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İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

**SOIL IMPROVEMENT USING JET GROUT
TECHNOLOGY**

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**YÜKSEKÖĞRETİM KURULU
TEZ KURULU
TAYİN VE TERS TAYİN
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PREFACE

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LIST OF SYMBOLS

a: Ratio of the flow rate from the injected grout to the flow rate of the back spoiled material [Q / Q_{back}]

Ca: Base soil adhesion

CPT: Cone Penetration Test

δ : Base soil friction angle

F: Friction force

FS: Factor of safety

L: Withdraw speed [m/s]

P_{os}: At rest soil force

P_{osl}: Surcharge force

P_{ps}: Passive force

P_w: Hydraulic force

Q: Flow rate [m^3/s]

ρ_z : Cement density [kg/m^3]

ρ_w : Water density [kg/m^3]

$\rho_{rück}$: Density of the back spoiled material [kg/m^3]

ρ_{saule} : Column density [kg/m^3]

ρ_{boden} : Ground density [kg/m^3]

SPT: Standart Penetration Test

W: Water portion of the suspension [kg/m^3]

W_B: Weight of structure footing load

W_{SC}: Weight of soilcrete

W_{SL}: Surcharge load

Z: Cement portion of the suspension [kg/m^3]

ÖZET

JET GROUTING YÖNTEMİ İLE ZEMİNLERİN İYİLEŞTİRİLMESİ

Geoteknik Mühendisliğinde zemin iyileştirme yöntemleri, gevşek veya yumuşak zeminlerin mühendislik özelliklerini geliştirmek için inşaat öncesi veya sonrasında yaygın olarak kullanılmaktadır. Son on yılda zemin iyileştirme yöntemleri üzerinde yoğun çalışmalar ve ilerlemeler olmuştur. Jet grout teknolojisi, temellerin oturmasını sınırlandırmak, kontrol etmek ve bir dereceye kadar sıvılaşma riskini azaltmak için son yıllardaki gelişimden etkilenen ve günümüzde sıkça kullanılan bir zemin iyileştirme yöntemi olarak tanımlanabilir.

Bu tez çalışmasında, jet grout teknolojisinin uygulama yöntemleri, uygulama alanları, avantaj ve dezavantajları incelenmiş ve jet grout kolonlarının imalatı sırasında yapılan kalite kontrol deneyleri konu edilmiştir. Çalışmada ayrıca, zayıf zemin koşulları yüzünden oturma yapan bir bina altındaki jet grout uygulaması incelenmiştir. Bu uygulama sırasında karşılaşılan sorunlar ve alınan önlemler ile enstrümentasyonun önemi konu edilmiştir.

ABSTRACT

SOIL IMPROVEMENT USING JET GROUTING TECHNOLOGY

Soil improvement techniques are widely used in geotechnical engineering to develop the engineering properties of loose or soft soils before or after construction. There have been extensive developments in this context during the previous decade. Jet grouting technology, which has also been effected from these developments, is one of the most common soil improvement techniques in order to control and limit excessive settlements of the foundations and in some to reduce the risk of liquefaction.

In this thesis; application methods, application range, advantages and disadvantages of jet grouting are presented as well as the quality control tests carried out during jet grout installation. The study also discusses a jet grouting application performed under a building subjected to differential settlement due to poor soil conditions. Troubles and precautions taken against these encountered problems during the jet grouting application were also subjected to this thesis as well as the importance of monitoring for the success of such kind of an application.

1 SOIL IMPROVEMENT IN GENERAL

1.1 Introduction

Although soil improvement is the oldest execution method in civil engineering, it is still one of the most intriguing and growing techniques.

More than 3000 years ago, for the construction of the Babylonian Temple, soil improvement was already in use. In the past also Chinese have used wood to improve soil. Even elephants have been used in some countries to compact soil.

For the present, our attention is particularly drawn to soil improvement by the use of explosives, impact energy, thermal treatment of the soil or increasingly ingenious grouting materials and grouting systems.

Throughout the past years it has become conventional to remove and replace the soil layers or to bypass them with expensive deep foundations, for sites underlain by soft or loose soil. But with the developments in soil improvement techniques this has changed and it has become much more economical to mitigate the undesirable situation with soil improvement techniques. Also, the reliability of these techniques, which is indicated with evident test results showing an increasing safety factor, makes soil improvement the first choice.

Especially during the last decade, soil improvement has come of age and reached a new level of acceptance in the geotechnical community. Now a day, the soil improvement is the most routinely used method in foundation engineering, where poor, unstable soils are encountered.

The growing market of the vibro technologies connected with soil improvement allows us today to increase the soil properties of large areas and deep layers. These results with lower liquefaction risk in earthquake zones, which provides a safer life.

We can distinguish the soil improvement methods under the following three subtitles:

- Temporary soil improvement techniques, which are limited to the period of the construction.
- Permanent soil improvement without the addition of any material
- Permanent soil improvement by adding materials

Soil improvement methods can also be classified for granular and cohesive soils or as shallow and deep soil improvement, which has also an important meaning from an engineering point of view.

“There are several advantages, which occur through ground improvement. Detrimental settlements can be reduced or prevented. Soil strength increases and bearing capacity of pavement subgrades can be improved. Undesirable volume changes, for example, caused by frost action, swelling, and shrinkage may be controlled.” [1]

“Ground improvement using deep densification, replacement, or in situ hardening of potentially liquefiable soils has been used at many sites for reducing settlement and lateral spreading caused by earthquakes. There is clear evidence that ground improvement can provide significant protection against earthquake induced liquefaction and ground failure and that settlements and lateral displacements of treated ground can be reduced to tolerable levels.”[2]

Table 1.1 shows the most preferred soil improvement methods.

Table 1.1 Soil Improvement Methods in Use

Dynamic Compaction
Vibro Compaction
Grouting
Jet Grouting
Preloading
Vertical Drains
Blasting
Micropiles
Stone Columns
Vibro Concrete Columns
Geotextiles
Electro-Osmosis
Freezing
Heating
Dewatering
Deep Mixing

As it can be seen from Table 1.1 above, civil engineering enables to choose the best soil improvement method, which will be most efficient for the encountered soil type.

Dynamic compaction is becoming one of the more routinely used ground improvement techniques because of its relatively low cost, especially when improving large areas. The dynamic compaction market is becoming a mature market; with steady growth expected because of the number of old fill sites that were previously bypassed when being considered for development.

The vibro – compaction market, in conjunction with stone columns, has also achieved universal professional acceptance, and has seen a steady growth because of more concerns about liquefaction. The vibro compaction industry now uses more powerful vibrators, that allow for wider probe spacing, and more economical improvement.

The grouting market is also in a mature marketplace, with many different applications available other than the more routine fixes for soft ground tunneling and

settlement related problems, both to remedied failures and in advance of new construction. [3]

1.2 Definitions of Soil Improvement Techniques

1.2.1 *Dynamic Compaction*

Due to its success in obtaining engineering property improvements, as well as the economics of the process, dynamic compaction has evolved into one of the more commonly used ground improvement methods.

Dynamic compaction is a procedure of compacting marginal materials in – place without the need for removal. It is used to improve the engineering properties of in – place soils at depth, both above and below groundwater level. Soil strength is increased, and compressibility and settlements are decreased as a result of the densification. The primary goal of dynamic compaction is to change a heterogeneous soil into one that has more uniform, stronger engineering properties. Basically, dynamic compaction consists of providing high-energy impacts at the ground surface repeatedly dropping steel or concrete tampers, 5 to 32 tones in weight from heights ranging from 12 to 112m. The energy is generally applied in phases on a grid pattern over the entire area using either single or multiple passes. Following each pass, the craters are either leveled with a dozer or filled with granular fill material before the next pass of energy is applied.

Dynamic compaction works best on deposits where the degree of saturation is low, the permeability of the soil mass is high, and drainage is good. Deposits considered most appropriate for dynamic compaction include pervious granular soils. If these deposits are situated above the water table, densification is immediate as the soil particles are forced into a denser state of packing. If these deposits are situated below the water table, the permeability is sufficiently high, excess pore water pressures generated by the impact of the tamper dissipate almost immediately, and densification is nearly immediate. Pervious granular deposits include not only natural sands and gravels but also fill deposits consisting of building rubble, some mine spoil, some industrial waste fill such as slag, and decomposed refuse deposits.

Deposits in which dynamic compaction is not appropriate would clayey soils, either natural or fill, that are saturated. In saturated deposits, improvements cannot occur

unless the water content of the deposit is lowered. Generally, clayey soils have permeabilities of less than 10^{-8} to 10^{-9} m/s, so dissipation of excess pore water pressures generated during dynamic compaction cannot occur, except perhaps over a lengthy period of time. This makes dynamic compaction impractical for these deposits. Furthermore, the degree of improvement is generally minor. [3, 4, 5]

1.2.2 Vibro Compaction

Vibro compaction is a method of deep densification of in situ granular soils by means of rearranging loose cohesionless sand grains into a denser array by insertion of vibratory probe. Spacing of the probe compaction points depends upon soil type, density requirements, and probe/vibrator characteristics. Typical spacings range from 1.5 to 4m.

“The effect of vibro compaction is a direct improvement of the existing soil by densification. It depends firstly on the compactibility of the soil and secondly on the efficiency of the vibrator being used. Due to the complexity of the parameters involved, it is not possible to establish accurate design criteria in advance. Without preceding large-scale compaction trials, a design depends mainly on the experience of the contractor who may be influenced by competitive considerations. The drawback of the technique in advancing an optimum design is compensated by the fact that in – situ quality control tests exist like soundings which are simple to perform and unequivocal in result. In particular, Static Cone Penetration Tests have proved to be economical and reliable.” [6]

The term vibro replacement or stone column uses the same equipment as in vibro compaction. The stone is forced laterally by the vibrofloat into the formation, creating stone columns.

Even though it has been around for a long time, there is still general confusion as to what the term vibro compaction means. There are several systems that have been identified as vibro compaction. They include vibroflotation, terra probe and vibrating beam.

Vibroflotation refers to compaction by means of a vibroflot, which densifies soils by horizontal motion from the vibrator inserted into the ground. The compaction work under this technique is conducted in accordance with the following procedure:

1. With the assistance of a mixture of water and compressed air injected from the end of the flot, penetration is initiated using horizontal vibration.
2. Laboratory tests: grain size analysis, water content test, and particle density test.
3. Working conditions: required depth of soil improvement, working efficiency of vibro flot shape.

Other vibratory techniques utilize a top pile-driving vibrator imparting vibration in a vertical mode to the ground, which normally requires closer spacing and is less efficient in finer granular formations.

The vibro compaction technique is most suitable for medium to coarse-grained sand with silt content of less than 10%, passing sieve size of 0.074mm, and clay content of less than 2%, passing sieve size of 0.005mm. Cohesive soils consisting of silt and clay material will not respond to vibratory compaction. [9, 7, 8]

1.2.3 Preloading

Preloading has been used for many years without change in the method or application to improve soil properties. Preloading or precompression is the process of placing additional vertical stress on a compressible soil to remove pore water over time. The pore water dissipation reduces the total volume causing settlement. This will reduce post – construction settlements due to densification, whereby the soil bearing capacity is increased. Surcharging is an economical method for ground improvement. However, the consolidation of the soils is time dependent, delaying construction projects making it a non-feasible alternative. [7]

By surcharging, the technique can accelerate the precompression and can also reduce settlements due to secondary compression. When an embankment or other area load is applied rapidly to a deposit of saturated, cohesive soils, the resulting settlement can be divided into three idealized components:

1. Initial (or "immediate") settlement occurs during application of the load as excess pore pressures develop in the underlying soil. If the soil has a low permeability and is relatively thick, the excess pore pressures are initially undrained. The foundation soil deforms due to the applied shear stresses with

essentially no volume change, such that vertical compression is accompanied by lateral expansion.

2. Primary consolidation settlement develops with time as drainage allows excess pore pressures to dissipate. Volume changes, and thus settlement, occur as stresses are transferred from the water (pore pressures) to the soil skeleton (effective stresses). The rate of primary consolidation is governed by the rate of water drainage out of the soil under the induced hydraulic gradients. The drainage rate depends upon the volume change and permeability characteristics of the soil as well as the location and continuity of drainage boundaries.
3. Secondary compression settlement is the continuing, long-term settlement, which occurs after the excess pore pressures are essentially dissipated and the effective stresses are practically constant. These further volume changes and increased settlements are due to drained creep, and are often characterized by a linear relationship between settlement and logarithm of time.

For purposes of analysis it is usually assumed that these three components occur as separate processes, in the order given. Experience has shown that the actual deformation behavior of soft foundation soils under embankment loadings is more complex than this simplified representation. In some cases the magnitude of one or more of these components may be insignificant. However, in most cases this simplifying assumption is reasonable and designs developed accordingly are appropriate.

By preloading design the bearing capacity of soil, the stability of the slope if exists and of course the degree of consolidation has to be considered. Excessive preloading on soils with insufficient bearing capacity will result with total collapse.

Soils that can be treated with preloading are organic silts, varved silts and clays, soft clays and dredged materials. [7, 9]

1.2.4 Vertical Drains

Vertical drains shorten the drainage path for the purpose of reducing the consolidation time. The drains increase the rate at which excess pore water pressure is dissipated during consolidation of compressible soils. Since the 1970's vertical

drains have primarily been constructed with prefabricated vertical drains. Vertical drains are used to densify soft compressible soils in conjunction with preloading.

When used in conjunction with precompression, the principal benefits of a vertical drain system (i.e., of accelerated consolidation) are:

1. To decrease the overall time required for completion of primary consolidation due to preloading,
2. To decrease the amount of surcharge required to achieve the desired amount of precompression in the given time,
3. To increase the rate of strength gain due to consolidation of soft soils when stability is of concern.

Vertical drains can also be used as pressure relief wells to reduce pore pressures due to seepage, such as below natural slopes, and to improve the effectiveness of natural drainage layers below loaded areas. [9]

1.2.5 Blast – Densification

Blasting is the process of detonating small charges within loose cohesionless soils for the purpose of densification. Blasting as a ground improvement alternative has been used in the Netherlands and USSR where empirical data is used for prediction of results.

Blast – densification increases the density of loose granular deposits, above or below the water table, to depths beyond the practical limits of dynamic compaction, vibro – compaction or other ground improvement techniques. Blast densification destroys the existing soil structure, causing rearrangement of soil particles into a more dense arrangement through the volumetric strains and shearing induced by the blast. Also, the explosive shock waves temporarily liquefy the soil, causing the soil particles to rearrange to a higher relative density as excess pore pressures dissipate.

Blast – densification has been used to treat soils to depths of up to 40m. The maximum depth to which this technique will may work is not known. As depth increases, the size of the charge necessary to destroy the soil structures and liquefy the soil increases, and effective radius of the improvement decreases because of the increased strength and stresses in soil. Practical difficulties in placing large charges

at great depths may be the limiting factor how deep the blast – densification is effective. [3]

1.2.6 Micro Piles

Micropiles were invented in Italy in the 1950's as a means to underpin sensitive historic buildings. The pile, called the “Palo Radice” or root pile, was a small diameter cast-in-place, lightly reinforced drilled pile developed by the Italian geotechnical construction company, Fondedile. The “palo radice” was generally designed to achieve 10 tons capacity.

Piles are divided into two general types: displacement piles and replacement piles. Displacement piles are members that are driven or vibrated into the ground, thereby displacing the surrounding soil laterally during installation. Replacement piles are placed or constructed within a previously drilled borehole, thus replacing the excavated ground. A micropile is a small-diameter (typically less than 300 mm), drilled and grouted replacement pile that is typically reinforced. Drilling a borehole, placing reinforcement, and grouting the hole construct a micropile. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed.

Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access- restrictive environments and in all soil types and ground conditions. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects.

Micropiles can be installed in a wide range of soils from non-cohesive, poorly graded granular soils, to cohesive plastic clays. Drilled micropiles are a viable solution to underpinning or structural support projects.

Micro Pile applications serve usually for structural foundation support, structural underpinning and soil or slope stabilizations. Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to underpin existing structures.

Micropiles must be designed to take the required compression, tension, and lateral load applied. This is accomplished by analyzing the pile's structural and geotechnical capacities. The structural capacity of the pile is governed by the grout and reinforcing steel strengths. The geotechnical capacity of the pile is developed in friction between the grout and the in situ soils. Published values are used in the design of the bond length for the pile; these values are proven with the load test.

Drilled micropiles are installed with rotary drilling techniques, using external water flush with flush joint casing and a sacrificial drill bit. The piles are drilled to their design depth, including a bond zone into load bearing soils (a criteria developed and proven from the load test). Upon completion of advancement of the drill string to the design depth, the grouting process commences. The casing is tremie grouted then pressure grouted through the drill head until the grout return is observed about the annulus between the casing and the existing ground. The result is a grouted-in-place micropile with reinforcing, able to carry loads from 10 to 200 tons. [7, 3, 10].

1.2.7 Stone Columns

Stone column construction involves the partial replacement of unsuitable subsurface soils with a compacted vertical column of stone that usually completely penetrates the weak strata. When jetting water is used the process is named vibro-replacement (or the, wet process). When used without jetting water in partially saturated soils, such as old rubble fill, the process is known as vibro-displacement (or the dry process). To date only the wet process has been used in the U.S., although both the wet and dry processes have been used in Canada and Europe. These techniques have been used since the late 1950's to construct columns of stone in marginal soils.

The stone is densified by the use of a vibrating probe originally developed in 1935 for the compaction of granular, noncohesive soils. The use of vibro stone columns for in situ reinforcement of soft cohesive soils and densification of granular soils has been accepted for over 30 years. Advances in the technique in the past ten years have included the development of more powerful vibrators, more frequent use of full-scale load tests, and development of variations on the installation technique using non – vibratory methods. The greatest advances are related to the use of vibro stone columns to reduce liquefaction.

Vibro stone column construction is accomplished by down the hole vibratory methods typically referred to as vibro replacement and vibro displacement. A vibratory probe penetrates the ground, either under its own weight or aided by water jetting. Horizontal vibrations are produced close to the base of the vibrator, induced by rotating eccentric weights mounted on a shaft driven by a motor housed within the casing. The vibrations produced by this unit are generated close to the tip of the probe and weave radially in the horizontal plane away from the unit. Stone backfill is introduced in controlled lifts, either from the surface down the annulus created by penetration of the probe or through feeder tubes directed to the tip of the probe. Repenetration of each stone backfill lift forces the stone radially into the surrounding in situ soil, forming a vibro stone column and the in situ soil form an integrated system having low compressibility and high shear strength.

In stone column construction, usually stone replaces 15 to 35 percent of the weak soil volume. Design loads on stone columns typically vary from 20 to 50 tons. The presence of the column creates a composite material of lower overall compressibility and higher shear strength than the native soil alone. Confinement, and thus stiffness of the stone, is provided by the lateral stress within the weak soil. Upon application of vertical stress at the ground surface, the stone and weak soil move downward together resulting in an important concentration of stress within the stone column. The resulting stress concentration in the stone is primarily due to the column being stiffer than the soil. An axial load applied at the top of a single stone column produces a large bulge to a depth of 2 to 3 diameters beneath the surface. This bulge, in turn, increases the lateral stress within the clay, which provides additional confinement for the stone. An equilibrium state is eventually reached resulting in reduced vertical movement when compared to the unimproved soil. Stone column groups loaded over the entire area undergo less bulging than for a single stone column.

Stone columns have been used to improve a wide range of soils from very soft clays and peats to materials less suitable for vibro compaction. In areas with seismic activity, vibratory ground improvement minimizes the potential for liquefaction.

The stone column technique of ground treatment has proven successful in;

1. Improving slope stability of both embankments and natural slopes,
2. Increasing bearing capacity,
3. Reducing total and differential settlements,
4. Reducing the liquefaction potential of sands and
5. Increasing the time rate of settlement.

Stone columns are used to support structures overlying both very soft to firm cohesive soils and also loose silty sands having greater than about 15 percent fines. At the present time, more stone column projects in the U.S. have been constructed in silty sands rather than cohesive soils; worldwide the reverse is true.

By adding crushed stone through a separate duct along side the vibroprobe creates Vibrostone columns. The stone column serves as a vertical drain, thus reduces excess pore water pressures caused by rapid loading or earthquakes, making soil liquefactionless likely.

Dry bottom feed stone columns are employed to densifying sandy soils in place by means of an electric vibrating unit, and installing granular columns of preferably crushed gravel in the same process.

Dry bottom feed stone columns are most suitable for the reduction of differential settlements, as the column installation adapts itself to changing soil conditions and acceleration of ground settlement. [3, 7, 11]

1.2.8 Vibro Concrete Columns

“The bottom feed vibro replacement system represented a major advance in ground improvement technology with its dry construction of high integrity compacted stone columns in weak cohesive soils. The bottom feed concept has been extended by the construction of vibro concrete columns (VCC) through very weak cohesive and organic soils which are unsuitable for the proper applications of conventional vibro replacement techniques.”[3]

“Vibroconcrete columns are a piling technique by means of an electrical vibrating unit. Most of the in situ soil is displaced sideways in the installation process,

therefore the vibro concrete columns is classified as a cast-in-place displacement pile.”[7]

1.2.9 Geosynthetics

Geosynthetics are factory manufactured polymeric materials which are typically produced in roll form, transported to the job site and appropriately placed and seamed together in the field. Their inherent characteristics are thinness, light weight, excellent quality control, ease of installation and general cost effectiveness in comparison to natural soil materials. They also have the critical characteristics of being very unforgiving in their installation thereby requiring constant vigilance via construction quality control and construction control accuracy. The current types of geosynthetics available along with the common used and a brief description are in the following paragraphs. [3, 12]

1.2.9.1 Geotextiles

Geotextiles form one of the two largest groups of geosynthetics. Their rise in growth during the past fifteen years has been nothing short of awesome. They are indeed textiles in the traditional sense, but consist of synthetic fibers rather than natural ones such as cotton, wool, or silk. Thus biodegradation is not a problem. These synthetic fibers are made into a flexible, porous fabric by standard weaving machinery or are matted together in a random, or nonwoven, manner. Some are also knit. The major point is that they are porous to water flow across their manufactured plane and also within their plane, but to a widely varying degree. There are at least 80 specific applications area for geotextiles that have been developed; however, the fabric always performs at least one of five discrete functions:

1. Separation
2. Reinforcement
3. Filtration
4. Drainage
5. Moisture barrier (when impregnated)

1.2.9.2 Geogrids

Geogrids represent a rapidly growing segment within the geosynthetics area. Rather than being a woven, nonwoven or knit textile (or even a textile-like) fabric, geogrids are plastics formed into a very open, grid like configuration, i.e., they have large

apertures. Geogrids are either stretched in one or two directions for improved physical properties or made on weaving machinery by unique methods. By themselves, there are at least 25 application areas, however, they function almost exclusively as reinforcement materials. [3, 12]

1.2.9.3 Geonets

Geonets, called geospacers by some, constitute another specialized segment within the geosynthetic area. They are usually formed by a continuous extrusion of parallel sets of polymeric ribs at acute angles to one another. When the ribs are opened, relatively large apertures are formed into a netlike configuration. Their design function is completely within the drainage area where they have been used to convey fluids of all types. [3, 12]

1.2.9.4 Geomembranes

Geomembranes represent the other largest group of geosynthetics and in dollar volume their sales are probably larger than that of geotextiles. Their growth has been stimulated by governmental regulations originally enacted in 1982. The materials themselves are "impervious" thin sheets of rubber or plastic material used primarily for linings and covers of liquid- or solid-storage facilities. Thus the primary function is always as a liquid or vapor barrier. The range of applications, however, is very great, and at least 30 individual applications in civil engineering have been developed. [3, 12]

1.2.9.5 Geosynthetic Clay Liners

Geosynthetic clay liners (or GCLs) are the newest subset within geosynthetic materials. They are rolls of factory fabricated thin layers of bentonite clay sandwiched between two geotextiles or bonded to a geomembrane. Structural integrity is maintained by needle punching, stitching or physical bonding. They are seeing use as a composite component beneath a geomembrane or by themselves as primary or secondary liners. [3, 12]

1.2.9.6 Geopipe

Perhaps the original geosynthetic material still available today is buried plastic pipe. This "orphan" of the Civil Engineering curriculum was included due to an awareness that plastic pipe is being used in all aspects of geotechnical, transportation and environmental engineering with little design and testing awareness. This is felt to be

due to a general lack of formalized training. The critical nature of leachate collection pipes coupled with high compressive loads makes geopipe a bona-fide member of the geosynthetics family. The function is clearly drainage. [3, 12]

1.2.9.7 Geocomposites

A geocomposite consists of a combination of geotextile and geogrid; or geogrid and geomembrane; or geotextile, geogrid, and geomembrane; or any one of these three materials with another material (e.g., deformed plastic sheets, steel cables, or steel anchors). This exciting area brings out the best creative efforts of the engineer, manufacturer, and contractor. The application areas are numerous and growing steadily. The major functions encompass the entire range of functions listed for geosynthetics discussed previously: separation, reinforcement, filtration, drainage, and liquid barrier. [3, 12]

1.2.9.8 "Geo-Others"

The general area of geosynthetics has exhibited such innovation that many systems defy categorization. For want of a better phrase, *geo-others*, describes items such as threaded soil masses, polymeric anchors, and encapsulated soil cells. As with geocomposites their primary function is product-dependent and can be any of the five major functions of geosynthetics. [3, 12]

1.2.10 Deep Mixing

“The deep mixing method is an in situ soil mixing technology that mixes existing soil with cementitious materials using mixing shafts consisting of auger cutting heads, discontinuous auger flights, and mixing paddles. The mixing equipment varies from single to eight shaft configurations depending on the purpose of the deep mixing. The soil cement produced generally has higher strength, less compressibility, and lower permeability than the native soils.”[3]

1.2.11 Electro Osmosis

When electrodes are placed across a clay mass and a direct current is applied, water in the clay pore space is transported to the cathodically charged electrode by electro-osmosis. Electro-osmotic transport of water through clay is a result of diffuse double layer cations in the clay pores being attracted to a negatively charged electrode or

cathode. As these cations move toward the cathode, they bring with them water molecules that clump around the cations as a consequence of their dipolar nature. In addition, the frictional drag of these molecules as they move through the clay pores help transport additional water to the cathode. The macroscopic effect is a reduction of water content at the anode and an increase in water content of the clay at the cathode. In particular, free water appears at the interface between the clay and the cathode surface. This excess of free water at the cathode has lubricating effects.

