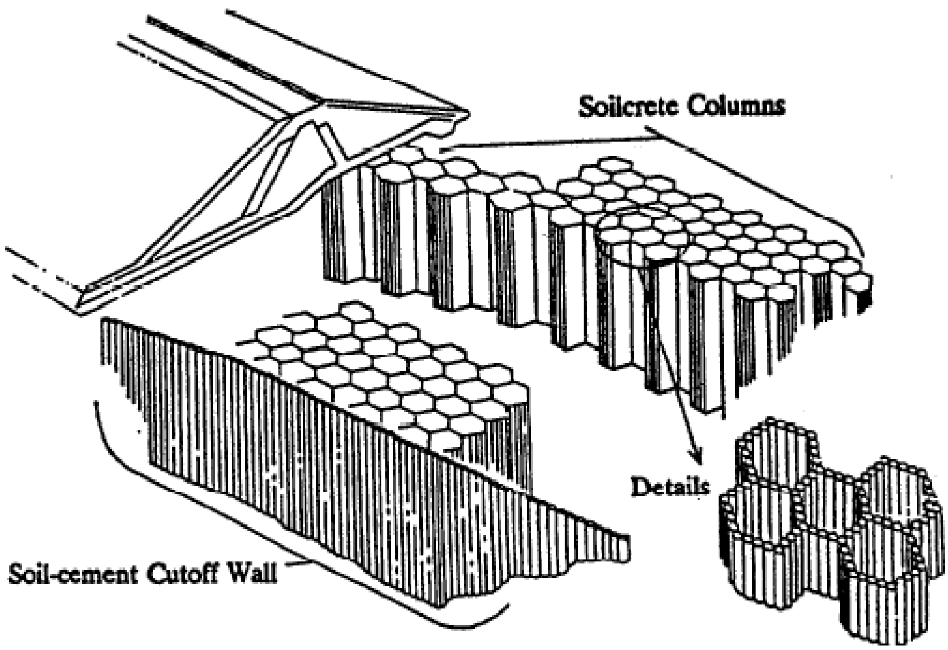




**Figure 2.22 :** DSM for Jackson Lake Dam Modification Project (Taki and Yang, 1991).

**4. In situ reinforcement, piles, and gravity walls (Sweden, Finland, France)**

Deep mixing method structures are formed as columns, walls or lattices. The purposes of these structures are reducing settlement under embankments, improving slope stability and supporting buildings and bridges.

In the United States, there are many application examples as small diameter columns. In Salt Lake City, Utah where more 1 meter settlement is predicted in the clay foundation soils without improvement. Figure 2.23 illustrates the concept, and these columns accelerates the settlement and generally used with preloading.

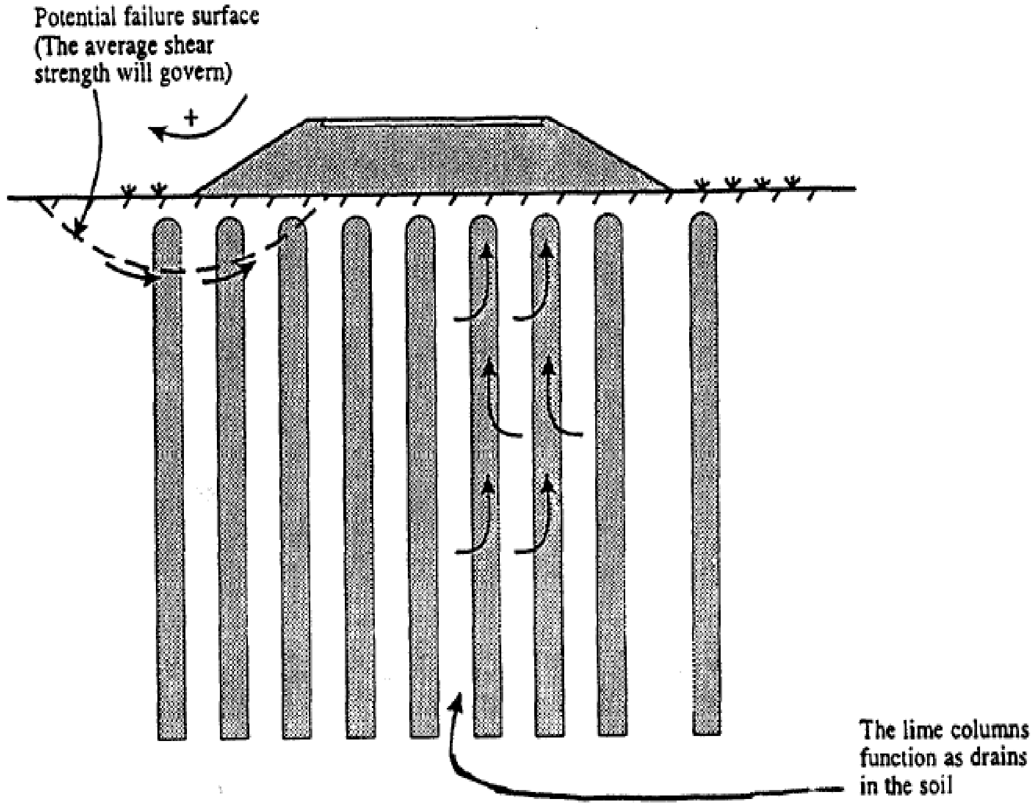



**Figure 2.23 :** Lime columns used in situ reinforcement (Stabilator, 1992).

**5. Environmental remediation (U.S., U.K.)**

Lime cement columns were installed to support a 42 m diameter, 20 m high fuel tank at an oil refinery in Singapore (Ho, 1996). The columns which are 0.4 m in diameter were installed to a depth of 13 m in the arrangement shown in Figure 2.24.

The piles were installed through 2.7 to 6.6 m of fill material over 4.8 to 9 m of peaty soft clay. The water table was 0.3 m below ground surface. These columns were installed to reduce differential settlement and increase the bearing capacity.

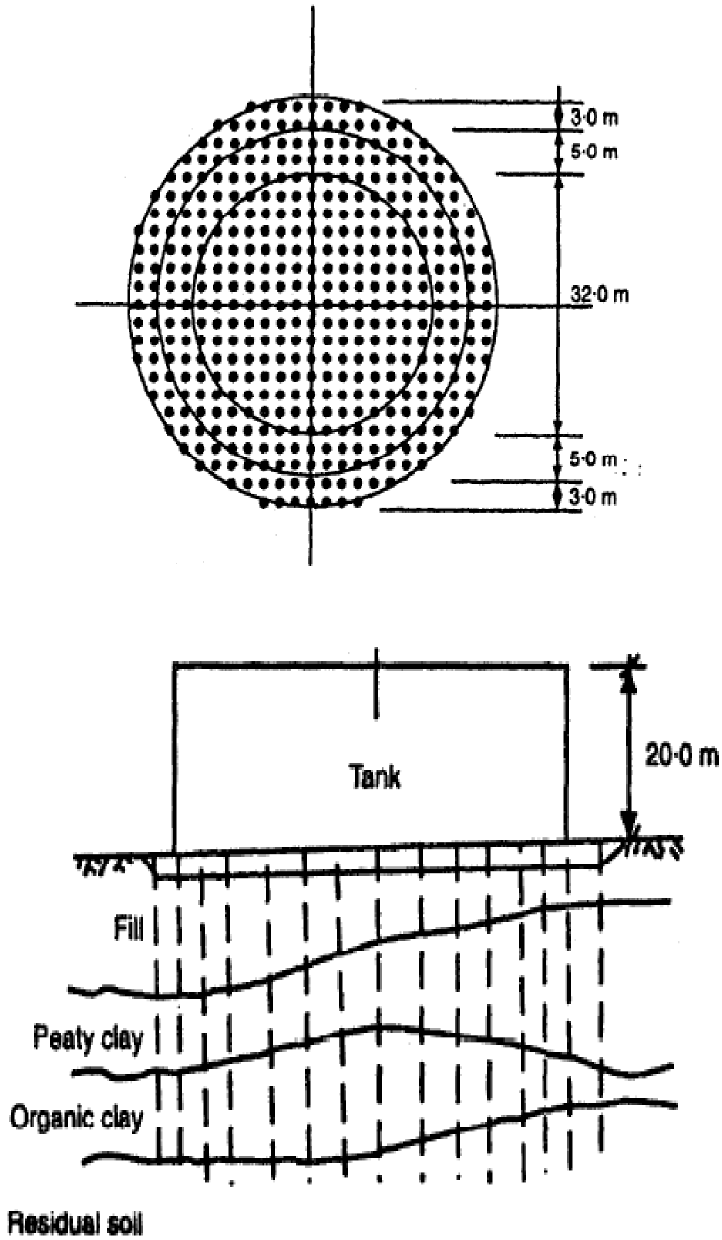



**Figure 2.24 :** Foundation stabilization for a fuel tank in Singapore (Ho, 1996).

### 2.2.11 Mini piles

Mini piles which is known as micro-piles have small diameter. Mini piles can be used to support structures or withstand loads. Mini piles have various applications and their diameters are ranging from 100 mm to 250 mm. They can be formed vertically or inclined. Besides concrete cast-inplace piles' behaviour is rely on the concrete, mini piles' load bearing capacity can be reached high levels because of the steel elements which is used in installation of mini piles.

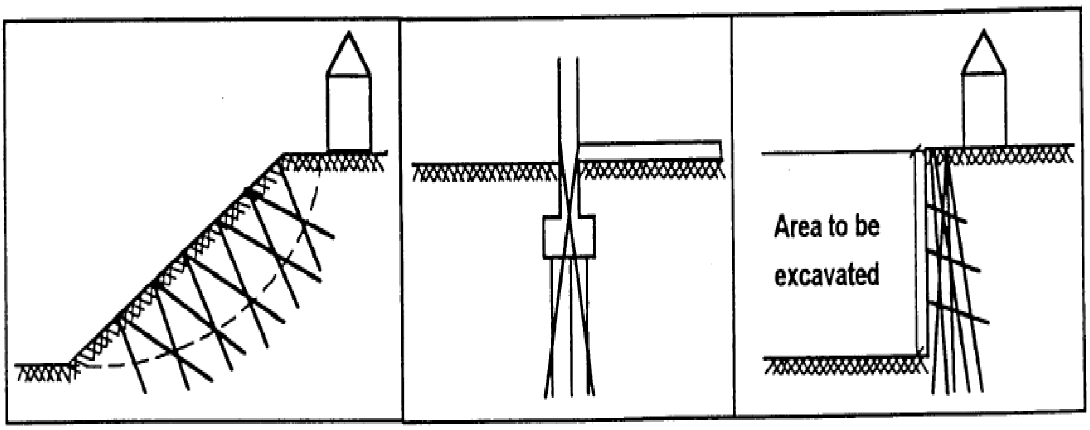



**Figure 2.25 :** Mini-pile applications (modified from Lizzi, 1983).

### 2.2.12 Soil nailing

Soil nailing is a ground supporting method by using steel rods. This application can be explained simply as reinforcing bars are driven to predrilled holes and are grouted. These reinforcing bars which is also called as nails are generally 20-30 mm in diameter. Therefore, nails are installed typcially 10 to 15 degrees down from the horizontal direction. In this application, face of the excavation is covered with a shotcrete layer. Drainage from the soil is obtained by strip drains. (U.S. Army Corps of Engineers, ETL 1110-1-185, 1999).

Nails work generally in tension but in specific conditions they can also work in bending and shear. The main principible of soil nailing system is that to improve stability. In frictional soils; it increases the normal force and accordingly increases soil shear resistance. In both frictional and cohesive soils; it lessens the driving force in failure surface. (FHWA, 1996b).

The purpose of soil nailing application is supporting the excavations and slopes. There are two mechanisms involved in the stability of nailed soil structures. Resisting tensile forces are generated in the nails in the active zone. These tensile forces must be transferred into the soil in the resisting zone through friction or adhesion mobilized at the soil-nail interface. The second mechanism is the development of passive resistance against the face of the nail. Soil nailing is most suitable for dense granular soil or stiff silty clay soils. (Mitchell and Christopher, 1990).

**Application of Soil Nail Walls**

Soil nail walls are used usually in highway cuts; for the repair, stabilization, and reconstruction of existing retaining structures; and also for supporting tunnel portals.

They are appropriate for these temporary and permanent applications which are given below:

• Roadway cut excavations

• Road widening under an existing bridge end

• Repair and reconstruction of existing retaining structures

• Temporary or permanent excavations in an urban environment.

In Europe and North America soil nail walls use as permanent and temporary stabilization of slopes, regeneration of failing retaining walls, temporary supporting for excavations, temporary or permanent supporting element for cuts. Generally, soil nail is installed into pre-drilled hole then the hole is grouted. In France, there is an another installation method that is percussion-driving nails are installed into soil without post-grouting. This application is making with a special jet nailing technique. In U.S. soil nailing is commonly used to support building excavations in urban areas. (FHWA-SA-96-069R, 1998).

**Ground Conditions**

Soil nailing system is mostly suitable for vertical or steepy slopes. The ground types which are suitable for soil nailing are weathered rock, stiff cohesive soils, dense sands and gravels with some cohesion. Therefore, the soil layer which is desired to install nails should be above the ground water table.

Loose granular soils, granular cohesionless soils that is poorly graded, soils have excessive moisture, organic soils, highly frost susceptible and swelling soils and highly fractured rocks with voids that ground types are not appropriate for soil nailing.

**Advantages of Soil Nailing**

- It provides less environmental impact and economical application compared to other construction methods.
- Additional structural element such as soldier beam is not used for this application.
- Manufacturing of soil nail walls is a rapid process.
- Adjustment of nail location and inclination is easy when compared to ground anchors.
- Soil nail walls accommodate large total and different settlements.
- Soil nail walls deflections are generally in tolerable limits.
- This application is more economical than other conventional ones like concrete gravity walls.

**Disadvantages of Soil Nailing**

- If the deformation control is very strict soil nailing will not be an appropriate application.
- Soil nailing is not suitable where the soil has large amounts of groundwater seep.
- Permanent soil nails need permanent easements.
- Soil nail walls demand experienced contractors.

**Construction Sequence**

**Excavate Initial Cut**

Firstly, it is important to ensure that the whole surface water will be controlled during the construction process. Approximately 1 to 2 meters of excavation depth is enough for the first excavation depending on the soil type. Excavated face can be stabilized by shotcrete during the construction if there is a stabilization problem.

**Drill Hole for Nail**

Nail holes are located to a specified length and inclination. It is important to select proper drilling method depending on the soil properties. There are cased methods for less stable ground and uncased method for more fragment materials.

**Install and Grout Nail**

The nails are installed into the drilled hole then it is filled with cement grout. Generally, the nails are 19 to 35 mm steel bars. To center the nail bar in the hole plastic centralizers are used. For permanent nails, corrugated grout filled plastic sheating or heavy epoxy coating is used for corrosion protection.

**Place Drainage System**

Drainage strips control the seepage during the construction. These drainage strips are extended down to the bottom of wall or weep holes can be occured that is penetreating the final wall face.

**Place Construction Facing and Install Bearing Plates**

Depending on the project, construction facing mainly consists of shotcrete layer and reinforced mesh. Therefore construction facing involves steel bearing plates and nuts that are placed at each nail head. It is also important that the nut is tightened at the nail and the bearing plate should be embed enough into shotcrete face.

**Repeat Processes to the Final Grade**

Until the final excavation depth is obtained these steps that are explained are repeated. If there is doubt about face stability shotcrete can be applied before drilling the nail hole.

**Place Final Facing**

For applications of permanent soil nailing cast-in place concrete is generally used. However, second of shotcrete may also be used as a final facing depending on the conditions. Figure 2.27 shows an example of permanent soil nail section of a road widening.

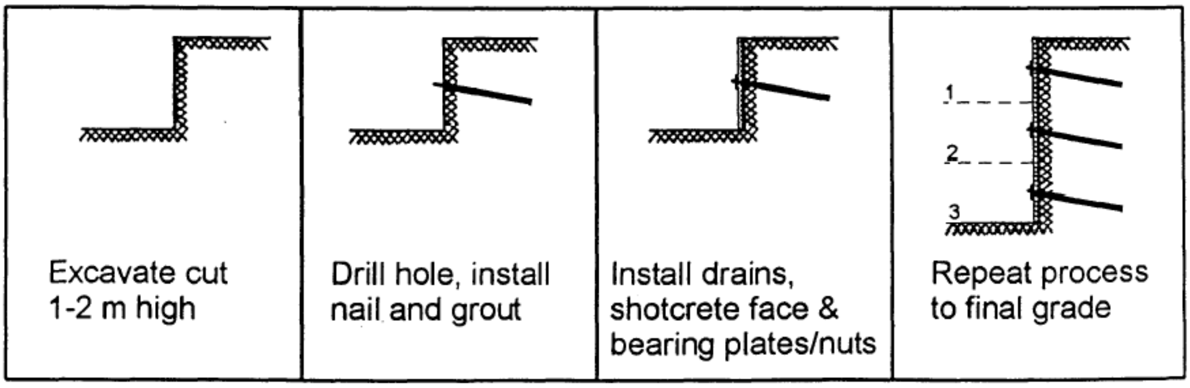



**Figure 2.26 :** Soil nailing for excavation support (after Walkinshaw and Chassie, 1994).

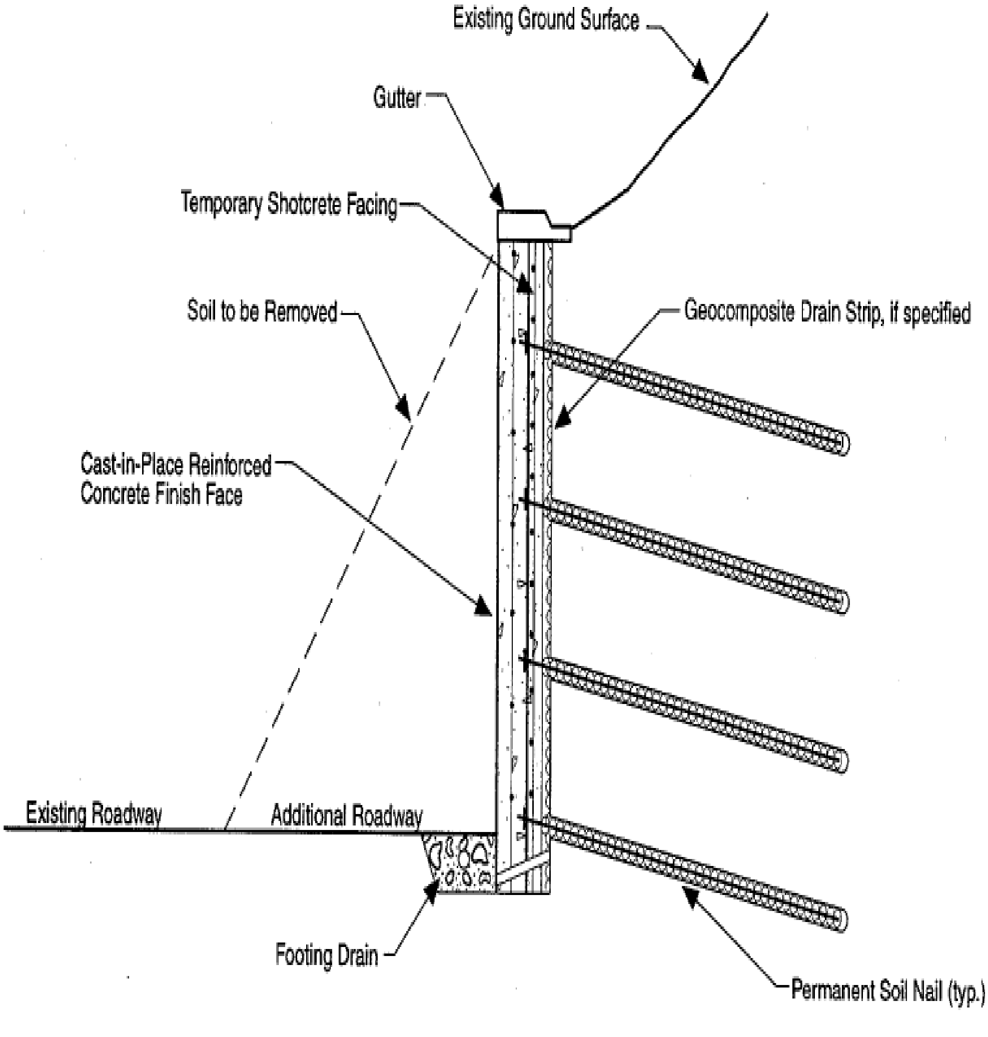



**Figure 2.27 :** Permanent soil nail wall section (FHWA-SA-96-069R, 1998).

### 2.2.13 Prefabricated vertical drains (PV) with or without surcharge fills

Prefabricated vertical drains are commonly installed in soft cohesive soils. The purpose of this method is to increase the rate of consolidation settlement. The rate of consolidation settlement is proportional to the square of the length of the drainage path to the drain. The rate of consolidation settlement depends on the drainage path length. If drainage path shortens the rate of settlement will increase. Geocomposites are used as drains because they have high flow capacity and also they are inexpensive. (U.S. Army Corps of Engineers, ETL 1110-1-185, 1999).

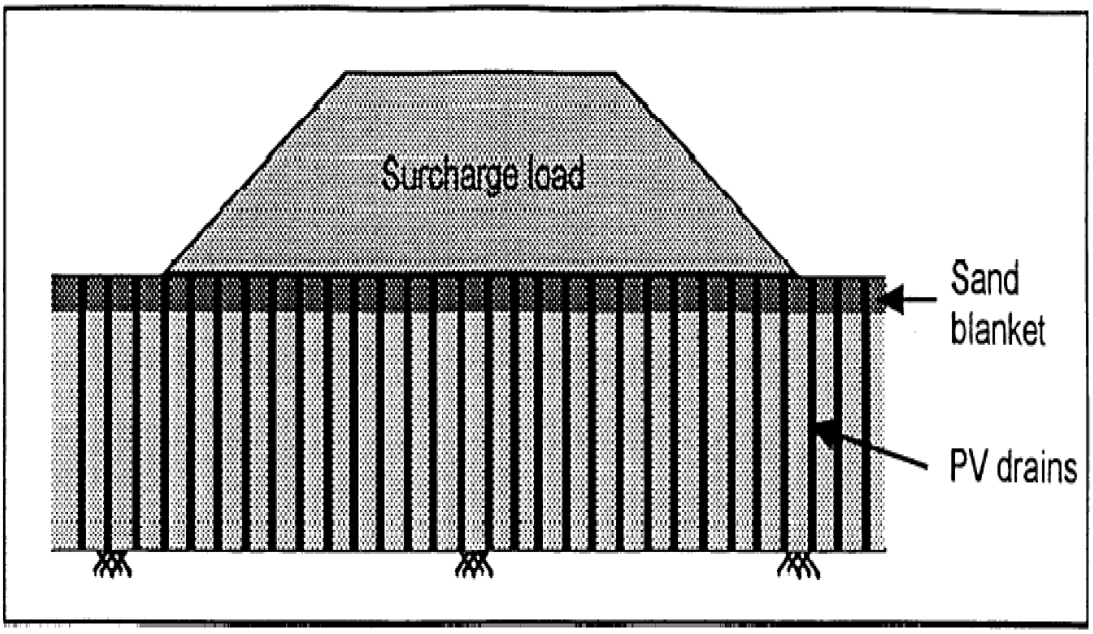



**Figure 2.28 :** PV drains with surcharge load (U.S. Army Corps of Engineers, ETL 1110-1-185, 1999).

To determining prefabricated vertical drain spacings, the time for the consolidation of the subsoil is considered. Generally the time for 90% consolidation ($t_{90}$) is used.