The original electro-osmotic experiments showed that flow of water through a clay-water system (with the applying of an electric field to the soil) is initiated by the movement of cations that are closely attracted to the surface of the clay particles. Electro osmosis is possible in clay soils since clay minerals have a net negative charge. As a result of this charge, a gradient in electrical potential forms (O) that extends into the water surrounding the particle. Cations are attracted to the surface of clay particles when the soil is hydrated to balance the negative potential.

The cations that are attracted to the surface of the particle can be separated into two regions. The first region consists of cations, which are held tightly to the particle. This region is called the Stern layer. The second layer is the diffuse layer, where a layer of attracted but mobile cations extends into the surrounding liquid. These cations are attracted to the surface by the electrical potential but contumely move away from the particle because of thermal fluctuations. The concentration of cations in the double layer diminishes as the distance from the surface of the particle increases. Ultimately, the double layer blends into free water.

Electro-osmosis occurs in clay soils when cations in the double layer are driven by the application of an electrical field, and as a result, a velocity field in the pore fluid develops. The velocity distribution change rapidly near the particle's surface, but then becomes flat at the edge of the double layer. Therefore, electro-osmotic flow appears as plug flow through the pores of soil. [7, 3]

1.2.12 Dewatering

Common reasons to lower groundwater levels are for construction excavations and for permanent structures which are below the water table and are not waterproof or are waterproof but not designed to resist the hydrostatic pressure. Permanent

dewatering systems are far less commonly used than temporary or construction dewatering systems.

1.2.13 Grouting

Grouting is the injection of stable suspensions or liquid into pores, fissures or voids, or the jetting of cement mixtures at high flow rate and pressure into the soil to create soil-cement. Grouts must be carefully selected to provide cost-effective placement and long-term benefits such as reduced permeability, increased strength or decreased deformability.

Grouting can be classified under different subgroups. One of the most common classifications is after the used grouting material. After this classification the grouting groups are named as; cement and chemical. Each process provides different levels of treatment allowing variation of permeability, extent and strength of resulting material.

1. **Cement grouting** requires fluid grouts to be injected into existing pore spaces in the soil or fissures within rocks. Wide ranges of grout designs are used depending on soil classification and the intended application.
2. **Chemical grouting** requires injection of specially formulated chemical solutions into finer grained soils, which cannot be permeated by cement. The main reason hindering the treatment of fine soils with cement is because of the grain sizes of the soil medium. Grouting material has to be finer than the soil due to its improvement effect by the penetration of grout into the soil.

Improvement technologies using injection can also be summarized as follows:

1. **Permeation Grouting:** Grout is injected into the soil at low pressure and fills the voids without significantly changing the soil's structure and volume. A wide variety of binders are used with this technique, the choice of which is dictated mainly by the permeability of the soil.

As a general rule, where the coefficient of permeability K is greater than 10^{-2} cm/sec, water/cement mixes are used and for permeability's as low as 10^{-5} cm/sec, the more expensive resin based grouts are used. Soils with K values lower than 10^{-6} cm/ sec are normally not groutable by permeation.

2. **Compaction Grouting:** A highly viscous grout with high internal friction is injected into a compactable soil; the grout acts as a radial, hydraulic jack and physically displaces the soil particles thus achieving controlled densification. Compaction is the process injecting a stiff mortar - like grout, capable of high internal friction, into a soil mass where grout does not enter the pores of the soil but displaces it in a manner that the grout stays in an expanding homogeneous mass. The soil becomes denser as water and / or air are forced out and soil particles are rearranged by incoming grout. Grout injection can be continued until grout forces overcome, overburden or containment pressures, and lift occurs.

Placement procedures take into account the entire site conditions and consist of top – down, bottom – up stages or combinations thereof to accomplish specific stabilizations and lift requirements. Top – downs are typically used for shallow applications and releveling operations. Bottom – ups work best for applications where stabilization applications require minimal lift. A combination application may use the bottom – up procedure to stabilize a deep formation problem before the weight of top – down stages is introduced.

3. **Claquage:** Grout is injected into the soil at high pressure through a special, valved tube (canne-a-manchettes), thereby hydrofracturing the soil; the resulting fissures are filled with the grout and the surrounding soil is compressed creating a densified mass.
4. **Jet grouting:** This system differs substantially from the other grouting techniques in that it breaks-up the soil structure completely and mixes the soil particles in-situ with a binder to create a homogenous mass which in time solidifies.

Jet Grouting is a technology in which high-pressure jets of cement grout are discharged sideways into the borehole wall to simultaneously excavate and then mix with the soil.

As a short description, jet grouting is an in-situ mixing of soils with a stabilizer, usually neat cement grout. The stabilizer is injected at very high pressures between 300 and 600 bar through a nozzle of small diameter. The grout obtains high injection

velocity, which enables the jet grouting process to destroy the natural matrix of the soil and creates a mixing of the stabilizer with the in-situ soils. The result is a homogeneous and continuous structural element with predetermined characteristics.

Every improvement method has limitations for specific soil types. Only a few of them are successfully applicable for a wide range of soils. One of these special and new improvement methods is Jet Grouting. Jet Grouting is one of the most used ground improvement systems, showing a huge success for a wide range of soils and accommodating advantages like fast and easy improvement also in limited areas.

The technique can be used regardless of soil type, permeability, grain size distribution, etc. Theoretically therefore, it is possible to treat most soils by jet grouting, from soft clays and silts to sands and gravels. Jet grouting makes it possible to inject any type of binder. In practice uses this technique water/cement mixes as grout, but where impermeabilization of the soil is required, water/cement/bentonite mixes are normally used. [14, 15, 16, 17]

2 JET GROUTING

2.1 Introduction

“Civil engineering projects are often designed and constructed on sites that are constrained by existing urban developments, heavily congested areas, or where their locations are predetermined by other factors. In some cases, soil improvement of weak underlying soils is required to support loads for the intended project. Soil improvement modifies and increases the mechanical characteristics of the treated soil, most notable its compressive strength and modulus of elasticity, while simultaneously decreases its permeability. The jet grouting method is one of the most successful systems to achieve these results” [7]

Jet grouting is one of the widest used reconnaissances in the field of soil improvement techniques. It is currently utilized throughout the world. From its conception in Japan in 1973 by Yahiro and Yashido, its use has spread, first to Europe, then to South and North America, the Far East, Asia and Africa.

Jet grouting can be defined as a method based on the introduction of hydraulic (sometimes combined with pneumatic) energy in order to erode soil and mix or replace the eroded material with an engineered grout to form a solidified in situ element known as Soilcrete. The name Soilcrete is given to the end product of the jet grouting technique by The Keller Group to distinguish it from the injected grout. It is not truly grouting but rather a hydrodynamic mix-in-place technique producing a soil-cement material. [3, 14]

Another definition for jet grouting is given by The American Society of Civil Engineers as a technique utilizing a special drill bit with horizontal and vertical high speed water jets to excavate alluvial soils and produce hard impervious columns by

pumping grout through the horizontal nozzles that jets and mixes with foundation material as the drill bit is withdrawn.

Welsh, Rubright and Coomber, defining jet or replacement grouting as the simultaneous controlled injection of cement grouts to replace soils eroded by high-pressure water jets, have proposed a more general definition. [18]

In another words, jet grouting is an in-situ mixing of soils with a stabilizer, usually neat cement grout. The stabilizer is injected at very high pressures between 300 and 600 bar through a nozzle of small diameter. The grout obtains high injection velocity, which enables the jet grouting process to destroy the natural matrix of the soil and creates a mixing of the stabilizer with the in-situ soils. The result is a homogeneous and continuous structural element with predetermined characteristics. Fig 2.1 and Fig 2.2 shows jet grout columns. [16]



Fig. 2.1 Jet Grout Columns [19]



Fig. 2.2 Jet Grout Columns [20]

Geotechnical grouting is normally employed to achieve improvements in watertightness, strength or stiffness of natural soil or rock formations. The main point in grouting is to ensure that grout enters the ground sufficiently so that the desired improvement can be achieved. Permeation grouting with low pressure using cementitious grout is effective in granular soils with gradings no finer than the medium sand range. For finer soils permeation requires the use of chemical grouts. Against the wide range, offered by chemical grouting, they show limited strength and stiffness as well they are expensive and may pose problems of toxicity. Also chemical grouts extend the treatable grain size only into the silty size. [21]

As an alternative to permeation, high-pressure grout is injected to the ground, which is known as Soilfrac grouting [14]. This provides deeper penetration of grout into soil that alter (improve) ground properties. Also compaction grouting similarly uses high pressure to expand low slump high viscosity bulbs of grout, which compacts and strengthens the ground.

Jet grouting has a radically different approach than the above-described grouting techniques. As mentioned in the definition of Jet grouting, it causes erosion to enable grouting in a very wide range. A comparison of the grain size ranges of cement and chemical injection against jet grouting is shown in Fig 2.3 In theory therefore it is possible by jet grouting to treat most soils, from soft clays and silts to sands and gravels. Although it is possible by jet grouting to inject any type of binder. In practice water / cement mixes or, where impermeabilization of the soil is required, water/cement/bentonite mixes are normally used.

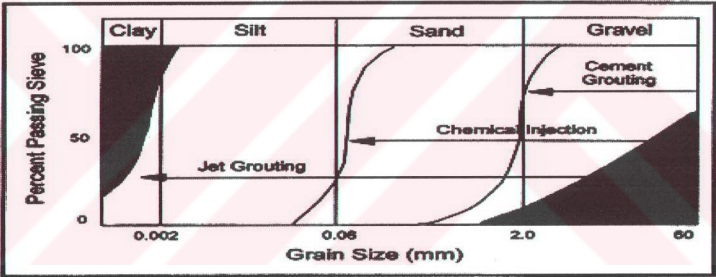


Fig. 2.3 Comparison of Jet Grouting With Other Grouting Methods [22]

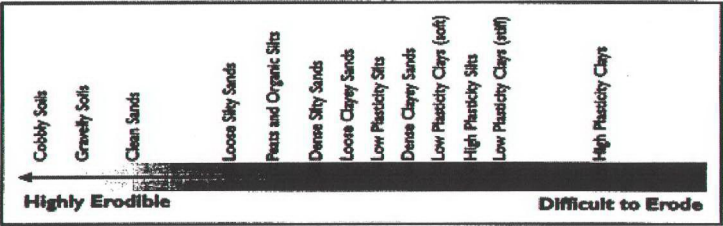


Fig. 2.4 Erodibility Scale [22]

The change in the difficulty of the Erodibility of soils is shown in Fig 2.4.

A second advantage of jet grouting is that large diameter columns or panels can be created, starting from relatively small diameter boreholes. Therefore, cutoff walls can be constructed beneath buildings with limited disruption of the structure itself.

A comparison of the costs of common grouting and of jet grouting is difficult. It is occasionally stated that jet grouting would be less costly. However, this is not easy to confirm since no cases are known where both methods have been tendered and offered for the same project.

The construction time will in many cases be shorter for jet grouting than for common grouting. This follows partly from the obligation to limit the grouting rate with pore filling injection to avoid cracking. With jet grouting an upper limit of the jetting flow rate is given for other reasons, for instance too high flow velocities in the borehole.

With all these above-mentioned jet-grouting skills, the following description of the system and the process is a summarization. "The Jet Grouting Technique employs high pressure erosive jets of water or grout to break down the soil structure, removes varying proportions of the resultant soil particles and replaces them with cement based grouts placed during the erosive operation. Soil particles not removed become mixed with the grout in situ to form the treated mass. [14]"

Jet grouting applications can be seen in variety of soil improvement cases like soil stabilization, excavation support systems, structural underpinning, seepage barrier, cut – off walls, soil consolidation for the excavation of shallow tunnels, environmental remediation or groundwater control.

"The advantages of improving soils by using jet grouting systems can be listed as follows [22]:

- Nearly all soil types groutable
- Specific in situ replacement possible
- Designable strength and permeability
- Treatment to specific subsurface locations
- Only inert components
- No harmful vibrations
- Limited working space required
- Any cross-section of soilcrete possible
- Maintenance free
- Ability to work around buried active utilities
- The most effective means of direct underpinning of structures and utilities
- Much faster than alternative methods

As it can be seen from the listed advantages, jet grouting seems to be a perfect improvement system for all soil types. But against all these, jet grouting has also some limitations. "General disadvantages of all jet grouting methods are related to the fluid mechanics requirement that there must be continuous flow of cuttings from the point of injection (nozzle) up to the surface, where atmospheric conditions exist. This requirement insures that the pressure head at the point of injection is only due to the weight of the cuttings. If the path is restricted, for instance soft clay creeping in around the drill rods, then the pressure head at the point of injection can become the same as the pump pressure (up to 600 bar). Obviously, the no – flow condition cannot continue for long. Either the restriction would clear by the pressure blowing the soil from around the drill rods, or the operator must stop the jetting before significant damage is done: ground heave, and lateral soil movements. If severe Hydro – fracturing of the soil occurs, then no columns would be formed. If the continuity of flow condition is satisfied, then the jet grouting method is a high velocity erosion process. This mechanism requires relatively large volume of spoil to be handled at the surface. [18]"

2.2 Construction Methods and Equipments

In general, all systems of jet grouting have similar basic construction procedures [3]:

- Set up on location at design angle of penetration
- Drill to depth by hydraulic rotary methods
- Commence jet grouting at the base of the design depth, lifting (and rotating if required) uniformly until the design top is reached

Many variations to these procedures have been developed to overcome drilling difficulties or ensure waste return. Also for the success of the erosive attack, much work is done. These include; pre – drilling, cased hole drilling, using a Down – the – Hole Hammer beneath the monitor, high flow drill fluid injection, high-pressure drill fluid injection and drill fluid variations for access, where for erosion precutting, double cutting, multiple nozzles, angled nozzles, high-pressure air, increased high-pressure air, increased high pressure fluid, ultra – high pressure of fluid injection, relief hole placement have been developed.

2.2.1 Working Sequence of Jet Grouting Applications

“The sequence of work starts usually with a small (100-200mm diameter) hole, which is drilled to the required depth, followed with a high pressure (several hundred Bars) fluid pump through one or more small (1-10mm) nozzles on a monitor at the foot of a 70-100mm diameter drill string and ends with that the drill string is slowly raised and rotated to form a column of soil cement. [3]”

“Following the work sequence it should be taken care that the borehole is drilled to the desired depth, using common methods adapted to the type of substrata. At the end of the drill rods the monitor is located in which the drill bit and one or a number of nozzles are fitted. The nozzles transform the high-pressure flow in the string into the high-speed jet directed to the soil. After reaching the final depth soil erosion by disintegration and mix – in – place procedure start at high pressure under rotation and translation of the string of rods The jet of suspension or an additional air / water jet erode the substrata. The eroded material is partly mixed with the suspension. A column – like element of subsoil and suspension develops. [21]”

During jetting, material in excess of the soil cement mix must rise freely to the hole collar to prevent the excess material fracturing and disturbing the surrounding ground. It is removed from site as it emerges.

The result, diameter, composition and strength of the columns, is dependent on drill string rotation and raising speeds, jet pressure and flow, grout mix; soil type, grain size distribution, composition and compactness; and jet configuration (single, double or triple jet set-up). Fig 2.5 shows the sequence of jet grouting in a schematic way. The procedure is repeated with overlapping of the elements to form a wall – like structure. A narrow wall – like structure made of planar panels may also be produced without rotation of the rods, by the production of overlapping panels.

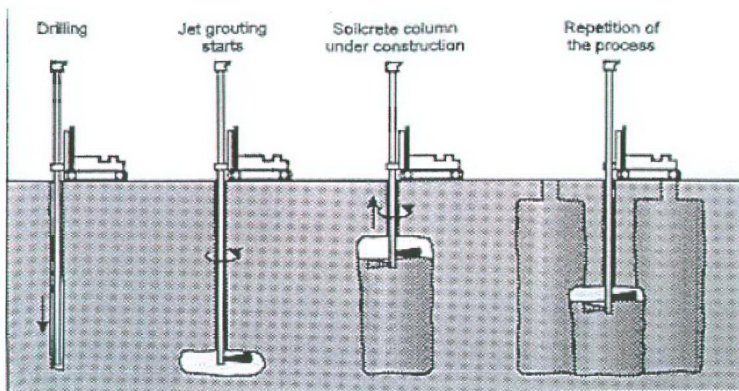


Fig. 2.5 Schematic Presentation of Jet Grouting Construction Sequence [22]

2.2.2 Jet Grouting Equipment

A general knowledge of the used equipment in jet grouting, will supply the understanding of the varying jet grouting methods and used equipment for each method. The required equipment consists of the drilling and grouting equipment including carrier, material silos, and batching plant, of compressor and high pressure pump. There may also be machinery for installation of casings, if required.

2.2.2.1 Jet Grouting Rig

Jet grouting rigs must be able to automatically regulate the rotation and the translation of the string of rods. Proving this ability all common drill rigs and drilling procedures are in use, which are adequate for the subsoil in question (Fig 2.6).

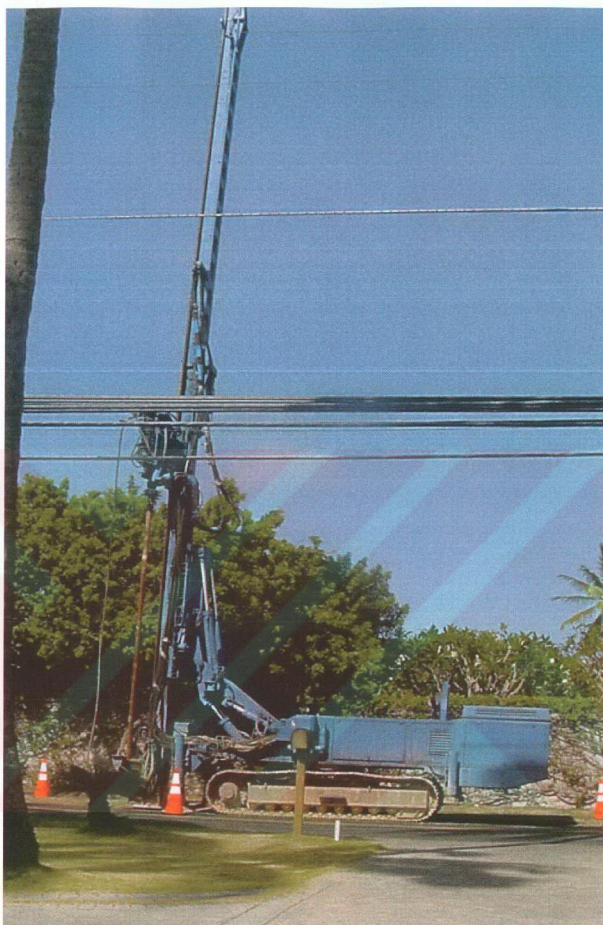


Fig. 2.6 Jet Grouting Rig

2.2.2.2 The Monitor and Nozzles

The monitor at the end of the string is equipped with the nozzles (Fig. 2.7, Fig 2.8). The number of nozzles depends on the system applied. For each jet one or more nozzles are arranged to keep the jet radial to the rod. The diameter of the nozzles is responsible for the velocity of the jet. The diameter is arranged in the range of 2 to 8mm. Care should be given on this nozzle diameter throughout the application, because these nozzles are broadening due to the high pressure and they have to be

checked and changed if necessary. A reliable way is to work with two monitors and change after every column production, so during the one monitor is in use, the other one can be prepared for the next column. This provides the control of nozzles continual and prevents any loose of time.



Fig. 2.7 Monitor Equipped with Nozzles



Fig. 2.8 Jet Nozzles in Process

2.2.2.3 The Batching Plant

The batching plant is similar to that for common grouting. Batching plants are designed continuously to guarantee correct processing and to maintain the specified quality of the grouting material. The equipment should be concentrated in a central station, also to lower the processing costs. Batching plants consist of silos, dosing equipment, colloidal mixers, agitators, measuring and recording devices. All units must be adjusted to the required high capacity. In any case jet grouting requires the ability to batch slurry at a rate of 5 to 25 m³/hr, depending on the system applied, at specific gravities of 1.4 to 1.8.

Silos are used for storage of cement and other dry bulk materials such as bentonite, which may be constituents of the grout mix. Screw feeds transfer the correct proportions of the powders for mixing with water. Each batch of grout is transferred to agitation tanks before passing through high output grout pumps into the grout delivery lines. In respect of the selected jet grouting system a high-pressure high volume water pump and a suitable compressor respectively deliver water and air into their separate lines. The lines are fed into the grout pipe stem for injection via swivel head mounted on the mast of the jet grout drill unit.

Control of various operations, monitoring of the jetting pressure and flow rates, and use of space, is made more efficient by containerizing the critical plant items (Fig 2.9, Fig 2.10). The unit occupies an area of up to 6,7m x 2,6m, depending on the specification, and weighs between 2 and 3 tones. Upon arrival on site the system requires only connection to water, electricity and grout lines before becoming operational.

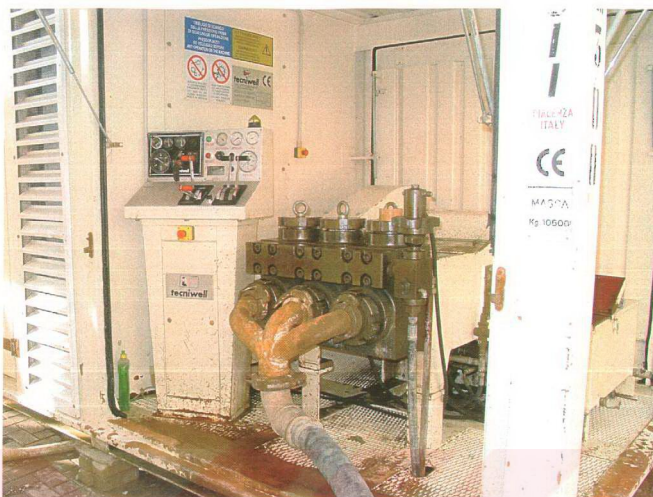


Fig. 2.9 Pressure Control Panel of a Pumping Station

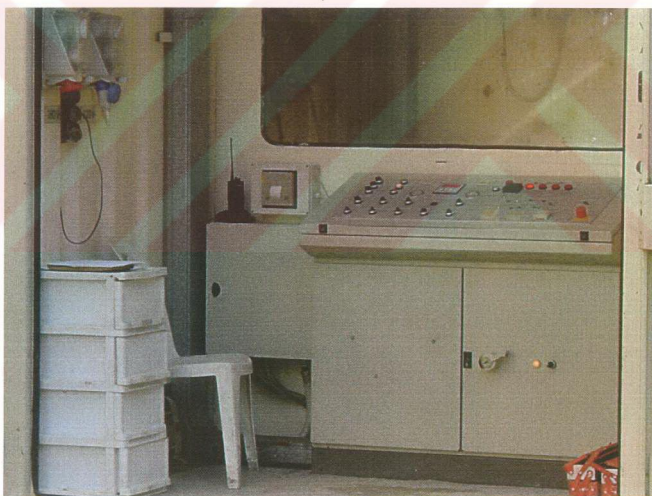


Fig. 2.10 Control Panel of the Batching Plant

The jet grouting pumps are far exceeding the capacity of pumps for common grouting. These pumps are developed for petroleum engineering and have pressure up to 1000 bar with a discharge up to 500 l/min. The weight is up to 14000 kg, with

dimensions up to 6m x 3m x 3m (length x width x height). They don't have an independent control of pressure, so discharge is required. The discharge relates to the stroke frequency, which must be adapted to the effective radius. The controlling parameters for the dimensions of the column are the speeds of rotation and translation. Pressure and discharge should be automatically recorded.

2.2.3 Jet Grouting Systems

Three known jet-grouting systems are used in practice. These are named as Jet 1 – Single Fluid System, Jet 2 – Double Fluid System, Jet 3 – Triple Fluid System. Skills of all systems and the necessary equipment used for all systems are described in the following paragraphs.

2.2.3.1 Jet 1 – Single Fluid System

This system is the most simple, as well as the method of choice of most qualified specialty contractors. The system has been tested and developed in Italy since the mid 1970's to its present state-of-the-art. The single fluid referred to in this method is neat cement grout. This method produces the most homogeneous soil-cement element with the highest strength. A high-pressure pump conveys the binder through a drill string to a set of nozzles located just above the drill bit. Grout is forced into the ground through the nozzles at pressures as high as 600 bars. The high velocity, high energy jet breaks-up and mixes the soil surrounding the drill string to create a column of stabilized material whose diameter ranges from 40 cm to 120 cm. The properties of this mixture once set; determine the characteristics of the treated ground. Extraneous suspension, under pressure, travels up the annulus between the grout pipes and the borehole sides for collection and disposal at the surface. The strength tests of the created soilcrete are made on this returned spoil material. The main equipment required for this system is:

- A high pressure, high flow pump (700 Bar, 300 l/min);
- A drilling rig fitted with a special drill string and a suitable timer with which step-raising of the drill string can be accurately controlled;
- An efficient batching plant with sufficient capacity for the binder.

Rotation speeds range between 10 and 20 rpm and the withdrawal rates vary between 20 and 50 cm/min. The high-pressure jet of binder breaks up and mixes the soil,

surrounding the drillstring to produce a column of stabilized material. Fig 2.11 shows the equipment used in this system.

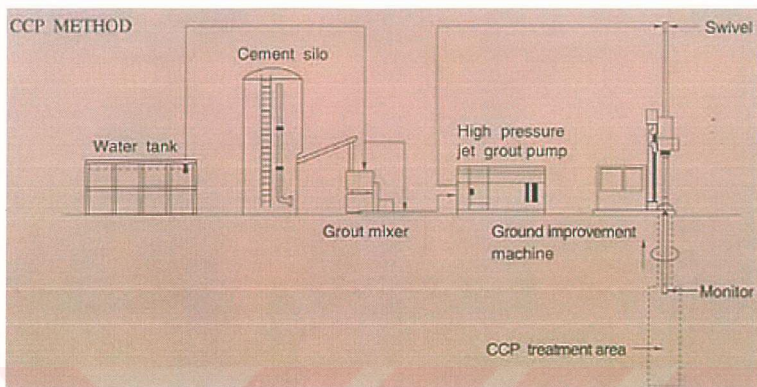


Fig. 2.11 Single Jet System (JET 1) [23]

2.2.3.2 Jet 2 - Double Fluid

The two fluids referred to in this method are neat cement grout and air. The neat cement grout is injected at a lower pressure and is aided by a cone of compressed air, which shrouds the grout injection. The compressed air addition is typically between 2 and 15 bars. This enables a greater treated volume during each jetting lift when compared with the single jet approach. The air reduces the friction loss allowing the cement grout to travel further from the injection point thereby producing greater column diameters. However, the presence of the air reduces the strength of the column as compared to the single fluid method, and produces more spoil return. Typically, the diameter of a column of soil treated performed by the monofluid system can be increased from 2cm to 4.5 cm by adding the air component.

Equipment for the application of this system is the same as for the monofluid system with the exception of the drillstring, which has to be made up of coaxial 2-way rods and with the addition of an air compressor. Fig 2.12 is a diagram of the JET 2 system, showing the sequence and use of the necessary equipment.

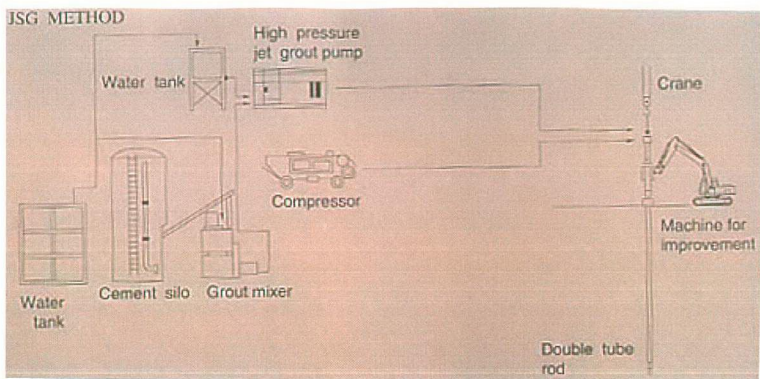


Fig. 2.12 Double Jet System (JET 2) [23]

2.2.3.3 Jet 3 - Triple Fluid

The three fluids referred to in this method are neat cement grout, air and water. Unlike Jet 1 and Jet 2, water is injected at high pressure and is aided by a cone of compressed air, which shrouds the water injection. This process produces an airlifting effect, which evacuates the soil within the intended column diameter. The grout is injected through a separate nozzle below the water and air nozzles to fill the void created by the air lifting process. Water pressures of up to 500 bars are employed, with air pressures of 2 – 15 bars. Jet 3 is a soil replacement method not an in-situ mixing method. The effect of the compressed air is two-fold; it increases the radius of influence of the jet of water and it "lightens" the mixture of soil and water in the zone of influence of the jet creating an air-lift which pumps excess water and soil fines, through the annular space between the borehole wall and the drill rods, to the surface. At the same time the binder is injected into the soil/water mix at a range between 5 and 50 bars through a second nozzle positioned just below the air/water nozzle. The soil and binder mix and in time set to create a column of stabilized material whose diameter may exceed 2 m. The main equipment required for this system is:

- A high pressure, high flow pump for the water jet (700 bar, 300 l/min);
- A low-pressure pump for the binder (70 bar, 120 l/min);
- A 3-way, coaxial drill string made up of a drill bit, a drill bit adapter, a nozzle holder for the cement nozzles, a high-pressure nozzle holder with the coaxial air/water nozzle, the drill rod, and a 3-way swivel
- A drilling rig, fitted with a suitable timer with which step-raising of the drill string can be accurately controlled;
- An efficient batching plant with sufficient capacity for the binder;
- A reservoir for water.

The procedure for drilling is the same as for the monofluid system. The flushing fluid is pumped to the drill bit through the binder line. Once the required depth is reached, water and air are injected through their respective lines to break up the soil surrounding the drill string while this is rotated and to start the airlift, which will displace the finer particles of soil. Injection of the binder then starts and the drill rod is slowly withdrawn. The fundamental difference between the two systems is in the diameter of the stabilized column and thus in the volume of the treated soil per hole. The "3-fluid" system becomes more advantageous where soils are difficult to drill, as fewer holes are required for the same volume of treatment. The 3-fluid system is however more complex, it requires more equipment and as a result also of the large volumes of material discharged from the boreholes; the system becomes expensive and difficult to apply, especially in restricted areas or where soils have low permeability. Fig 2.13 is showing the sequence of the above-described equipment, which is necessary for the application of the JET 3 system.

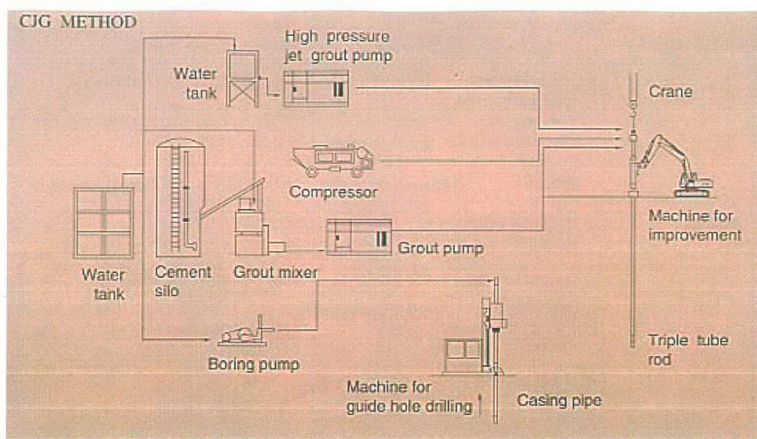


Fig. 2.13 Triple Jet System (JET 3) [23]

2.2.3.4 Comparison of All Systems

Every jet grouting system has its advantages and disadvantages. The point is, that the system on an application should be chosen after the aim of the project. In areas where the effective thickness of a created jet grout panel is more important than the strength of the end product, an application with Jet 2 or Jet 3 may be preferable. But when soilcrete strength and time plays the major role, than Jet 1 system is more versatile. It can be applied at any inclination and consequently finds wide use in the stabilization of existing structures and in tunnels where space is restricted. Setting up and excavation times are considerably shorter; the method is also cheaper, cleaner and less noisy to apply.

Table 2.1 summarizes the range of parameters commonly commonly obtained by use of the 3 jet grouting systems and Fig 2.14 shows the difference of the three system, via the different monitors and varying nozzle types used by them.

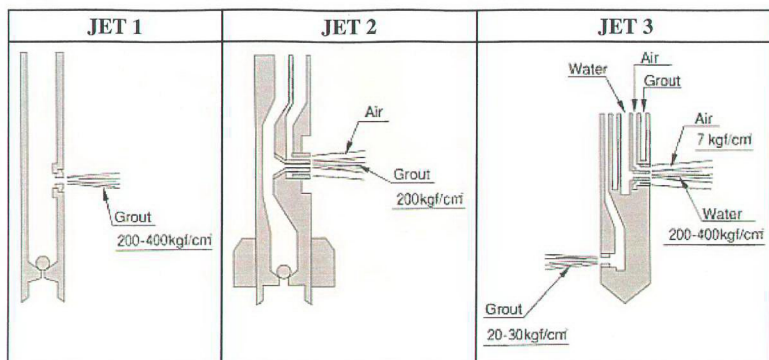


Fig. 2.14 Jet Grouting Systems [23]

Table 2.1 Range of Jet Grouting Properties

Soil Type	Column Diameter	Soilcrete Strength
Jet 1 Sands and Gravels Clay	60 cm – 90 cm (up to 110 cm) 60 cm – 90 cm	70 – 250 kg/cm ² 20 – 100 kg/cm ²
Jet 2 Sands and Gravels Clay	90 cm – 180 cm (up to 300 cm) 90 cm – 150 cm	35 – 140 kg/cm ² 10 – 70 kg/cm ²
Jet 3 Sands and Gravels Clay	150 cm – 240 cm 90 cm – 180 cm	35 – 105 kg/cm ² 10 – 50 kg/cm ²

Table 2.2 shows the rate of the jet grouting parameters, which selection and determination will explained in the following section, for all three jet grouting methods. This table is obtained from the used parameters on several applications by the jet grouting company BCI. As it can be seen from the Table 2.2 the binder injection pressure is very low for the JET 3 like mentioned before. This is pointing again on that, that the JET 3 system uses in comparison to JET 1 and JET 2 the air and water mix for eroding, where the other systems are eroding soils via injected grout.

Table 2.2 Parameters for Jet Grouting

Parameters For Jet Grouting	JET 1		JET 2		JET 3	
	Min.	Max.	Min.	Max.	Min.	Max.
Binder injection pressure (MPa)	20	60	30	60	3	7
Binder flow (l/min)	40	120	70	150	70	150
Air pressure (MPa)	-	-	0.6	1.2	0.6	1.2
Air flow (l/min)	-	-	2000	6000	2000	6000
Water injection pressure (MPa)	-	-	-	-	20	50
Water flow (l/min)	-	-	-	-	70	150
Binder nozzle diameter (mm)	1.5	3	1.5	3	4	8
Water nozzle diameter (mm)	-	-	-	-	1.5	3
Coaxial air nozzle opening (mm)	-	-	1	2	1	2
Speed of rotation (rpm)	10	25	5	10	5	10
Speed of withdrawal (cm/min)	10	50	7	30	5	30

2.3 Jet Grouting Design

“The volume and extent of ground modification is generally the point at which the process begins” says Welsh [25]. If jet grouting is to be used the applicability of the site must be ascertained. Welsh also suggests that “accurate qualitative soil description and, to a lesser extent, quantitative assessment of in situ density in a predominantly granular soil or shear strength in a cohesive material remains the key to achieving optimum jet grouting design. In situ permeability of the soil has little bearing on jet grouting unlike historical grouting methods. [18]”

2.3.1 The Effect of Jet Grouting on the Soil

“The effect of jet grouting on the soil is essentially either to increase its strength or decrease its permeability. [18]” Jet grouting is an easy and fast way to improve soil, against other systems serving on the same course. The system gains its success from the achieved strength of the end product. While the high-pressure injection gets mixed with the in situ soil and strengthens it, also the pores get filled via to the high-pressure effect, so that the soil starts to act as an impermeable wall. But it shouldn't be forgotten, that a complete impermeable cut off is only to be obtained by intersecting jet grout columns or linear grouting on low spaced points, without the rotation of the drill rig. “Miki et. Al. (1980) has reported an increase of 30 times of the unconfined compressive strength of a soft clayey soil, which reached a number of 200 times for the secant value. Also in a test section carried out in Pakistan, the permeability of jet-grouted columns was reported to be between a hundredth and a

ten thousand of that for the untreated sandy soil [18].” These values prove us how effective ground treatment with jet grouting is.

Against all explanations from the further paragraph, jet grouting can only strengthen the soil with a correct design, which is suitable for the soil parameters on the site. Also the application program is a part of the design and must be applied correct. False injection pressure or withdrawal speed will cause an uneconomical result for the best, but much worse, too high pressure can cause a total collapse on the site, because of an unsuitable eroding effect. For the prevention of a not required error, production sequence should be monitored carefully.

2.3.2 Preparation for an Efficient Design

Before the major step of the jet grouting design, the need and requirements of a jet grouting program must be established using traditional geotechnical analysis like settlement analysis, bearing capacity calculations, earth pressure or water inflow rate determinations regarding to the intention of jet grouting used on the site. Also an approximate design, reaches its goal only with a good knowledge of the type and condition of the soil. Factors, which affect the results of treatment of soils by jet grouting, are:

- Soil type
- Soil consistency
- Bulk density
- Grain size distribution
- Water content

All of these would indicate the need for in situ reinforcement or improvement. Once this need is established, the requirements of jet grouting can be determined based on whether the jet-grouted material will support structural loads, earth loads or water pressure. The strength or permeability of jet-grouted soil will dictate whether columns or panels are used and the layout and dimensions required. As a summary, the main design criteria's are:

1. Diameter of soilcrete column or dimension of the panel
2. Strength of the elements
3. Permeability of the elements

The above mentioned design factors are playing major roles in determining the achievable strengths or permeability's in the natural soil deposit after treatment. The effect of the construction method and equipment has also a huge importance in treatment performance. A complete jet grouting system design includes always the verification of the calculated or predicted end conditions, with the help of field tests.

The construction costs plays also one of the major roles by the decision of the treatment method or the approval of a project. They depend on the work progress, i.e. on the speed of rotation and translation, on the labor and material costs and on the costs of the operating equipment.

Welsh has indicated the main points, which is to be taken into account in jet grouting design as follows [25]:

- For any soil, grout or water pressure and probe withdrawal rate are the most significant design factors regarding the volume of jet-grouted soil.
- At a given grout pressure and withdrawal rate, jet grouted volume decreases with increasing clay content.
- As clay content increases, grout pressure must be increased and/or withdrawal rate decreased for a given jet grouted volume.
- It is difficult to obtain jet grout column diameters in excess of 1,5m in stiff to hard clays using typical grout or water pressures.
- If the uniformity coefficient (D_{60}/D_{10}) is equal to or greater than 8, the jet-grouted volume is not significantly affected by grain size distribution.
- If the uniformity coefficient for granular soils is less than 8, column diameters up to 3,0m are possible to obtain using typical operating parameters.

- If the gravel size and larger particle content of the soil is greater than 50 percent, grout penetration may be reduced and be more irregular owing to the tendency for large particles to reflect the jet stream.

Successful practical application of jet grouting recognizes three realities:

- Variations to the axis of jetting due to drilling alignment tolerances.
- Variations to the intended column diameters, panel lengths, widths, etc. due to soil non – uniformities.
- The properties of the treated ground are determined by the in situ soil, as well as by the grout injected.

Given this range of variables and the complexity of natural soils, it is not surprising that no reliable fundamental theoretical relationship governing jet grouting have been produced. The selection of parameters to be used is guided by experience, enhanced by semi – empirical relationships.

2.3.3 Jet Grouting Parameters and Design Procedures

Much experimental work is being done by contractors throughout the world to develop formulas which will assist in determining what parameters should be used in the jet grouting procedure in order to obtain the desired results of column diameter, speed of execution, economy and final strength for different soils. Although there are a few design methods for the determination of the jet grouting parameters, an establishment made via test columns is still the safest method.

2.3.3.1 Overview to Jet Grouting Parameters Used on Applications

For a general idea Table 2.3 collected by Welsh [25] has a wide spectrum showing the used jet grouting parameters and their grout consumption against this parameters for different soil types. It shows especially the effect of the probe withdrawal rate on grouted volume. This is a summarized table from past experiences and projects. Although taking it as basic knowledge, local experience has always more reliability in jet grouting design. Even the form of the foundation on a soil improvement project from inside of a constructed building for example could affect the success of the treatment and force the engineer to change the parameter.

Table 2.3 Effect of Probe Withdrawal Rate on Grouted Volume [25]

<u>Soil</u>	<u>Water / Grout Pressure (bar)</u>	<u>Withdrawal Rate (cm / min)</u>	<u>Grouted Volume (m³)</u>
Soft Rock	390 – 490	2,3 – 10,0	40 – 50
Dense Sand and Gravel	390 – 490	3,0 – 12,0	45 – 55
	350 – 420	30,0	85 – 550
Medium Dense Sand	390 – 490	13,0 – 23,0	50 – 73
	200 – 400	10,0 – 50,0	40 – 465
	380	90,0	60
	275	40,0	85
	390 – 490	50,0 – 120,0	25 – 185
	55 – 70	40,0 – 60,0	60 – 305
Loose Sand	390 – 490	23,0 – 28,0	67 – 76
Clay and Silt	390 – 490	30,0 – 38,0	73 – 82
	200 – 400	10,0 – 50,0	40 – 465
	275	40,0	85
	390 – 490	50,0 – 120,0	20 – 163
	55 – 70	40,0 – 60,0	23 – 120

2.3.3.2 General Design Procedures of Jet Grouting

The below explained design procedure is feasible because of its fundamental sequence and easy understanding. Therefore it is useful to follow this method in jet grouting design.

The first step is to choose the approximate, final strength for the treated soil. “With this parameter and with the aid of the enclosed chart (Fig 2.15) determine the amount of cement, which should be injected into each cubic meter of treated soil. It has to be noticed that this step becomes easier and more accurate with increased local experience. Because of the different behavior of the treated soil by jet grouting in each case, local experience and construction of test columns is always insurance for an approximate solution. This is the case, because sometimes unexpected heave on locally loose points of the ground or around the mat foundations can found place, which can not be predicted with the sounding data. If binders other than cement are

used, local experience from laboratory or site tests must be developed to determine this parameter. [24]"

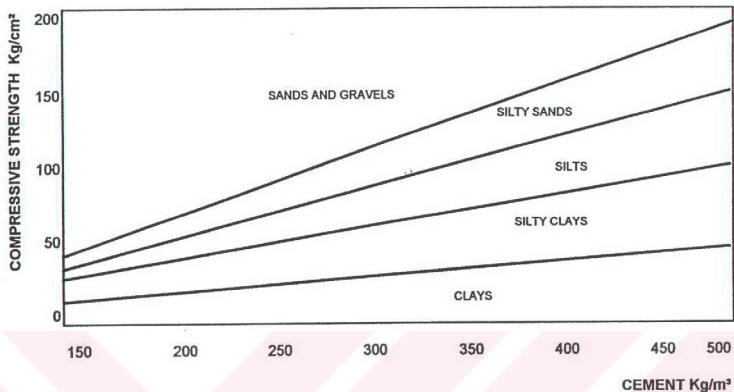


Fig. 2.15 Correlation Between Strength and the Quantity of Cement Injected Per Cubic Meter Of Treated Soil

Choose the desired diameter of the column of treated soil and calculate the amount of cement, which should be injected per meter of column.

“As next the composition of the binder has to be chosen. The principal characteristic of the binder is that it must be pumpable. In the case of a water/cement mix, the proportions of water and cement will affect both its pumpability and its strength; the higher the water/cement ratio, the higher the pumpability of the mix but the lower its final strength. So the desired ratio must be selected based on the aimed final strength and the erodibility of the local soil. Other aspects upon which the mix composition must be based are: the nature of the soil, its grain size distribution, its permeability and water content.

In permeable, granular formations, in-situ water and water from the binder may drain away from the treated area; mixes with higher water/cement ratios may therefore be used; conversely in a cohesive, low permeability soil very little drainage will occur and mixes of lower water/cement ratio will give better strength. In high permeability soils and where strength is of secondary importance, bentonite may be added to the binder to reduce the effects of drainage. In general, the water/cement ratio of

commonly used binders ranges from 1.0 to 1.5. The ideal mix however must be determined from in-situ tests. [24]"

The amount of binder to be injected per meter of treated column must be calculated from the steps explained below, as a further step.

At least a pressure level for the injection has to be selected. This is commonly between 400 and 500 bar. The ideal injection pressure is determined according to past experience and from in-situ tests. This factor is also a function of the capacity of the high-pressure pump and of the nature of the soil. Attention has to be taken on that, that the higher the pressure of the binder, the higher the energy of the jet and consequently the higher its capacity to fracture the soil.

Against Welsh's offered rules in 1987, Steve Birdwell says that the diameter of the column is not consequently directly related with the pump pressure. So it is a myth that the higher the pressure, the larger will be the diameter of the column of treated soil. Although of its effect, the diameter of the column is largely a function of time. The duration of time that the jet of binder is kept at one fixed level and so the volume of binder injected at that level is designating the column size mainly. Steve Birdwell could have right against Welsh, regarding to 14 years more experience with jet grouting systems.

Size and number of nozzles has to be selected, which fit the drill string and from the enclosed 'pressure/flow' (Fig 2.16) chart, determination of the flow of binder can be made.