In order to find the time, it is needed to estimate an equivalent sand drain diameter for the prefabricated vertical (wick) drain used. The equations suggested by Koerner (1999) are;

$$dsd = \sqrt{\left(\frac{dv^2}{ns}\right)} \tag{2.4}$$

$$dv = \sqrt{\left(\frac{4btn_d}{\Pi}\right)} \tag{2.5}$$

dsd = equivalent sand drain diameter

dv = equivalent void circle diameter

b,t = width and thickness of the wick drain

ns - porosity of sand drain

$$n_d = \frac{Void\ area\ of\ wick\ drain}{total\ cross\ sectional\ area\ of\ strip} = \frac{void\ area\ of\ wick\ drain}{bxt} \qquad \textbf{(2.6)}$$

The equation for estimating the time t for consolidation is (Koerner, 1999);

$$t = \frac{D^2}{8c_h} \ln \frac{D}{d} - 0.75 \ln \frac{1}{1-U} \qquad \textbf{(2.7)}$$

t = time for consolidation

ch = coefficient of consolidation of soil for horizontal flow

d = equivalent diameter of strip drain

$$= \frac{circumference}{\Pi} \qquad \textbf{(2.8)}$$

D = sphere of influence of the strip drain;

a) for a triangular pattern, D = 1.05 x spacing Dt

b) for a square pattern, D = 1.13 x spacing D

D= distance between drains in triangular spacing and

Ds = distance for square pattern

U = average degree of consolidation

Surcharge preloading can be used with vertical drains to increase the prior settlement of construction. A new application for PV drains is in the area of mitigation of liquefaction risk. PV drains were installed in conjunction with stone columns in a test section at Salmon Lake Dam in Washington (Luehring, 1997). The prefabricated drains were used to treat drainage and dissipated the excess pore pressure. They also prevent the rupture of the foundation soil and were installed before creating stone columns. The stone columns were installed considering that the rupture problem of the foundation soil was improved. During the installation of columns, water and air were ejected from drains. Finally, the conclusion is that the wick drains relieved excess air and water pressure. On the other hand, prefabricated drains protect dam and foundation soil from rupturing and disturbance. (ASCE, Geotechnical Specification Publication No.69, 1997).

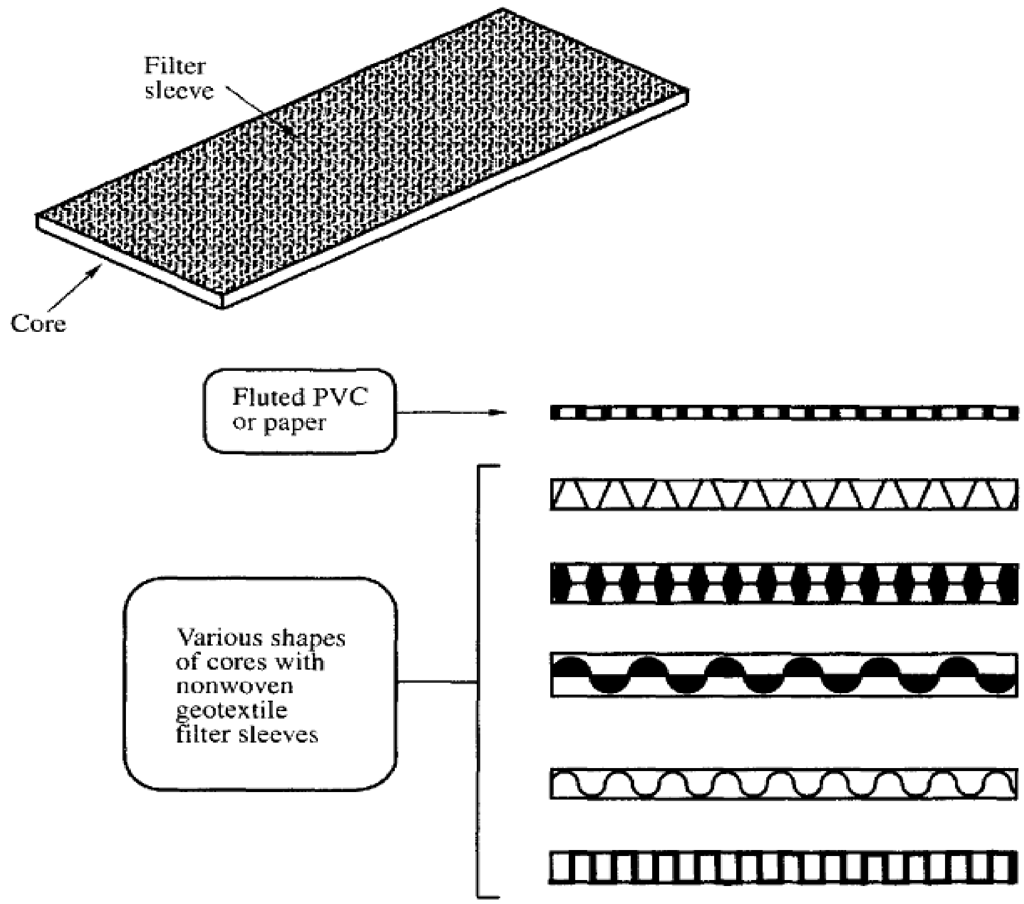



**Figure 2.29 :** Typcial core shapes of string drains (Hausmann, 1990).

Advantages of Using Wick Drains (Koerner, 1999);

1. The analytic procedure is available and straightforward in its use.

2. Tensile strength is definitely afforded to the soft soil by the installation of the wick drains.

3. There is only nominal resistance to the flow of water if it enters the wick drain.

4. Construction equipment is generally small.

5. Installation is simple, straightforward and economic.

### 2.2.14 Preloading

Preloading method is most effective for soft clay soils. This method is used in construction sites which involve weak silts, clays, organic materials, sanitary landfills and other kind of compressible soils. The water in pores of soil is removed by preloading in time. However, surcharge loads must remain for months or years to get the desired settlement which means construction is delayed. In order to remove water faster from soil horizontal and vertical drains may be used. The purpose of the method is to evoke consolidation process. The greater surcharge load automatically decreasing the time for consolidation. The consolidation process can be observed by settlemet plates and piezometers.

There are two types of vertical drains which are cylindirical sand drains and prefabricated vertical (wicked) drains.

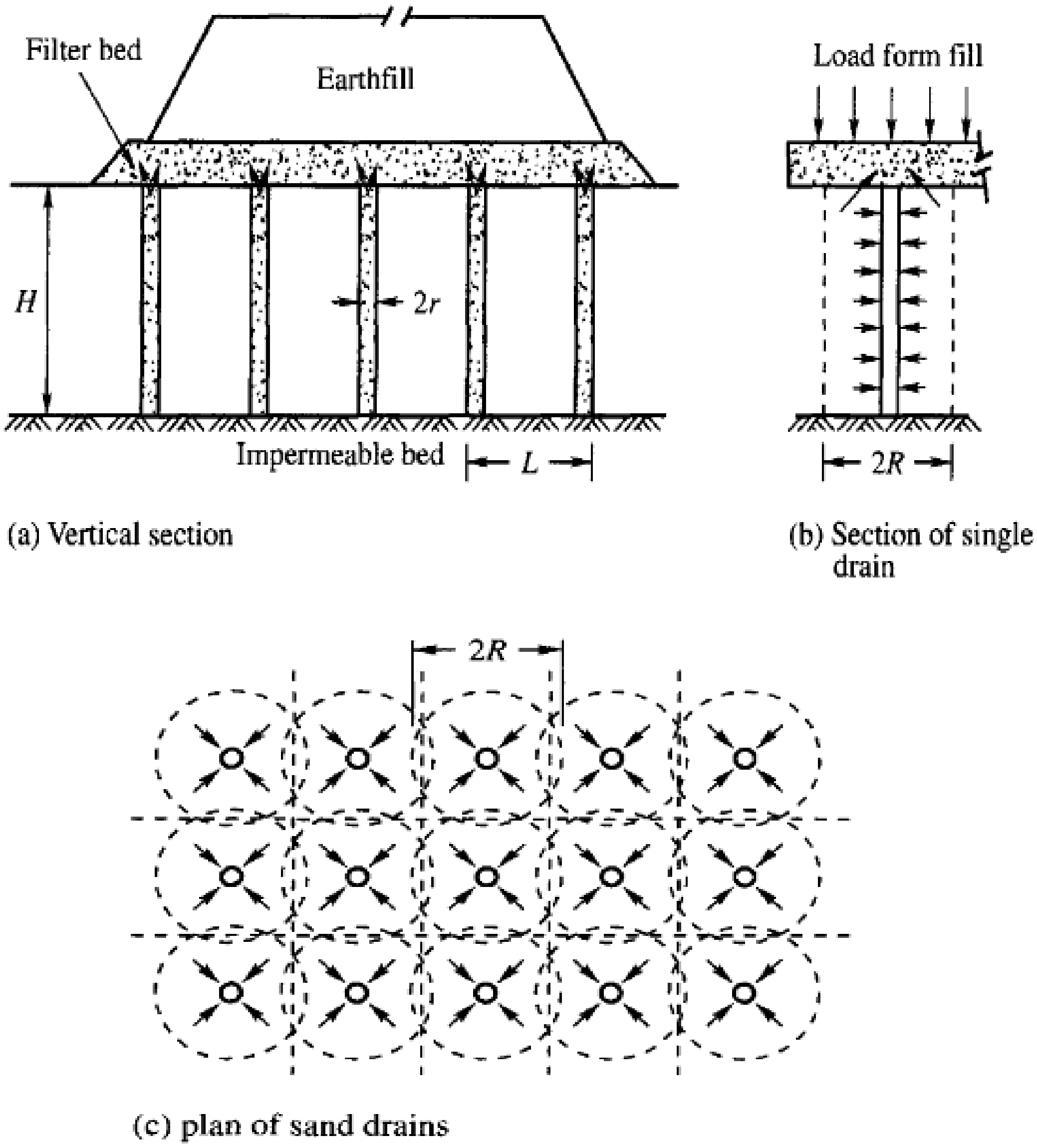



**Figure 2.30 :** Consolidation of soil using sand drains (V.N.S. Murthy, Principles and Practices of Soil Mechanics and Foundation Engineering).

### 2.2.15 Sand drains

Sand drains are installed both vertical and horizontal direction for consolidating compressible materials. Principally, they are placed under the surcharge loads to accelerate the consolidation process by occuring quick drainage. Vertical drains provide a shorter path for the water flow. Accordingly, consolidation process occurs in a short time. The arrangement of sand drains shown in Figure 2.30 (c-plan of sand drains).

Commonly, medium to coarse sand is used as a drain. The diameter of drains is more than 30 cm and effective drain spacing should be determined carefully for economy. Generally, they are placed 2 to 3 meters apart from each other. Depth of the vertical drains are depend on compressible soil thickness.

Surcharge load causes water to squeeze out in radial direction to sand drain and in the vertical diection to the sand blanket. The dashed lines shown in Figure 2.30 are drawn midway between the drains. The planes passing through these lines may be considered as impermeable membranes. The water flows out to the center of drain. The problem of computing the rate of radial drainage can be simplified without appreciable error by assuming that each block can be replaced by a cylinder of radius R such that; (V.N.S. Murthy, Principles and Practices of Soil Mechanics and Foundation Engineering).

$$\Pi R^2 = L^2 \tag{2.9}$$

L is the side length of the prismatic block.

$$U_z\% = 100\, f(T) \tag{2.10}$$

degree of consolidation Uz%

$$T = \frac{c_v t}{H^2} \tag{2.11}$$

If the bottom of the compressible layer is impermeable, then H is the full thickness of the layer.

For radial drainage, Rendulic (1935):

$$U_r\% = 100\, f(T) \tag{2.12}$$

$$T_r = \frac{c_v r}{4R^2} t \tag{2.13}$$

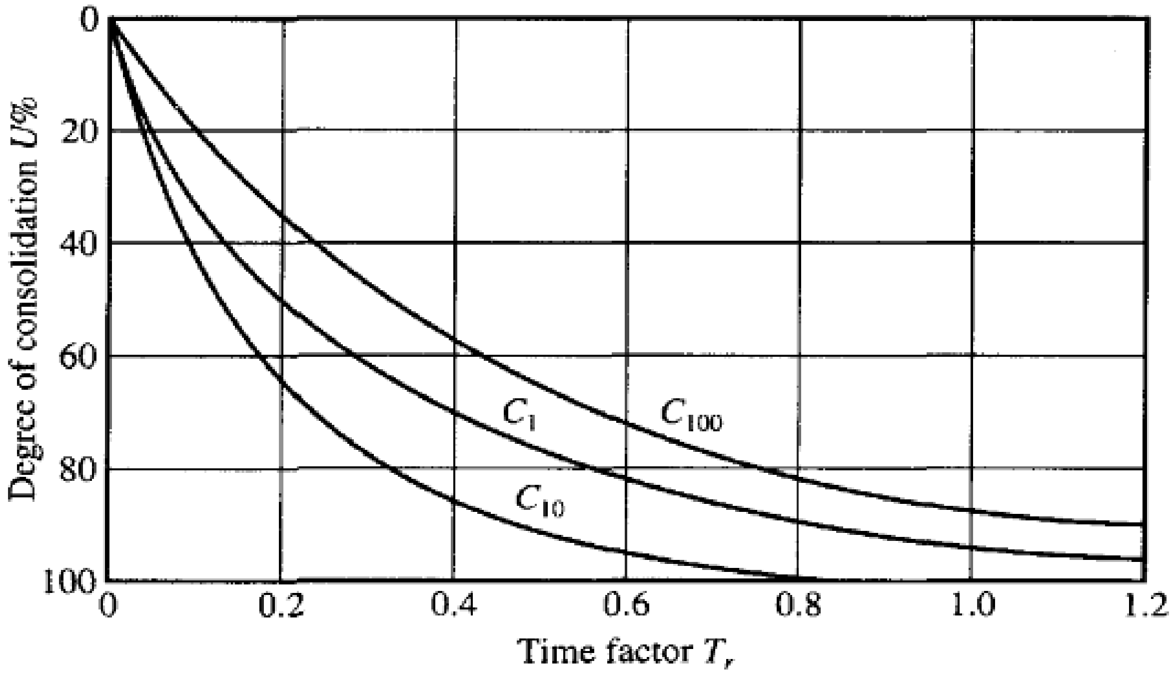



**Figure 2.31 :** Time consolidation ratio relationship (V.N.S. Murthy, Principles and Practices of Soil Mechanics and Foundation Engineering).

The degree of consolidation U% and the time factor Tr depends on the ratio Rlr. The relation between Tr and U% for ratios of Rlr equal to 1, 10 and 100 in Figure 2.31 and they are expressed by curves C1, C10 and C100 respectively.

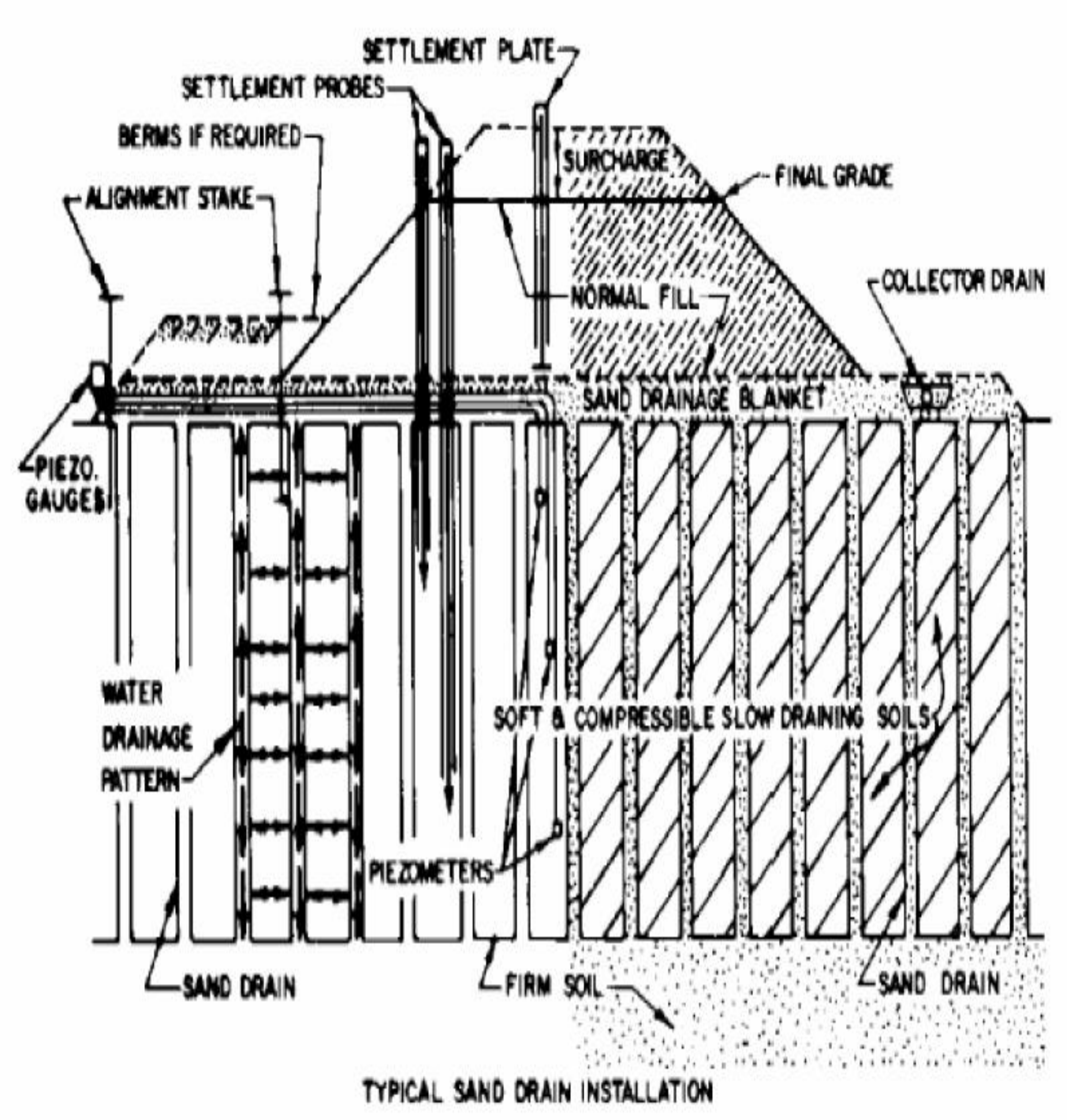



**Figure 2.32 :** Typical sand drain installation (FHWA RD 86-186, August 1986).

### 2.2.16 Electro osmosis

The aim of electroosmois process is that to get lower compressibility and higher strength. This method is applied to saturated clay soils. When electric potential is applied to soil the cations will be attracted to the cathode and the anions will be attracted to the anode. The cations and anions carry water as they move so, the net flow of pore water will be toward to cathode. The water is collected at cathode and it can be removed. This means that the soil which is between electrodes will be consolidated. Consolidation occurs best at anode and least at the near of cathode.

### 2.2.17 Buttress fills

Buttress fills are used to decrease the risk of liquefaction and to improve the stability of slopes. The buttress forms an aditional weight which increases resisting force for slopes and it increases confining pressure to increase the resistance of liquefaction.

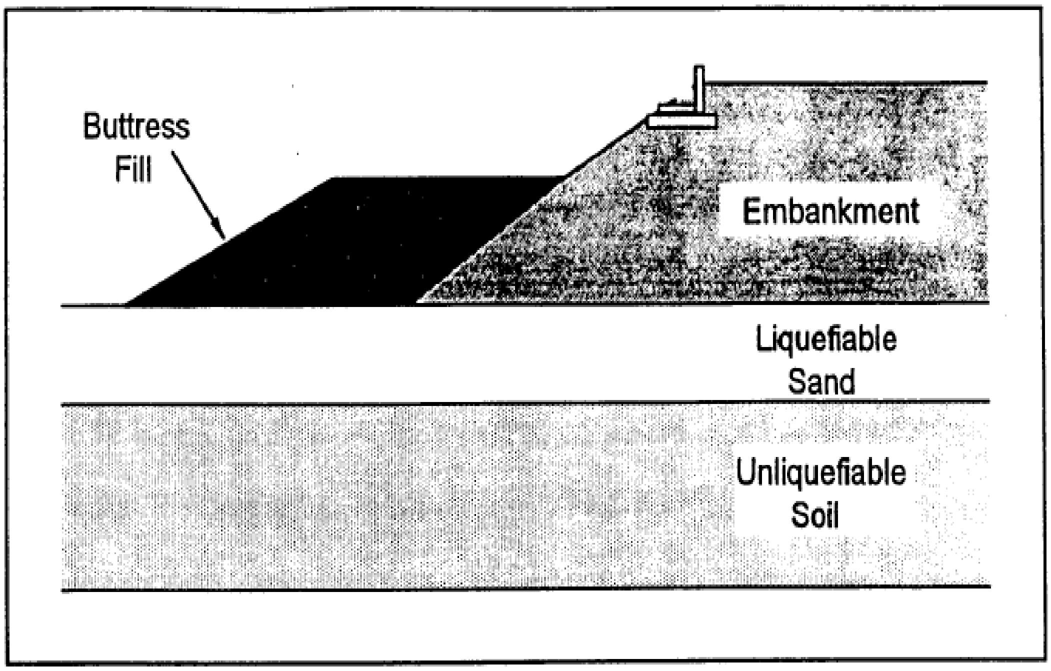



**Figure 2.33 :** Butress fill at toe embankment (U.S. Army Corps of Engineers ETL 1110-1-185, 1999).

### 2.2.18 Biotechnical stabilization and soil bioengineering

Biotechnical stabilization and soil bioengineering are used to improve the stability of slopes aganist erosion. The biotechnical stabilization method consists of using live vegetation in combination with inert structural or mechanical components, such as retaining structures, revetments and ground cover systems (ASCE, Geotechnical Specification Publication No.69, 1997).

The live vegetation and structural/mechanical elements work together to improve soil stabilization. The live plants are used in soil bioengineering as soil reinforcement, hydrolic drains and barriers.(U.S. Army Corps of Engineers ETL 1110-1-185, 1999).

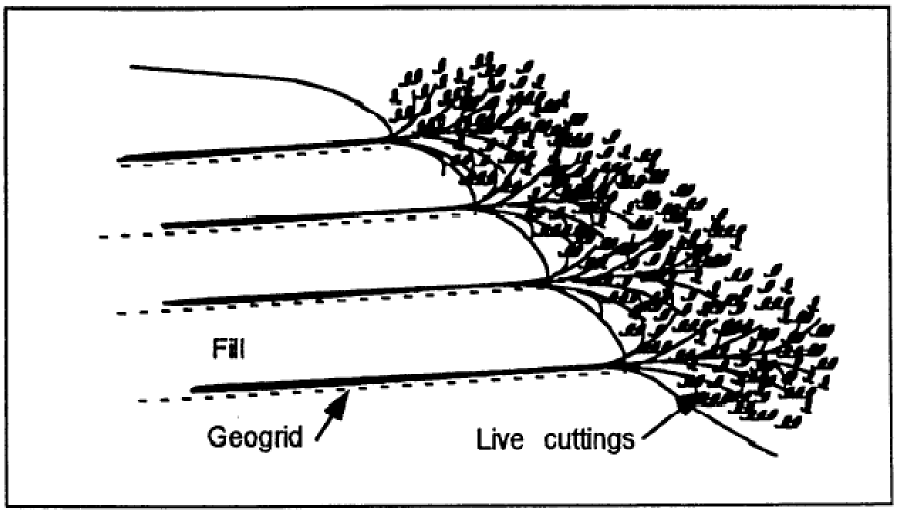



**Figure 2.34 :** Biotechnical stabilization by brush layering (after Gray and Sotir, 1996).

### 2.2.19 Soil reinforcement

A geosynthetic-reinforced soil contains horizontally placed layers of geosynthetic reinforcement. Geosynthetic-reinforced soils present higher stiffness and higher load carrying capacity than a soil mass without the reinforcement. This increasing is occured by geosynthetic reinforcement on the soil mass and it restrains deformation of the mass of geosynthetic reinforced soil along the axial direction of the reinforcement. (FHWA-RD-97-142, 1997).

Geosynthetics are commonly used in following applications:

• Underground drainage, including prefabricated drainage strips

• Soil separation

• Soil stabilization

• Permanent erosion control

• Silt fences

• Base reinforcement for embankments over soft ground

• Geomembranes

- **Development of design parameters for geosynthetic application:**

For underground drainage design, sources of water which is desired to drain and the gradiation and density of soil in the vicinity of the geosynthetic drain. Geotechnical site investigation  will be adequate for the drainage design, generally. However, for shallow systems, hand holes will be adequate, either.

For permanent erosion control, measurement of the groundwater and gradiation of soil that is under the geotextile layer are important to the geosynthetic design.

Investigation for silt fences can generally be done by soil examination.

For designing geomembrane, groundwater information and soil gradation information is sufficient. If the geomembrane design is planing for a slope, the soil stability information is also need.

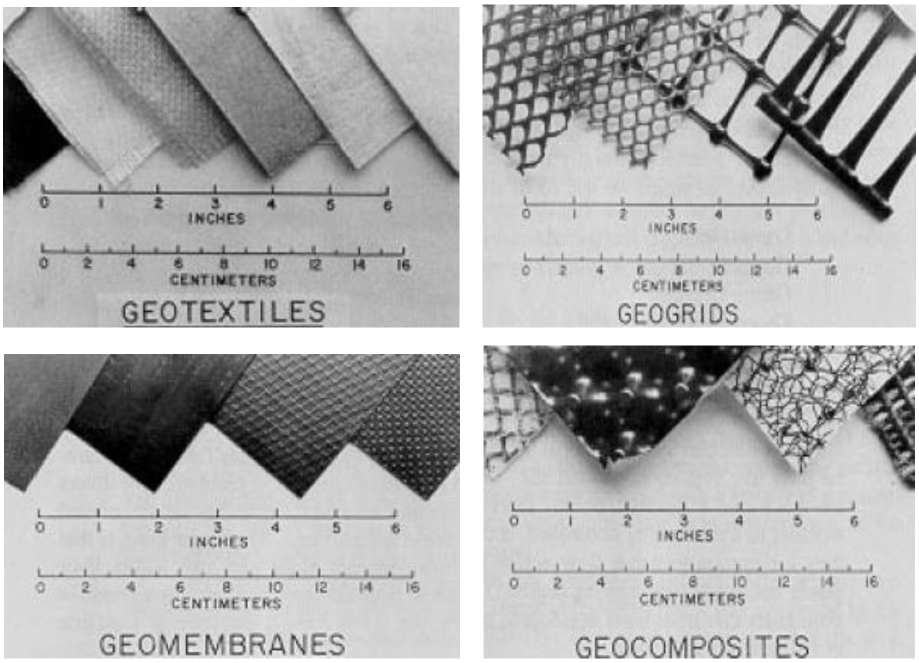



**Figure 2.35 :** General geosynthetic types ( U.S. Army Corps of Engineers EM 1110-1-3500, 1995).

# 3. CASE HISTORIES FOR DEEP SOIL IMPROVEMENT METHODS

## 3.1 Iron and Steel Plant

### 3.1.1 General

In this section case history of the construction of an Iron and Steel Plant in the South of Turkey is given. There would be different buildings that have various functions and sizes are designed within the scope of the project. The field area is totally 400 000 $m^2$ and total floor area of the buildings is approximately 179 000 $m^2$. Construction area is placed on both the natural soil which is 130 000 $m^2$ and man-made soil which is 49 000 $m^2$. Iron and Steel Plant mainly consists of Continuous Pickling Line, Cold Mill and Hot Dip Galvanizing Line.

### 3.1.2 Site description

A total 88 soil investigation boreholes in variable depths that were ranging from 20.45 to 50.45 were carried in December 2008 – March 2009 and in August 2009 additionally 15 boreholes were carried out that were 25.45 m to get more information. During the drilling process SPT was carried out each 1.50 m and disturbed samples were taken. Further, during the drilling in clay soil undisturbed soil samples (UD) were taken in each 3 m. The specified depths of disturbed and undisturbed soil samples were taken is shown in borehole layout plan in Appendix-A.

While drilling, to obtain ground water table in the field, soil investigations were made. When elevation of the field is +8.00 m it is accepted that ground water level is varying from +3.00 to +1.50 meters.

### 3.1.3 Site specific soil investigations

**Field experiments**

In the field for soil investigation SPT is made each 1.50 meter as mentioned. SPT results are high because of the soil which involves silty clay-clayey silt layers and alteration of coarse grained sandy gravel layers. Thus, the relative densities of the granular layers' were not obtained correctly.

At the shallow depths SPT values (N30) are varying from 5 to 8. However, at deeper depths the results are varying from 15 to 20. Moreover, clayey layers' consistency changes depending on the depth that at the surface consistency is soft-medium besides it is hard-very hard at lower depths. Sandy-gravel layers which are encountered in soil are generally dense.

**Laboratory experiments**

In Soil Mechanics Laboratory various lab tests were performed on disturbed and undisturbed soil samples. By using disturbed soil samples to classify soil consistency limits, soil size distribution tests were done. With undisturbed clayey soil samples to get shear strength unconfined compression and triaxial (UU) tests were performed. Therefore, on clayey undisturbed samples consolidation test was done. Results of the laboratory experiments are given in Appendix A.

### 3.1.4 General soil properties

If the soil is classified corresponding to USCS soil classification system; generally silty sand-gravel layers which are in SM, GM and GC group and shows non-plastic (NP) property. Also, silty clay – clayey silt soil layers' properties are stick to CL and CL-ML corresponding to USCS. Plasticity index (Ip) is %4 - %7 which means soil's plasticity is low.

Triaxial and unconfined pressure tests were performed on undisturbed silty-clay soil samples which were taken from surface of the soil and at the end of these experiments undrained shear strength (cu) was found among 20-40kPa. Undrained shear strength was found approximately 50kPa from test results that were done on soil samples which were taken from deeper than 20.00 meter.

### 3.1.5 Cold mill-galvanized coating-dyehouse building

#### 3.1.5.1 Introduction

In this thesis, Cold Mill-Galvanized Coating-Dyehouse Building which is a part of Iron and Steel Plant is explained in detail. Iron and Steel Plant is 6 km away from town and, its location is between government highway and railway.

#### 3.1.5.2 General and regional geology

Cold Mill-Galvanized Coating-Dyehouse Building Field Study Report was prepared about site's geographic location, climate and general geology in January 2008.

Briefly, on the soil surface generally sandy gravel - gravelly sand and silty clay - clayey silt layers which have medium hard-hard consistency were encountered. By drilling maximum 50 meter depth was achieved and in these borings under the alluvial storing Haydar formation was not found.

#### 3.1.5.3 Boreholes

In Iron and Steel Plant field first soil investigations were made in June 2006. Within the scope of this investigation 11 boreholes were made. Generally, their depths are 15.00 m.

At the site of Iron and Steel Plant construction 28 boreholes were made between the dates of December 2007 and January 2008. Borehole layout plan is given in Appendix A. Borehole depths and elevations are illustrated in Table 3.1 SPT Test was performed in every 1.50 m and disturbed soil samples were taken during the SPT Test regarding to ASTM 1586 and TS 901. Undisturbed samples were also taken in clayey soil in each 3.00 meter. The specified depths of disturbed and undisturbed soil samples were taken; ground water level and elevation are shown in borehole logs.

**Table 3.1:** Cold Mill-Galvanized Coating-Dyehouse Building Boring information.

| Borehole No | Top elevation (m) | Depth (m) | GW Depth (m) | GW Elevation (m) |
|---|---|---|---|---|
| S3 | 7,81 | 50,45 | 4,25 | 3,56 |
| S4 | 8,03 | 50,45 | 5,2 | 2,83 |
| S5 | 8,19 | 20,45 | 4,55 | 3,64 |
| S6 | 8,15 | 50,45 | 5 | 3,15 |
| S7 | 7,96 | 20,45 | 5,9 | 2,06 |
| S8 | 8,06 | 50,45 | 4,65 | 3,41 |
| S9 | 7,92 | 20,45 | 5,7 | 2,22 |
| S10 | 8,27 | 50,4 | 5,05 | 3,22 |
| S11 | 8,03 | 20,45 | 4,95 | 3,08 |
| S12 | 8,38 | 50,85 | 5,2 | 3,18 |
| S13 | 7,93 | 20,45 | 4,5 | 3,43 |
| S17 | 8,17 | 20,45 | 5,95 | 2,22 |
| S18 | 8,17 | 50,45 | 5,65 | 2,52 |
| S19 | 8,17 | 20,45 | 4,95 | 3,1 |
| S22 | 8,18 | 20,45 | 6,5 | 1,68 |
| S23 | 8,14 | 20,45 | 6,5 | 1,64 |
| S24 | 8,14 | 50,45 | - | - |
| S25 | 8,14 | 20,45 | 6,5 | 6,5 |
| S26 | 8,23 | 20,43 | 6,7 | 6,7 |
| S67 | 7,99 | 25,45 | - | - |
| S68 | 7,99 | 25,45 | - | - |
| S69 | 8,00 | 25 | - | - |

### 3.1.5.4 Groundwater conditions

During drilling ground water was also investigated. As it can be seen from table the field was straigthened to +8.00 m elevation. Ground water is generally 5.00 m deep from the soil surface. Ground water elevation is accepted as +3.00 m for this field.

### 3.1.5.5 Field experiments

During boring SPT test was carried out in each 1.50 m. Gained high SPT results in sandy-gravelly levels is caused by coarse grained material and it doesn't give correct SPT results. Besides this unclarity, SPT test results which were obtained from silty clay – clayey silt layers gives adequate information about consistency. To obtained SPT N30-soil elevation relation is given in Appendix-A. Therefore, sandy-gravelly soil layers are generally dense-medium dense.

### 3.1.5.6 Laboratory experiments

Regarding to TS 1900 and ASTM Standards experiments were performed in soil mechanics laboratory. To classify soil and to obtain consistency limits, soil size distribution tests and consistency tests were done. With undisturbed clayey soil samples to get shear strength unconfined compression and triaxial (UU) tests were performed. To obtain clayey soil's compressibility consolidation tests were performed.

Corresponding to USCS soil classification system; the soil is generally in SM, GM and GC groups and shows non-plastic (NP) property. Silty clay – clayey silt soil layers' properties are stick to CL and CL-ML corresponding to USCS. For this soil strata Plasticity index is in the vicinity of %3 - %7 which means soil's plasticity is low.

Triaxial and unconfined pressure tests were performed on undisturbed silty-clay soil samples which were taken from surface of the soil and at the end of these experiments undrained shear strength was found between the ranges of 10 - 45 kPa. Undrained shear strength was found between the ranges of 40 – 80 kPa from test results that were done on soil samples which were taken from deeper than 20.00 meter.

#### 3.1.5.7 Soil profile

By using boreholes which were carried out at the site of Cold Mill-Galvanized Coating-Dyehouse Building soil sections were drawn and they are illustrated in Appendix-A.

Soil Profile I-I, was drawn with the correlation of boreholes S03, S67, S68 and S04, S69, S70, S05, S06, S01, S02. In this part, field surface elevation is in the vicinity of +8.00 m. In this soil profile, generally among the soil surface and 8.00-12.00 m depths medium hard – hard consistency silty clay – clayey silt layers were found. Dense packed sand-gravel layer was passed among 12.00 – 16.00 m depths during boring. In some boreholes (S03, S69, S05, S06) coarse material was encountered at deeper depths. Under the sand-gravel soil layers, hard silty clay layers were found. Almost the whole borehole at 30.00-36.00 meters deep, again dense-very dense sand-gravel storing was achieved. Boreholes which are 50.00 m have ended in this soil layer.

In the soil profile II-II, silty clay layer which has medium hard – hard consistency was found among the soil surface and 8.00-12.00 m depths. At 8.00-12.00 m depths sand-gravel soil layer begins. Under this sand-gravel soil layers, hard-medium hard 4.00m-6.00m thickness silty clay layers was found. In this granular unit, hard silty clayey layers were also found. Deep boreholes that are S08, S10, S12 at the depth of 40.00 m very hard- hard consistency silty – clayey soil was encountered and these boreholes were ended in this cohesionless soil unit.

Soil profile III-III is very similar to soil profile I-I. 8.00-12.00 m thickness medium hard silty-clay layer which starts from the surface exists in this soil profile, too. Generally, under this soil sand-gravel and hard silty-clay intercalation were encountered. Briefly this soil profile involves on the surface medium hard silty-clay layer, with depth hard silty-clay layers. At the depth of 38.00-40.00 m dense sand-gravel units were encountered.

#### 3.1.5.8 Soil model and geotechnical parameters for design

To perform engineering analyses of Cold Mill-Galvanized Coating-Dyehouse Building a soil profile is created and it is shown in Figure 3.1.

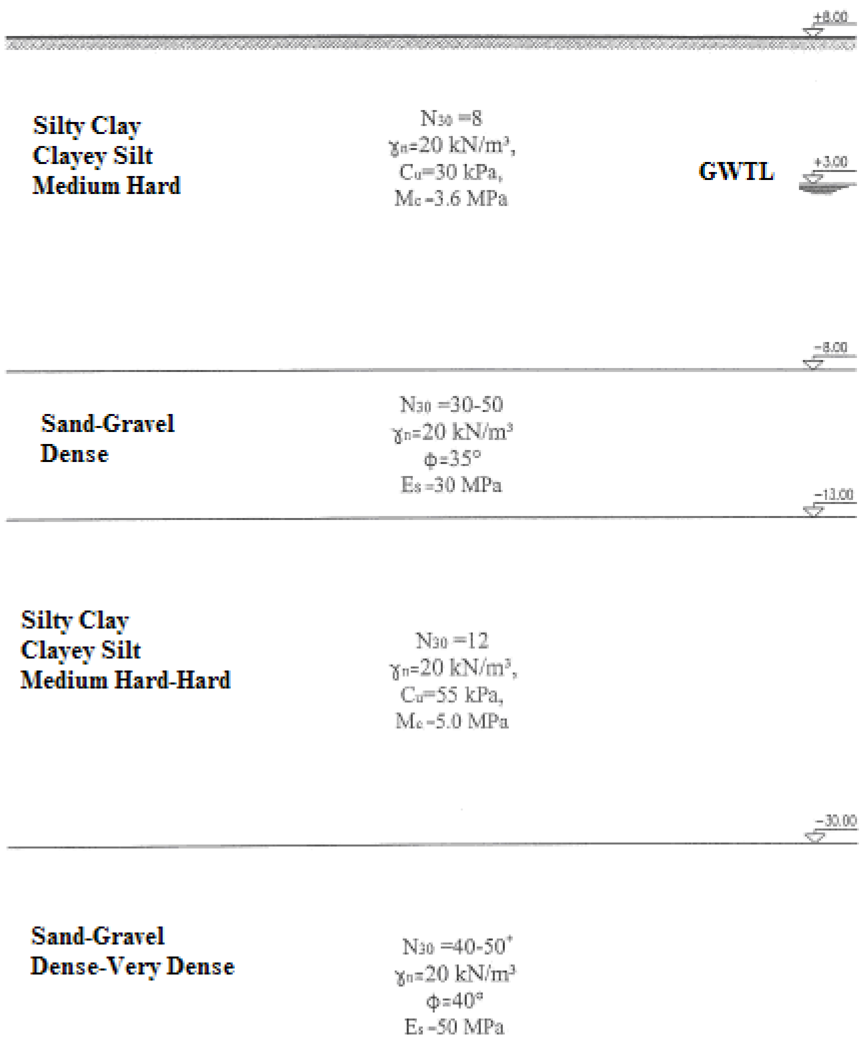



**Figure 3.1:** Cold Mill soil profile.

Regarding to this soil model, generally on the soil surface 16.00 m thickness medium hard consistency silty clay - clayey silt unit is exist. Under this compressible unit there is a dense sand-gravel 5.00 m thickness unit. At deeper depths 15.00-17.00m thickness medium hard-hard silty clay – clayey silt soil layers were found. Generally, at 38.00-40.00 m depth dense-very dense sand-gravel storing was found. Engineering parameters of soil layers are illustrated in Figure 3.1. According to field exploration results; soil profile and soil parameters were defined.

**Silty Clay-Clayey Silt (I):** Cohesive soil is found closed to the surface of soil in Cold Mill's construction site. This unit is generally medium hard. For silty clay – clayey silt these values can be selected.

**Table 3.2:** Soil profile-I engineering properties.

| | |
|---|---|
| Natural unit weight | $\gamma n$ = 20.0 kN/m3 |
| Avarage SPT impact number | N30 = 8 |
| Undrained Cohesion | cu = 30 kPa |
| Deformation Modulus | Mc = 3.6 MPa |
| Vertical bearing coefficient | kv = 2 000 t/m³ = 20 000 kN/m³ |

**Sand-Gravel (I):** Values for dense packed granular soil units that are encountered at depths 16.00-21.00 m is given below.

**Table 3.3:** Soil profile-I engineering properties.

| | |
|---|---|
| Natural unit weight | $\gamma n$ = 20.0 kN/m3 |
| Avarage SPT impact number | N30 = 30-50 |
| Friction angle | $\phi$ = 35° |
| Deformation Modulus | Es = 30.0 MPa |
| Vertical bearing coefficient | kv = 5 000 t/m³ = 50 000 kN/m³ |

**Silty Clay-Clayey Silt (II):** In this soil profile among 21.00-38.00 m depths there is thick silty clay – clayey silt unit. This cohesive unit has medium hard – hard consistency. Soil parameters for this soil unit are given below.

**Table 3.4:** Soil profile-II engineering properties.

| | |
|---|---|
| Natural unit weight | $\gamma n$ = 20.0 kN/m3 |
| Avarage SPT impact number | N30 = 12 |
| Undrained Cohesion | cu = 55 kPa |
| Deformation Modulus | Mc =5.0 MPa |

**Sand-Gravel (II):** Values that have chosen for this soil profile that is dense-very dense packed granular unit at the depth of 40.00 m is given below.

**Table 3.5:** Soil profile-II engineering properties.

| | |
|---|---|
| Natural unit weight | $\gamma n$ = 20.0 kN/m3 |
| Avarage SPT impact number | N30 = 40-50 |
| Friction angle | $\phi$ = 40° |
| Deformation Modulus | Es = 50.0 MPa |

#### 3.1.5.9 Engineering properties of Cold Mill-galvanized coating-dyehouse building

The size of Cold Mill-Galvanized Coating-Dyehouse Building is 146.00 m x 402.00 m. Cold Mill soil sections are given in Appendix-A. Top elevation of the pavement surface is projected as +8.55 m.

Steel columns that are placed among 26A-34 axes are carried by T1 and T3 individual footings. Foundations are going to construct 2.80 m deep from the top elevation of pavement which means it will be on +5.75 m elevation. Foundation base elevation is above the ground water table level so there won't be any problem about ground water. However, there are channels which are placed on +1.00 m elevation. This means that there will be excavation under 2 m of ground water level thus inside of these channels should be kept dry.

Average stress is defined as 140 kPa and maximum stress that is transmitted to the foundation was defined as 230 kPa for this building.

#### 3.1.5.10 Geotechnical considerations

a. Foundation system and settlements

If the soil model which is given in Figure 3.1 is observed 12.00-16.00 m thickness medium hard silty clay – clayey silt carries the whole shallow foundation of the Cold Mill-Galvanized Coating-Dyehouse Building. In this situation, bearing capacity verification should be made with silty clay layer's shear strength. For silty clay layer according to SPT test results, undrained cohesion is chosen around 30 kPa.

In this condition bearing capacity of shallow foundation is calculated as:

qult = 6 cu + $\gamma$ Df = 6 x 30 + 20 x 2.80 = 180 + 56 = 236 kPa

Fs = 3.0;

qall = 236 / 3 $\cong$ 80 kPa < 200 kPa

In the project for shallow foundations average base stress is 200 kPa so the failure safety is not obtained. By considering $\sigma_z$= 200 kPa foundation's long term settlement is also calculated. Settlement calculations are given in Appendix-A. Calculated settlement value for T3 foundation is 14.00 cm. For this reason it is expected that there would be different settlement between these foundations and this settlement value would be around 5.00 cm. Considering the structure's working principles that different settlements and settlements cannot be allowed. Finally, a part of Cold Mill-Galvanized Coating-Dyehouse Building that is between 26A-34 axes cannot be carried by T1 and T3 shallow foundations. Thus, to carry foundation loads safely, foundation soil must be improved by jet grout columns.

b. Improving soil by jet columns

Jet grout columns bearing capacities were calculated for foundations that place on +5.60 m and +2.00 m elevation. These calculations are given in Appendix-A. In either case, jet columns that have 18.00 m length and 0.80 m diameter with Fs=2.5 carry Qall = 650 kN .