The speed of rotation of the drill string, which should rotate one or two complete turns per step, is also affecting the column diameter and strength. It's highly effective for the homogeneity of the column. It has also to be noticed that different jet grouting plan geometries need different production sequences. The plan geometries are shown on Fig 2.17. Different behaviors of the drill string give the ability to produce different jet grout element types. For example, instead of rotation, oscillation will end with a half column. Raising jetting monitor without rotation forms panels of treated soil and employing twin nozzles similarly forms wings. A development allows the formation of part columns by causing a 'windscreen wiper' motion of the monitor during lifting. These basic geometries have all been found helpful in practice, but they have to be planned and selected for the necessity of the project.

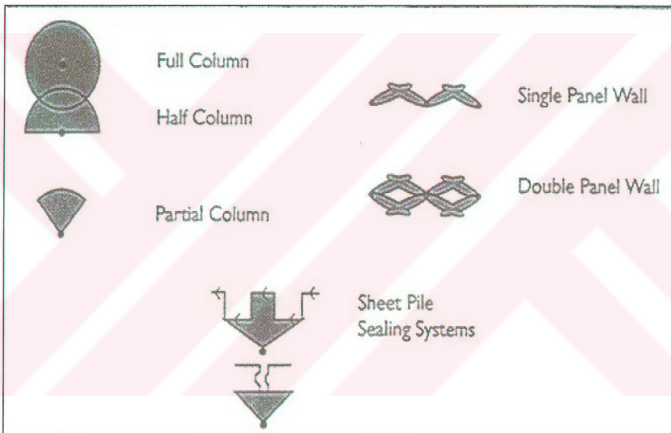


Fig. 2.17 Jet Grouting Plan Geometries [22]

Following the calculation of the above-mentioned parameters, trials on site should be carried out. They should consist of jetting 4 or more sets of 3 columns, each set having different values of mix composition, flow and step-time.

After a pre-established setting time, the test columns should be exposed so that their diameter and the strength of the soil-binder mix can be verified. Where the trial columns are deep, coring techniques can be used to recover samples for measuring and testing.

One another design method, is shared on a publication of M. Lesnik. This method is originated from a formula, which is based on the following theoretical supports [26]:

- The Continuity Equation, Bernoulli Equation and the General Energy Equation are the used base theories for the description between the pumping pressures, flow rate and jet grouting energy.
- From the ratio of theoretical to real flow rate, a prediction of casualties of the system can be determined.
- The spread distance of grouted injection under high pressure can be formulated with the help of the boundary layer theory from Karman, Prandtl and Balsius.
- The restrain articles of mechanics serve for the description of masses and energy balance.
- With the help of the mass storm formulas it is possible to determine a speed relationship for the spreading beam.
- Hydrodynamic effects are taken to account for a correct establishment of the spreading value and speed from grout in the ground and the area around.

The formulation, obtained due to the theoretical base factors described above, is given below.

$$D = \sqrt{\frac{Q \left\{ \frac{\rho_z * \rho_w}{\rho_w + \rho_z * \left(\frac{W}{Z} \right)} * \left[1 + \left(\frac{W}{Z} \right) \right] - a * \rho_{rück} \right\}}{L * \frac{\pi}{4} * (\rho_{saüle} - \rho_{Boden})}} \quad \text{(Equation 2.1 [26])}$$

Where:

Z: Cement portion of the suspension [kg/m^3]

W: Water portion of the suspension [kg/m^3]

Q: Flow rate [m^3/s]

ρ_z : Cement density [kg/m^3]

ρ_w : Water density [kg/m^3]

a: Ratio of the flow rate from the injected grout to the flow rate of the back spoiled material [Q / Q_{back}]

$\rho_{\text{rück}}$: Density of the back spoiled material [kg/m^3]

ρ_{saule} : Column density [kg/m^3]

ρ_{boden} : Ground density [kg/m^3]

L: Withdraw speed [m/s]

D: Jet Grout Column Diameter [m]

It is clear, that this is an easier way for the prediction of the jet grouting parameters for the desired column diameter, than the before described design procedure given in Steve Birdwell's article. After this formula, is the designer able to calculate a specific column diameter, where the preceding design, has only the ability to predict it.

Again here local experience and in – situ tests are basically the most important point of the whole design, and are of necessity to be sure of the reliability of the formula.

The considerations of the engineering design of jet grouting are given in groups after the purpose of treatment.

2.3.3.3 Jet Grouting Design in Underpinning

"In comparisons to others is underpinning the application for which jet grouting has been most used. This is probably due to the cost and schedule competitiveness of the system, as well as the safety of jet grouting versus conventional systems. This is particularly so when underpinning is also combined with excavation support and groundwater control. In general, underpinning design focuses on developing geometry of interconnected columns, which are capable of resisting overturning and sliding. So the underpinning design considerations could be listed as:

- Bearing capacity of the system
- Retaining system evaluation for lateral earth pressure and surcharge loads
- Settlement review
- Strength adequacy of the system

When the treatment serves for an excavation support then are the considerations broaden. The exceeding considerations are:

- What depth is necessary and what shear strength and geometry of soilcrete will resist surcharge, soil and water pressure imposed after excavation?
- Are soil anchors or internal bracing necessary?

The conventional underpinning and excavation design is made according to the earth pressure diagram in Fig 2.18 The calculation of safety factors for overturning and sliding are indicating reliability for values over 1.5. These can be calculated as:

$$F = (W_B + W_S) * \tan \delta + g * Ca \quad \text{(Equation 2.2)}$$

$$FS_{\text{overturning}} = [(W_B + W_S) * a + P_{ps} * b] / [(P_{os} * c) + (P_w * d) + (P_{osl} * e)] \geq 1.5 \quad \text{(Equation 2.3)}$$

$$FS_{\text{sliding}} = [P_{ps} + F] / [P_{os} + P_w + P_{osl}] \geq 1.5 \quad \text{(Equation 2.4)}$$

Where:

a, b, c, d, e: Moment arms

Ca: Base soil adhesion

δ : Base soil friction angle

F: Friction force

FS: Factor of safety

P_{os} : At rest soil force

P_{osl} : Surcharge force

P_{ps} : Passive force

P_w : Hydraulic force

W_B : Weight of structure footing load

W_{SC} : Weight of soilcrete

W_{SL} : Surcharge load

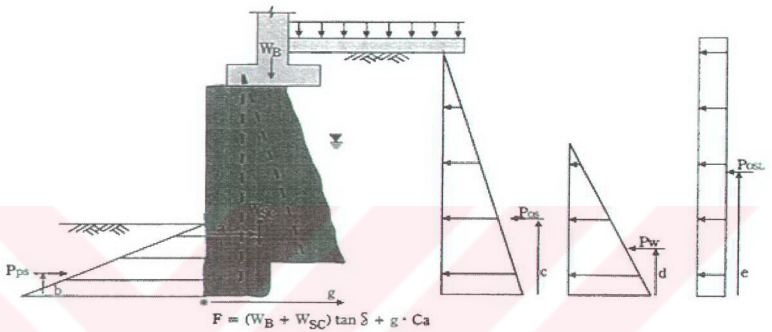


Fig. 2.18 Conventional Underpinning / Excavation Support Design Schema [3]

In underpinning design, it has been assumed that soilcrete act as a homogenous mass not unlike gravity wall. Internal stresses are checked at critical locations and the average desired unconfined compressive strength is typically set at three times the design requirements.

The work plan for jet grouting is also important and is somewhat integrated into the sequence of operations. It has to be noted that, jet grouting beyond 30° from vertical should not use air as component, as it tends to fracture the ground and cause heave.

In general work should progress from low in situ stress regions to high in situ stress regions; that is to say, when underpinning, start beyond the edge of a column footing and progress daily closer to the center until the design coverage is achieved. This will allow soil to “arch” and minimize structure deformations until the soilcrete cures sufficiently to accept load [3].

Where groundwater control alone is required, the permeability and wall thickness of the soilcrete plays a major role. Permeability is one of the most difficult parameters

to test for in the field. The degree of the integrity possible from interconnected soilcrete elements and the quantity of water, which can be tolerated through the soilcrete barrier, has to be considered.

Drilling tolerances are also relevant to the quality of the treatment and become increasingly important with depth. For example, a single row of interlocking vertical columns, with nominal 1m diameter columns spaced at 800mm centers may be required. If the column boreholes are drilled at a tolerance of 1 in 75, at 5m depths, a potential worst error in spacing of 133mm arises. At only 10m depths the error is such that overlap of columns may not take place, leaving gaps in the intended block of soil to be treated, and spacing or column diameters may need to be altered to allow for this. Drilling tolerances of up to 1 in 100 can be achieved in favorable circumstances but more practical considerations will dictate lower tolerances in the 1 in 30 to 1 in 75. Factors dictating tolerances include the drilling method selected, the mass height and drill rig type, the skill and experience of the operators, the depths and inclination of the holes and spoils penetrated.

2.4 Properties of the Elements

The procedure applied for construction, the mix composition of the suspension and the content of suspension are major effective factors on the properties of the soilcrete elements, in another words of the jet grout columns. From a structural point of view it is mainly the strength, the permeability and the erosion stability, which are of interest.

2.4.1 Composition of Jet Grouting Material

The composition of jet grouting material, or grouting material in general consist of water and cement at W/C ratios of 0.8 to 1.5. Ratios above this upper limit are considered as unsuitable suspensions.

The cement, used in grout suspensions, is generally made of Portland cement, blast – furnace cement, finely ground grouting cement and ultra fine cement. Future Slag cements, fly ash and swelling clays have also been used successfully. Decisive properties are fineness of grinding and grain size distribution in the upper range. The fineness of grinding is mainly responsible for the ability of a suspension to enter joints and to pass the channels between them. The cement used in grouting or jet

grouting has to be able to prove the following skill; a specific surface equal or bigger than $3000\text{cm}^2/\text{g}$, 0.1mm maximum grain size and at least 90% of the particles has to be smaller than 0.05mm in diameter. For grouting of fine soils, these skills are for the used ultra fine cement $8000\text{cm}^2/\text{g}$ specific surface, 0.02mm maximum grain size and 90% of the material has to be smaller than 0.015mm in diameter.

In some application is the case that bentonite is used. Care has to be given for the water adsorption potential of bentonite. It shall be at least 500%. The bentonite slurry shall be pre – activated over a period of about 10 hours in a mixture of 1kg bentonite to 10L water. Pre – activation means soaking and homogenizing of the slurry.

In general is the classical rule for grouting materials to apply the coarsest material that is able to enter and to penetrate the existing voids. This is valid for economic and for technical reasons. A thin material of low viscosity – inappropriate for large voids – will travel to an undesired distance. Density and strength of the hardened materials increase with the content of coarse particles. Also the workability of the material plays one of the major roles.

C. Kutzner says that, tests have been made to use suspensions of clay and cement, for economic reasons. He states that the compressive strength decreases considerably in use of such suspension and the effect of clay should carefully be evaluated before a decision is given. [21]

2.4.2 Compressive Strength of the End Product

The compressive strength of the end product has huge importance from an engineering point of view. A high compressive strength is in most case the reason of ground treatment. The type of soil and the composition of the suspension has are the ruling factors of the compressive strength, appropriate construction procedure and jetting parameters provided. Table 2.4 gives values of achievable uniaxial compressive strengths and Fig 2.19 shows them graphically. The first four columns represent the values for well-graded soils, where the others represent poorly graded soils.

Table 2.4 Compressive Strength (MPa) of Jet Grouted Soils [21]

Column No:	1	2	3	4	5	6	7
Cement Grout:	Gravel	Sand	Silt, Clay	Organic Soil	Gravel sandy	Sand silty	Silt clayey
W/C = 0,6	≤ 20	≤ 15	≤ 12	≤ 3	12 to 18	10 to 14	6 to 10
W/C = 1,0	≤ 20	≤ 15	≤ 12	≤ 3	6 to 10	5 to 7	3 to 5

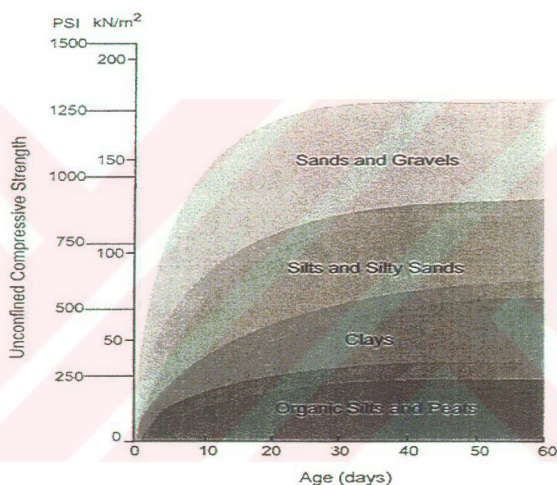


Fig. 2.19 Typical Soilcrete Strengths [21]

As it can be seen from the table above, the reachable compressive strengths for the applications on well-graded soils are slightly higher than on poor graded soils.

If the aim is stabilization like underpinning, the desired strength values are higher than 10MPa. This is achievable for all type of soils, except organic soils. Up to a layer thickness of about 1.5m such organic soils may be completely eroded and be replaced by suspension. If the thickness is over 1.5m, piles or concrete elements must be applied for a satisfactory result.

The development of strength versus time is shown in Fig. 2.20. C. Kutzner points out that the strengthening of jet grout end products on non – cohesive soils are similar to that of pure cement suspensions. The reduced and delayed strength development of cohesive soils after being jet grouted is due to remaining pore water and to remaining excess water of the suspension and, possibly, of the cutting jet.

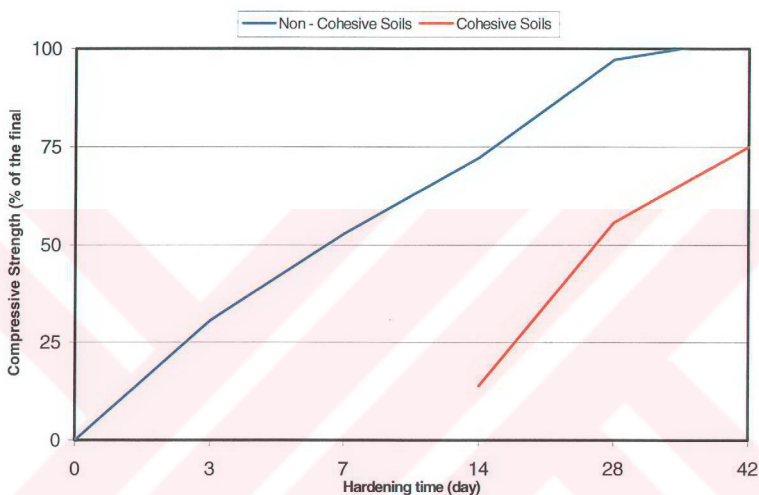


Fig. 2.20 Strength Development of Jet Grouted Soils [21]

The direct effect of the suspension content over the compressive strength is a knowledge, which is known as one of the main rules. So, some information can be taken from the unit weight of the element and also of the spoil return. Usually the weight of the soil is higher than the weight of the suspension. Therefore, the weight of the element is higher than that of the suspension.

The strength of a homogeneous element depends on the content of the suspension. For non – cohesive soils the relation of strength versus content will be proportional, as can be derived from concrete techniques. 50% strength of pure suspension corresponds to 50 % content of suspension in the element. For cohesive soils the decrease of strength will be more pronounced. The strength will be about 40% of that

of pure suspension at 80% content of suspension in the element, and 10% strength at 30% content.

For economic reasons an optimum strength would be achieved with complete pore filling of a non – cohesive soil. This corresponds to a percentage of suspension in the element of 25 to 30%. Then the strength of the element will be about 12 to 16MPa, which is 25 to 30% of the strength of pure suspension taken to be 50MPa. Such strength level is required for underpinnings.

Taking 20kN/m^3 as the weight of the in situ soil and $16,7\text{kN/m}^3$ the weight of hardened suspension, the corresponding element weight is expected as $19,2\text{kN/m}^3$, which fits the measured values of soilcrete weights in the range of 18 to 21kN/m^3 . [16, 21]

2.4.3 Homogeneity

The range of values of soilcrete follows from differences in pore volume and differences in the actual content of suspension. The best result is to achieve the most possible homogeneity status.

These considerations stress the importance of checking the spoil return, in addition to the control of the jetting flow rate. This serves not only to estimate the diameter of the element but also its quality. Homogeneity plays a major role in the soilcrete quality. Limitations can be enumerated as the homogeneity of the soil in situ and the applied construction procedure. It is impossible in practice to achieve perfect adaptation to the given conditions by varying all details of the procedure.

Cobbles over about 60mm edge embedded in the soil mass are sources of inhomogeneity. The jets cannot erode and mix the soil behind such cobbles. In cohesive soil lumps of untreated material may occur. The risk that such inhomogeneities develop increases with the plasticity index of the soil. Varying strength, in the range between 1,3 to 20MPa, is reported to be found from core samples of jet grouted high plastic clay.

Also stratification is a reason of inhomogeneity in jet grouting applications. An element produced in clayey silt has volume ratios of 1 to 3, against an element produced in sandy gravel. With the same jet flow rate the diameter of the element in sandy gravel can be expected as about 2.65m, where this parameter for clayey silt is

only 1,6m. But the element in clayey silt consists of pure suspension, where the suspension ratio for the gravel is about 35%. So the strength of the element in the gravel layer is about 30 to 40% of that in the silt layer.

2.4.4 Permeability

The permeability will essentially depend on the type of soil, on the composition of the grout and on the mix proportion of soil and suspension. General experience gives values $k \leq 10^{-8}$ m/s.

The permeability of a sand decreases after treatment from $k = 10^{-4}$ m/s to a range of 10^{-9} to 10^{-8} m/s, with 1,5 coefficient of uniformity. Jet grouting of cohesive soils will also result in permeability's around the same range.

As an example to the excellent performance of horizontal jet grouting, a structure made in Düsseldorf Germany in 1985 with huge seepage problems, had after the treatment a remaining seepage, which did not exceed 8 l/s per 100m².

Also the repair of the Söse Dam in Germany could be taken as a good example. A concrete core with an upstream layer of silty clay is the base of the dam. After 55 years of operation the concrete showed deficiencies due to leaching. The clay layer was jet grouted to regain satisfactory permeability of the system clay / concrete. A grouted layer had been produced about 0.5m thickness, with strong bond to the concrete. The grouted suspension was composed of 300kg blast – furnace cement, 50kg bentonite and 1000l water. The permeability achieved finally was $k = 3.3 \times 10^{-9}$ m/s after 75 days of application. [21, 16]

2.5 Range of Application

Against the wide application range of jet grouting given in Fig 2.3, like all ground improvement systems also jet grouting has some limitations. A part of these limitations can be seen on the mentioned figure, gravel sized over 60mm are not compatible to be improved with jet grouting. But size is not the relevant object for speaking of the limitations of jet grouting.

2.5.1 Limits of the Application Range

“The range of application of jet grouting is limited mainly by the strength of the substratum to be eroded. This is one of the main differences between common and jet

grouting. Common grouting refers to the size of joints and pores, which is irrelevant for jet grouting. [21]”

2.5.1.1 Limitations of Cohesive Soils

A number of parameters reflect the strength of the substrata, depending on the type of soil. For instance cohesion indicates the strength of cohesive soil. The erosive efficiency is influenced by even small amounts of cohesion in soils being treated. Cohesion increases with the plasticity index and with the consistency. In relatively insensitive cohesive soils column diameters are dependent on soil shear strength. Undrained cohesion of 40kPa and a liquid limit of 40% are seen as limiting values for the application of jet grouting. In cohesive soils stronger than mentioned as limit, the cutting width might not exceed 2 to 5mm only. Economic reasons also effect the limitation for cohesive soils, because of the low translation speed translation causing a huge spend of time for a column production. Depending on the application, soils with higher shear strengths might be treated with abnormally high energy inputs using the double – jet system, or a two – phase approach of initial cutting with an air – water jet followed by more standard jetting.

In sensitive cohesive soils relatively large diameters can easily be cut. However, particularly if diameters above 1,5m are attempted, control of the waste slurry and large variations in the geometry and properties of treatment will arise due to variation in sensitivity within the deposit.

The risk of partial or complete blocking of the waste slurry returns is considerably enhanced with cohesive soils. The slurry is much thicker than in granular soils, and with softer soils the borehole diameter may reduce under the action of lateral earth pressures. The effect of structure is also more marked in cohesive soils. For example, the erosion of fissured clay may cause small lumps of clay to be formed, which will fall into the slurry and restrict escape to the surface. Pre – flushing with water may assist in these circumstances.

2.5.1.2 Limitations of Non – Cohesive Soils

For non – cohesive soils, density is the limiting parameter for jet grouting applications. Although, sand and gravel of high density have been treated successfully in practice, according contractor information an upper limit of application is given by gravel of 60mm in diameter. Obviously, it is impossible to

move larger pieces of stones from the natural location and to bring them to the surface through the annular space. The problem is changed to pore filling without change of the soil structure. This is better achieved by conventional grouting.

In general, sands are seen to be best suited to treatment. Erosion of individual particles takes little energy due to their lack of cohesion with surrounding particles, and escape of the extraneous slurry to the surface is also assisted by the flowability and lack of cohesion. In these soils, relatively large column diameters or panel lengths can be formed. The largest column diameter will be formed in loose, badly graded materials. Relative density is usually more important than grading, especially when the uniformity coefficient is high.

2.5.1.3 Application on Multilayered Substrata

Where treatment is required to extend through two or more distinct strata, the jetting parameters can often be altered to maintain the geometry throughout the full treatment depth. However, some change in treatment properties relating to the change in soil type may have to be accepted. In more mixed soil conditions which bands, lenses or laminations of one soil type intrude into another, it will be impractical to alter grout or jetting parameters during any one lift. In these cases, the parameters are set to meet the minimum critical requirements in any soil expected.

2.5.2 Application Range of Jet Grouting in General

Taking all the limitations and considerations into account, the range of application covers almost all kinds of soil, as shown in Fig 2.3. The application of common grouting is enriched by jet grouting, particularly in the case of cohesive soils. This is achieved by changing from pore filling to mix – in – place, to some extent with soil replacement.

After C. Kutzner, “in solid rock the compressive strength opposes the erosive attack. In Germany jet grouting was used to seal jointed, but rather weak sandstone. The water jet, surrounded by compressed air, was able to cut the sandstone. The compressive strength of the material was 8 MPa. The sandstone was jointed at medium spacing. The procedure was applied to cut the solid banks between existing joints and then to fill the joints with grout material. [21]”

2.6 The Jet Grouting Process; Drilling, Eroding and Grouting

As mentioned before a typical jet grouting system starts with drilling, following eroding, grouting and quality control at least. A typical sequence of jet grouting is given in Fig 2.5

2.6.1 Drilling

“Jet grouting starts with drilling a borehole, usually 100 – 150 mm in diameter, to the required base depth of the proposed jet grouted mass. This is to enable the jetting pipes (or ‘monitor’) to be inserted. It is not necessary for the boreholes to be vertical and inclined drilling may be required to gain access.

2.6.1.1 Drilling Types

Boreholes may be pre – drilled with any one of a number of proved overburden pressure rock drilling systems prior to monitor insertion. Alternatively, they can be self – drilled by attaching drill bits, appropriate to the ground conditions, to the monitor base. Selection of pre – drilling versus self – drilling is dictated by the soil or rock types to be penetrated, by the balance of drilling time versus jetting time, or by other factors such as working space limitations.

Maintenance of the stability of diameter of the bores is important in jet grouting, since the waste slurry generated during jetting is required to reach the surface via the boreholes. Pre – drilled hole stability and size is usually maintained, prior to monitor insertion, by means of a bentonite suspension. However, other means can be employed, including temporary casing. With self – drilling, stability is maintained primarily by the presence of the grout pipes and monitor in the hole. The drill flushing medium (water, foam, mud or air) is normally kept flowing during the short delay between drilling and the commencement of jetting to maintain borehole diameter. Whatever the means of drilling, once jetting commences; the escaping waste slurry properly maintains stability. [14]

2.6.1.2 Drilling Equipment

Drilling is usually facilitated by compact but powerful and articulated diesel – hydraulic crawler drill rigs fitted with rotary or rotary percussive drill heads. The machines typically weigh 6 – 20 tones depending on size and mast height, although special units weighing less than 1.5 tones, with remote power packs, have also been

used when headroom or and / or spacing space restrictions dictate. Purpose – built jet grouting units have mast and heads designed to drill, handle and install the grout pipes efficiently, and complete with on board electronic monitoring equipment have proved most effective. [14]

2.6.2 Eroding and Grouting

After the drill phase eroding is the following step of the jet grouting process, to enable the placement of the grout slurry. For eroding and grout placement, a choice of the jet grouting system has to be made, taking caution on the requirements of soil. Once the decision is given, the necessary application parameters, like withdraw speed, swivel speed and nozzle diameter has to be determined, so the designed column diameter can be obtained.