Total foundation load will be N=6000 kN if the foundation transfers $\sigma_0$ = 200 kPa to soil. In this case, 6000 / 650 = 9 jet columns are adequate under each shallow foundation. For T3 foundation a typical jet grout location is illustrated in Figure 3.2. In order to other foundations jet grouting placement can be done the same.

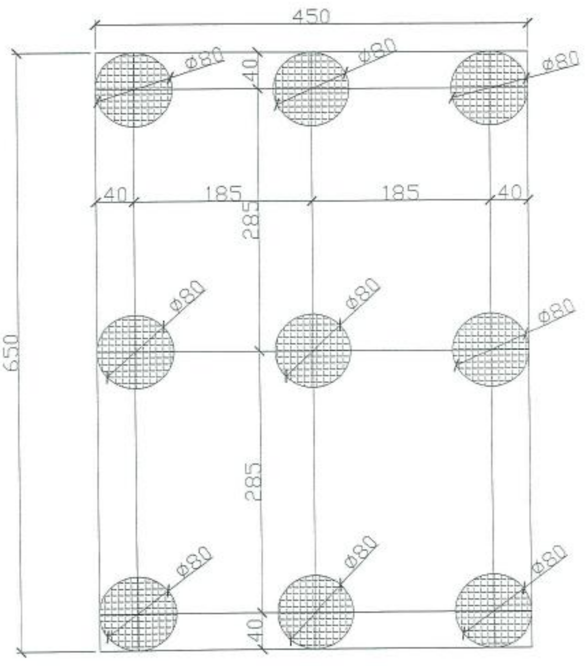



**Figure 3.2:** Jet grout layout plan for T3 foundation.

Concrete slabs which sit on soil are affected by stresses around 140-230 kPa. These concrete slabs are placed on medium hard silty clay soil layer. To prevent occurring damage on concrete slabs because of the long time settlement (consolidation settlement) silty clay soil which is under the slabs is at least 1 m soil is removed. Than, by using sand-gravel or crushed stone 4 x 25 cm thickness engineering fill must be done. This fill has to be compressed till 95% modified proctor density is achieved. It is also recommended that concrete slab thickness that sits on compressed granular soil should be selected at least 20 cm, joints on concrete should not exceed 6 m and two lines of steel mat should be used.

c. Application specifications of jet grout columns

Before starting the application, working platform must be occurred. Jet grout equipments are brought to the field then it should be checked. Cement which is used for jet grout production must be min Portland cement 42.5 and it has to be homogeneous and also, well crushed.

Cement and water mixing ratio should be 1/1. Injection mixing is not waited in mixer more than an hour.

By using drilling set and water jet, 4 inch or 90 mm diameter holes are occurred at the specified depths according to project. When the desirable depth is achieved cement/water mixing which is mixed at 1/1 ratio is applying to soil with high pressure.

At the beginning jet grout columns which are designed in the project would be tried to occur by considering soil conditions in the construction field. Obtained jet columns will be tested and in the view of the fact that these results jet pressure, rotation value and constant pull velocity are determined.

d. Test jet grout columns manufacturing parameters

Under the foundation of Cold Mill building that is the part of Iron and Steel Plant it is decided to improve soil with jet grout columns. Before starting the application of jet grout columns there will be produced 6 test jet grout columns to get exact manufacturing parameters that are proper for the soil. Test parameters are given in Table 3.6.

**Table 3.6: Test** parameters of jet grout columns.

| Manufacturing Parameters | Column Number: 1 | Column Number: 2 | Column Number: 3 | Column Number: 4 | Column Number: 5 | Column Number: 6 |
|---|---|---|---|---|---|---|
| Desired Column Diameter (cm) | 80 | 80 | 80 | 80 | 80 | 80 |
| Water / Cement Ratio (By weight) | 1 | 1 | 1 | 1 | 1 | 1 |
| Jet Grout Pressure (bar) | 450 | 450 | 400 | 400 | 400 | 400 |
| Rotation Velocity (rpm) | 25 | 20 | 25 | 25 | 20 | 20 |
| Nozzle Diameter (mm) | 2.2 | 2.0 | 2.2 | 2.2 | 2.0 | 2.2 |
| Rod Pull Velocity (cm/min) | 50 | 40 | 50 | 40 | 50 | 40 |

Three days after their manufacturing upper part of these columns are revealed then to produce 80 cm diameter jet columns proper parameters will be chosen.

e. Test jet grout columns for Cold Mill foundation

To obtain jet grout columns manufacturing parameters in March 2008 totally 8 test jet grout columns were done. Manufacturing parameters of these test columns are given in Table 3.7.

**Table 3.7:** Manufacturing parameters of jet grout columns.

| Manufacturing Parameters | Column Number: 1 | Column Number: 2 | Column Number: 3 | Column Number: 4 | Column Number: 5 | Column Number: 6 | Column Number: 7 | Column Number: 8 |
|---|---|---|---|---|---|---|---|---|
| Desired Column Diameter (cm) | 80 | 80 | 80 | 80 | 80 | 80 | 80 | 80 |
| Water / Cement Ratio (By weight) | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| Jet Grout Pressure (bar) | 450 | 400 | 400 | 400 | 500 | 450 | 400 | 450 |
| Rotation Velocity (rpm) | 25 | 25 | 25 | 20 | 25 | 20 | 20 | 20 |
| Nozzle Diameter (mm) | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 2.0 | 2.0 | 2.4 |
| Rod Pull Velocity (cm/min) | 50 | 50 | 40 | 50 | 50 | 40 | 50 | 50 |
| Total Injection (kg) | 2000 | 2000 | 2160 | 2200 | 2400 | 2000 | 2000 | 2600 |

After 3 days of manufacturing 8 test jet grout columns diameters were tested then it was found that jet columns diameters were ranging from 45 to 50 cm. Thus, it was decided to produce 4 additional test columns. Additional jet grout columns parameters are given in Table 3.8.

**Table 3.8:** Test parameters of additional jet grout columns.

| Manufacturing Parameters | Column Number: 9 | Column Number: 10 | Column Number: 11 | Column Number: 12 |
|---|---|---|---|---|
| Desired Column Diameter (cm) | 80 | 80 | 80 | 80 |
| Water / Cement Ratio (By weight) | 1 | 1 | 1 | 1 |
| Jet Grout Pressure (bar) | 500 | 500 | 550 | 550 |
| Rotation Velocity (rpm) | 16 | 16 | 16 | 16 |
| Nozzle Diameter (mm) | 2.4 | 2.4 | 2.4 | 2.4 |
| Rod Pull Velocity (cm/min) | 40 | 40 | 40 | 40 |

4 additional test jet grout columns diameters were controlled in 16.03.2008 and the results are given in Table 3.9.

**Table 3.9:** Manufacturing parameters of additional jet grout columns.

| Manufacturing Parameters | Column Number: 9 | Column Number: 10 | Column Number: 11 | Column Number: 12 |
|---|---|---|---|---|
| Desired Column Diameter (cm) | 80 | 80 | 80 | 80 |
| Water / Cement Ratio (By weight) | 1 | 1 | 1 | 1 |
| Jet Grout Pressure (bar) | 500 | 500 | 550 | 550 |
| Rotation Velocity (rpm) | 16 | 16 | 16 | 16 |
| Nozzle Diameter (mm) | 2.4 | 2.4 | 2.4 | 2.4 |
| Rod Pull Velocity (cm/min) | 40 | 40 | 40 | 40 |
| Measured Column Diameter (cm) | 66.25 | 62.5 | 61.25 | 90.5 |


**Figure 3.3:** Test jet grout column.

Within the scope of the project jet columns that have D=0.80 m diameter, L=18m length can be produced with these parameters that are given below:

| | |
|---|---|
| Water / Cement Ratio (By weight): | 1 |
| Jet Grout Pressure (bar): | 500 |
| Rotation Velocity (rpm): | 16 |
| Nozzle Diameter (mm): | 2.4 |
| Rod Pull Velocity (cm/min): | 40 |

f. The revision for Cold Mill foundation jet grout columns production parameters

On the date of 12.04.2008 field investigation was made on construction site and 5 jet grout columns that are under G-26 foundation axis diameters were controlled. The whole columns are produced with these parameters:

| | |
|---|---|
| Water / Cement Ratio (By weight): | 1 |
| Jet Grout Pressure (bar): | 550 |
| Rotation Velocity (rpm): | 16 |

Nozzle Diameter (mm): 2.4

Rod Pull Velocity (cm/min): 40

**Table 3.10:** Manufacturing parameters of jet grout columns.

| Manufacturing Parameters | Additional Column Number: 1 | Additional Column Number: 2 | Additional Column Number: 3 | Additional Column Number: 4 | Additional Column Number: 5 |
|---|---|---|---|---|---|
| Pre-drilling Pressure (bar) | 400 | 400 | 350 | 350 | 300 |
| Pre-drilling Rotation Velocity (cm/min) | 25 | 25 | 25 | 25 | 25 |
| Obtained Column Diameter (cm) | 67 | 63 | 60 | 55 | 52 |

For obtaining suitable jet grout column diameters to project pre-drilling parameters were revised as given below:

Pre-drilling Pressure (bar): 500

Pre-drilling Rotation Velocity (rpm): 25

g. The Revision of jet method for Cold Mill foundation jet grout columns

Jet-2 method test column parameters and measured average column diameter values are given in Table 3.11.

**Table 3.11:** Test jet column parameters with jet-2 method.

| Manufacturing Parameters | Column Number:1 | Column Number: 2 | Column Number: 3 | Column Number: 4 | Column Number: 5 | Column Number: 6 |
|---|---|---|---|---|---|---|
| Desired Column Diameter (cm) | 80 | 80 | 80 | 80 | 80 | 80 |
| Water / Cement Ratio (By weight) | 1 | 1 | 1 | 1 | 1 | 1 |
| Jet Grout Pressure (bar) | 350 | 350 | 350 | 400 | 400 | 400 |
| Rotation Velocity (rpm) | 16 | 16 | 16 | 16 | 16 | 16 |
| Nozzle Diameter (mm) | 3.0 | 3.0 | 3.0 | 3.0 | 3.0 | 3.0 |
| Rod Pull Velocity (cm/min) | 40 | 40 | 40 | 40 | 40 | 40 |
| Obtained Column Diameter (cm) | 90 | 85 | 105 | 86 | 105 | 113 |

There are three main jet systems that are advanced to create required jet column geometry. One of them is called as single fluid system which is the simplest one. In this system grout is ejected and then mixed with soil. Double fluid system involves compressed air that enhances erosive effect. It is very useful when drilling is significantly under ground water.  Triple fluid system which is the third method uses three fluids that are grout, jetting water and compressed air.

This system consists of a grouting nozzle and water jetting nozzle in order to carry much excavated soil particles to the surface while limiting the grout ejected. The triple system achieves erosion and grout injection independently. Thus, it is a system superior to the other two systems and this can be a view point of quality control.

In this project, firstly single fluid system (Jet-1) was preferred as a jet system. Afterwards, it has been seen that 5 jet grout columns which were under G-26 foundation axis diameters has not reached desired length. For this reason as an effective alternative single fluid jet system has changed to double fluid jet system.


**Figure 3.4:** Test column with jet-2 method.

It can be seen from the gained results that Jet-2 method provides suitability to working schedule and efficiently manufacturing. Accordingly, D=0.80 m diameter jet columns can be produced with these parameters that are given below:

| | |
|---|---|
| Water / Cement Ratio (By weight): | 1 |
| Jet Grout Pressure (bar): | 350 |
| Rotation Velocity (rpm): | 16 |
| Nozzle Diameter (mm): | 3 |
| Rod Pull Velocity (cm/min): | 80 |

| | |
|---|---|
| Air pressure (bar) | 10 |

h. Quality control and validation of jet columns

There are three main factors that must be controlled:

. Column diameter

. Column position

. Column property.

It is important to record these parameters for validation of individual jet columns during installation:

. Depth

. Withdrawal rate

. Air pressure

. Grout or water pressure and volume

. Rotation.

Additionally, inclinometers can build into the jet grout monitor that measure deviation of the drill string. It is important to carry out quality control testing on the grouts which is used. This control testing includes specific gravity, viscosity and strength tests by 28 day cube strengths.

Site engineer can understand easily the correctness of column installation by these parameters. Franz (1972), Fritsch and Kirsch (2002), Kirsch and Sonderman (2002) list standards and publications relating to control and execution of jet grouting.

In order to validate a project jet column, its diameter, position and strength or permeability must be checked.

**Column diameter:** The most common method to control column diameter is to construct trial columns and expose them. Exposed columns diameters are measured directly. This method is very effective at shallow depths however, expense increases with depth. Coring of columns can be an alternative but interpretation of diameter or strength with this method is often difficult. Borehole callipers also can be used to measure the extent of a column but it is restricted to shallow depths.

**Column position:** Column position relates with drilling tolerance. It is succeed by installation of inclinometers.

**Column parameters:** Column properties are generally measured by coring techniques. Cylindrical samples are taken from jet columns in the field to get their pressure strength. If column gain its strength core sample is taken. However, if the column is wet sample is taken by piston sampler (Durgunoğlu vd., 2003).

**Pile loading test:** To control jet grout column manufacturing quality and project loads pile loading test is applied. With results of this test load-settlement, load-time and settlement-time graphs are drawn. Test load is selected as 50% more than jet grout load capacity. Test column is loaded until it failures or failure condition is acceptable. When load has a constant value if deformation increases or the load which causes deformation more than 10% of column diameter it is accepted that failure load has achieved. Test equipments are reaction beams, hydraulic jack, hydraulic pressure pump, manometer, and a measurement mechanism.

**Pile integrity test:** Jet column lengths are controlled by integrity tests. In integrity tests a hammer impact is given on the top of the pile. A signal is occurred by an impact and it is affected by variations of pile shape or material. This signal is recorded as a velocity signal when it is refraction. For determining refraction points signal is converted to a function which depends on depth.

## 3.2 Iron and Steel Plant Slitting Cutting Package Service

### 3.2.1 General

In this section case history of the construction of a Cold Mill and an Iron Plate Service Centre in West part of the Turkey is given. Iron and Steel Plant was constructed at the north of the town. It is totally 247 670 m2 and its elevation is 300-350 m above the sea level. Original field inclination is 10% however, before starting the construction with excavation and filling operations south part of the field elevation is made 318.00 m, north part of the field elevation is made 339.00 m.

At the first step, Slitting Cutting Package Service and Cold Mill are desired to construct 318.00 m platform. For this reason, with excavation, filling and wall construction operations south part of the field elevation was smoothed to 318.00 m. General view of the field in March 2008 is given in Figure 3.5. Two-staged reinforced soil wall which is 24.00 m in total has done throughout the south border. Vertical excavation surface that is 21.00 m height has supported with soil nail wall and reinforced soil wall. Field layout plan is given in Appendix-B.

**Figure 3.5:** General view of the field in March 2008.

There would be different buildings that have various functions and sizes are designed within the scope of the project. The field area is totally 400 000 m2 and total floor area of the buildings is approximately 179 000 m2. Construction area is placed on both the natural soil which is 130 000 m2 of and man-made soil which is 49 000 m2.

### 3.2.2 General and regional geology

Field Study Report was prepared about site's geographic location, climate and general geology. Briefly, construction field consists of white, beige coloured, thin-medium stratified, biomicrite, calcarenite and marn storing. Thickness of this unit is around 350 m.

### 3.2.3 Boreholes

25 boring were carried out at the site of an Iron and Steel Plant in February-March 2008. Boring depths are ranging from 10.00 to 20.00 m. During drilling rock samples were taken by core barrel. Type of the rock and fracture, values of TCR and RQD are defined and shown in boring logs. To obtain ground water level perforated pipe were placed to whole boring holes. Boring layout plan is given in Appendix-B. Borehole depths and elevations are illustrated in Table 3.12

**Table 3.12:** General information of an Iron and Steel Plant borings

| Borehole No | Top elevation (m) | Depth (m) |
|---|---|---|
| S1 | 320,17 | 10 |
| S2 | 324,04 | 13,5 |
| S3 | 328,07 | 17 |
| S4 | 330,91 | 20 |
| S5 | 319,21 | 10 |
| S6 | 322,96 | 12 |
| S7 | 326,70 | 16 |
| S8 | 314,57 | 10 |
| S9 | 318,41 | 10 |
| S10 | 322,13 | 11 |
| S11 | 323,86 | 13 |
| S12 | 312,14 | 10 |
| S13 | 315,93 | 10 |
| S14 | 306,53 | 11 |
| S15 | 308,66 | 10 |
| S16 | 308,66 | 11,5 |
| S17 | 310,79 | 10 |
| S18 | 301,66 | 10 |
| S19 | 304,80 | 10 |
| S20 | 307,85 | 12 |
| S21 | 307,48 | 10 |
| S22 | 293,00 | 10 |
| S23 | 310,17 | 10 |
| S24 | 313,64 | 10 |
| S25 | 312,34 | 10 |

### 3.2.4 Ground water conditions

Ground water investigations were made during drilling and it was found that biomicrite does not carry groundwater.

### 3.2.5 Laboratory experiments

In soil mechanic laboratories unconfined compression and point load tests were performed to rock core samples. For each test 31 samples has been used. Therefore, 12 samples were used to determine elasticity modulus.

According to experiments, qu is ranging from 16.0 MPa to 63.0 Mpa in unconfined compression test. Average qu is selected as 40.0 Mpa. Biomicrite soil’s natural unit weight is 24 kN/m3.

Is(50) is ranging from 1.10 Mpa to 4.60 Mpa according to test results. Average Is(50) is selected as 2.50 Mpa.

### 3.2.6 Soil profile and soil model

Soil sections were drawn by using Slitting Cutting Package Service and Cold Mill drilling results. They are given in Appendix-B.

A-A’ soil section which is shown in Appendix-B has prepared by using SK1, SK2, SK3 and SK4 boreholes. Soil profile consists of beige-light gray coloured, thin-medium layered micrite limestone storing. Limestone core samples TCR values were found as 100% and RQD values was found as 40-50%. These values mean that limestone has adequate engineering properties.

B-B’ soil section which is shown in Appendix-B has prepared by using SK8, SK9, SK10 and SK11 boreholes. In this section, it can clearly see that in the middle of field filling operations were made as mentioned at the beginning. Predominantly, this soil section consists of micrite limestone.

Similar soil properties are seen in C-C’, D-D’ and E-E’ sections. In these sections, compressed filling has encountered. Soil profile predominantly consists of medium hard-hard limestone and rock fill storing which is placing on hard lime stone unit. Soil profile properties are given below.

**Rock fill:** With explosion method and crusher rock excavations are made to make south part of the field elevation is 318.00 m and north part of the field is 339.00 m. By using obtained rock material rock filling is made at variable thickness (Figure 3.6). Thickness of the south-west field part is 20.00-22.00 m besides south-east field part is only a couple of meters. After placing the rock fill it must be compressed with a vibratory cylinder.

**Figure 3.6:** The view of rock material in field.

There are various sizes of soil particles in the rock fill. However, it was determined that before compressing laid fill thickness was higher than 25-30 cm. For this reason, compressing could not do effectively. To determine rock fill's engineering properties field experiments should be done. Their names are given below:

- Five drilling investigation holes should be formed to control rock fill
- Pressiometer tests in boreholes
- Area loading test at two points (after dynamic compaction)
- Seismic investigation (before and after dynamic compaction)

To prepare foundation project, rock fill storing geotechnical parameters are given below.

However, after performing field experiments additional opinions will pronounce about rock fill bearing capacity, compressibility and foundation which sits on the fill.

Natural Unit Weight $\gamma n$ = 20.0 kN/m3

Shear strength resistance angle $\varphi = 35°$

Elasticity Modulus Es = 20 MPa

Vertical Bearing Coefficient kv = 20 000 kN/m³ (2 000 t/m3)

Safe Bearing Capacity qall = 150 kPa (1.5 kg/cm2)

**Limestone:** At the construction site soil profile consists of limestone. 25 boreholes were applied in this site and during the complete drilling lime stone was encountered. This Limestone is defined as beige-light gray coloured, thin-medium layered and medium hard rock. TCR and RQD values are very high for limestone. Limestone rock has very high bearing capacity for civil engineering applications and also very low compressibility. It is an ideal foundation material. However, there will be an excavation difficulty during smoothing and foundation excavations. For this reason, explosion method is performing and hydraulic crushers are working on the construction site. Geotechnical parameters of limestone are given below.

Natural Unit Weight $\gamma n$ = 24.0 kN/m3

One dimensioned Pressure Strength qu=40 MPa

Pointed load index Is(50)= 2.50 MPa

Elasticity Modulus Es = 500 MPa

Total Core Percent TCR=100%

Rock Quality Index RQD = 40%

Vertical Bearing Coefficient kv = 100 000 kN/m³ (10 000 t/m3)

Safe Bearing Capacity qall = 0.60 MPa (6 kg/cm2)

To obtain engineering analyses of Slitting Cutting Package Service and Cold Mill soil sections are given in Appendix-B. As can be understood from the soil model construction field consists of two major soil units. On the soil surface there is a compressed rock fill storing which has varied thickness. Second unit of the soil model is micritice limestone that is presented at the basement.

### 3.2.7 Engineering properties of slitting cutting package service and cold mill

Engineering properties of Slitting Cutting Package Service and Cold Mill within the scope of project are summarized below.

**Slitting Cutting Package Service**

Plan sizes of this building are 72.00 m x 197.00 m. Horizontal spacing is 23.00 m and longitudinal spacing is selected as 12.00 m. Foundation of the building is placed on 318.50 m elevation. There is a dilatation on the longitudinal direction.

**Cold Mill**

There are four horizontal spacing that are 28.00 m and sizes of this building are 112.00 m x 402.00 m. Longitudinal spacing is 12.00m. Cold Mill section and plan are given in Appendix-B. Therefore, there will be cranes which have Q=10-60 t capacity in Cold Mill. There are three dilatations on the longitudinal direction in building. Building foundations will be constructed under 2.80 m depth from ground surface.

### 3.2.8 Geotechnical considerations

**Foundation system and settlements**

- **Iron plate service centre**

Building foundation is shown in Appendix-B. Rock surface elevations of Slitting Cutting Package Service construction are ranging from 302.00 m to 315.00 m. It means that all foundations of the building sit on rock fill soil. Rock fill thickness that is presented under the foundations is among 14.00 m to 1.00 m.

There are main subjects that must be considered during projecting the foundations of Slitting Cutting Package Service.