2.6.2.1 Sequence

The eroding phase materializes in single and double jet systems simultaneously with the grouting phase, because the injected grout under high-pressure cuts the soil in situ produces the jet grout element due to the mix in place finding place. If the triple system is selected, than the sequence has a further step, because the grout is injected via the lower placed nozzle after the air / water mix has eroded the soil in nature.

2.6.2.2 Influence of the Progress Speed and Material Demand

The work progress results from the speeds of rotation and translation. They must both be adjusted to the type of soil and to the desired size of the elements. Rotation and translation are practiced within a wide range of speed. Many combinations result in high quality elements.

It should be noted that only the combination of rotation and translation speed determines the work progress. In combination with the soil properties the two speeds are responsible for the dimensions of the produced elements. Finally, the type of soil and the two speeds in combination with the properties of the suspension gives the base for the element properties and the homogeneity. For reliable prediction and result, the large number of influences stresses tests.

“Rapid progress at the higher translation speeds leads to high demand for suspension per unit of time and large pump capacity. For instance, a column (0.8m) in gravel at a mix proportion of suspension / soil of 40 / 60% requires 700 liter of suspension for

section 1.0m length. [21]” The translation speed is taken as 20 to 40 cm/min and the jetting flow rate amounts to 140 to 280 l/min in a single jet system. An excess demand results from the requirement that part of the soil and suspension are brought to the surface as spoil return. So, a column 1,0m in length needs about 1000 liter of suspension. For instance as it will be mentioned in the last chapter of this thesis, the demand in KEDAŞ General Directorate Building in Izmit was in average 195kg suspension per m length of jet grout column. The period of construction is 2,5 to 5min only, due to the assumed translation.

As a comparison, a grouting zone of equal size by conventional grouting with a pumping rate of 10 to 15 l/min requires 40min for a section 1.0m in length, but it is completed with only 500 liters of suspension.

The material demand for jet grouting is about twice that for conventional grouting. However, with the necessary material for only 1,0m length of treatment by conventional grouting, a column of 10m can be constructed by jet grouting. This should be considered in comparing costs and other economic aspects. [21, 16]

2.6.2.3 Applications on Stratified Substratas

It shall be considered that related to stratified substrata, with constant work progress the effective radius of the eroding jet decreases with increasing strength of the in situ soil layer.

With constant supply of suspension the following disadvantage occurs: In weak strata a large column develops, with low content of suspension and low strength. In contrast, in strong strata a column that is small in diameter develops, with high content of suspension and high strength. Also high translation speed in weak soils can cause lateral movement, which could be very dangerous at sites where other buildings or pipelines are in the near area.

Taking such kind of disadvantages into account, the translation speed has to be adapted to the strength of the strata. This requires careful soil investigation and evaluation of the strata before giving the start for the treatment. Other means to cope with the problem are tests at the beginning of the work and consistent control of the return spoil, pointing out to the importance of local experience. [10, 6]

2.6.2.4 Spoil of Placed Grout

Suspension not remaining in the ground travels to the surface via the borehole annulus as for the single system. However, an airlift effect enhances the exhaust of the spoil when double or triple system is used. Any restriction or blockage to the free flow of the waste, with any of the three systems, may result in over – pressuring of the ground, with possible associated ground heave or lateral deformation. Underpinning projects of existing buildings requires careful handling, because the spoil will be blocked by the foundation, especially when mat foundations are in question, leading to lateral movement. The pumping pressure has to be decreased to hindrance the problem.

The spoil generated by jet grouting can be channeled under gravity, or directly pumped from the borehole exit point into skips or tanks for off – site disposal. Alternatively, the spoil can be fed to handling lagoons where the slurry sets into a cohesive material for easier handling and disposal. For slurries with high water / solids content, centrifugal filters can be used to form sludge plus water for separate disposal.

2.6.2.5 Monitoring

Full control of jet grouting from commencement of drilling to the completion of grouting is fundamental to its success and requires understanding, monitoring and control of the site technical objectives and limitations. Skill and experience of the site crew in jet grouting is also a major factor affecting the quality of the treatment. Also necessary for an efficient result is monitoring of batching, mixing, drilling and jetting, a reliable plant and equipment well matched to the performance requirements.

The use of electronic equipment has proved helpful. This enables the jetting operator to monitor easily the vital control variables of lift speed, rpm, and the flows and pressures of the erosive and placement jets. The information allows earliest indications of any system malfunction or blockage in the ground to be obtained and ensures timely and appropriate action. In addition, the data is stored automatically and may be printed out to provide a complete record of the panel or column constructed. [5, 6, 10]

2.7 Quality Control and Assurance

A successful jet grouting application requires a full quality control program, involving tests prior to application, monitoring and tests during application and tests after application, which can be described as quality assurance. Only, with this kind of quality control is it possible to create an end product as planned.

2.7.1 Quality Control Prior to Application

A jet grouting application can be described as procedural; that means, one should maintain the jet grouting parameters and materials consistent with the approved test section. The installation of test columns prior to production is a necessity, to have a proper knowledge about the geometry and quality of the end products. To make a prediction about the required jet grouting parameters with precision, it has to be worked before on the specific site. It is however, possible to estimate these values empirically from past experiences in similar conditions. These parameters to be determined are:

- Drilling: Hole diameter, drilling slurry makeup
- Drill Setting: Lift speed, rotation speed, number of lifts
- Monitor: Nozzle sizes for all components injected, angle of the nozzles, number of nozzles
- Injection: Volume and pressure of all components injected
- Material: Method of mixing and material components and their considerations

In some cases, grout mixes should be tested in the laboratory to ensure that the desired quality can be achieved. Ordinarily, this can be predicted with reasonable accuracy by knowing the material type and amount of cement injected.

As a summarization, prior to grouting some soil samples from the test site must be taken, for compatibility testing of the resulting soil-grout mixture. "This will determine the susceptibility to permeation by chemical contaminants. The permeability of the soil matrix and the available void space will dictate the water/cement ratio of the grout, grout pressures, and hole spacing used. Wall thickness will determine the spacing of the grout hole array.

2.7.2 Sampling and Testing of Jet Grout Columns

Visual inspection, inspection with the help of monitoring devices and sampling are the methods available to create a monitoring or quality assurance program. Care should always be given to sampling and testing.

2.7.2.1 Mixture Quality

As important as the monitoring program is the assurance of the quality of the grout mixture. A perfect application with an incapable mixture wouldn't have a meaning at all.

The quality of the grout mixture can be determined by pouring a sample into a tray 1-inch deep and leaving it to set overnight. Break the sample open the next day and observe a vertical section. If banding is present within the section, it is an indication of poor mixing. It is also suggested that a pump test be performed at the field site to determine baseline conditions for later comparisons. [27]"

Additionally, by measuring waste return unit weight one should be able to assess the mixture quality, as well the geometry of soilcrete. However, this geometry prediction method has not proven reliable, and visual inspection remains the best method to assess geometry. The methods of sampling and testing soilcrete are given in Table 2.5.

Table 2.5 Sampling and Testing Methods

Requirement	Sample Methods	Test Methods
Strength	a) Wet grab (in situ) cast in to molds b) Cast in place plastic pipe retrieved after cure c) Core drilling	a) Unconfined Compression b) Triaxial c) Tension d) Direct Shear e) CPT (in situ) if soft enough
Continuity		a) Pile Integrity Test
Permeability	a) Wet grab (in situ) cast in to molds b) Cast in place plastic pipe retrieved after cure c) Core drilling d) Cast – in – Place Piezometer e) Drilled and Cast Piezometer	a) Permeameter b) Rising or Falling Head (in situ) c) Packer Testing

2.7.2.2 Wet Grab Sampling

Devices have been developed to retrieve wet grab samples at any depth, provided the sampler can be inserted in the uncured soilcrete the depth desired. If the ground is very sandy or includes cobbles and boulders, this will restrict deep insertion. Wet grab samples are usually poured into small molds and cured and tested like a concrete cylinder.

When permeability is a requirement, it can be determined in the laboratory on samples taken from the structure. It may also be checked in situ by water pressure tests made in boreholes penetrating the structure. Both procedures are applied.

2.7.2.3 Core Drilling

Core drilling is one of the most used sampling methods in jet grouting. The taken core sample offers the opportunity to identify the created soil grout mixture. Because of its easy applicability core drilling is also one of the cheapest sampling methods on the field. Fig 2.21 shows taken jet grout samples with core drilling.



Fig. 2.21 Core Samples

Care has to be given to the application time of sampling. The jet grout column has to gain its strength, so minimum 7 days after grout placement is recommended to take core samples.

2.7.2.4 Pile Integrity Test

Pile integrity testing is also one of the quality assurance tests, which is usual to be made after jet grouting applications.

Modern non-destructive methods based on small strain impulse techniques enable the integrity of all the piles on a site to be established rapidly and economically to enhance confidence in the foundation greatly.

The test is performed using a portable, battery powered unit, which requires no backup vehicle or vulnerable cables across the site that disrupt normal construction activity. The test equipment is based upon the most modern digital data acquisition and signal processing techniques currently available, thus maximizing the resolution and effectiveness of the test system (Fig 2.22)



Fig. 2.22 Pile Integrity Testing Equipment

In excess of two hundred piles per day may be tested with this unit, depending on access and pile head condition.

Prior to testing, the pile head should be exposed and cut back to sound concrete with a reasonably level surface, free from standing water and pile cap reinforcing steel. It is recommended that the shaft concrete be allowed to cure for at least seven days before testing.

Striking the pile head with either an instrumented or conventional hammer to propagate a compressive stress pulse traveling longitudinally within the pile shaft performs the test. Fig 2.23 shows a picture taken during the test.



Fig. 2.23 Pile Integrity Application

A transducer held against the pile head registers the initial iMPact and the subsequent vibration response of the pile head. The measured signals are digitized by the test unit and stored in memory. As the stress pulse travels down the pile shaft, reflected waves are generated from any changes in the pile shaft acoustic impedance, which in most cases may be interpreted as changes in pile shaft area. These reflected waves arrive some time later at the pile head and are registered by the pile head transducer. The time of arrival of these reflected waves and their form allows the elevation of the change in impedance and its sense (i.e. an increase or decrease) to be determined easily.

The test results can be analyzed in both the time domain and the frequency domain. In time domain analysis (also known as the sonic echo test), pile head velocity is plotted against time. Reflected waves generated by pile shaft anomalies are readily identifiable with this method of presentation of the test results. It is also possible to compensate analytically for the effects of the attenuation of the stress pulse by the soil resistance supporting the pile shaft.

Frequency domain analysis may also be performed. This involves converting the time domain signals via Fourier analysis into frequency spectra. If the information

obtained from the instrumented hammer is included in the analysis, a small strain pile head dynamic stiffness value for the pile can be calculated.

This is a very valuable index parameter, a useful in determining defects in the upper two to three diameters of pile shaft, which is often a critical region.

Adoption of test result analysis in both the time and frequency domains maximizes the reliability of the test interpretation. In addition to the vibration response of the pile head during the test, information regarding the pile dimensions, concrete mix design, reinforcing steel details and the soil conditions are desirable to complete the interpretation of any test result. [28]

2.7.2.5 Cone Penetration Test

A full quality control program for a jet grouting application involves also the Cone Penetration testing. CPT is performed with a cylindrical penetrometer with a cone penetrating into the ground at a constant rate of penetration. During the penetration, the forces on the cone and the friction sleeve are measured. The measurements are carried out using electronic transfer and data logging, with a measurement frequency that can secure detailed information about the soil conditions.

The results from a cone penetration test can in principle be used to evaluate stratification, soil type, soil density and in situ stress conditions, shear strength parameters. In jet grouting applications, shows such kind of an analyzes the end condition of the treated soil.

Generally, the results from cone penetration tests may also be used, directly, for design of piled foundations in sand and gravel. Indirectly it can be used (shear strength) for piles in clay. [29]

A CPT test result chart is presented as shown in Fig. 2.24.

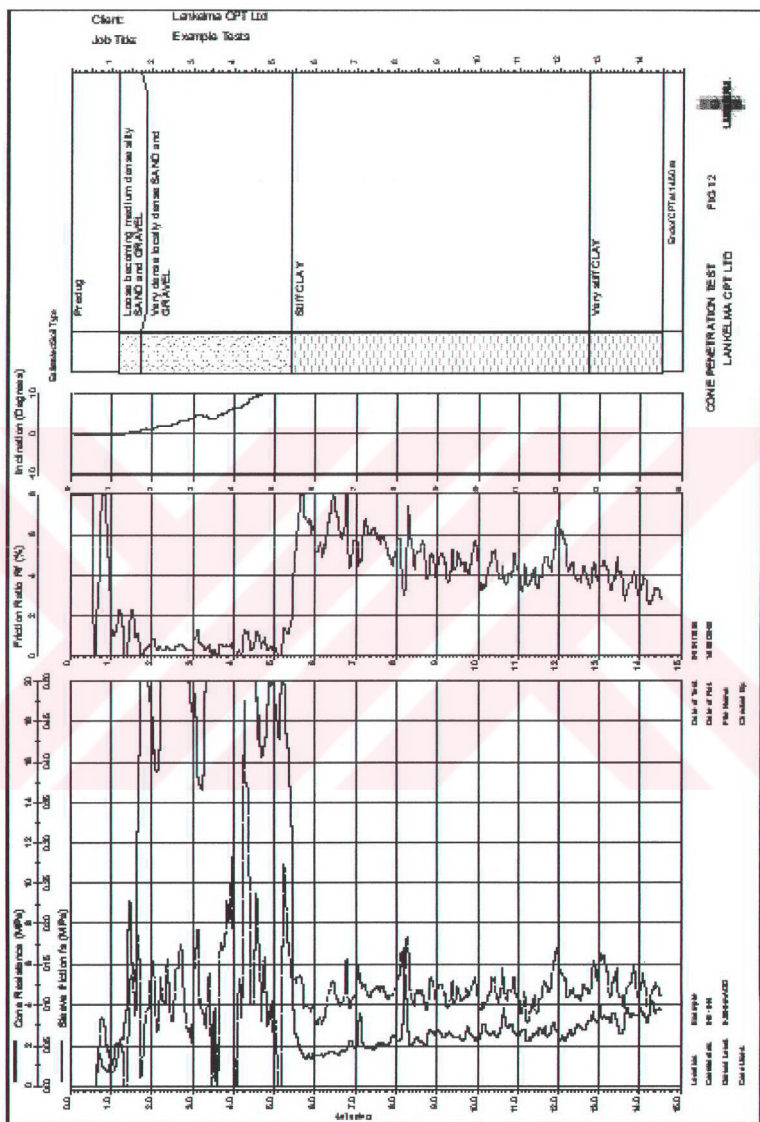


Fig. 2.24 CPT Result Charts [29]

Cone penetrometers can be divided in subtraction cones and compression cones. A subtraction cone measures the total force on the penetrometer (sleeve + tip) and the tip resistance. The sleeve friction is calculated by subtracting the tip resistance from the total force. A compression cone measures the tip resistance and the sleeve friction separately. This results in a lower maximum value (1 MPa) for the sleeve friction.

There is no restriction for the sleeve friction of the subtraction cone. The only restriction is the total force on the penetrometer. It is mentioned to be better to use subtraction cones because a sleeve friction higher than 1MPa is common, especially in very stiff clay. [29]”

There are cone penetrometers with 10 and 15 tones capacity. Table’s 2.6, 2.7 and 2.8 give specifications for both cones

Table 2.6 Subtraction Cones [29]

	10 tonnes	15 tonnes
Apex angle of cone:	60°	60°
Diameter:	35.7 mm	43.8 mm
Projected area of cone:	1000 mm ²	1500 mm ²
Length of friction sleeve:	134 mm	164 mm
Area of friction sleeve:	15000 mm ²	22500 mm ²
Max. force on penetrometer:	100 kN	150 kN
Max. cone resistance q_c (if $f_s = 0$):	100 MPa (100 kN)	100 MPa (150 kN)
Max. sleeve friction f_s (if $q_c = 0$):	6.6 MPa (100 kN)	6.6 MPa (150 kN)
Diameter of push rods:	36 mm	36 mm

Table 2.7 Compression Cones [29]

	10 tonnes	15 tonnes
Apex angle of cone:	60°	60°
Diameter:	35.7 mm	43.8 mm
Projected area of cone:	1000 mm ²	1500 mm ²
Length of friction sleeve:	134 mm	164 mm
Area of friction sleeve:	15000 mm ²	22500 mm ²
Max. force on penetrometer:	100 kN	150 kN
Max. cone resistance q_c (If $f_s = 0$):	100 MPa (100 kN)	100 MPa (150 kN)
Max. sleeve friction f_s:	1 MPa (15 kN)	1 MPa (22.5 kN)
Diameter of push rods:	36 mm	36 mm

Table 2.8 General Information [29]

Rate of penetration:	20 ± 5 mm/s
Max. inclination:	15°
Sensors:	Cone resistance Sleeve friction Inclination Depth Pore water pressure (only for CPTU) with a max. reach of 1 MPa, 2 MPa or 5 MPa
Position of thrust machine:	At least 1 m from previous CPT, or at least 20 times the diameter of a previous borehole.
Level of thrust machine:	Vertical ± 2o
Distance between measurements:	Maximum 20 mm
Calibration:	Every 3 month or 3000 m of sounding
Max. wear:	Angle of cone: 60° ± 5° Sleeve: 35.7 ± 0.4 mm
Symbols used on some Cone Calibration Certificates:	C10 = 10 tones compression cone C15 = 15 tones compression cone S10 = 10 tones subtraction cone S15 = 15 tones subtraction cone C = measurement of cone (tip) resistance F = measurement of sleeve friction P = measurement of pore water pressure Example: Cone number: S10-CF.038 means: 10 tones subtraction cone, number 038, with measurement of tip resistance and sleeve friction.

The CPT has three main applications:

1. To determine sub-surface stratigraphy and identify materials present.
2. To estimate geotechnical parameters.
3. To provide results for direct geotechnical design.

To determine the identification of the soils present, many correlation charts are in use.

The interpretation of the CPT for geotechnical parameters can be made for fine-grained soils as clay to estimate the undrained shear strength and coefficient of

volume change, where for coarse-grained soils as sand CPT values are used for the reckoning of relative density.

It is possible to evaluate the equivalent SPT-values for all soil types

The preliminary undrained shear strength (C_u) of clay can by the help of the following formula:

$$C_u = q_c / N_k \quad \text{(Equation 2.5)}$$

Where:

q_c = measured cone resistance

N_k = 18 for normally consolidated (NC) clays 20 for overconsolidated (OC) clays

For NC and lightly OC clays and silts up to firm in consistency (q_c less than about 1.2 MPa), a first approximation can be obtained by using the relationship between the cone resistance (q_c) and the coefficient of volume change (m_v):

$$s = H \Delta p m_v = (1/\alpha \cdot q_c) \quad \text{(Equation 2.6)}$$

Where:

s = settlement of soil layer H

H = thickness of soil layer

Δp = applied load midway soil layer H

m_v = coefficient of volume change

α = Coefficient

q_c = cone resistance

The value of α depends soil type and the Over Consolidation Ratio

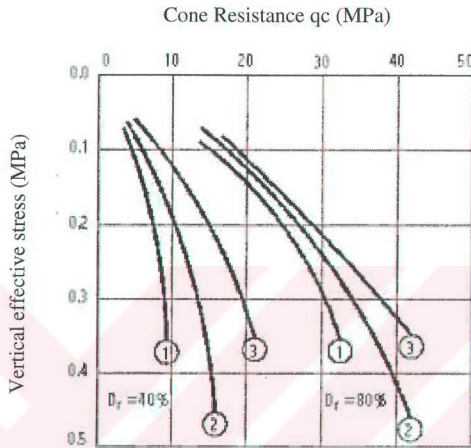
For non-cohesive soils, the relative density is often used as a soil parameter. Fig. 2.25 below shows a chart, which gives the relationship between the measured cone resistance and the relative density (D_r) or density index (I_D).

$$D_r = I_D = (e_{max} / e) / (e_{max} / e_{min}) \quad \text{(Equation 2.7)}$$

Where:

e_{max} and e_{min} are the maximum and minimum void ratios that can be determined in the laboratory using appropriate standards;

e is the in situ void ratio. [29]"



- 1- Schmertmann (1976) Hilton Mines sand – high compressibility
- 2- Baldi et al., (1982) Ticino sand – moderate compressibility
- 3- Villet & Mitchel (1981) Monterey sand – low compressibility

Fig. 2.25 Effect of Sand Compressibility [29]

2.7.2.6 Pull Out Tests

Also in some applications, after its service reason, pull out tests; on jet grot columns are possible to be made. Horizontal or angled jet grout columns need high pull capacities due to their working status.

2.7.2.7 Laboratory Testing

The recovered samples of grout mixtures or end product, are subjected to known geotechnical tests like unconfined compression test, the triaxial test, direct shear test or permeability tests for the determination of their properties. These tests are a kind of measurement of the success of an application, in that they are showing how near the obtained properties to design properties are.

Sampling must be planned for every application by the design phases, so that the tests could shed light on the complete application. Also the rate of the tests is important. Enough laboratory testing must have been done, for correct generalization and interpretation over the result data's.

2.7.2.8 Coordination of the Application

Because the equipment used in jet grouting is highly computerized, a trained and experienced operator is also required to monitor the process during the application. While the injection process, readouts of flow rates, injected quantities, and materials used are generated, and based on data interpretation, which make the engineer able to modify the process to obtain a desired end product.

2.7.3 Quality Assurance Programs in Jet Grouting

With the fact that the quality assurance program for jet grouting applications will change for every project, an example are given below, based on a case history. A comparison between the aimed and achieved soil strength and permeability on the given jet grouting applications shows the success of soil improvement by jet grouting.

2.7.3.1 Ford – Otosan Automobile Plant

“The ground improvement project of the Ford – Otosan Automobile Plant is one of the recent largest jet grouting applications in Turkey. The plant site is located to the east of the town of Golcuk bounded by the Izmit – Golcuk Highway on the south and the Marmara Sea on the north. The total area of the site is approximately 1 600 000m². The complex was consisting of five major facilities. The soil profile under the plant was soft to medium clay or loose sand at the upper 5m, and medium dense silty sand with liquefaction potential below these upper strata.

Foundation design for the various buildings in the complex consisted of jet grout columns and bored piles. The jet grout columns were of 800mm diameter and 10m effective lengths. These columns transferred uniform base pressures of 20 to 50kPa under the continuous or footing foundations to denser strata, thus controlling settlements and reducing the risk of liquefaction.

The quality control program of the above-described application on the Ford – Otosan Automobile Plant, was consisting of preliminary parameter tests, a number of tests

implanted during the production and of coring from the head of the jet grout columns as main quality control test. Integrity testing was also implemented before the Kocaeli Earthquake on August 17, 1999, however, the test was performed rarely as a secondary test after the earthquake. In the interpretation of the integrity testing performed on jet grout columns, was observed that there is a difficulty in receiving clear toe reflections since the modulus of elasticity of a jet grout column is very close to that of the surrounding soil layers. Continuous coring was mainly performed after the earthquake. Three to four samples from each cored column were selected for strength tests.

Six 100 mm cube samples of grout were taken on a daily basis and strength tests were performed at 7 days and 28 days.

Since the proposed site is very large and the soil conditions vary abruptly, additional confirmatory jet grout columns were also installed before the commencement of some of the structures in order to confirm the jet grouting parameters, complying with the required nominal diameter of 800mm. Those test columns were excavated for visual inspection. They were also cored, and unconfined compression tests were performed on selected jet grout core samples.