Because of rock fill doesn't show homogeneous characteristic properties and rock fill thickness is various foundations should be projected as continuous footing to prevent different settlements. Maximum different settlement is 2 cm and maximum angular rotation is 1/1000 for foundation design.

Safe bearing capacity can be selected as 150kPa. For earthquake loads this value can be 50% increased. With additional field experiments this values will be controlled. If this bearing capacity is not obtained, dynamic compaction will be applied to improve soil.

During increasing rock fill on 314.00 m elevation, uncompressed thickness doesn't be higher than 25 cm and maximum aggregate size should be 10 cm.

- **Cold Mill and equipment foundations**

In this building, foundations that are especially present on D and E axes sit on rock filling. These foundations also carry Slitting Cutting Package Service building. West part of the F axis some foundations sit on rock fill. For this reason, F axis foundations should be projected as an individual footing and between rock and foundation base, lean concrete mat should be occurred where it is necessary.

Foundations which are on the G, H and L axes will be carried by rock soil. Thus, there won't be serious problem with Cold Mill Foundations. There are main subjects that must be considered during projecting the foundations of Cold Mill Foundations.

Safe bearing capacity can be selected as qall=600 kPa for individual foundation design. In this condition, foundation sizes can't be minimum and so safe bearing capacity must be selected as qall=200-250 kPa. For earthquake force analyses safe bearing capacity can be 50% increased. On rock soil and lean concrete mat there won't be any settlements.

If rock excavations make with explosion method it should be done before starting the construction of reinforced soil wall, soil nail wall and building foundation construction.

**Geotechnical earthquake engineering considerations**

These geotechnical earthquake engineering parameters can be considered for Slitting Cutting Package Service which sits on rock filling.

Soil Group C

Local Soil Class Z3

Effective Ground Acceleration A0 = 0.40

Spectrum Characteristic Periods TA = 0.15 TB = 0.60

These geotechnical earthquake engineering parameters can be considered for Cold Mill which sits on limestone.

Soil Group A

Local Soil Class Z1

Effective Ground Acceleration A0 = 0.40

Spectrum Characteristic Periods TA = 0.10 TB = 0.30

### 3.2.9 Slitting Cutting Package Service dynamic compaction improvement application

Slitting Cutting Package Service will be carried by rock fill layer that has 10-15 m thickness. As mentioned before rock fill is not homogenous and its thickness is various under foundation. This condition causes different settlements when earthquake occurs. To increase safe bearing capacity and to decrease rock fill void ratio it is proper to choose dynamic compaction.

Before dynamic compaction seismic refraction experiments were performed and results are given below.

**Table 3.13:** Seismic refraction experiment results before dynamic compaction

| Profile No | 1.Layer P Wave Velocity (m/s) | 2.Layer P Wave Velocity (m/s) |
|---|---|---|
| 1 | 1111,00 | 2777 |
| 2 | 1538,00 | 1666 |
| 3 | 1428,00 | 1836 |
| 4 | 1111,00 | 1333 |
| 5 | 1237 | 1521 |
| 6 | 926,00 | 1812 |
| 7 | 736,00 | 1526 |
| 8 | 831,00 | 1451 |
| 9 | 1662,00 | 1891 |
| 10 | 1518,00 | 1927 |

Dynamic compaction was applied on rock fill soil which is going to carry Slitting Cutting Package Service Foundations. Concrete cylinder that is 12 tons was dropped 6 times from 20 m high and 80 cm depth craters were gained on rock fill soil surface so, rock fill was compressed (Figure 3.7). With dynamic compaction 28 000 $m^2$ field was improved. During compressing process 550 $tm/m^2$ energy was applied on unit field.

**Figure 3.7:** The view of Rock Fill after dynamic compaction

After compaction, to see its effectiveness seismic refraction experiments were performed at the same points. Results of these experiments are given in Table 3.12.

It can be understood from seismic refraction experiments on the soil surface there is a loose layer that is 1.00 m. This soil layer is among weight dropped areas and also it is heaved and loosened layer. However, it won’t be a problem because it will be removed during foundation construction.

**Table 3.14:** Comparing seismic refraction experiment results before and after dynamic compaction.

| | 1.Stage | | 2.Stage | |
|---|---|---|---|---|
| Profile No | 1.Layer P Wave Velocity (m/s) | 2.Layer P Wave Velocity (m/s) | 1.Layer P Wave Velocity (m/s) | 2.Layer P Wave Velocity (m/s) |
| 1 | 1111,00 | 2777 | 2425,00 | 2976 |
| 2 | 1538,00 | 1666 | 2602,00 | 3000 |
| 3 | 1428,00 | 1836 | 1500,00 | 2900 |
| 4 | 1111,00 | 1333 | 1356,00 | 2477 |
| 5 | 1237 | 1521 | 1700 | 2500 |
| 6 | 926,00 | 1812 | 1790,00 | 3200 |
| 7 | 736,00 | 1526 | 1584,00 | 2728 |
| 8 | 831,00 | 1451 | 1250,00 | 2941 |
| 9 | 1662,00 | 1891 | 1700,00 | 3033 |
| 10 | 1518,00 | 1927 | 1780,00 | 2975 |

As can be seen from Table 3.12 after dynamic compaction P wave velocities increase. It shows that soil layer is compressed. To classify soil regarding to earthquake regulations velocity of S wave is needed so, by using Vp=1.71Vs it can be calculated.

Before dynamic compaction rock fill is C soil group that is medium dense sand-gravel soil. After dynamic compaction according to earthquake regulations rock fill is B soil class that is dense sand-gravel soil.

Improving soil with dynamic compaction is succeeded for the construction field of Slitting Cutting Package Service building. Before compressing average Vp =1000 m/s besides after compressing it reaches Vp=1700 m/s. So that, engineering properties of rock fill are improved at least 50%.

Geotechnical earthquake engineering parameters of improved rock soil with dynamic compaction are given below.

Soil Group B

Local Soil Class Z2

Effective Ground Acceleration A0 = 0.40

Spectrum Characteristic Periods TA = 0.15 TB = 0.40

# 4. CONCLUSIONS

Soil treatment is applied to alter engineering properties of soil. Generally, the main aims of the soil improvement are increasing the shear strength of soil, decreasing permeability and undesired settlements, increasing density for cohesionless soil and consistence for cohesive soil. There are important factors while soil improvement method is selecting such as soil properties, field conditions, designed structure properties, applicability, time and economy. Furthermore, experience is very important to decide which soil treatment method will be most effective and to follow the effectiveness and the quality of project manufacturing.

There are many soil improvement methods that relates to soil type. Mainly these methods are divided two titles that are ‘shallow improvement methods’ and ‘deep improvement methods’. In this thesis, deep soil improvement methods are investigated in detail and also case histories are given for deep improvement methods which are jet-grout and dynamic compaction. Summarizes and considerations of these methods are given below.

## 4.1 Cold Mill-Galvanized Coating-Dyehouse Building

Cold Mill-Galvanized Coating-Dyehouse Building foundation sits on 12.00-16.00 m thickness medium hard silty clay – clayey silt.

In this situation, bearing capacity verification has done with silty clay layer’s shear strength. Factor of safety was selected as 3.0 and accordingly qall was calculated as 80 kPa. In the project for shallow foundations average base stress is 200 kPa so the failure safety is not obtained. By considering $\sigma z$= 200 kPa foundation’s long term settlement was also calculated. For T3 foundation calculated settlement value is 14.00 cm. Therefore, it is expected that there would be different settlement between these foundations and this settlement value would be around 5.00 cm. Predicted different and total settlements cannot be allowed. It was assumed that a part of Cold Mill-Galvanized Coating-Dyehouse Building that is between 26A-34 axes cannot be carried by T1 and T3 shallow foundations.

Verifications for failure and consolidation settlement are not safe. There are two alternatives to correct this event for this system that are; shallow foundations can be placed on piles or improving foundation soil by jet grouting method. In this condition, it was preferred to improve foundation soil by jet grout columns. Because, this solution will be safe and economic. It was decided that jet columns that have D=0.80 m diameter, L=18m length and S=2.85m spacing is proper for the project.

In this project, firstly single fluid system was preferred as a jet system. Afterwards, it has been seen that 5 jet grout columns which were under G-26 foundation axis diameters has not reached desired length. For this reason as an effective alternative single fluid jet system has changed to double fluid jet system. It can be seen from the gained results that Jet-2 method provides suitability to working schedule and efficiently manufacturing.

### 4.2 Slitting Cutting Package Service and Cold Mill

**Slitting Cutting Package Service,** all foundations of the building sit on rock fill soil. Rock fill thickness that is presented under the foundations is among 14.00 m to 1.00 m. Because of rock fill doesn’t have homogeneous characteristic and its thickness is not constant foundations should be projected as continuous footing to prevent different settlements. Safe bearing capacity was selected 150 kPa and it was controlled by field experiments. Bearing capacity didn’t obtain with tests and it was decided to improve soil by dynamic compaction. To increase safe bearing capacity and to decrease rock fill void ratio it is proper to apply dynamic compaction. Concrete cylinder that is 12 tons was dropped 6 times from 20 m high and 80 cm depth craters were gained on rock fill soil surface so, rock fill was compressed. With dynamic compaction 28 000 m2 field was improved. During compressing process 550 tm/m2 energy was applied on unit field. After dynamic compaction P wave velocities increase and it shows that soil layer is compressed.

Before dynamic compaction rock fill is C soil group that is medium dense sand-gravel soil. After dynamic compaction according to earthquake regulations rock fill is B soil class that is dense sand-gravel soil. Before compressing average Vp =1000 m/s besides after compressing it reaches Vp=1700 m/s. So that, engineering properties of rock fill are improved at least 50%.

**<u>Cold Mill and equipment foundations</u>**, in this building, foundations that are especially present on D and E axes sit on rock filling. Also, these foundations carry Slitting Cutting Package Service. Foundations which are on the G, H and L axes will be carried by rock soil. Thus, there won't be any problem with Cold Mill Foundations. Improvement is not necessary.

It can be seen from the results that with suitable improvement method and experience engineering properties of soil are altered. Soil improvement applications are increasing day by day in the last decade in Turkey. Because it is economic, quick and effective.

### 4.3 General Conclusion

Soil improvement methods such as jet grout, piling, soil replacement and dynamic compaction have been executed in both sites where there are adverse soil conditions. After implementation of such soil improvement methods, it has been observed that those methods have been very successful in terms of increasing the bearing capacity and decreasing the compressibility of soils encountered in both sites. Therefore, although the structures of steel plants that have been erected in both sites are relatively heavy, there have been no noticeable settlements so far. This reveals that soil improvement techniques that have been implemented at both sites have been very successful.

## REFERENCES


**ASCE,** July 1997. Ground Improvement, Ground Reinforcement and Ground Treatment Developments, Geotechnical Specification Publication No.69, Utah State University, Logan, Utah.

**Ayan, E.,** 2009. Derin Zemin İyileştirme Yöntemleri ve Uygulamadan Örnekler, *Yüksek Lisans Tezi*, İTÜ, İstanbul.

**Baker, H.,** 1996. Grouting in Geotechnical Engineering, New Orleans, L.A..

**Barron, R.A.,** (1948). Consolidation of fine-grained soils by drain wells, Trans. ASCE, 113(2346),718–742.

**Covil, C.S. & Skinner, A.E,.** (1994). Jet grouting, a review of some of the operating parameters that form the basis of the jet grouting process, in Proceedings Grouting in the Ground.

**Das, B., M.,** 2002, "Principles of Geotechnical Engineering" 5th Edition, California State University, Sacramento.

**De Silva, S.,** Implementation and Performance of Stone Columns at Penny's Bay Reclamation in Hong Kong, Maunsell Geotechnical Services Limited, Hong Kong.

**EM 1110-1-3500,** 1995. Engineering and Design Chemical Grouting, Department of the Army, U.S. Army Corps of Engineers, Washington DC 20314-1000.

**ENAR Müh. Mim. Danışmanlık Ltd. Sti.,** 2008. Demir Çelik Tesisleri Sac Servis Merkezi ve Soğuk Haddehane Tesisi Mühendisliği Değerlendirme Raporu, İstanbul.

**ENAR Müh. Mim. Danışmanlık Ltd. Sti.,** 2008. Demir Çelik Tesisleri Soğuk Haddehane Jet Grout Kolonu Yapım Teknik Şartnamesi, İstanbul.

**ENAR Müh. Mim. Danışmanlık Ltd. Sti.,** 2008. Demir Çelik Tesisleri Zemin ve Temel Mühendisliği Değerlendirme Raporu, İstanbul.

**ETL 1110-1-185,** 1999. Guidelines on Ground Improvement for Structures and Facilities, Department of the Army, U.S. Army Corps of Engineers, Washington DC 20314-1000.

**Fellenius, B.H. & Massarsch, K.R,.** (2001). Dynamic Compaction of Coarse-Grained Soils, A case history. Proceedings of the 54th Canadian Geotechnical Conference, Calgary, September 19–21, Canadian Geotechnical Society.

**FHWA-IF-99-015,** 1999. Geotechnical Engineering Circular No.4: Ground Anchors and Anchored Systems, U.S. Department of Transportation, Federal Highway Administration, Washington DC.

**FHWA0-IF-03-017,** 2003. Geotechnical Engineering Circular No.7: Soil Nail Walls, U.S. Department of Transportation, Federal Highway Administration, Washington DC.

**FHWA-RD-83-026,** 1983. Design and Construction of Stone Columns Volume I, U.S. Department of Transportation, Federal Highway Administration, Washington DC.

**FHWA-RD-99-138,** 2000. An introduction to the Deep Soil Mixing Methods as Used in Geotechnical Applications, U.S. Department of Transportation, Federal Highway Administration, Washington DC.

**FHWA-SA-95-037,** 1995. Geotechnical Engineering Circular No.1: Dynamic Compaction, U.S. Department of Transportation, Federal Highway Administration, Washington DC.

**Krogh, P. & Lindgren, A,.** (1997). Dynamic Field Measurements During Deep Compaction at Changi Airport, Singapore, Examensarbete 97/9. Royal Institute of Technology (KTH), Stockholm, Sweden.

**Kutzner, C.,** 1996. Grouting of Rock and Soil, Technical University Darmstadt, Hofheim, Germany.

**Küsin, C.C.,** 2009. Jet-Grout Yöntemi ile İyileştirilen Zeminlerin Sonlu Elemanlar Yöntemiyle Sayısal Analizi, *Yüksek Lisans Tezi*, Çukurova Üniversitesi, Adana.

**Murthy, V.N.S.,** 2002. Geotechnical Engineering: *Principles of Soil Mechanics and Foundation Engineering* , 5th Edition, UBS Publishers Distributors Ltd., New Delhi.

**ENAR Müh. Mim. Danışmanlık Ltd. Sti.,** 2008. Demir Çelik Tesisleri-Soğuk Haddehane-Galvaniz Kaplama-Boyahane Binası Zemin ve Temel Mühendisliği Değerlendirme Raporu, İstanbul.

**Toker Sondaj ve İnsaat Ltd. Sti.,** 2008. Demir Çelik Tesisi-Soğuk Haddehane-Asitleme-Galvaniz ve Boya Üniteleri Arazi Çalışma Raporu, Ankara.

**Toker Sondaj ve İnsaat Ltd. Sti.,** 2008. Kocaeli-Gebze VI. (İMES) Organize Sanayi Bölgesi 100 Nolu Parsel Arazi Çalışmaları Raporu, Ankara.

APPENDICES

APPENDIX A : Jet Grouting Project Documentations
APPENDIX B : Dynamic Compaction Project Documentations