Core samples were taken from the heads of jet grout columns, approximately one every 200 columns, which is described to be equivalent to a sample for every five slabs of 16 m x 20 m. Unconfined compression tests were performed on the core samples taken from the jet grout columns; q_u values of 3.5MPa to 25MPa were observed, with an average of 5.85MPa. In Table 2.9 are the results of uniaxial compression tests carried out on jet grout samples taken from the head of the columns given. [30]"

Table 2.9 Results of Uniaxial Compression Tests Carried Out on Jet Grout Samples Recovered From The Head of The Columns [30]

Building Name	No. of Core Samples	Type of Soil	Ave. Compressive Strength q_u (MPa)
New & Old Body Shop	26	Sandy Silt	5.8
Water House & Transformers	7	Sandy Silt	5.4
Switch Center	12	Sandy Silt	6.1
Paint Shop	6	Silt	5.4
Paint Shop	4	Silt	5.4
Paint Shop	6	Clayey Silt	5.3
Other Buildings	20	Sandy Silt	6.3

“In addition, continuous core samples were taken one in every 1000 jet grout columns, or one in every structure. Unconfined compression tests were performed on the core samples taken from the jet grout columns in order to investigate the variations of strength characteristics and densities of column with depth. The test results indicated the q_u values ranging from 2.6 MPa to 40.6 MPa. Corresponding sample densities of 15.2 kN/m^3 and 21.5 kN/m^3 were measured.

Also three deformation controlled uniaxial compression tests were carried out on core samples, to measure the modulus of elasticity (E_s) of the column. The cores were taken from jet grout columns of different structures in the complex, which were installed after the earthquake. The test results indicated values of modulus of elasticity (E_s) ranging from 2500 MPa to 3850 MPa. For the same samples, densities of 17.2 kN/m^3 and 20.3 kN/m^3 were measured. [30]”

2.8 Applications of Jet Grouting

“In addition to the key advantage over other grouting methods of being able to deal with a wide range of soils, jet grouting allows more accurate assessment in advance of grout quantities, volume of ground influenced and the properties of the treated ground. It has successfully been applied for both temporary and permanent functions in a wide range of applications, particularly for soil improvement cases like soil stabilization, excavation support systems, structural underpinning, seepage barrier/cutoff walls, soil consolidation for the excavation of shallow tunnels, environmental remediation or groundwater control. [14]” Fig. 2.26 shows the range of jet grouting applications.

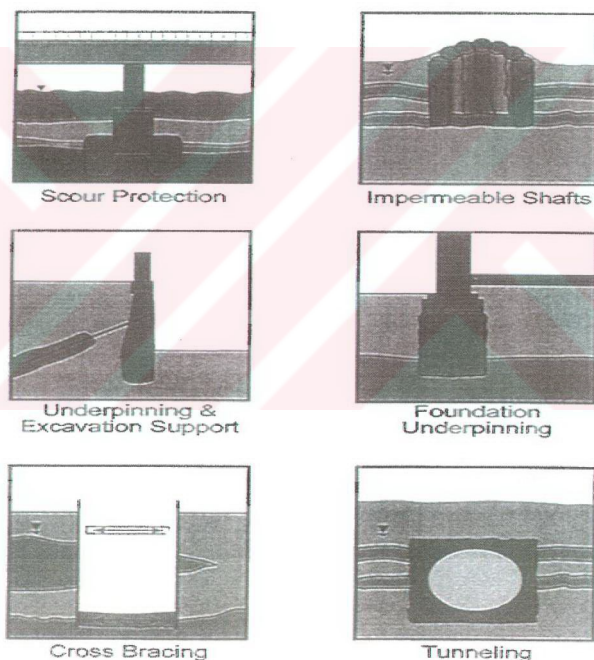


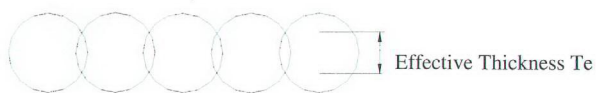
Fig. 2.26 Jet Grouting Applications

2.8.1 Groundwater Control

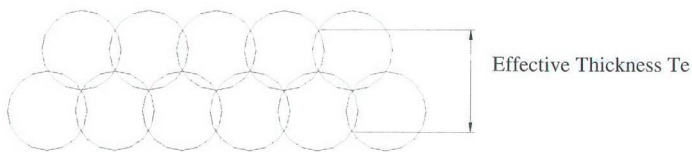
“Groundwater control applications vary from reducing inflows in temporary excavations, in which a rudimentary barrier is required, to permanent cut – off curtains in dams, for which low mass permeability and durability are relevant.

2.8.1.1 Cut – Off Curtains

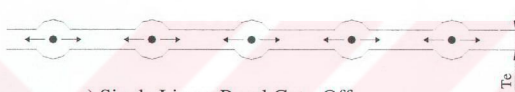
“Cut – off curtains can be formed by overlapping columns, panels or wings of soil treated using jet grout (Fig. 2.27). These can be installed in inclined planes as well as vertical and it is not necessary to take the treated zone all the way to the surface. The configuration depends on requirements. A thicker wall using columns may be needed when stability is required as well as sealing, where as a simple panel cut – off may be more appropriate when only water flows need to be reduced. The depth of the wall is also a consideration, since tolerances on treated volumes and drilling axes may dictate one type. Cut – off curtains formed using jet grouting have been employed in new dams, in remedial work for existing dams, in landfill leachate isolation, in preventing waterflow into excavations or tunnels, and in river or canal training and remedial works. Fig 2.28 shows the applications of jet-grouted cut – off curtains. [14]”



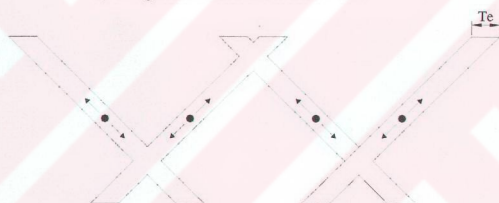
a) Single Row Cut - Off



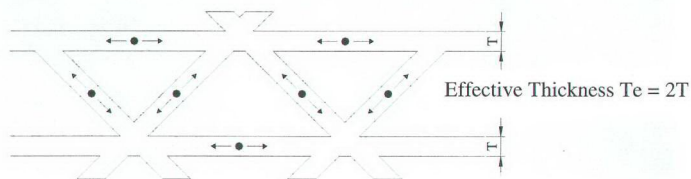
b) Double Row Cut - Off



c) Single Linear Panel Cut - Off



d) Single Overlap Panel Cut - Off



e) Double Panel Cellular Cut - Off

Fig. 2.27 Typical Column Panel and Layouts [14]

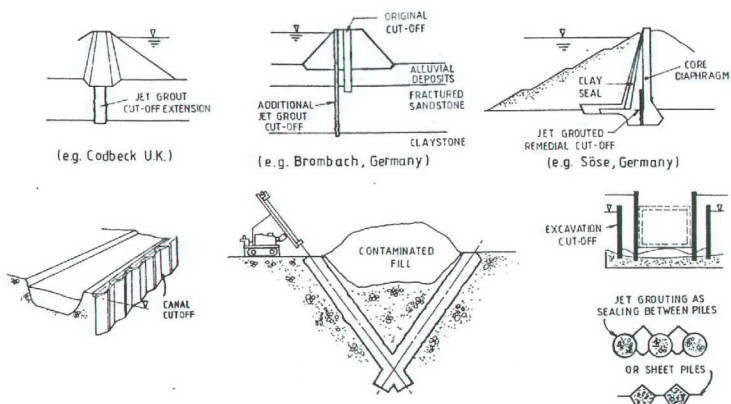


Fig. 2.28 Application of Jet Grouted Cut – Off Curtains [14]

2.8.1.2 Jet Grouted Base Seals

“Base seals constructed via jet grouting are finding wide place in the world, because of the fast and easy application profits. “Jet grouted base seals concerning relatively large areas have also found wide application (Fig 2.29). A typical column area layout for a base seal is shown in Fig 2.30. A minimum thickness of 0.5m is suggested for temporary excavations, and this can be increased with the closing columns as shown to ensure the integrity of the seal. Jet grouting enables construction of base slabs before excavation commences. These can be connected directly to the surrounding excavation walls (sheet piles, secant piles, diaphragm walls, etc.) to form an effective seal because the erosive jets locally clean the walls before or during grout placement. Obstructions can be encapsulated within the jet grout slab and may not require to be removed. Other applications include their use as temporary props in excavations and beneath landfills to limit migration of toxic leakages or radioactive contaminants. In these applications, much greater slab thickness than 0.5m will be necessary. [14]”

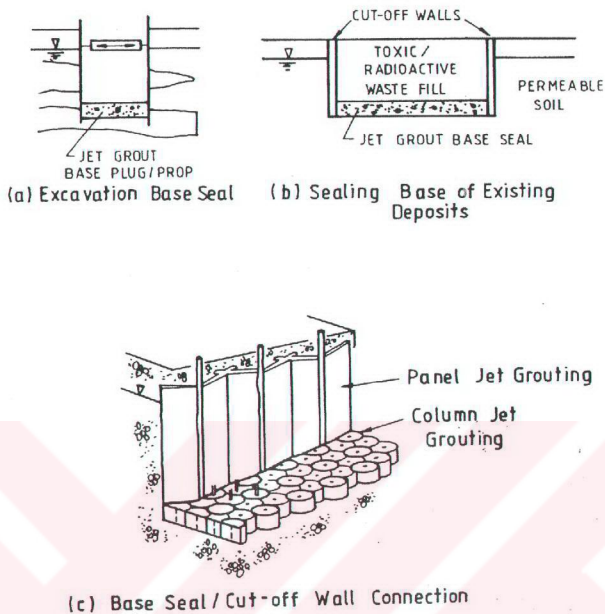


Fig. 2.29 Combined Wall and Base Seal Formed by Jet Grouting

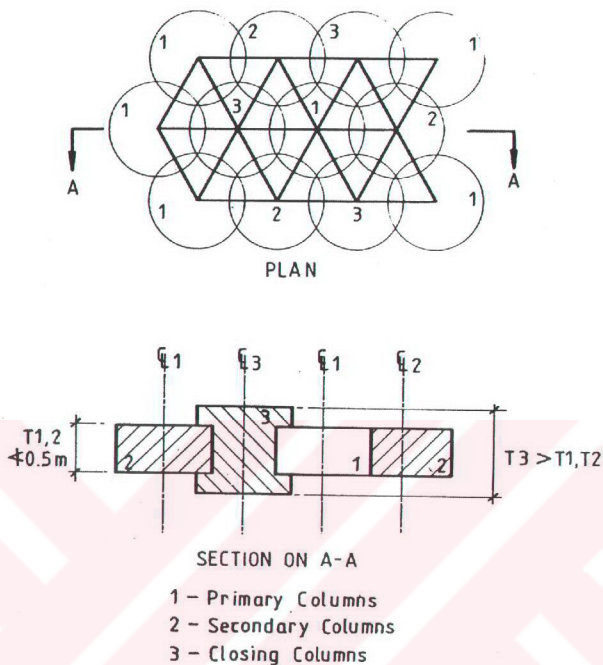


Fig. 2.30 Plan and Section for Jet Grout Bottom Seal [14]

The provisions of an effective mean of earth retention or barrier to water inflow has long been a problem in the vicinity of service conduits (gas, water, electricity, etc.) in major excavations. Jet grouting easily seals the necessary windows in the excavation support walls formed from diaphragm walls or piling.

2.8.2 Underpinning

Underpinning of buildings and structures using jet grouting has found wide application owing to its geometric flexibility, relative independence from soil variability and ease of access without the need to expose existing foundations. Strengths of treated ground are such that substantial loads can be carried. Access is gained to the required treatment zone by inclined drilling, where possible, through the lower exterior walls and foundations to ensure a good foundation – grout bond (Fig 2.31a). Ensuring that drilling is deep enough assists uniform transfer of load to deeper component strata. Underpinnings by jet grouting can also perform by drilling

from within the buildings in distress or inside tunnels and beneath bridge docks in headroom as low as 3m (Fig 2.31b).

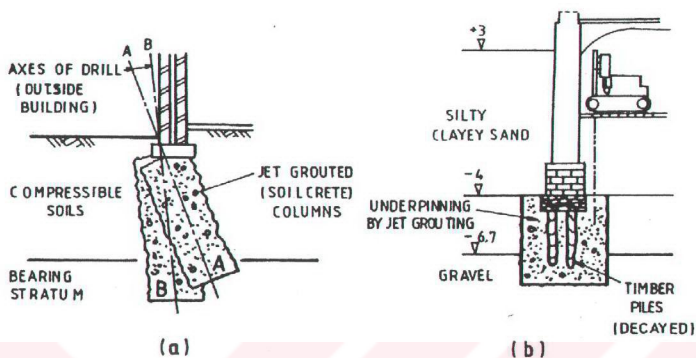


Fig. 2.31 Underpinning of Foundation Using Jet Grout Columns [14]

“Jet grouting has also been used in underpinning old timber – piled foundations. These can settle after many years of adequate performance due to the tendency for timber, located above the permanent groundwater level, to rot with time. Depending on the layout, jet grouting can often form an excellent structural connection between the underside of the pile cap and competent strata at depth without the need to remove the rotten piles. These become encapsulated in the jet grout mass. Similarly most existing service pipes can be encapsulated within the treatment zone without undue disturbance.

Jet grouting has found special application in the construction of basements or other large excavations adjacent to existing buildings or structures. The treated soils function not only as an underpinning support but also as temporary or permanent earth and/or water retention (Fig 2.32). The required lateral support in this type of application may necessitate ground anchors installed directly through the jet grouted mass. The number and spacing of the anchors should be determined by stability calculations, with anchor loads generally specified up to 400 kN. Wallings are not usually required as the treated ground is sufficiently strong to allow stressing of the anchors against plates, 250mm to 400mm square. [14]”

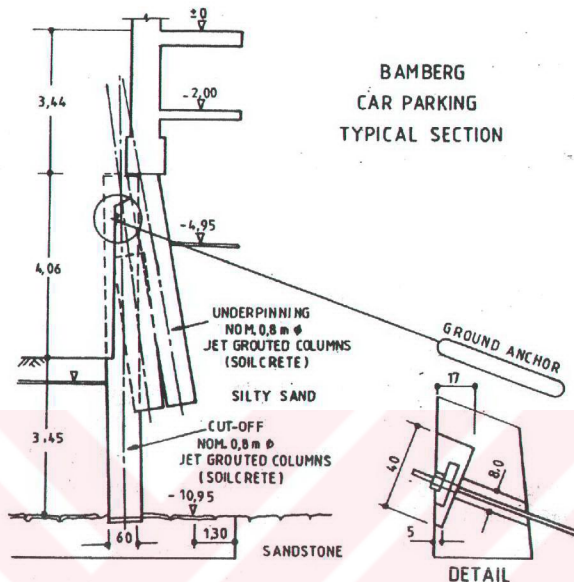


Fig. 2.32 Use of Jet Grouting to Underpin, Provide Excavation Support, and Control Groundwater [14]

Before employing jet grouting for underpinning, it has to be clarified; that the condition of the building or structure and the capacity of its foundations to span temporarily across freshly treated ground needs to be established. Also care has to be taken that columns small in diameter are employed, even when the system being used is able to construct much larger columns. This reduces the volume of unsupported ground during each lift. The columns should be designed to interlock together to form a mass of treated ground and minimize the effects of local possibly unrecognized, variations to the soil. The cement content should be matched to the ground conditions to ensure that the in situ mix will provide sufficient strength. The construction sequence should ensure that columns have developed sufficient strength to be uninfluenced by the formation of any adjacent column. Weakening of column strength, or reduction in size must be allowed for if the treatment zone passes through soils with silt content exceeding 15%, or any clay. The erosive jet cleans the underside of the existing foundation to enable a good connection with the grouted ground. To ensure contact, wet grout levels must be maintained above the foundation

base level until set occurs. It is essential that the foundations be monitored during all drilling and jetting operations. This can be done by optical leveling and provides the earliest indications of heave or settlement exceeding tolerances [14].

2.8.3 Tunnels and Shafts

Tunnel drives cover large distances and often a significant range in elevation during which the ground conditions are likely to vary quite substantially. Consequently, jet grouting is a particularly relevant form of treatment as a result of its ease of access and ability to treat a wide range of soils. It leaves no potential obstacles or hindrances behind, for instance in the form of pipes, lost points, etc. It has been used to facilitate tunneling in difficult ground, to minimize the effects of subsidence caused by tunneling, and in remedial works to existing tunnels.

To enable the safe passage of tunnels at depth, adjacent structures and buildings can be underpinned. Underpinning can also be applied to tunnels themselves. In some cases, this has been used to ensure existing tunnels in use are not damaged during nearby passage of new tunnels. In other cases underpinning has been used to prevent further settlement damage or distortion to existing tunnels, which have developed foundation problems after many years of use. Treatment can be achieved by drilling from the surface, or from within the tunnel as appropriate.

The formation of archers or vaults of grouted ground above the tunnel roof can achieve protection against the effects of tunneling subsidence. These can be formed in a variety of ways (Fig 2.33). Larger blocks of ground can be similarly formed to encompass the complete tunnel cross – section, or that part in unstable ground. This not only minimizes ground movements due to tunneling but also enables excavation to proceed using the chosen method through unsuitable or water – bearing conditions (Fig 2.33). Relatively large blocks of treated ground are needed in these cases and generally the Jet 2 or Jet 3 systems are used to form the maximum jet grout column sizes appropriate for the conditions, often in excess of 1.2m diameter. Strengths need to be high enough to stabilize the ground and low enough to facilitate tunnel excavation, and are often in the range of 1 to 5 N/mm². Cut – off walls have also been employed to isolate buildings close to the tunnel line from the effects of dewatering to assist tunnel excavation.

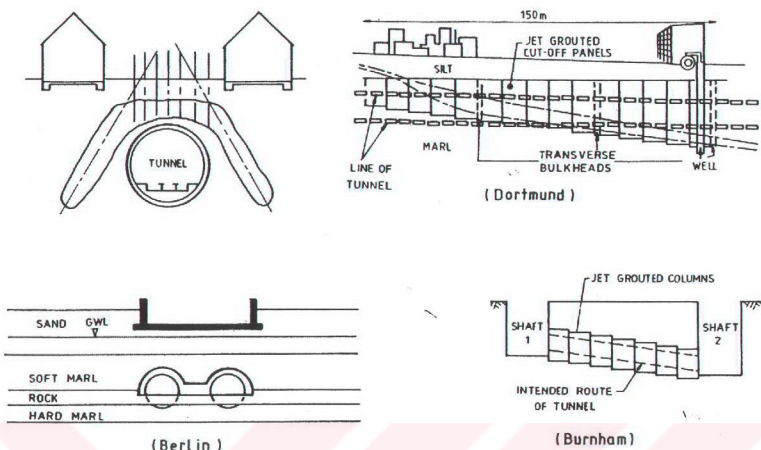


Fig. 2.33 Jet Grout Curtains or Umbrellas to Protect Tunnels [14]

When access from the surface is difficult as a result of steeply sloping ground or the presence of roads, railways or large buildings, it may be necessary to reach the zones to be stabilized from within the tunnel. This has been achieved by forming an arch using sub – horizontal jet grouted columns to distances of 10 – 15m ahead of the tunnel face (Fig 2.34). Once the arch is formed, tunnel excavation advances to within 2 – 3m of the end of the treated length, whence drilling and jetting a new arch commences. Excavation in the each section is immediately followed by the installation of steel supports and shotcreting. Fast setting columns, using the minimum of water injection and with the minimum of ground disturbance during jetting, dictate that small diameter columns, 0.4 to 0.7m, formed using the Jet 1 system, are appropriate [14].

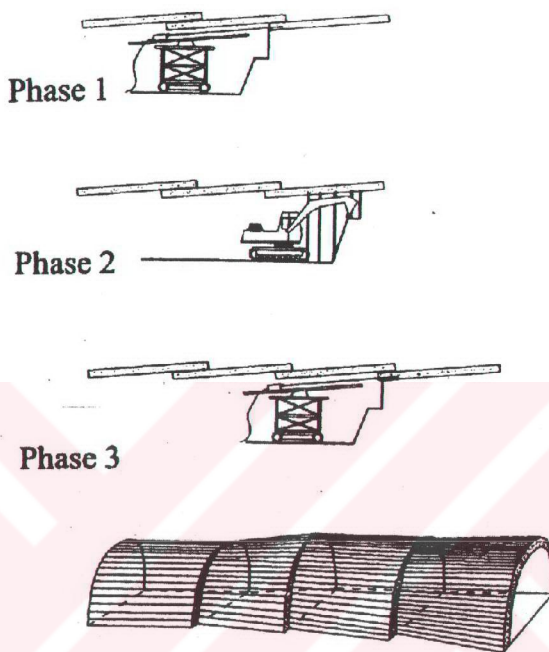
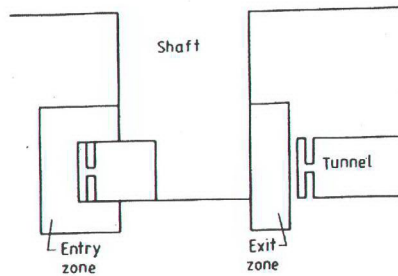


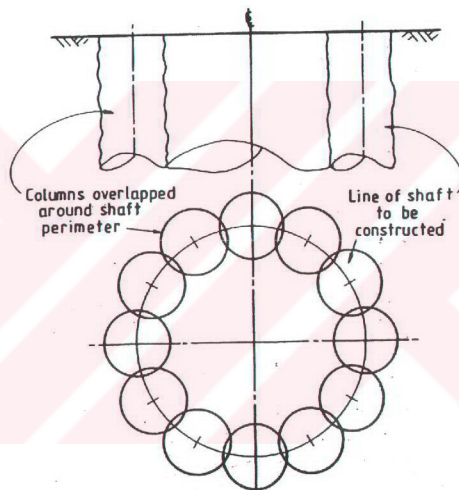
Fig. 2.34 Tunnel Stabilization Using Sub – Horizontal Jet Grout Columns [14]

“Jet grouting has also been useful at the start and finish of tunnel drives. Entrance blocks of treated ground ensure that transfer from access shaft to tunnel drive and the commencement of tunneling occurs in known conditions. Jet grouted thrust blocks have also been employed to provide sufficient reaction for machine starts. Similarly, reception blocks of treated ground adjacent to transit or retrieval shafts allow breakthrough in safe conditions of tunnel machines or men in compressed air conditions.

Shafts also can be effectively stabilized using jet grouting. Base sealing slabs can be employed to prevent piping in cohesionless soils and with increased thickness also to prevent base heave or even act as props. Indeed, jet grouting has frequently been used to maintain stability, or minimize groundwater ingress, during shaft construction or as a permanent measure. This is usually achieved by collar of intersecting columns (Fig 2.35).



a) TUNNEL ENTRY / EXIT TREATMENT



b) SHAFT STABILIZATION

Fig. 2.35 Shaft Stabilization and Tunnel Entry and Exit Blocks [14]

3 SOIL IMPROVEMENT PROJECT FOR THE GENERAL DIRECTORATE BUILDING OF KEDAŞ IN IZMIT

3.1 Introduction

Jet grouting technology, is one of the most common soil improvement techniques in order to control and limit excessive settlements of the foundations and in some cases to reduce the risk of liquefaction. It is also suitable for installation inside an existing building or structure, which was the matter on the application for the improvement of the foundation soil of the General Directorate Building of KEDAŞ in Izmit. The building had experienced differential excessive settlements due to poor soil conditions.

The building covers a plan area of 36.40m x 14.50m and consists of eight storeys (ground floor + 7 floors) in total.

Also, asymmetrically loading conditions of the building were an important factor affecting the occurrence of differential settlements. A shelter with 60 cm thick reinforced concrete wall was placed on one side of the ground floor, causing a significant difference on the transmitted loads to the foundation.

The settlements started during construction of this building and became obvious after the Marmara Earthquake on 17 August 1999.

The present situation was also evaluated as an important case, because it was obvious that further settlements would take place with the start of use of the building. It should be noticed also, that KEDAŞ's archive, which is consisting of thousands of folders and the office furniture that was going to be placed inside this building, would bring extra loads to the system.