## APPENDIX A

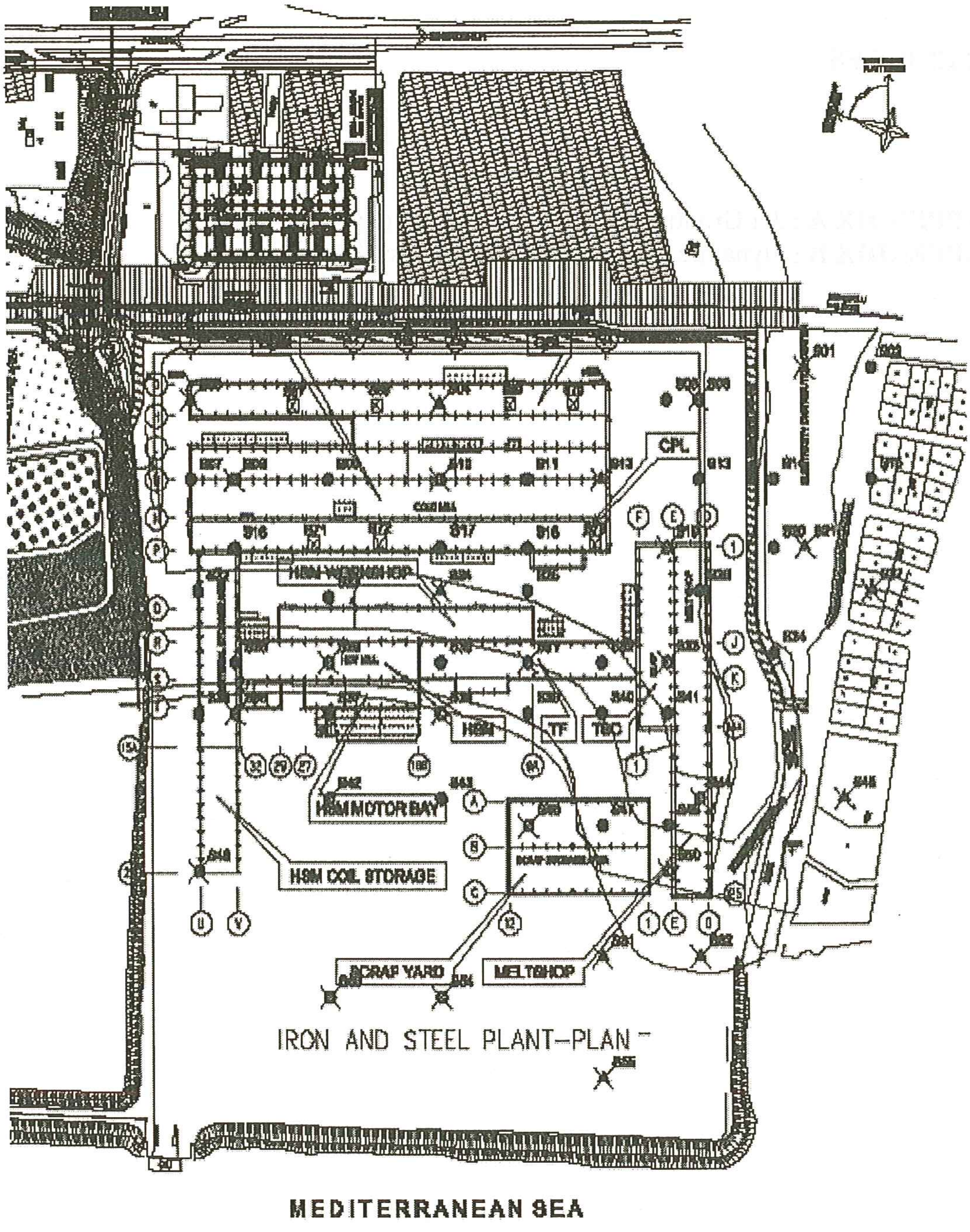



**Figure A.1:** Layout plan of Iron and Steel Plant.

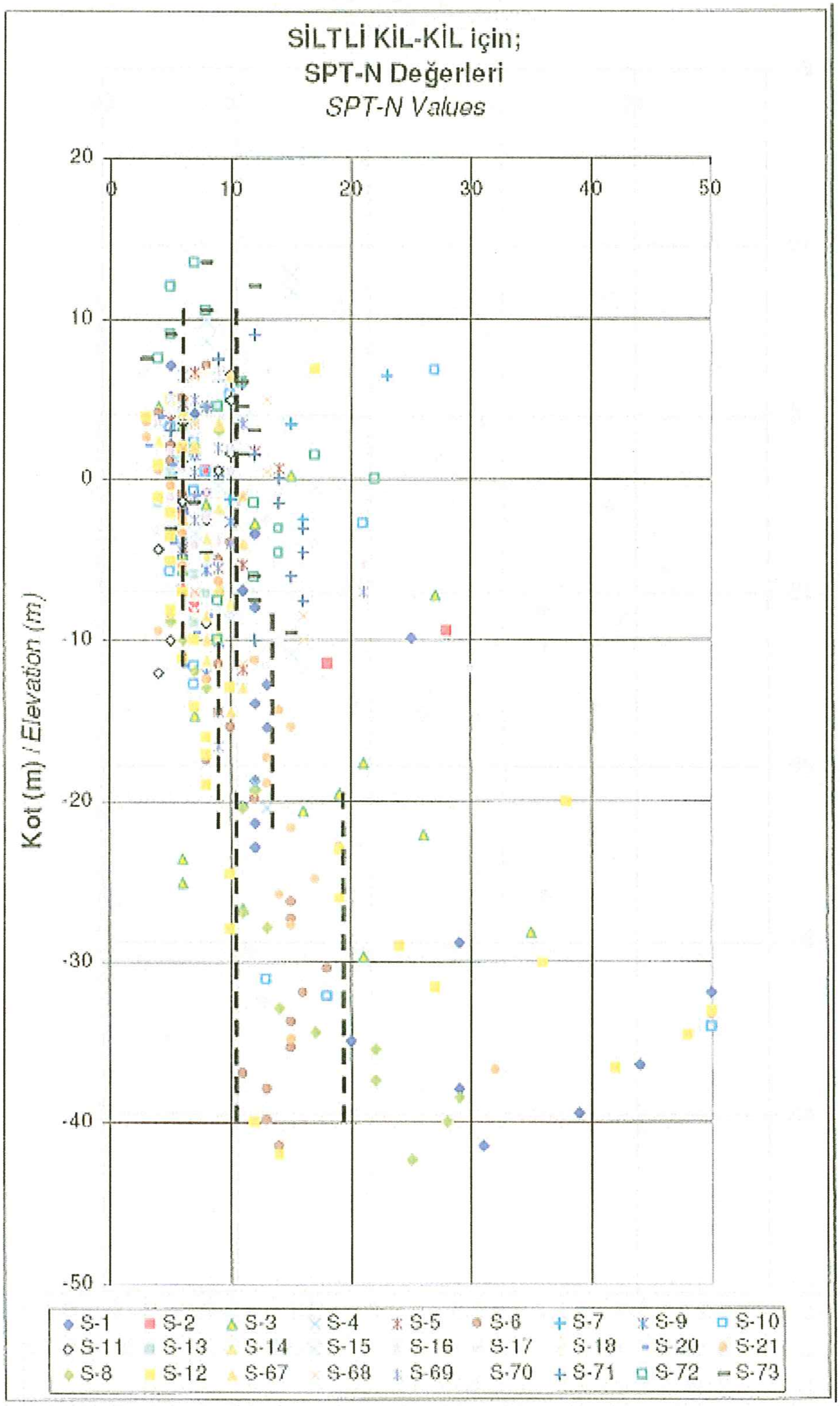



**Figure A.2:** SPT-N results for silty-clay soil.

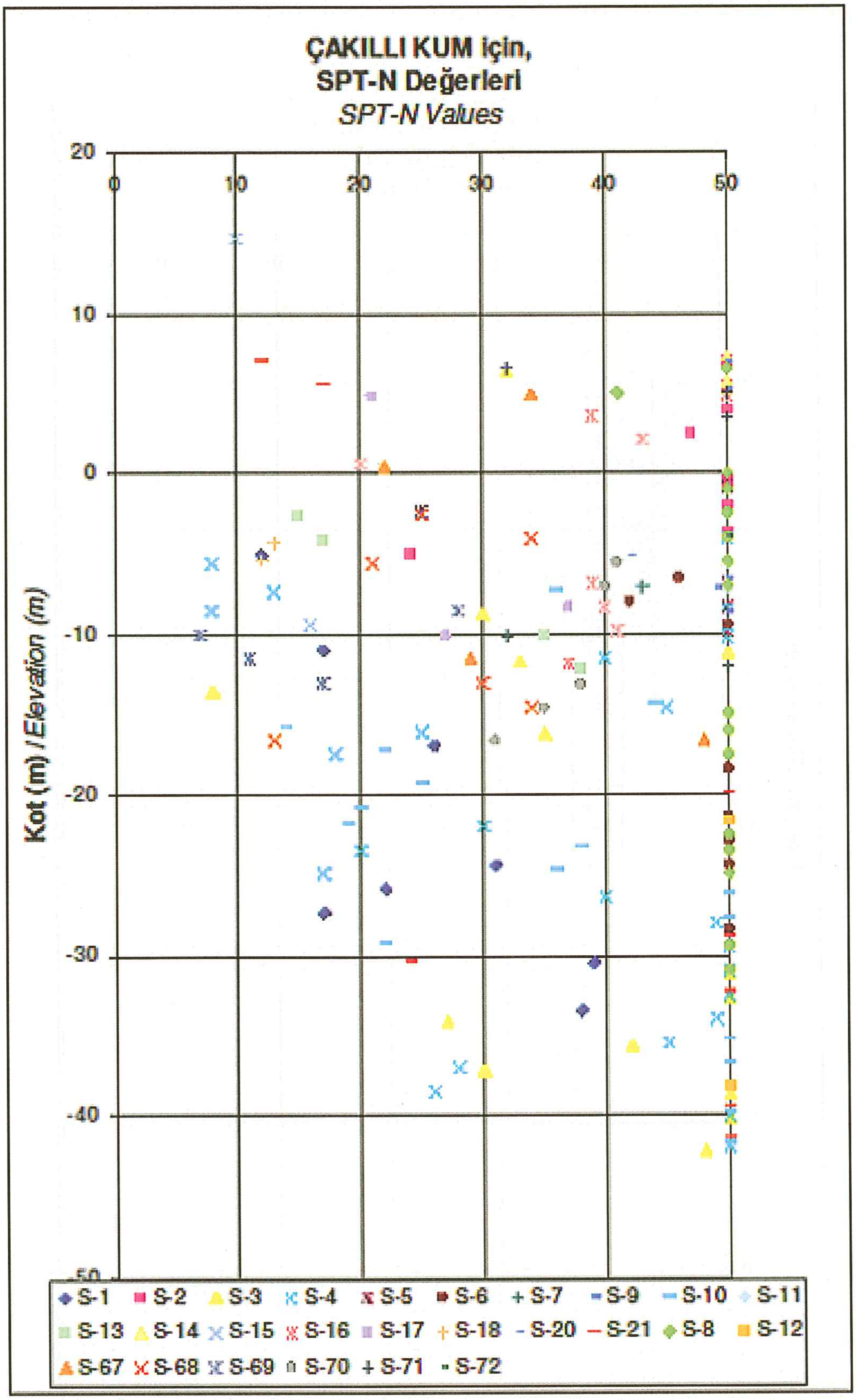



**Figure A.3:** SPT-N results for sandy-gravel soil.

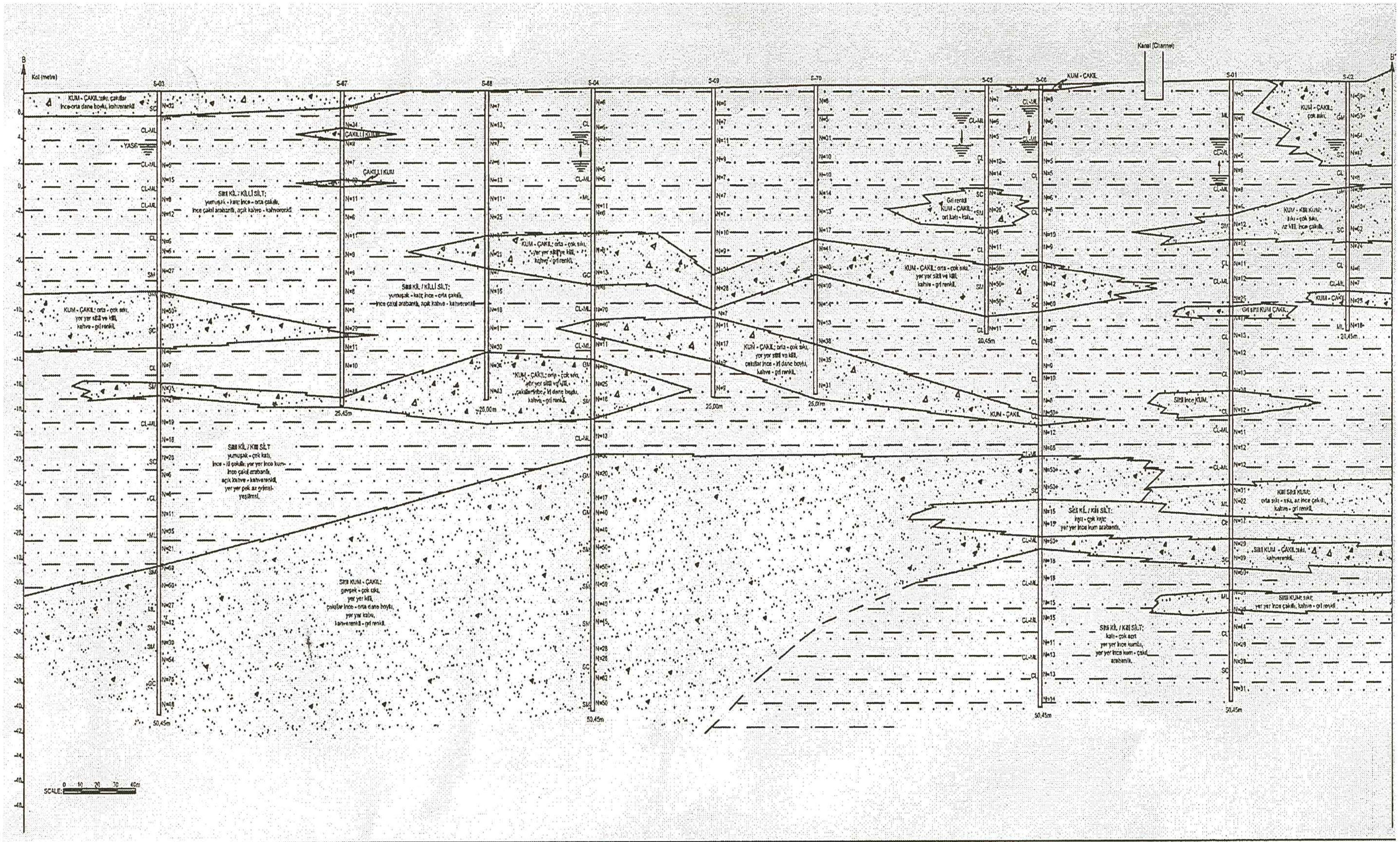


**Figure A.4 :** B-B soil section.

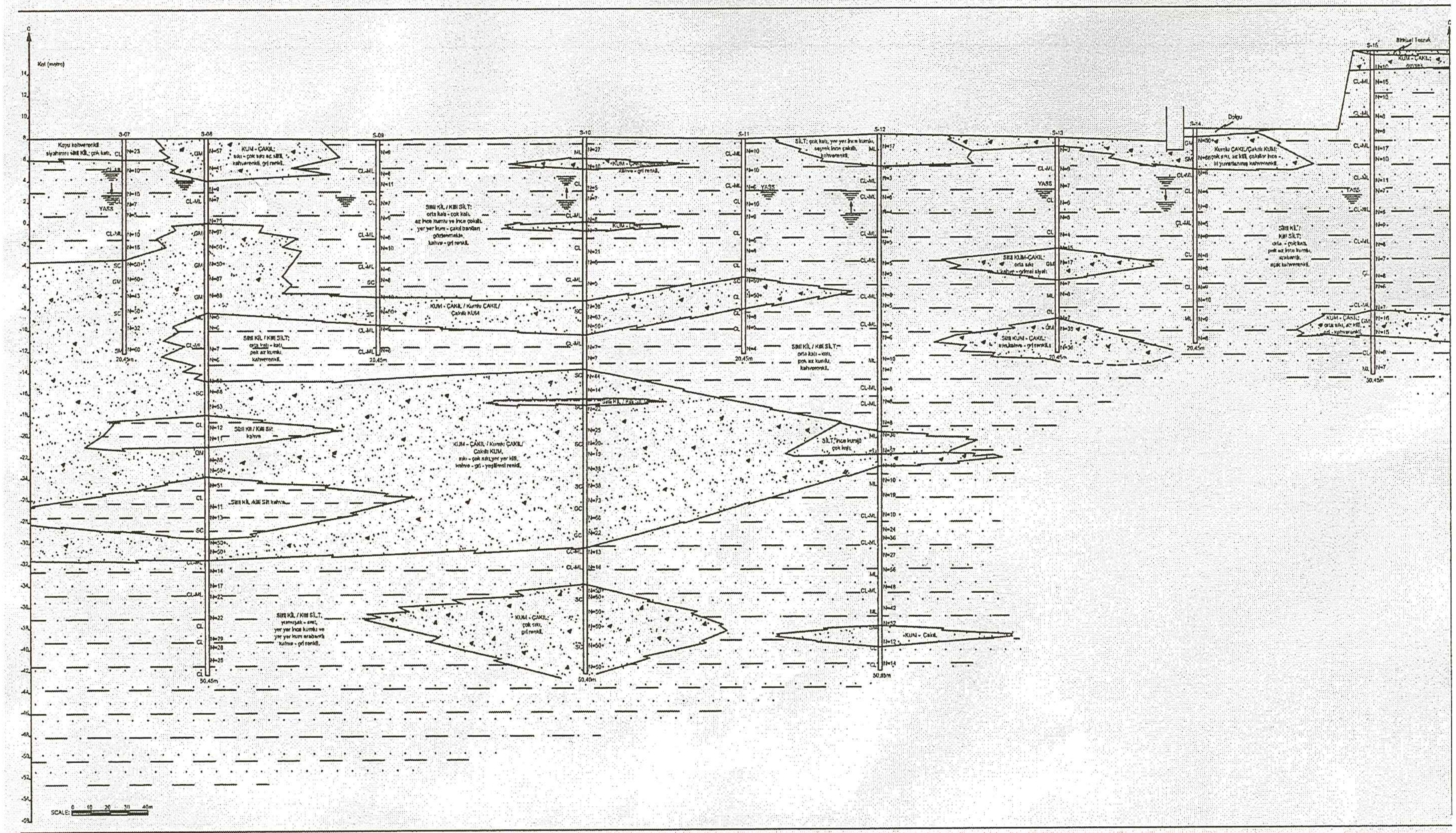


**Figure A.5 :** C-C soil section.

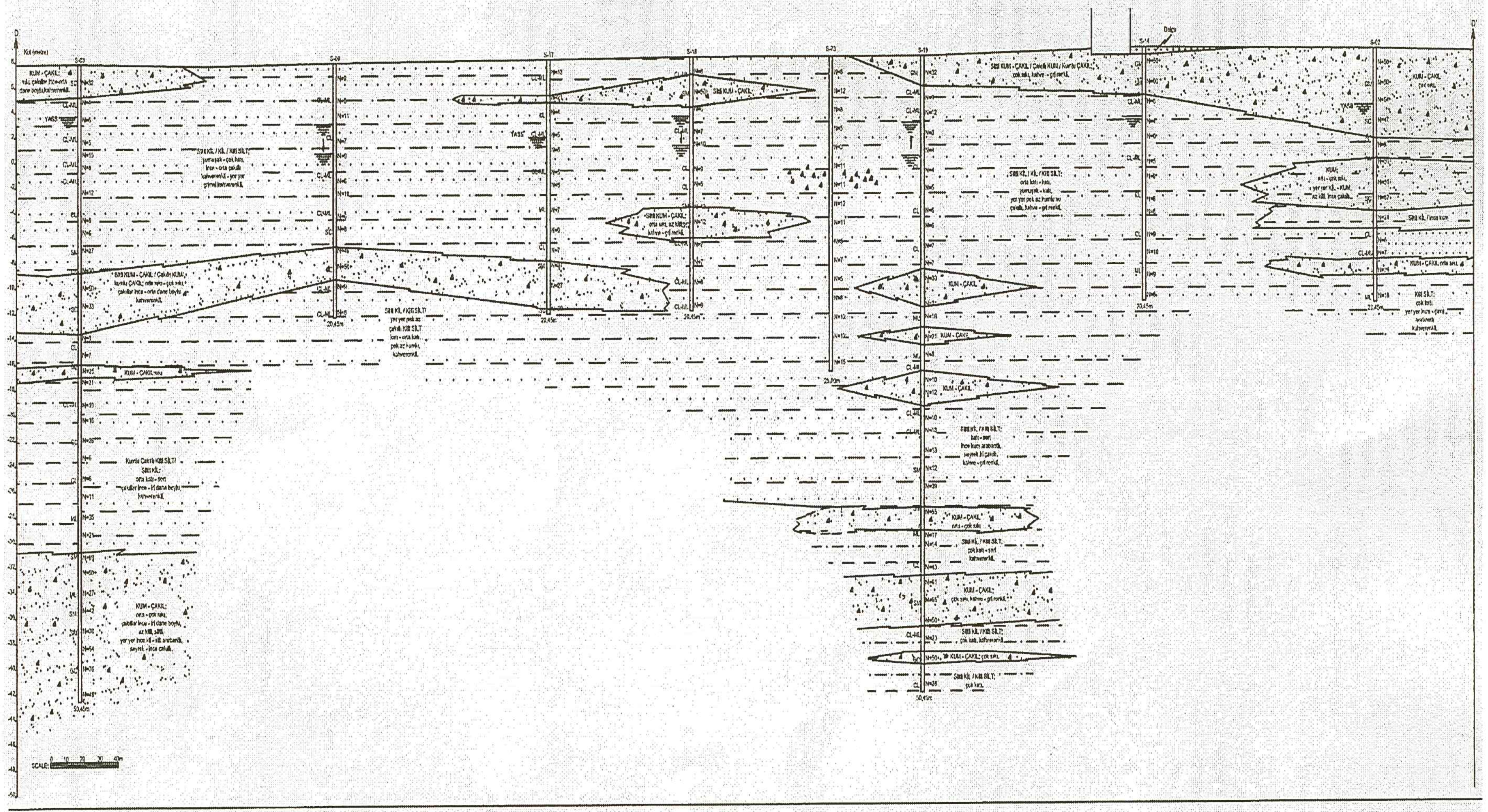


Figure A.6 : D-D soil section.

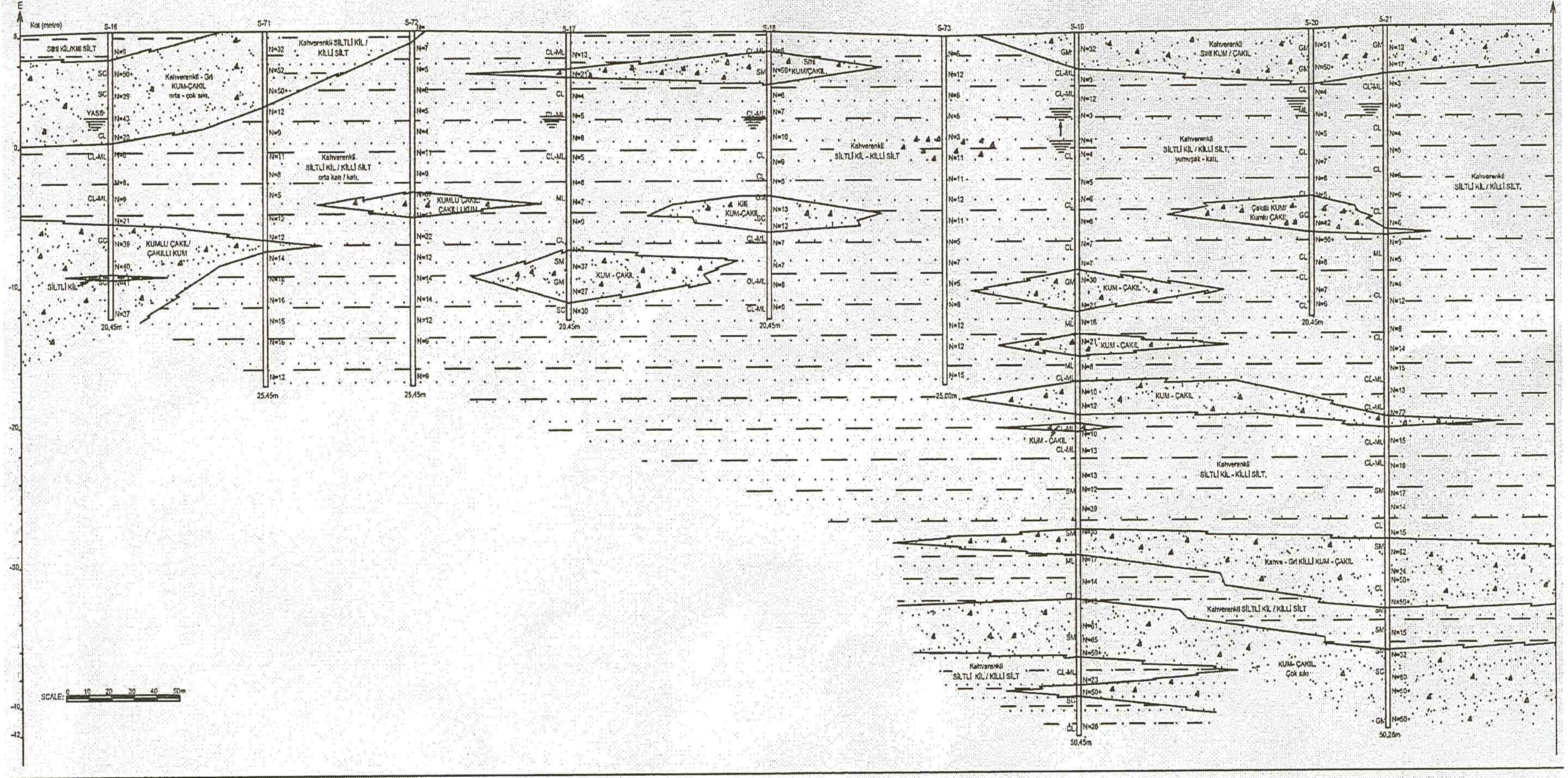


Figure A.7 : E-E soil section.

```
Project Information
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Project    : Atakas Soquk Haddehane
Project ID :
Engineer   :
Date       : 04 Feb 2008


All file   :
Soil file  :
Load file  :


Settings
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Ground elevation           :       0, (m)
G.W.T. initial condition   :      -5, (m)
G.W.T. final condition     :      -5, (m)
Water density              :    1000, (kg/m^3)
Gravity                    :     981, (m/s^2)
```

**Figure A.8:** T3 foundation (4.50m x 6.50m) settlement analysis (continued)

Soil Data

| Layer | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| Type | siltli kil | kum çakil | siltli kil | kum cakil | |
| Thick. (m) | 16,00 | 5,00 | 17,00 | 10,00 | 0,00 |
| Depth (m) | 16,00 | 21,00 | 38,00 | 48,00 | 48,00 |
| Density (kg/m^3) | 2000,00 | 2000,00 | 2000,00 | 2000,00 | 0,00 |
| Exp. j | 0,00 | 0,50 | 0,00 | 0,50 | 0,00 |
| m(top) | 4,00 | 150,00 | 15,00 | 300,00 | 0,00 |
| m(bot) | 4,00 | 150,00 | 15,00 | 300,00 | 0,00 |
| mr(top) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| mr(bot) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| dp(top) (kPa) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| dp(bot) (kPa) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| OCR(top) | 1,00 | 1,00 | 1,00 | 1,00 | 1,00 |
| OCR(bot) | 1,00 | 1,00 | 1,00 | 1,00 | 1,00 |
| Step (m) | 1,00 | 1,00 | 1,00 | 1,00 | 1,00 |

Soil Data

| Layer | 6 | 7 | 8 | 9 | 10 |
|---|---|---|---|---|---|
| Type | | | | | |
| Thick. (m) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| Depth (m) | 48,00 | 48,00 | 48,00 | 48,00 | 48,00 |
| Density (kg/m^3) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| Exp. j | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| m(top) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| m(bot) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| mr(top) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| mr(bot) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| dp(top) (kPa) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| dp(bot) (kPa) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| OCR(top) | 1,00 | 1,00 | 1,00 | 1,00 | 1,00 |
| OCR(bot) | 1,00 | 1,00 | 1,00 | 1,00 | 1,00 |
| Step (m) | 1,00 | 1,00 | 1,00 | 1,00 | 1,00 |

**Figure A.9::** T3 foundation settlement analysis (continued).

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Loading Data...

| | | | | |
|---|---|---|---|---|
| Load 1<br>Load type : Rectangle | X<br>(m) | Y<br>(m) | Load<br>(kN) | Stress<br>(kPa) |
| Appl.:Fin<br>Stress Distr.: Integration<br>Precision(m)= 1,00<br>Depth (m): 0,00<br>Elev. (m): 0,00 | 0,00 | 0,00 | 5850,00 | 200,00 |
| | Length (m): 6,50 | Width (m): 4,50 | | |

**Figure A.10:** T3 foundation settlement analysis (continued).

Differential Settlement Analysis - Boussinesq

| | (0,,0,) | | (3,25,2,25) | | |
|---|---|---|---|---|---|
| Depth (m) | Incr. S (mm) | Total S (mm) | Incr. S (mm) | Total S (mm) | Diff. S (mm) |
| Layer 1 siltli kil | 2000 kg/m^3 | j=0, | | | |
| 0,00 | 78,60 | 137,77 | 70,54 | 106,26 | 31,51 |
| 1,00 | 24,60 | 59,17 | 15,76 | 35,72 | 23,45 |
| 2,00 | 12,73 | 34,56 | 6,48 | 19,96 | 14,60 |
| 3,00 | 7,29 | 21,83 | 3,80 | 13,47 | 8,36 |
| 4,00 | 4,44 | 14,54 | 2,50 | 9,68 | 4,86 |
| 5,00 | 2,85 | 10,10 | 1,74 | 7,18 | 2,91 |
| 6,00 | 1,91 | 7,25 | 1,26 | 5,44 | 1,80 |
| 7,00 | 1,33 | 5,34 | 0,93 | 4,19 | 1,15 |
| 8,00 | 0,96 | 4,01 | 0,71 | 3,25 | 0,75 |
| 9,00 | 0,71 | 3,04 | 0,55 | 2,54 | 0,50 |
| 10,00 | 0,54 | 2,33 | 0,43 | 1,99 | 0,34 |
| 11,00 | 0,42 | 1,79 | 0,35 | 1,56 | 0,23 |
| 12,00 | 0,33 | 1,37 | 0,28 | 1,21 | 0,16 |
| 13,00 | 0,27 | 1,04 | 0,23 | 0,93 | 0,11 |
| 14,00 | 0,22 | 0,77 | 0,19 | 0,70 | 0,07 |
| 15,00 | 0,18 | 0,55 | 0,16 | 0,51 | 0,04 |
| 16,00 | 0,00 | 0,37 | 0,00 | 0,35 | 0,02 |
| Layer 2 kum çakil | 2000 kg/m^3 | j=0,5 | | | |
| 16,00 | 0,05 | 0,37 | 0,05 | 0,35 | 0,02 |
| 17,00 | 0,04 | 0,32 | 0,04 | 0,30 | 0,02 |
| 18,00 | 0,04 | 0,27 | 0,04 | 0,26 | 0,01 |
| 19,00 | 0,03 | 0,24 | 0,03 | 0,22 | 0,01 |
| 20,00 | 0,03 | 0,20 | 0,03 | 0,19 | 0,01 |
| 21,00 | 0,00 | 0,17 | 0,00 | 0,16 | 0,01 |
| Layer 3 siltli kil | 2000 kg/m^3 | j=0, | | | |
| 21,00 | 0,02 | 0,17 | 0,02 | 0,16 | 0,01 |
| 22,00 | 0,02 | 0,15 | 0,02 | 0,15 | 0,01 |
| 23,00 | 0,01 | 0,14 | 0,01 | 0,13 | 0,00 |
| 24,00 | 0,01 | 0,12 | 0,01 | 0,12 | 0,00 |
| 25,00 | 0,01 | 0,11 | 0,01 | 0,11 | 0,00 |
| 26,00 | 0,01 | 0,10 | 0,01 | 0,10 | 0,00 |
| 27,00 | 0,01 | 0,09 | 0,01 | 0,09 | 0,00 |
| 28,00 | 0,01 | 0,08 | 0,01 | 0,08 | 0,00 |
| 29,00 | 0,01 | 0,07 | 0,01 | 0,07 | 0,00 |
| 30,00 | 0,01 | 0,06 | 0,01 | 0,06 | 0,00 |
| 31,00 | 0,01 | 0,06 | 0,01 | 0,06 | 0,00 |
| 32,00 | 0,01 | 0,05 | 0,01 | 0,05 | 0,00 |
| 33,00 | 0,00 | 0,05 | 0,00 | 0,05 | 0,00 |
| 34,00 | 0,00 | 0,04 | 0,00 | 0,04 | 0,00 |
| 35,00 | 0,00 | 0,04 | 0,00 | 0,04 | 0,00 |
| 36,00 | 0,00 | 0,03 | 0,00 | 0,03 | 0,00 |
| 37,00 | 0,00 | 0,03 | 0,00 | 0,03 | 0,00 |
| 38,00 | 0,00 | 0,02 | 0,00 | 0,02 | 0,00 |
| Layer 4 kum cakil | 2000 kg/m^3 | j=0,5 | | | |
| 38,00 | 0,00 | 0,02 | 0,00 | 0,02 | 0,00 |
| 39,00 | 0,00 | 0,02 | 0,00 | 0,02 | 0,00 |
| 40,00 | 0,00 | 0,02 | 0,00 | 0,02 | 0,00 |
| 41,00 | 0,00 | 0,02 | 0,00 | 0,02 | 0,00 |

**Figure A.11:** T3 foundation settlement analysis (continued).

Differential Settlement Analysis - Boussinesq

| | (0,,0,) | | (3,25,2,25) | | |
|---|---|---|---|---|---|
| Depth | Incr. S | Total S | Incr. S | Total S | Diff. S |
| (m) | (mm) | (mm) | (mm) | (mm) | (mm) |
| 42,00 | 0,00 | 0,01 | 0,00 | 0,01 | 0,00 |
| 43,00 | 0,00 | 0,01 | 0,00 | 0,01 | 0,00 |
| 44,00 | 0,00 | 0,01 | 0,00 | 0,01 | 0,00 |
| 45,00 | 0,00 | 0,01 | 0,00 | 0,01 | 0,00 |
| 46,00 | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| 47,00 | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| 48,00 | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |

------------------------End of data----------------------

**Figure A.12:** T3 foundation settlement analysis.

### Jet Grout Bearing Capacity Analyses

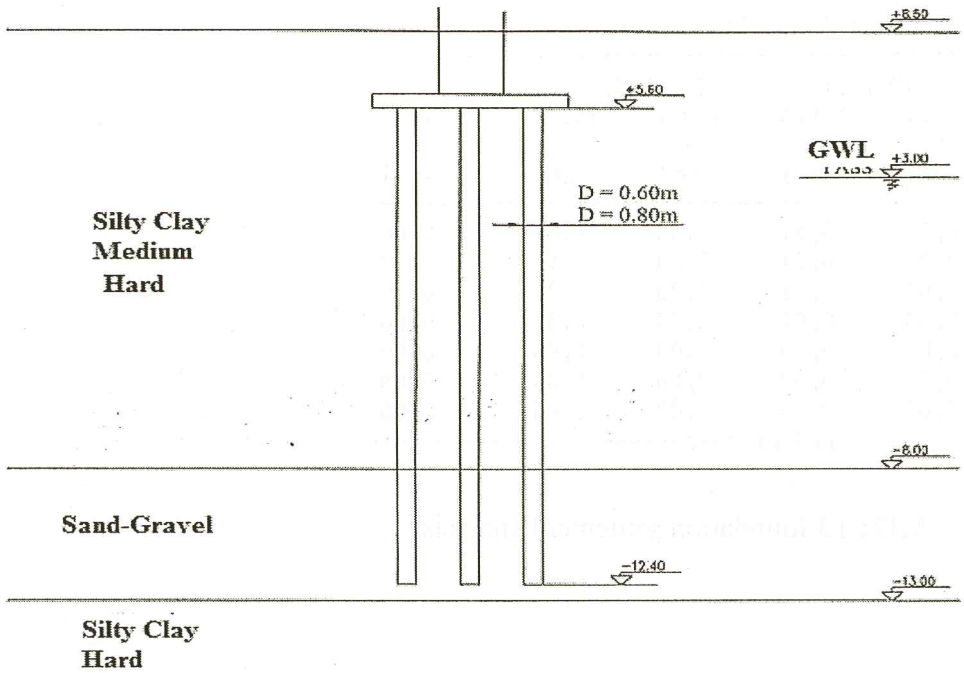


**Figure A.13:** Jet grout design (at +5.60m elevation).

*Kazık Tipi* {Çakma[1] , Delme[2]}
*Kazık Cinsi* {Beton [1] , Çelik [2]}

**Cinsi:** 1
K= 0,70
**Tipi:** 2
**Çapı (m)**= 0,80
**As(m²/m)**= 2,51
**Ab(m²)**= 0,50

$$Q_{all} = \frac{Q_s}{FS}$$

| Qs in FS= | 2,5 |
|---|---|

**JET GROUT TAŞIMA KAPASİTESİ; Temel Taban Kotu +5.60m**

| Derinlik | Su [1/0] | Tabaka | Cinsi | H [m] | γ [kN/m3] | cu [kPa] | ϕ | γ*H | σv [kN/m2] | α | Nq | Qs[kN] | QS[kN] | Qall[kN] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | siltli kil | 1,00 | 20 | 30 | | 20,00 | 10,00 | 0,96 | | 72 | 72 | 29 |
| 2 | | 1,00 | siltli kil | 1,00 | 20 | 30 | | 20,00 | 30,00 | 0,96 | | 72 | 145 | 58 |
| 3 | 1 | 2,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 45,00 | 0,96 | | 72 | 217 | 87 |
| 4 | 1 | 3,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 55,00 | 0,96 | | 72 | 290 | 116 |
| 5 | 1 | 4,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 65,00 | 0,96 | | 72 | 362 | 145 |
| 6 | 1 | 5,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 75,00 | 0,96 | | 72 | 434 | 174 |
| 10 | 1 | 6,00 | siltli kil | 4,00 | 10 | 30 | | 40,00 | 100,00 | 0,96 | | 290 | 724 | 290 |
| 12 | 1 | 10,00 | siltli kil | 2,00 | 10 | 30 | | 20,00 | 130,00 | 0,96 | | 145 | 869 | 347 |
| 14 | 1 | 12,00 | siltli kil | 2,00 | 10 | 30 | | 20,00 | 150,00 | 0,96 | | 145 | 1013 | 405 |
| 16 | 1 | 14,00 | kum çakıl | 2,00 | 10 | | 35 | 20,00 | 170,00 | | 8 | 295 | 1308 | 523 |
| 18 | 1 | 16,00 | kum çakıl | 2,00 | 10 | | 35 | 20,00 | 170,00 | | 8 | 295 | 1603 | 641 |
| 19 | 1 | 18,00 | kum çakıl | 1,00 | 10 | | 35 | 10,00 | 170,00 | | 8 | 147 | 1751 | 700 |
| 22 | 1 | 19,00 | siltli kil | 3,00 | 10 | 55 | | 30,00 | 170,00 | 0,75 | | 311 | 2062 | 825 |
| 24 | 1 | 22,00 | siltli kil | 2,00 | 10 | 55 | | 20,00 | 170,00 | 0,75 | | 207 | 2269 | 908 |
| 26 | 1 | 24,00 | siltli kil | 2,00 | 10 | 55 | | 20,00 | 170,00 | 0,75 | | 104 | 2373 | 949 |

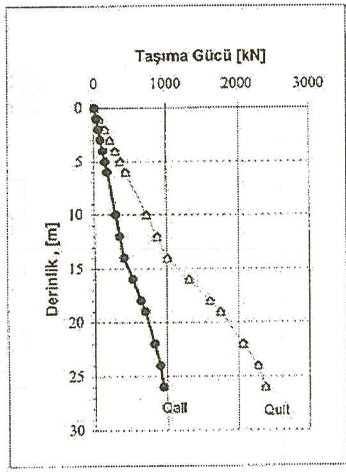


**Figure A.14:** Jet grout bearing capacity (at +5.60m elevation).

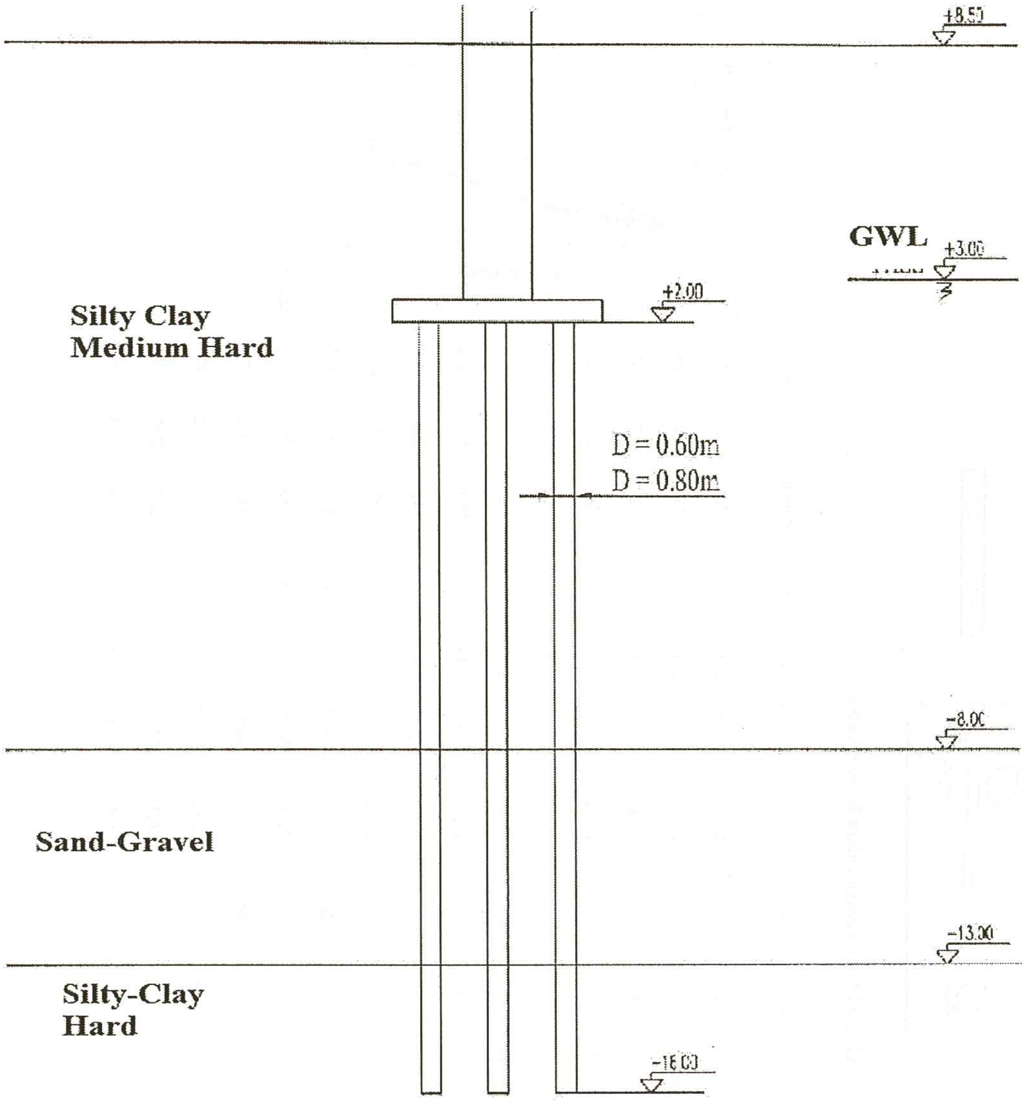


**Figure A.15:** Jet grout design (at +2.00m elevation).

*Kazık Tipi* {Çakma[1] , Delme[2]}
*Kazık Cinsi* {Beton [1] , Çelik [2]}

| |
|---|
| **Cinsi:** 1 |
| **K=** 0,70 |
| **Tipi:** 2 |
| **Çapı (m)=** 0,80 |
| **As(m²/m)=** 2,51 |
| **Ab(m²)=** 0,50 |

$$Q_{all} = \frac{Q_s}{FS}$$

| Qs in FS= | 2,5 |
|---|---|

**JET GROUT TAŞIMA KAPASİTESİ, Temel Taban Kotu +2.00m**

| Derinlik | Su [1/0] | Tabaka | Cinsi | H [m] | γ [kN/m3] | cu [kPa] | ϕ | γ•H | σv [kN/m2] | α | Nq | Qs[kN] | QS[kN] | Qall[kN] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1 | 1,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 5,00 | 0,96 | | 72 | 72 | 29 |
| 2 | 1 | 2,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 15,00 | 0,96 | | 72 | 145 | 58 |
| 3 | 1 | 3,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 25,00 | 0,96 | | 72 | 217 | 87 |
| 4 | 1 | 4,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 35,00 | 0,96 | | 72 | 290 | 116 |
| 5 | 1 | 5,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 45,00 | 0,96 | | 72 | 362 | 145 |
| 6 | 1 | 6,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 55,00 | 0,96 | | 72 | 434 | 174 |
| 7 | 1 | 7,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 65,00 | 0,96 | | 72 | 507 | 203 |
| 8 | 1 | 8,00 | siltli kil | 1,00 | 10 | 30 | | 10,00 | 75,00 | 0,96 | | 72 | 579 | 232 |
| 10 | 1 | 10,00 | siltli kil | 2,00 | 10 | 30 | | 20,00 | 90,00 | 0,96 | | 145 | 724 | 290 |
| 12 | 1 | 12,00 | kum çakıl | 2,00 | 10 | | 35 | 20,00 | 110,00 | | 8 | 191 | 915 | 366 |
| 13 | 1 | 13,00 | kum çakıl | 1,00 | 10 | | 35 | 10,00 | 125,00 | | 8 | 108 | 1023 | 409 |
| 15 | 1 | 15,00 | kum çakıl | 2,00 | 10 | | 35 | 20,00 | 140,00 | | 8 | 243 | 1266 | 506 |
| 17 | 1 | 17,00 | siltli kil | 2,00 | 10 | 55 | | 20,00 | 140,00 | 0,75 | | 207 | 1473 | 589 |
| 19 | 1 | 19,00 | siltli kil | 2,00 | 10 | 55 | | 20,00 | 140,00 | 0,75 | | 207 | 1681 | 672 |
| 21 | 1 | 21,00 | siltli kil | 2,00 | 10 | 55 | | 20,00 | 140,00 | 0,75 | | 104 | 1784 | 714 |