Two borings and two CPT tests with 32 m depth were carried out at the site for soil investigation purposes. Evaluating all these considerations and the results of the site investigation, a necessity for soil improvement was evident. Because of its speed, installation ability from inside the building and suitability for present soil conditions, the jet grouting technique was decided as the improvement technique. Prior to the

construction of jet grout columns, the differential settlements were decreased with a preloading of 40kPa on the south side of the building.

Finally, soil improvement has been carried out with 101 jet grout columns of each 20m lengths. The content of the application was consisting of 93 vertical columns, placed inside the building, and 18 five degrees angled columns, around the building.

At the end of the application, laboratory tests that were conducted on cores recovered from jet grout columns have shown that the goal of 1.5kN/cm² has exceeded and the columns have proven their quality. In average the columns had 20-day uniaxial compression strength of 7.5kN/cm² and 28-day strength of 80kg/cm².

3.2 General Status of Building

The General Directorate Building of KEDAŞ is on the north of the E – 5 highway in İzmit, Kocaeli. On the opposite side of the building is the Hafız Selim Street. On the east passes Hafız Binbaşı Street, also having the court building placed on the across side. The General Directorate Building has a plan area of 466m² and consists of ground floor + 7 storeys (fig 3.1). A mat foundation with 60 cm thickness exists beneath the building. The main problem of the building was differential settlements within a range of 30 cm, which is quite much further than the tolerable values.



Fig. 3.1 General Directorate Building of KEDAS

3.3 Geotechnical Investigations and Establishments

“After the Marmara Earthquake on 17 August 1999, Izmit Municipality conducted borings for soil investigation on different locations in the city. Two of these borings (S21, S63) and one CPT test were at the nearby of the KEDAS building. Also two additional CPT tests with 32 m depth were carried out on 04/05/2002 as a part of the site investigation. By means of these tests, necessary evaluations have been made and a soil profile is obtained. On the given site plan (Fig 3.2) are the locations of the borings and CPT tests are shown. Boring logs and CPT logs are appended to this thesis.” [31]

3.3.1 Borings

As mentioned, the Izmit Municipality has carried out two borings with numbers S 21 and S 63 at the site.

3.3.1.1 Boring S63

“The boring Si – S63 was carried out on the northeast corner of the building at elevation +2.50m. Passing a fill layer of a thickness of 3.20m, a desiccated green –

brown silty clay layer of 0.9m thickness was obtained. Boring followed on a medium stiff silty clay layer of 8.2m thickness and SPT blow counts $N_{30} = 4 - 6$. Between 12.50 – 16.00m depths a soft consistence silty clay layer is encountered, which has $N_{30} = 2$ SPT blow counts. Finally the boring terminates in a medium stiff silty clay layer with SPT blow count values of $N_{30} = 6 - 8$, on 19.95m depth.”[31]

3.3.1.2 Boring S21

The boring Si – S21 was carried out on the west of the building at elevation +1.89m. Below a fill layer of 2.20m thicknesses, a silty – clay with soft consistency and SPT blow count values of $N_{30} = 2 - 4$ were obtained. The boring terminated at 19.95m depth. [31]

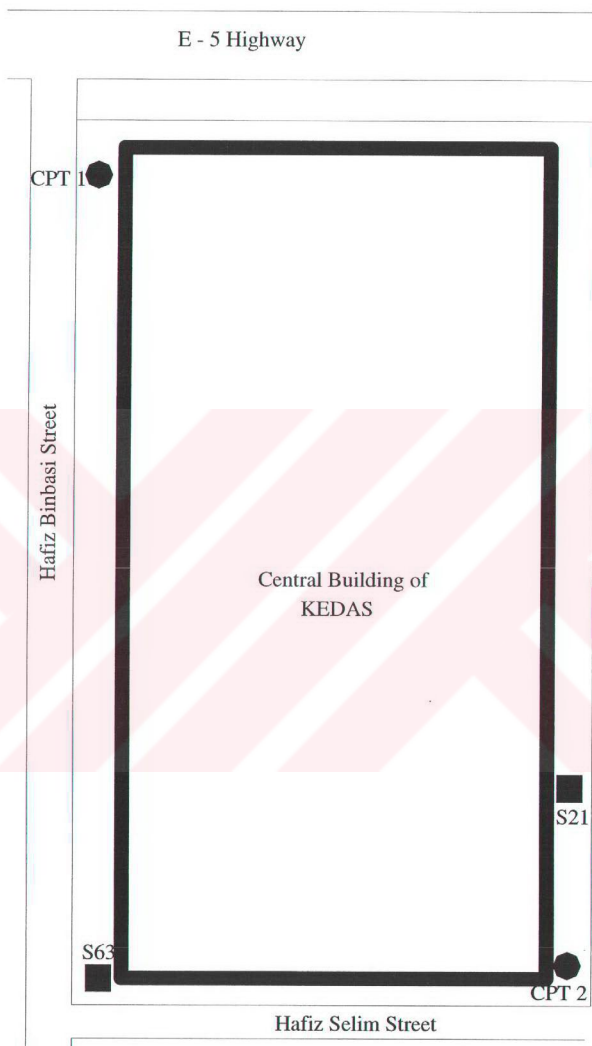


Fig. 3.2 Site Plan

3.3.2 CPT Tests

On the coverage of this work two CPT tests have been carried out at the investigation site on 04/05/2002. The locations of the CPT tests are shown on the plan in Fig 3.2 and both of them are 32m deep. Based on the boring data, a preliminary boring of 2.50m for both CPT tests, with a drill rig, has been made to pass the fill layer. Such kind of predrilling was necessary, as the Conic Penetrometer can only move ahead in soft soil conditions.

3.3.2.1 CPT 1

“The southeast corner of the building was the suggested location for this test. Following the 2.50m thick fill layer, a clay layer was encountered with a measured cone resistance of $q_c = 0.60$ MPa and a sleeve friction of $f_s = 0.023$ MPa, which ends at 5.25m depth. The following layer encountered was a loose, 50 cm thick silt – sand layer with measured cone resistance $q_c = 2.2$ MPa and sleeve friction $f_s = 0.027$ MPa, below which lies a sandy silt – silt layer with a thickness of 3.0m. The measured cone resistance and sleeve friction values of this layer are $q_c = 1.15$ MPa and $f_s = 0.019$ MPa. Between 8.75m and 28.00m depths is a silty clay layer, which has soft consistency in the upper parts and gaining medium consistency with depth. While the encountered cone resistance and sleeve friction values of this layer in the upper part are $q_c = 0.70$ MPa, $f_s = 0.027$ MPa, the deeper parts have values of $q_c = 1.00$ MPa, $f_s = 0.028$ MPa. The CPT test is finalized at 32.00m depth in medium dense silty sand layer.” [31]

3.3.2.2 CPT 2

“The second CPT test was performed on the northwest corner of the building. Taking into account that the soil layers encountered in CPT 2 test are weaker than the ones from the first test, it will be not false to say that nearly the same is encountered. Also the CPT 2 test is finalized at 32.00m depth.” [31]

3.4 Engineering Properties of Soil Units and Soil Profile

An idealized soil profile is obtained based on the two borings and CPT tests. This is appended to the thesis. The parameters of the layers constituting the soil profile are explained below.

3.4.1 Fill Layer

On all borings and CPT tests a fill layer with a thickness varying between 2.20m and 3.20m has been encountered. The parameters given in Table 3.1 may be accepted for this unit.

Table 3.1 Engineering Parameters of Fill Layer [31]

Natural Unit Weight	$\gamma_n = 19.0 \text{ kN/m}^3$
Deformation Modulus	$E_s = 10.0 \text{ MPa}$
Friction Angle	$\phi = 30^\circ$
Coefficient of Vertical Subgrade Reaction	$k_h = 1500 \text{ t/m}^3 = 15000 \text{ kN/m}^3$
Coefficient of Horizontal Subgrade Reaction	$k_v = 3000 \text{ t/m}^3 = 30000 \text{ kN/m}^3$

3.4.2 Silty Clay – Clayey Silt Layer

Under the fill layer, silty clay – clayey silt layer was encountered, interbedded with silty sand. Borings show that the observed silty clay layer accommodates shell in bands. SPT blow counts are higher in the deeper layers. Desiccation has caused a medium stiff state, in respect to the upper part of this layer. Nevertheless the SPT blow counts, which were recorded in this clay layer are, between $N_{30} = 2 - 4$. This unit with cohesion has a soft to medium stiff state with an increasing stiffness with depth. The following engineering parameters may be accepted for this layer:

Table 3.2 Engineering Parameters of Silty Clay – Clayey Silt [31]

	Soft Levels	Medium Stiff Levels
Natural Unit Weight	$\gamma_n = 17.0 \text{ kN/m}^3$	$\gamma_n = 18.0 \text{ kN/m}^3$
Average SPT Blow	$N_{30} = 2$	$N_{30} = 6$
Deformation Modulus	$M_c = 10.0 \text{ MPa}$	$M_c = 10.0 \text{ MPa}$
Cohesion	$c_u = 30 - 40 \text{ kPa}$	$c_u = 60 \text{ kPa}$
CPT Cone Resistance	$q_c = 0.60 - 0.70 \text{ MPa}$	$q_c = 1.00 \text{ MPa}$
CPT Sleeve Friction	$f_s = 0.022 \text{ MPa}$	$f_s = 0.025 \text{ MPa}$
Coeff. of Vertical Subgrade Reaction	$k_h = 750 \text{ t/m}^3 = 7500 \text{ kN/m}^3$	$k_v = 1250 \text{ t/m}^3 = 12500 \text{ kN/m}^3$
Coeff. of Horizontal Subgrade Reaction	$k_v = 1500 \text{ t/m}^3 = 15000 \text{ kN/m}^3$	$k_v = 2500 \text{ t/m}^3 = 25000 \text{ kN/m}^3$

3.4.3 Silty Sand Layer

As mentioned before a silty sand layer has been observed, which is interbedded with the silty clay – clayey silt layers. The parameters for loose to medium dense placed silty sand layer are shown in Table 3.3.

Table 3.3 Engineering Parameters of Silty Sand [31]

	Loose Levels	Medium Dense Levels
Natural Unit Weight	$\gamma_n = 18.0 \text{ kN/m}^3$	$\gamma_n = 19.0 \text{ kN/m}^3$
Deformation Modulus	$E_s = 5.0 \text{ MPa}$	$E_s = 5.0 \text{ MPa}$
Friction Angle	$\phi = 29^\circ - 30^\circ$	$\phi = 32^\circ$
CPT Cone Resistance	$q_c = 0.60 - 0.70 \text{ MPa}$	$q_c = 1.00 \text{ MPa}$
CPT Sleeve Friction	$f_s = 0.022 \text{ MPa}$	$f_s = 0.025 \text{ MPa}$
Coeff. of Vertical Subgrade Reaction	$k_h = 750 \text{ t/m}^3 = 7500 \text{ kN/m}^3$	$k_v = 1250 \text{ t/m}^3 = 12500 \text{ kN/m}^3$
Coeff. of Horizontal Subgrade Reaction	$k_v = 1500 \text{ t/m}^3 = 15000 \text{ kN/m}^3$	$k_v = 2500 \text{ t/m}^3 = 25000 \text{ kN/m}^3$

3.5 Geotechnical Evaluation

The General Directorate Building of KEDAŞ, located on the E – 5 Highway in İzmit, Kocaeli, has a base area of 466m² and constitutes of a ground floor and 7 floors. The building rests on a mat foundation with 60cm thickness. The building was constructed before Marmara Earthquake on 17 August 1999, and settlements up to 30cm's has occurred on the northwest side of the building. These settlements started with the construction of the building and became visible after the earthquake. Evaluation of the two borings, which were carried out by the municipality in 2000 and CPT tests have shown that the soil profile constitutes of soft clay and loose sand layers. Furthermore, it has been determined that the building rests over highly compressible layers. The mat foundation, which was placed on highly compressible layers without any soil improvement or deep foundation, was also incapable to resist the excessive settlements. [31]

3.5.1 Allowable Bearing Pressure and Foundation System

The General Directorate Building has a mat foundation system with a thickness of 60cm. Prior to the mat construction; a 2.0m thick layer of the uncontrolled fill had been placed as a controlled fill. The pressures transferred to the soil from an 8-storey building resting on 60cm thick mat foundation can be calculated as followed:

$$\sigma = 60\text{cm} \times 24\text{kN/m}^3 + 8\text{floor} \times 10\text{kN/m}^2 = 94.4\text{kN/m}^3 \cong 100\text{kN/m}^2$$

According to the projects, a shelter was constructed on the north side of the building with concrete walls of 60cm thickness as high as the ground floor. These shelter walls increases the pressure acting to soil by 18.4kPa. So the north side of the building transfers 118.4kPa pressure to the ground, where as this pressure is only 100kPa for the south side of the building.

The allowable bearing capacity of the Izmit Bay alluviums, on which the subjected building rests, is controlled by the soft clay layer between 2.50m and 5.25m depth and by the loose silty sand layer, which has a liquefaction risk.

A liquefaction analysis has been carried out with the corrected SPT values for sand layers and is appended to this thesis. The SPT blow counts used in this analysis and the results of this liquefaction analysis are summarized in Table 3.4.

Table 3.4 Results of Liquefaction Analysis

Layer	Soil Type:	γ_n (kN/m ³):	GWT(m)	N ₃₀	N'	Earthquake Magnitude (M)	ts/t ₀ :	End Condition:
1	CLAY	17,00	3	2		7	>1	No liquefaction
2	CLAY	17,00		2		7	>1	No liquefaction
3	SAND	18,00		4	5	7	0,18	Liquefaction
4	SAND	18,00		2	2	7	0,15	Liquefaction
5	CLAY	17,00		5		7	>1	No liquefaction
6	CLAY	17,00		5		7	>1	No liquefaction
7	CLAY	17,00		5		7	>1	No liquefaction
8	CLAY	17,00		5		7	>1	No liquefaction
9	CLAY	17,00		5		7	>1	No liquefaction
10	SILT	18,00		5		7	>1	No liquefaction

"The soil improvement, within 2.0m depth, was insufficient to provide the consolidation of these above mentioned layers, which have caused differential settlements under the building. The allowable bearing capacity of the present state is:

$$q_{ult} = 5.53su + \gamma D_f = 5.53 \times 40 + 18 \times 1.20 = 221 + 21.6 = 242.6\text{kPa}$$

With a factor of safety of $F_s = 3$;

$$q_{all} = 242.6 / 3 \cong 80\text{kPa}.$$

The calculations show that the allowable bearing capacity of this soil profile is not enough to cover the base pressures transferred from the building. Regarding to the calculations above, the failure was to construct the building over a mat foundation,

which should have had a deep foundation system with piles. In addition to this, no settlement calculations had been made although the subsoil was Izmit Bay alluviums with high compressibility. [31]

3.5.2 Settlement of the Mat Foundation

In the report by Prof. Dr. Ahmet Saglamer dated May 2002, settlement analysis was carried out using UNISETTLE software program, which is appended to this thesis. Also a secondary analysis is appended, which is made after the “Stress Distribution Formula for Point Loading” of Westergaard. In both analyses is the north part of the foundation transfers 118,4 kPa pressure to the soil, against 100 kPa pressure in the south part. Fig 3.3 shows the four points, which are selected for the settlement analyses.

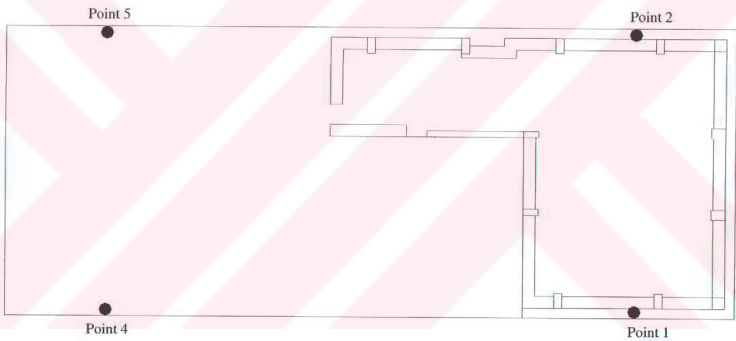


Fig. 3.3 Plan of the Mat Foundation Showing the Points, on which the Settlement Analyses have been made.

On the settlement calculations using the “Stress Distribution Formula of Westergaard”, the pressures acting from the building are transformed to point loads. For this purpose, 2 areas, in the shelter side, where the acting pressure is 118,4kPa (North side) and 2 areas in the South side of the building has been selected. The formula, which has been used, is as follows:

$$\Delta p = Q / ((z^2 \pi) (1/(1+2(r/z)^2))^{3/2}) \quad \text{Equation 3.1}$$

The pressures acting to these areas have been taken on the calculations as point loads located at the centers of these selected areas. These areas are shown on Fig. 3.4.



Fig. 3.4 Pressure Areas 1,2,3 and 4

After the results of the calculations using Westergaard's formula, settlements between 250mm and 340mm are predicted.

"The settlements calculated under the General Directorate Building, vary between 318mm and 410mm, after the results of UNISETTLE." [31] The results of both analyses are summarized in Table 3.5

Table 3.5 Results of Settlement Analyses

Point	Settlement (mm)	
	Unisettle	Point Load Analysis
1	371	340
2	410	290
4	318	250
5	349	250

The calculations made with UNISETTLE are corresponding with the measured settlement values determined by geodetic methods. A surveying company named Denge made the measurements on 26/06/2000.

The analyses with UNISETTLE give proper results than the point load analysis. The reliability of UNISETTLE can also be verified in that; the UNISETTLE results correspond with the measured values.

As mentioned before, the existence of soft clay layers, over which the building rests, are the main cause of settlements. Because of the weaker soil state on the northwest corner, the building has settled more on this corner and on the Marmara Earthquake on 17 August 1999 even increased this differential settlement. The addition of the asymmetrical loading condition was enlarging the settlements.

The use of the Shelter as archive, after the earthquake, even increased the differential settlements by causing an additional uniform load of 10kPa.

The differential settlement is the source for the rigid angular rotation that had occurred on the mat foundation of the building. The value of this rotation is now at $\delta = 0,3\text{m} / 31,40\text{m} \cong 1/120$. As it known, angular rotation of $\delta = 1/100$ will cause structural failures on the concrete framework of the building.

After the report of Prof. Dr. Ahmet Sağlam, a concluding assessment is made that the building is overloaded by 30% at the northwest corner. It is also stated that the building will achieve a much worse condition with the start of its use, since additional dynamic and static loads will come into the system. [31]

3.5.3 The Precautions, Taken Against the Continuing Settlements

Against the above-described state of the General Directorate Building of KEDAŞ, Prof. Dr. Ahmet Sağlam has recommended a soil improvement application with jet grout columns. Prior to the construction of jet grout columns, the range of the differential settlement had been decreased with a preloading of 40kPa on the south side of the building. After the settlement analyses carried out, a recover of 8cm is achievable with preloading. The results of the analyses, compared with the former settlement analyses carried out with UNISETTLE are given in Table 3.6.

Table 3.6 Results Showing the Effect of Preloading [31]

Point	Settlement (mm)		B – A (mm)
	Present State (A)	After Preloading (B)	
4	318	399	81mm
5	349	428	79mm

The location of the preloading is shown on Fig. 3.5.

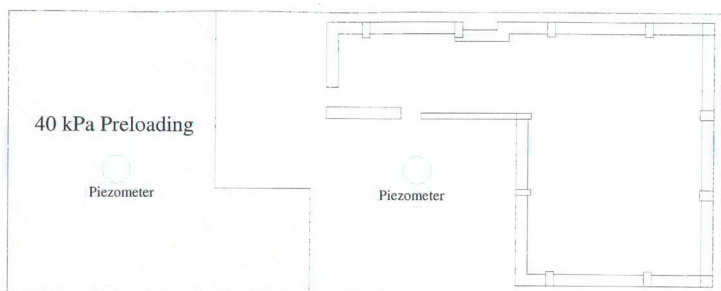


Fig. 3.5 Plan Showing Preloading Area [31]

Preloading has been carried out in stages, and only after the preceding stage has reached ultimate primary settlement, the following stage is applied. Following this concept, loading steps of 20kPa was applied to the building. The first step was loaded on 18/07/2002, while the final step was applied on 19/09/2002.

In conclusion, preloading has aimed to bring the building in balance, so the major improvement application could be implemented. This improvement was intending the prevention of liquefaction and to end further settlements, which would have find place after the management moves to the building.

Two piezometers have been placed inside the building, to monitor the changes of the pore water pressure. Therefore was it possible to monitor the present situation of the consolidating soil layer. The location of the piezometers is shown in Fig. 3.6.

Although after a duration of four months, only 2 – 3cm settlement occurred related to preloading, which was much lower than the calculated settlement value. So it has been decided to continue with the construction of the jet grout columns. [31]

The change in the pore water pressures can be seen on Fig 3.4.4. This graph shows also the recorded data, after the jet grouting application. The date of the application can clearly be seen from the huge increase of the pore water pressures. The table containing all monitoring data from piezometers is appended to this thesis. Unfortunately, the data collected via the piezometers was reliable, by estimating the consolidation stage.

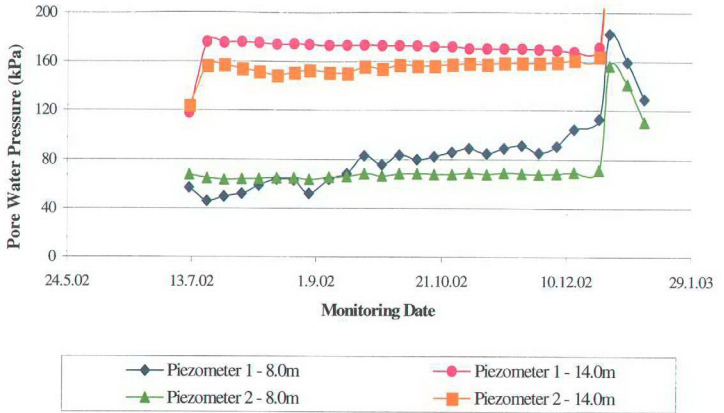


Fig. 3.6 Graphic Due to the Change in Pore Water Pressures

Jet grout columns were able to prevent further settlements and a possible liquefaction occurrence, according to the liquefaction analysis appended to this thesis. Soil improvement was planned to carry out, with jet grout columns of 80cm in diameter, under the mat foundation with a thickness of 60cm. The columns were planned to be constructed in a length of 20m. 100 jet grout columns were considered for the improvement project.

Retrofilling of the building was the second step after soil improvement. The first goal of this project was to balance the loads acting on soil from the building. So it has been considered to build a reinforced concrete wall of 40cm thickness to the opposite side of the shelter. Although this wall would be helpful for a better-arranged load distribution, it would cause a total collapse if it were constructed before the soil conditions are improved. In summary, one of the affecting points, starting the jet grouting application, was the plan of this wall construction.

3.6 Jet Grouting Application

3.6.1 Design, Application and Properties of the Constructed Columns

The soil improvement application of the General Directorate Building of KEDAŞ is consisted of jet grout columns, which are expected to prevent further settlements and

any liquefaction to occur during an earthquake. Every column is planned to be 80cm in diameter and to have 20m lengths.

The mat foundation has a thickness of 60cm, between beams with 100x120 cm dimensions. A cross section of the foundation system is shown in Fig. 3.7 and a picture of the foundation is shown in Fig. 3.8.

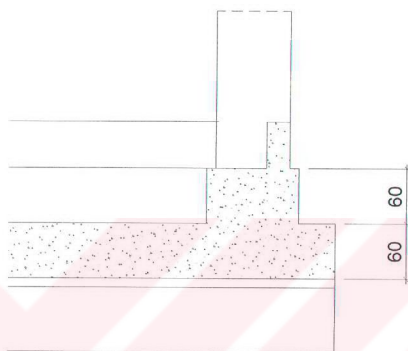


Fig. 3.7. Cross Section of the Foundation System

Taking the framework constituting of beams into account, the location of the jet grout columns has been selected so that no boring was carried out in these beams. As it can be seen from the applied jet grout design appended to this thesis, total 111 jet grout columns have been constructed. 93 of these columns were inside the building. The last 18 jet grouts were added almost at the end of the execution and they have been constructed from outside right from the perimeter walls, inclined 5° from vertical, so that the columns end beneath the foundation. Every column had 20m lengths, which points out of a total production of 2220m jet grout columns.