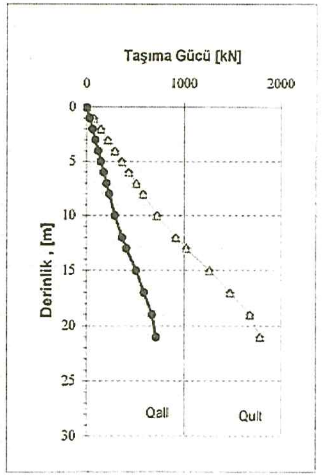


**Figure A.16:** Jet grout bearing capacity (at +2.00m elevation).

APPENDIX B

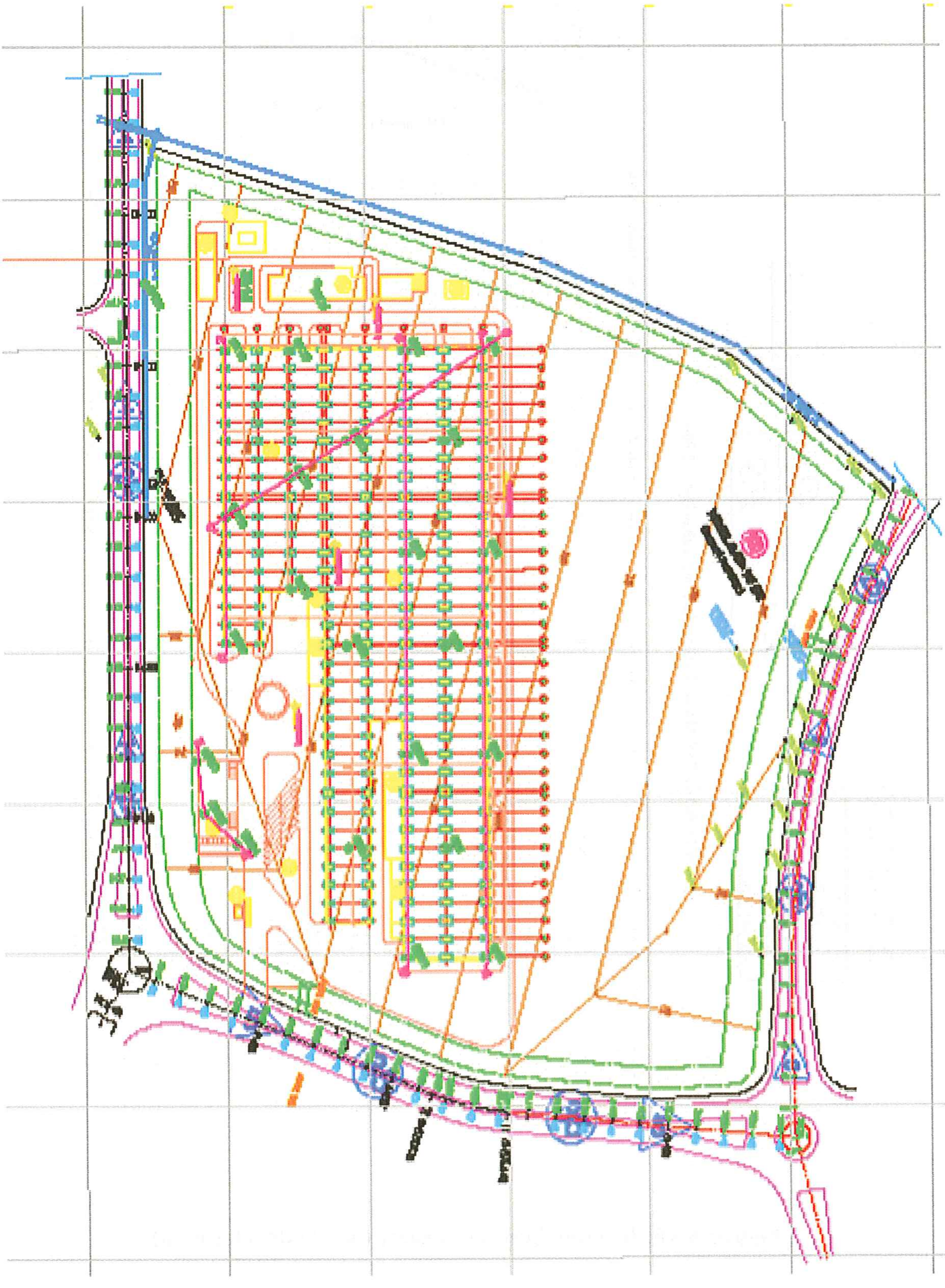

**Figure B.1:** Dynamic compaction layout plan.

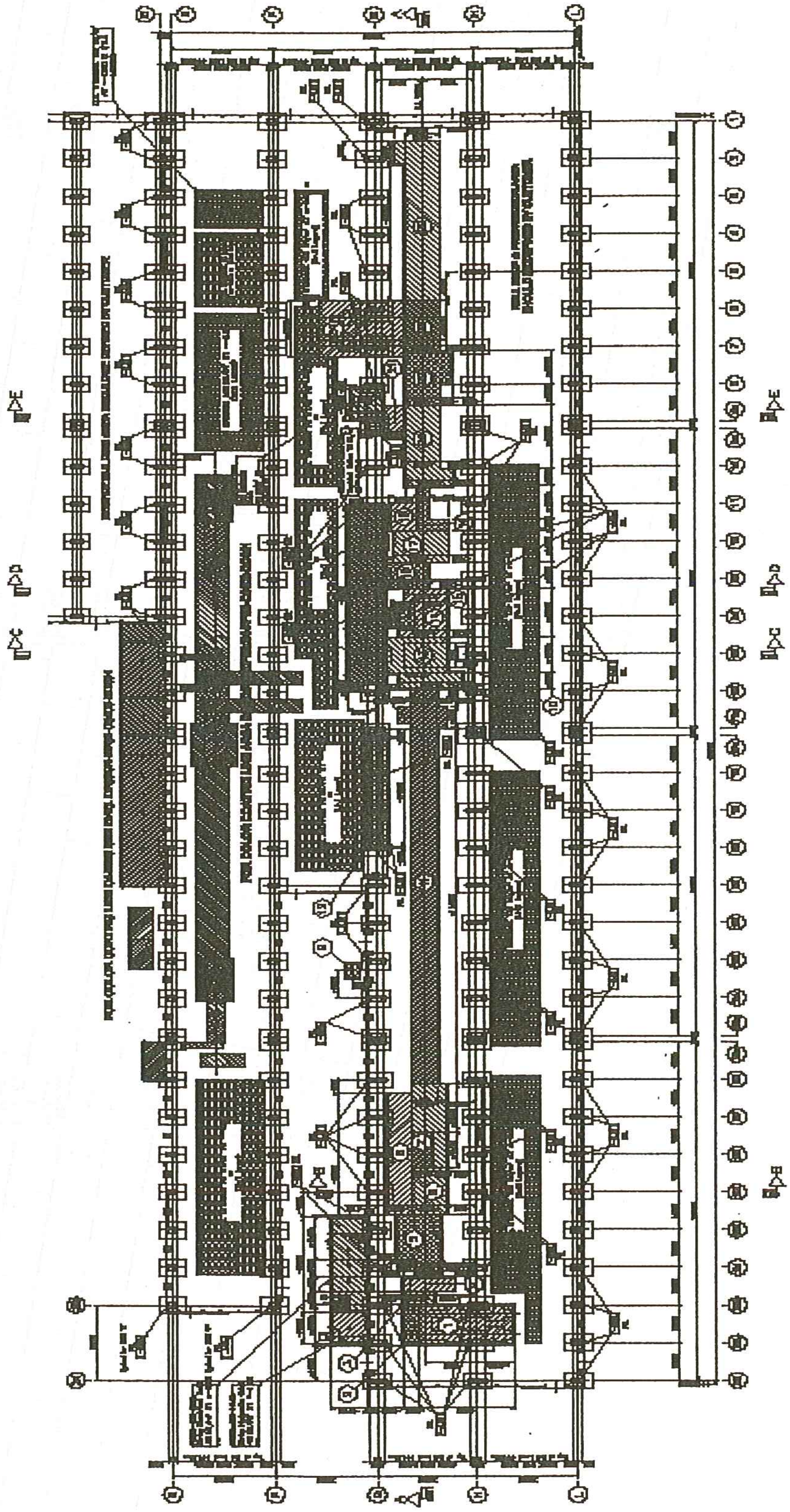

**Figure B.2:** Foundation system.

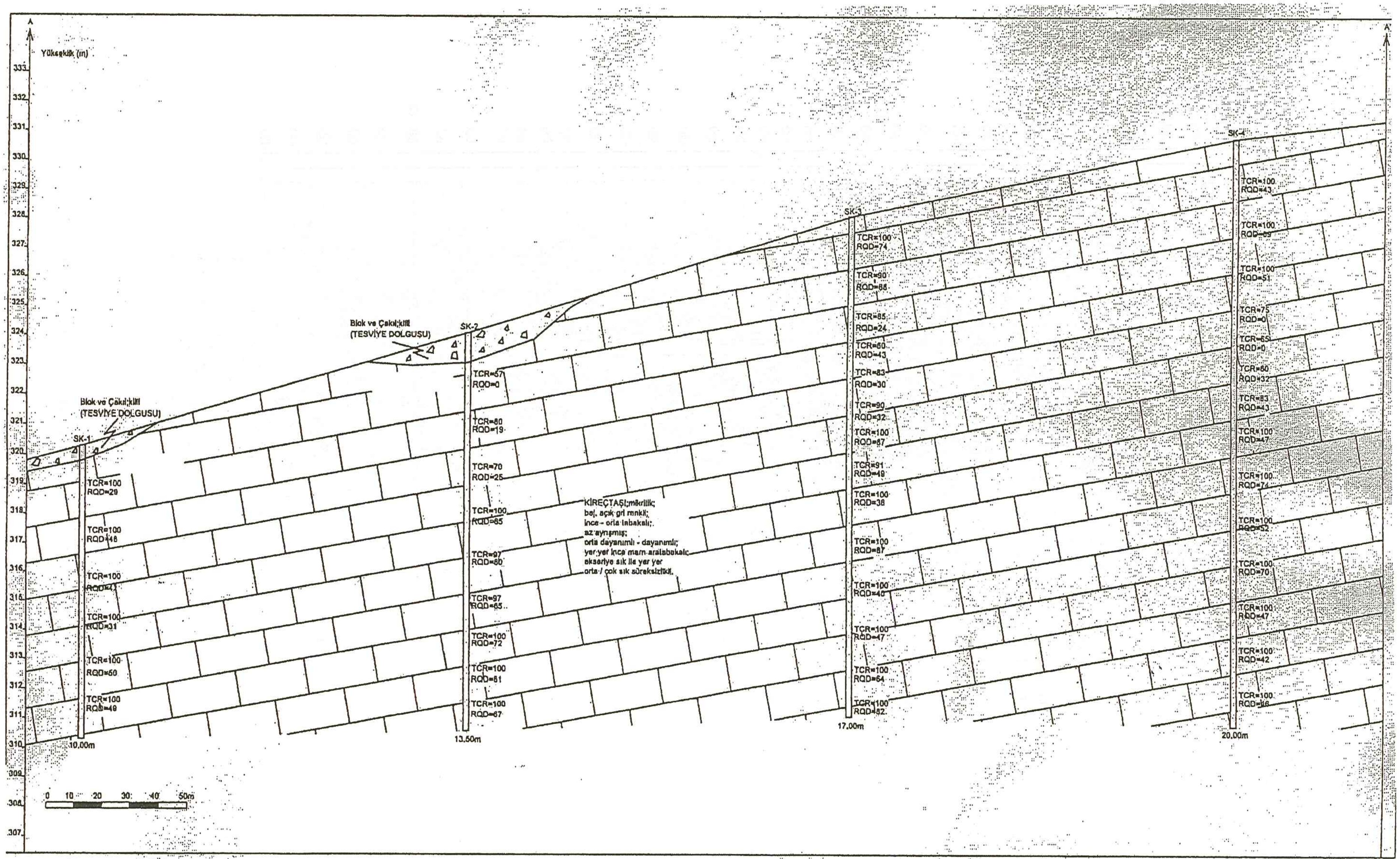


**Figure B.3:** A-A soil section.

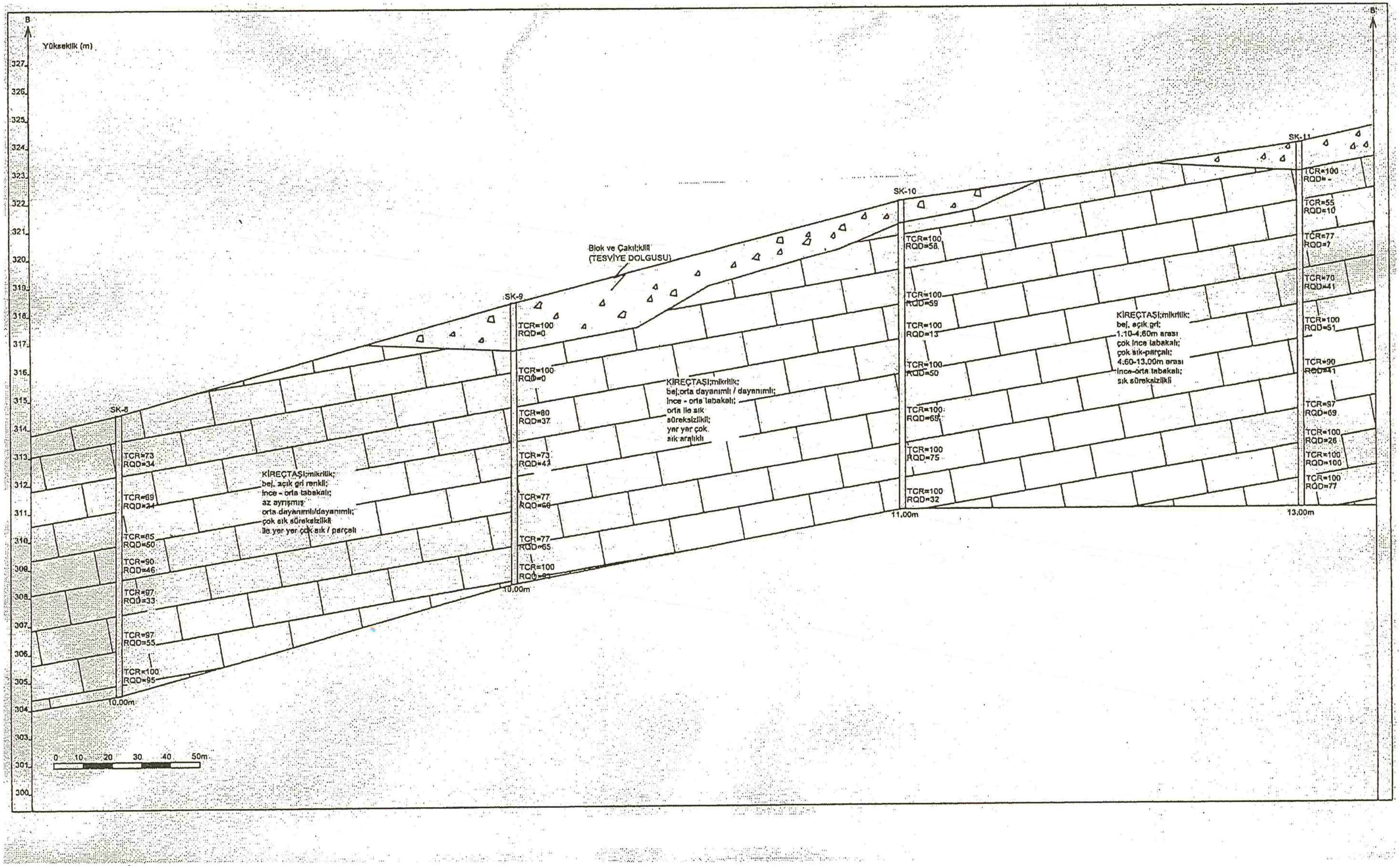


Figure B.4: B-B soil section.

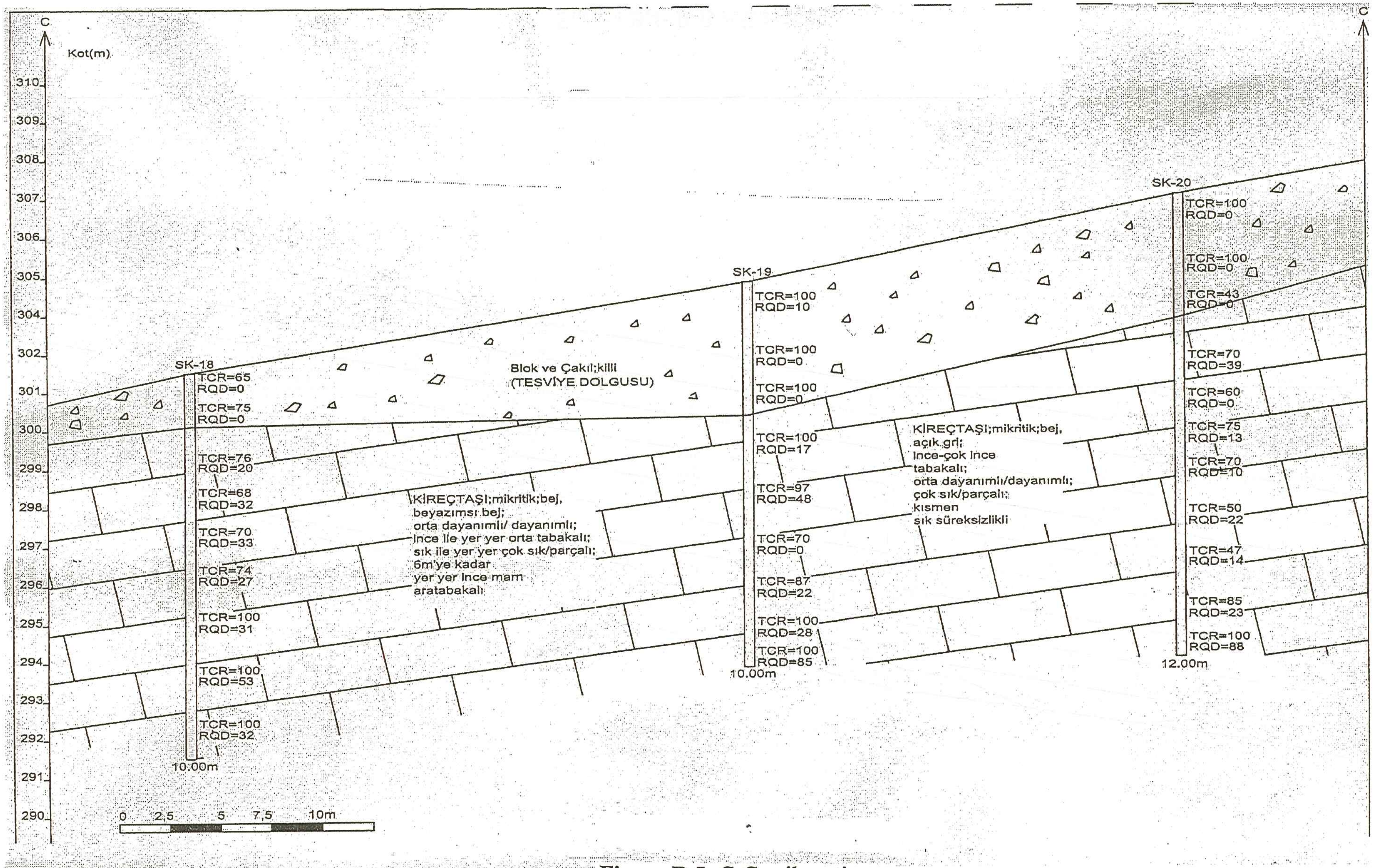


**Figure B.5:** C-C soil section.

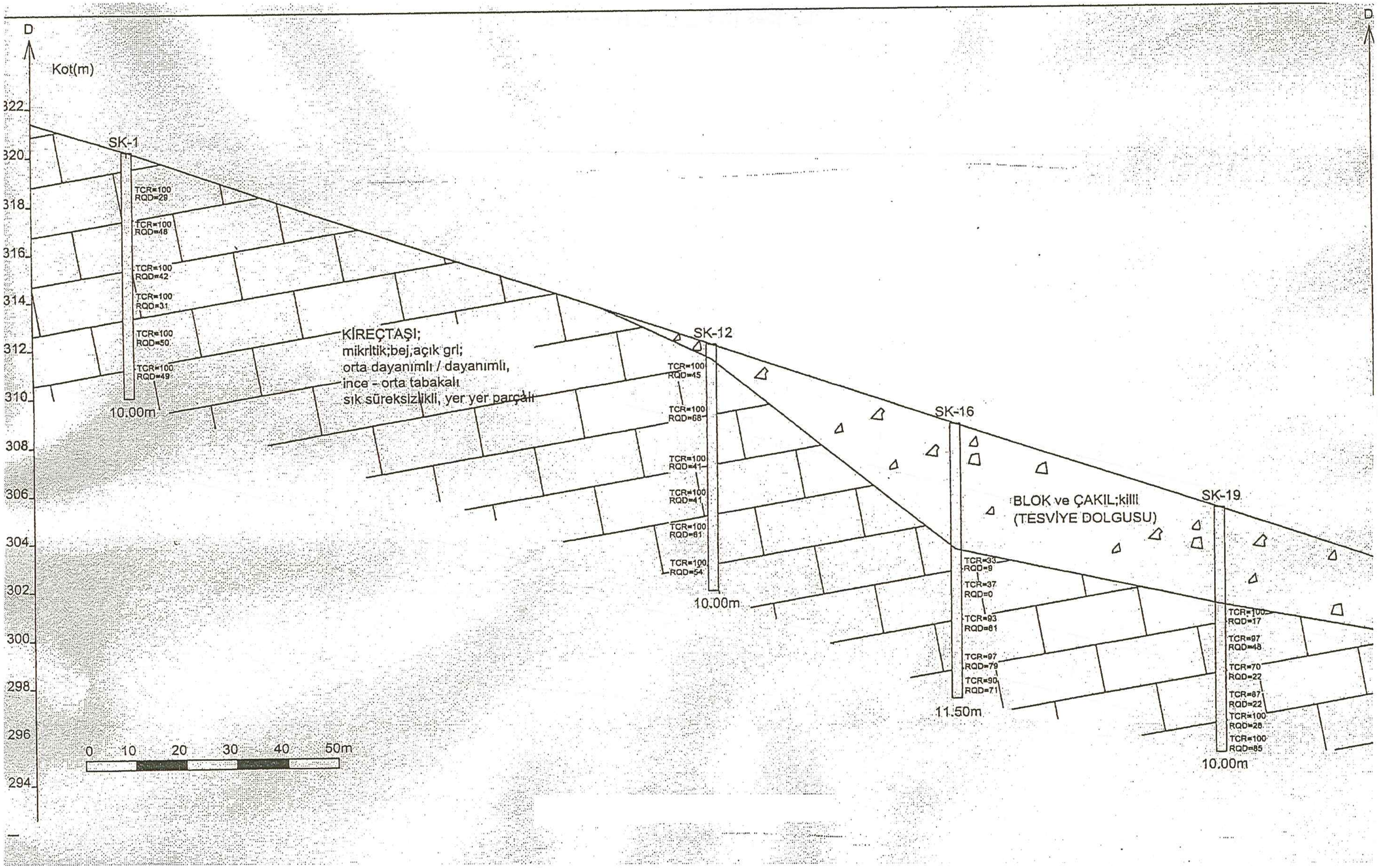


**Figure B.6:** D-D soil section.

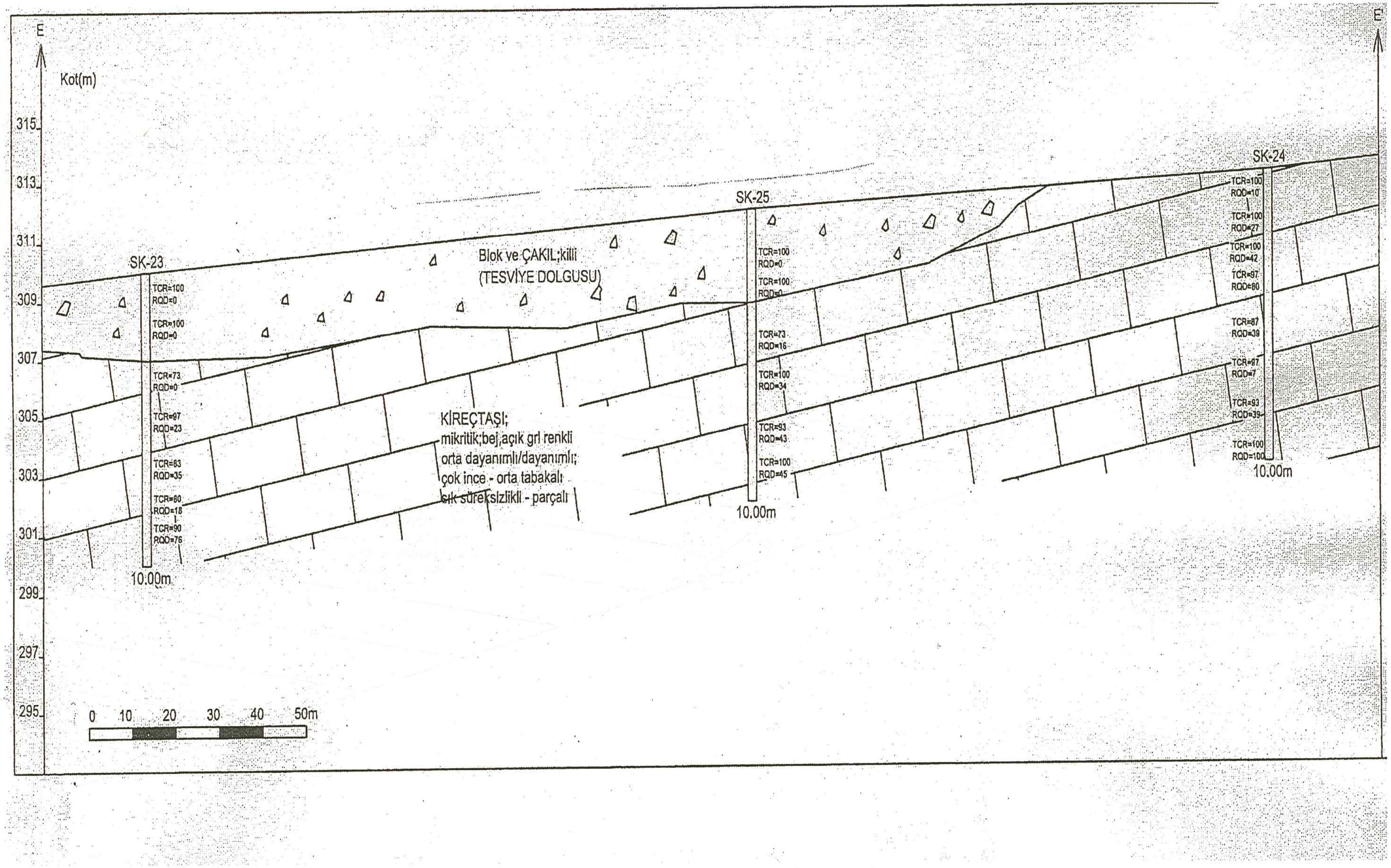


**Figure B.7:** E-E soil section.

## CURRICULUM VITAE

| | | |
|---|---|---|
| **Candidate's full name** | : | Gökçe Ceylan |
| **Place and date of birth** | : | Ürgüp – 11.06.1988 |
| **Permanent Address** | : | Mecidiye Mah. Turakeş İsmail Hakkı sok. Erdem apt. No:14/4 Ortaköy Beşiktaş/İstanbul Turkey |
| **Bachelor Degree** | : | Selçuk University Engineering and Architecture Faculty, Civil Engineering Department |