Fig. 3.8 Foundation System of the General Directorate Building of KEDAŞ

3.6.2 The Jet Grouting Equipment Selected For This Application

During the jet grouting application at KEDAŞ building the Casagrande M3D type drill rig has been used. This drill rig had a drilling diameter of 10cm, to prevent disturbance on the foundation. The height of rig tower was restricted to 2.40m and was therefore suitable to work inside the building. Also, the width of the rig was restricted, so that it could pass through 90cm wide door spaces. The drill rig was able to work within 45 cm distances to wall borders (Fig 3.9). The rods were of 1.0m lengths, allowing to work in low heights (Fig 3.10). Because the necessity was to strengthen the soil in short time, the Jet 1 system was selected for the application. [20]



Fig. 3.9 Jet Grouting Drill Rig

The jet grouting application is implemented following the instructions of the technical specification guide, which has been prepared prior to the application.

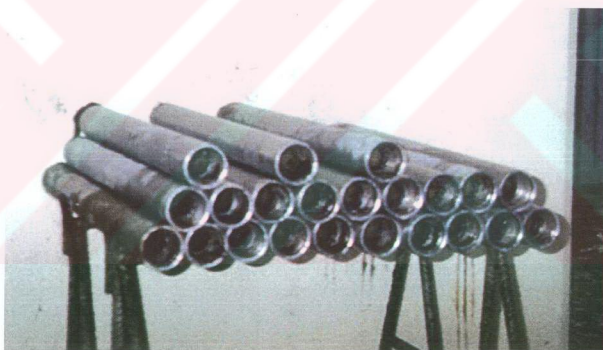


Fig. 3.10 Parts of Drilling Rig

After the specification guide, the pumping unit was capable to work up to 600-bar pressure, with a flow rate of 180 l/min. The manometer of this unit was expected to be in 10-bar sensitivity. The pumping unit of Techniwell complied with technical specs. The pumping, selected unit for this application, is shown on Fig.3.11.

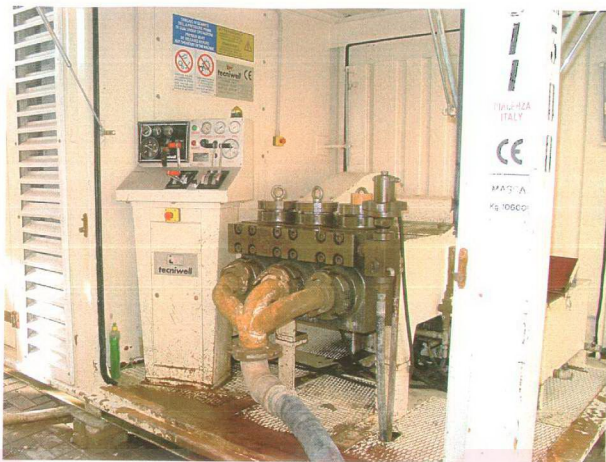


Fig. 3.11 Pumping Unit

An automatic mixing unit with weighing, mixing and resting ability was also required. The mixing unit had to be capable to feed the grout pump with minimum 15m^3 grout per hour. The selected mixing unit is shown on Fig 3.12.



Fig. 3.12 Mixing Unit

After the equipment was ratified and mobilized to the site, the test columns were constructed on 01.12.2002 for the selection of the suitable jet grouting parameters, to reach the specified column diameter.

3.6.3 Working Sequence and Limitations of Application

The application was carried out to comply with the instructions of the Specification. After these requirements, columns were prepared with a grout mixture of 1/1 W/C ratio and the Jet 1 system was used. The requirements for the mixture were PC 42.5 cement and clean water.

Other limitations are listed below:

- Because that the grouting rig had 1.0m long rods, the columns were constructed in stages. For the continuity of the columns, every grouting stage started with grouting 10 cm inside the already constructed column part.
- During the construction, the deviation of the drill rig was restricted in a range of 1/75 from vertical.
- Jet grout columns adjacent to each other with a distance less than 2.0m center-to-center spacing were constructed only with an interval of 24 hours.
- At the end of the application, a number of core specimens from the jet grout columns were recovered, which were subjected to unconfined compression tests.

The complete implementation was carried out under the control of two civil engineers, who have monitored the application, so that every regulation and production standard has been matched.

3.6.4 Test Columns and Application Parameters

“As mentioned before, prior to application on 01.12.2002, test columns had been constructed to establish grouting parameters. All of these columns were constructed with a grout constituting of 1/1 W/C (water / cement) ratio. For the construction of these columns a convenient area was selected in front of the building.

Three columns had been constructed with different parameters, which were considered to be able to produce the desired column. The specifications of these columns are summarized on Table 3.7.

Table 3.7 Jet Grouting Parameters of Test Columns

Test Column Nr:	Grouting Pressure (Bar)	Nozzle Diameter (mm)	Withdrawal Speed (Minute/m)
1	450	2.2	3.2
2	450	2.0	2.5
3	475	2.2	2.5

After duration of one week, the columns became ripe and have obtained their strength. The columns had been uncovered on 09.12.2002. Test Column Nr. 1 and Nr. 2 were 4.00m and Nr. 3 was 2.50m long. The reason for a shorter third test column was a concrete mass under its location, which couldn't be penetrated. A picture of test columns is given on Fig. 3.13.

Diameters of each column were measured and finally the parameters of Test Column Nr. 1 were selected as they were considered to be suitable to obtain the specific column properties. Test Column Nr.1 is shown on Fig. 3.14.

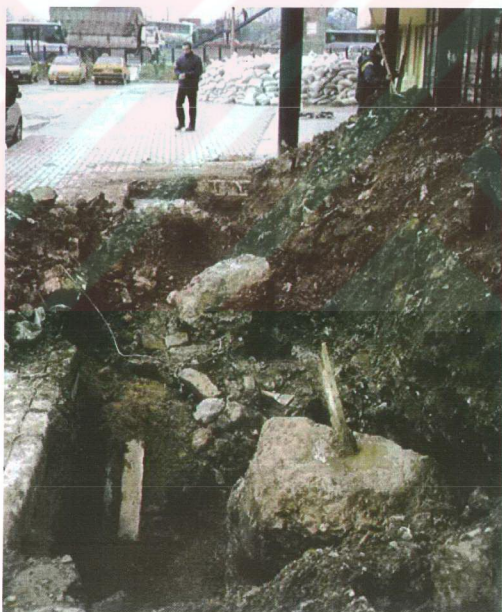


Fig. 3.13 Test Columns



Fig. 3.14 Test Column Nr. 1

3.6.5 Preparations for the Application

Preparations for the application were consisting in majority of extraction and demolishment of the partition walls inside the building and floor tiles. The goal of this subject was to supply the work ability of the drill rig inside the building. The result of these facilities interior can be seen on Fig. 3.15.



Fig. 3.15 Interior of KEDAŞ General Directorate Building Prepared For Jet Grouting Application

3.6.6 The Construction Phase

“Following the preparations, start was given on 10.12.2002 for the application. Because of the shelter side of the building is over loaded, the implementation started from this section. The idea was that the grouting pressure could lift it up a little, so that the differential settlements decreases.”[33]

The produced columns terminated 60 cm below the mat foundation as shown on the cross section in the drawings. So the columns started from 22.30m deep and finished 2.30m under the ground floor elevation. After 14 columns were installed, it has been observed that swellings took place at some areas around the building. Fig.3.16 shows heaved soil under the transformer building, which is located adjacent to the General Directorate Building. The soil under the foundation was soft soil and grout, which couldn't pass the mat foundation, pushed it laterally. Since, the soil located under the building had moved and finally squeezed out the soil on its final destination upwards so that it swells.



Fig. 3.16 Swelling under The Transformer Building Right Behind The General Directorate Building

As a first precaution, the column head elevation had been transferred 1.0m deeper, so that the upper elevations of the columns were 1.60m deeper than the mat foundation. These would exterminate the lateral push effect caused by the high grouting pressure and rigid mat foundation. Although precautions have been taken, swellings continued. With the increasing number of columns, swellings started to occur on remote areas, for instance at the other side of the street. Where Fig 3.17 shows how the pavement stones has raised, because of the pressure effect, Fig 3.18 shows a crack on the park wall caused by the same reason.



Fig. 3.17 Risen Pavement Stones



Fig. 3.18 Crack Caused By Grouting Pressure

All these pointed out that the soil was softer than it had been estimated. So further precautions had to be taken. It was decided to decrease the grouting pressure to 350 Bar, and also one decision was to produce the columns primarily, which are adjacent to the wall of the building, so that they act as a block wall and hinder any more grout to force the lateral soil movement. The inclined jet grout columns at the wall border outside the building had been added because of these circumstances. The experience obtained, were that columns installed only inside the building couldn't be sufficient to control the settlements. So an increase on the column number of 20% were decided and 18 columns were added. Columns placed outside the building, have an angle of 5° from vertical.

Finally the problems have been solved with the additional measures. The application ended on 04.01.2003 with a production summary of total 111 columns. The length of the produced columns in total is 2220m. At the end of the application cement expenditure has been calculated as 195 kg/m of each column as an average.

3.6.7 Quality Control Program

The quality control program was including two borings made on column H – 6 and J – 6. Borings were carried out with Creallius 500 type drilling machine, which works with rotary system (Fig.3.19).



Fig. 3.19 Drilling Machine Taking Core Samples From The Jet Grout Columns

The purpose was to take Core samples, and subject them to unconfined compression test. Core samples are shown on Fig. 3.20 and Fig.3.21. A second cause for taking cores from the completed columns were to prove the continuity of the columns. These were carried out with drilling rig, because the columns were constructed under the mat foundation, from a hole with a diameter of 75cm. The condition of the columns was unsuitable to carry out a pile integrity test. [33]



Fig. 3.20 Core Samples of Column J – 6



Fig. 3.21 Core Samples of Column H – 6

The samples shown above are sufficient to consider that the jet grout columns have been constructed in continuity and with adequate quality.

Seven unaxial compression tests have been carried out. The samples were 20 days old, and because of that the 28-day strengths of the samples were found with the help of a short calculation. 28 – day strength is considered, as the 95% of the total strength of grout, where the 20 – day strength is 80% of overall strength. [33]

Table 3.8 Results of Uniaxial Compression Tests

Column Number	Sample Depth (m)	Diameter (mm)	Sample Height (mm)	Failure Force (kN)	20-Days Pressure Strength (N/mm ²)	28-Days Pressure Strength (N/mm ²)
J – 6	15.0	54.5	106	19.0	8.1	8.6
J – 6	15.0	54.5	102	22.5	9.6	10.2
J – 6	22.0	54.5	92	21.5	9.2	9.8
H – 6	10.0	54.5	98	13.0	5.6	6.0
H – 6	10.0	54.5	90	17.0	7.3	7.7
H – 6	18.5	54.5	102	12.5	5.4	5.7
H – 6	20.0	54.5	103	17.5	7.5	8.0

After the Technical Specification Guide, pressure strength of minimum 1.5kN/cm² was to be obtained for each column. The results have shown that this goal has been exceeded much further in this application and the columns have proven their quality. In average, the columns have shown a 20-day pressure strength of 7.5kN/cm² and a 28-day pressure strength of 80kg/cm².

Also, to uncover a part of the floor and to get a picture of the constructed columns was a part of the quality control program (Fig.3.22). It has been observed that although the grout were not pumped over 1.60m depth from the ground floor, it filled all blanks and holes in the mat foundation and risen to the ground floor elevation. Because of this reason, there was no necessity to repair the mat foundation with any mortar. [33]



Fig. 3.22 Uncovered Section of Floor

Soil improvement under the foundation of the General Directorate Building of KEDAŞ has been successfully fulfilled. Serving this purpose 93 vertical and 18 inclined jet grout columns have been constructed. These jet grout columns transfer the super structure loads acting to the hard clay layers at 23.0m depths. This improvement of the foundation soil can be considered as a precaution for any further settlements and any liquefaction potential. Monitoring that has been conducted on the building show that there is no further increase in settlements. [33]

4 CONCLUSION

Soil improvement techniques are widely used in geotechnical engineering to develop the engineering properties of loose or soft soils before or after construction. There have been extensive developments in this context during the previous decade. Jet grouting technology, which has also been affected from these developments, is one of the most common soil improvement techniques in order to control and limit excessive settlements and increase bearing capacity of foundations and in some to reduce the risk of liquefaction.

In this thesis; application methods, application range, advantages and disadvantages of jet grouting are presented as well as the quality control tests carried out during and after jet grout installation. The study also discusses a jet grouting application performed under a building subjected to differential settlement due to poor soil conditions and mean loading. Remedial measures taken against these encountered problems during the jet grouting application were also subjected to this thesis as well as the importance of monitoring for the success of such kind of an application.

Soil improvement is one of the oldest execution methods in civil engineering. But it is still one of the most intriguing and growing techniques. Especially during the last decade, soil improvement has come of an age and reached a new level of acceptance in the geotechnical community. Now a day, soil improvement is the most routinely used method in foundation engineering, where poor, unstable soils are encountered.

Jet grouting is a respectively new developed technology in coMParison to other soil improvement methods. It is currently utilized throughout the world. From its conception in Japan in 1973 by Yahiro and Yashido, its use has spread, first to Europe, then to South and North America, the Far East, Asia and Africa.

One can distinguish the soil improvement methods under the following three subtitles:

- Temporary soil improvement techniques, which are limited to the period of the construction.
- Permanent soil improvement without the addition of any material
- Permanent soil improvement by adding materials

Jet grouting finds its place in this classification as; “Permanent soil improvement by adding materials” like all other grouting methods.

Soil improvement methods can also be classified for granular and cohesive soils or shallow and deep soil improvement, which has also an important meaning from an engineering point of view. In such kind of a classification, jet grouting occupies a wide range due to its suitability for both granular and cohesive soils.

Jet grouting has a radically different approach than other existing grouting methods, because of its applicability range due to soil types and end product. Jet grouting is not filling voids in soil; it causes erosion and mixes grout with soil under high pressure. All these make jet grouting to be a step ahead in the first look.

“Jet grouting can be defined as a method based on the introduction of hydraulic (sometimes combined with pneumatic) energy in order to erode soil and mix or replace the eroded material with an engineered grout to form a solidified in situ element known as Soilcrete [3].” It is not truly grouting but rather a hydrodynamic mix-in-place technique producing a soil-cement material.

Another definition for jet grouting is given by The American Society of Civil Engineers as a technique utilizing a special drill bit with horizontal and vertical high speed water jets to excavate alluvial soils and produce hard impervious columns by pumping grout through the horizontal nozzles that jets and mixes with foundation material as the drill bit is withdrawn. The sequence of work starts usually with a small (100-200mm diameter) hole, which is drilled to the required depth, followed with a high pressure (several hundred bars) fluid pump through one or more small (1-10mm) nozzles on a monitor at the foot of a 70-100mm diameter drill rod and ends with that the drill rod is slowly raised and rotated to form a column of soil cement. [3]”

Three known jet-grouting systems are used in practice. These are named as Jet 1 – Single Fluid System, Jet 2 – Double Fluid System, Jet 3 – Triple Fluid System

The single fluid referred to Jet 1 method is neat cement grout. This method produces the most homogeneous soil-cement element with the highest strength.

The two fluids referred to JET 2 method are neat cement grout and air. The neat cement grout is injected at a lower pressure and is aided by a cone of compressed air, which shrouds the grout injection. This enables a grater treated volume during each jetting lift when coMPared with the single jet approach.

The three fluids referred to JET 3 method are neat cement grout, air and water. Unlike Jet 1 and Jet 2, water is injected at high pressure and is aided by a cone of compressed air, which shrouds the water injection. This process produces and airlifting effect which evacuates the soil within the intended column diameter.

The procedure applied for construction, the mix composition of the suspension and the content of suspension are major effective factors on the properties of the soilcrete elements, in another words of the jet grout columns. From a structural point of view it is mainly the strength, the permeability and the erosion stability, which are of interest.

The composition of jet grouting material, or grouting material in general consists of water and cement at W/C ratios of 0.8 to 1.5. Ratios above this upper limit are considered as unsuitable suspensions.

In addition to the key advantage over other grouting methods of being able to deal with a wide range of soils, jet grouting allows more accurate assessment in advance of grout quantities, volume of ground influenced and the properties of the treated ground. It has successfully been applied for both temporary and permanent functions in a wide range of applications, particularly for soil improvement cases like soil stabilization, excavation support systems, structural underpinning, seepage barrier/cutoff walls, soil consolidation for the excavation of shallow tunnels, environmental remediation or groundwater control. [14]

Jet grouting is also suitable for working inside existing building and structures. So it enables to improve the soil after construction and provide support to the foundation of an existing building. This is possible because of the ability of this method to construct large diameter columns from small diameter boreholes. As it has been seen in the case history reported in this thesis, columns with 80 cm in diameters have been successfully constructed inside the building.

Although all advantages that jet grouting technology offers, a coMParison of the costs of common grouting and of jet grouting is difficult. It is occasionally stated that jet grouting would be less costly. However, this is not easy to confirm since no cases are known where both methods have been tendered and offered for the same project.

The construction time would in many cases be shorter for jet grouting, than for conventional grouting. This follows partly from the obligation to limit the grouting rate with pore filling injection to avoid cracking. With jet grouting an upper limit of the jetting flow rate is given for other reasons, for instance to limit too high flow velocities in the borehole.

With all these above-mentioned jet-grouting skills, the following description of the system and the process is a summarization. "The Jet Grouting Technique employs high pressure erosive jets of water or grout to break down the soil structure, removes varying proportions of the resultant soil particles and replaces them with cement based grouts placed during the erosive operation. Soil particles not removed become mixed with the grout in situ to form the treated mass. [14]"

Another point that has to be taken into account is that jet-grouting applications are not suitable for soils with cobbles over 60mm diameter. Against very high jet grouting pressure, the erodibility of such material isn't sufficient. That causes lateral movements on the softer soils, which turns the improvement to a material consumption far away from matching the goal.

The case history reported in this thesis is a jet grouting application, which emphasizes the importance of quality tests and monitoring. On soft grounds, like under the General Directorate Building of KEDAŞ, high grouting pressures and extensive grout consumptions can cause lateral movements of soil layers, which lead to damages on adjacent structures and utilities. The crack on the neighboring park wall or the swellings under the pavement stones can be considered as instances for such damages. Such circumstances, as in the prior sentences, require very good monitoring during the application to prevent any undesired damage.

The General Directorate Building of KEDAŞ in Izmit had been subjected to differential excessive settlements due to poor soil conditions and uneven loading. The building rests over an area of 36.40m x 14.50m and consists of eight storeys (ground floor + 7 floors) in total.

The asymmetrically loading conditions of the building were an important factor on the occurrence of differential settlements. A shelter was placed on one side of the ground floor, causing a huge difference on the transmitted loads to the foundation due its 60 cm thick perimeter walls. The settlements initiated during construction of the building and became obvious after the Marmara Earthquake on 17 August 1999.

Soil improvement has been carried out with 111 jet grout columns of each 20m lengths. 93 vertical columns are placed inside the building and 18 inclined columns are supporting the building from outside. Prior to the construction of jet grout columns, the ranges of the differential settlement were decreased with a preloading of 40kPa on the south side of the building.

At the end of the jet grouting application, laboratory test results from the cores of the columns have shown that the application goal of 1.5kN/cm^2 has been exceeded and the columns have proven their quality. In average columns had 20-day unaxial compression strength of 7.5kN/cm^2 and 28-day strength of 80kg/cm^2 , which have been accepted as satisfactory.

The only problem was due to the soft soil layers. Some swellings took place at neighbor areas. Adequate monitoring added with on time intervention surpassed these swellings, so that the application could have resulted with accomplishment.

As a summary, the jet grouting is one of the most advanced technologies on the field of soil improvement applications. As every engineering application, without correct regulations and monitoring jet grouting can cause catastrophic results. Quality monitoring, like reported in the case history, enables the engineers to take precautions against undesired consequences.

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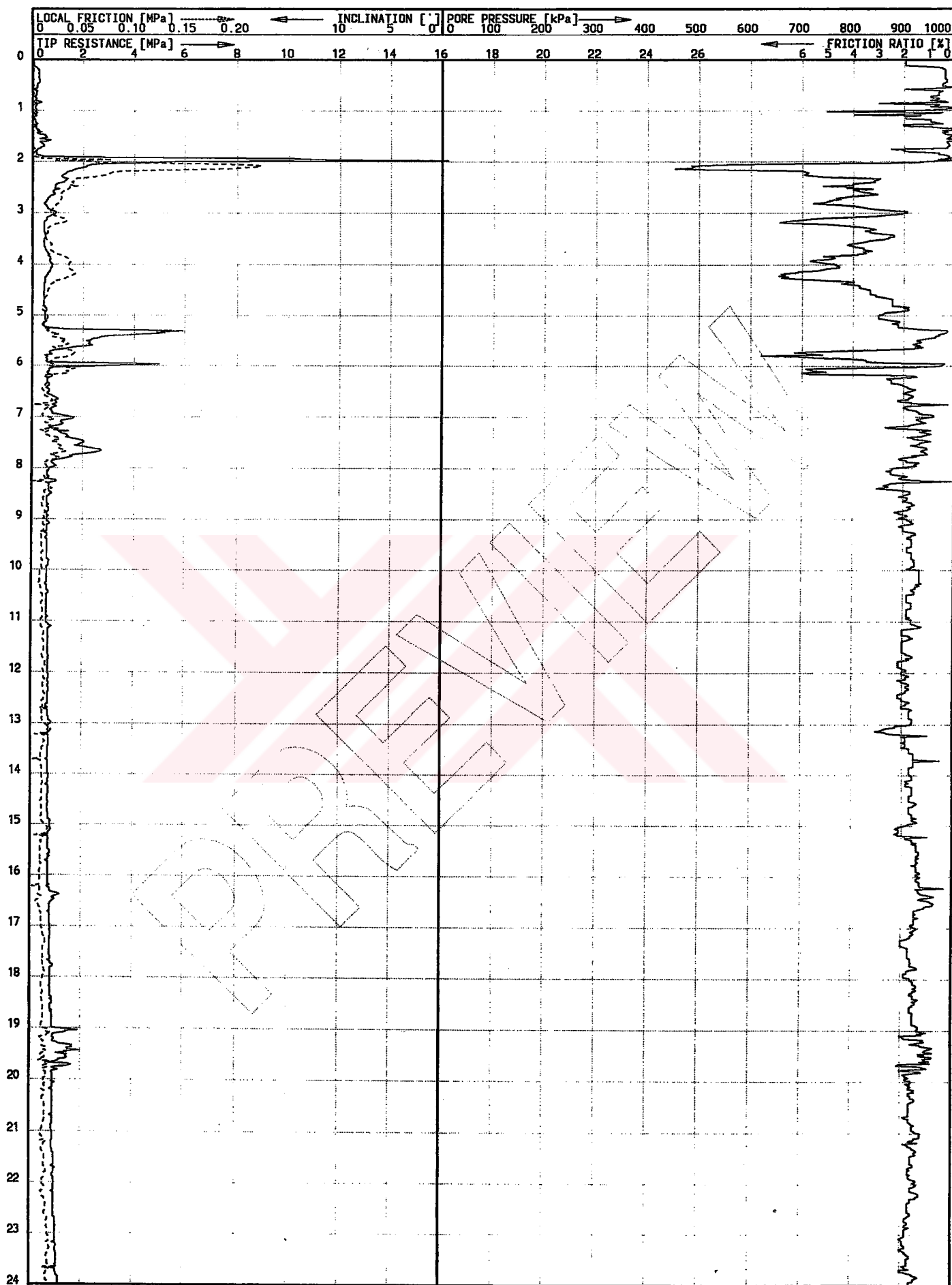
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APPENDIX

- **CPT Logs of KEDAŞ Building**
- **Boring Logs of KEDAŞ Building**
- **Idealized Soil Profile for KEDAŞ Building**
- **Liquefaction Analysis for KEDAŞ Building**
- **Settlement Analyses after Westergaard's Point Loading Formula**
- **Settlement Analyses Made with Unisettle**
- **Settlement Analyses Made with Unisettle after Preloading**
- **Piezometer Data Table**
- **Jet Grout Application Plan for KEDAŞ Building**



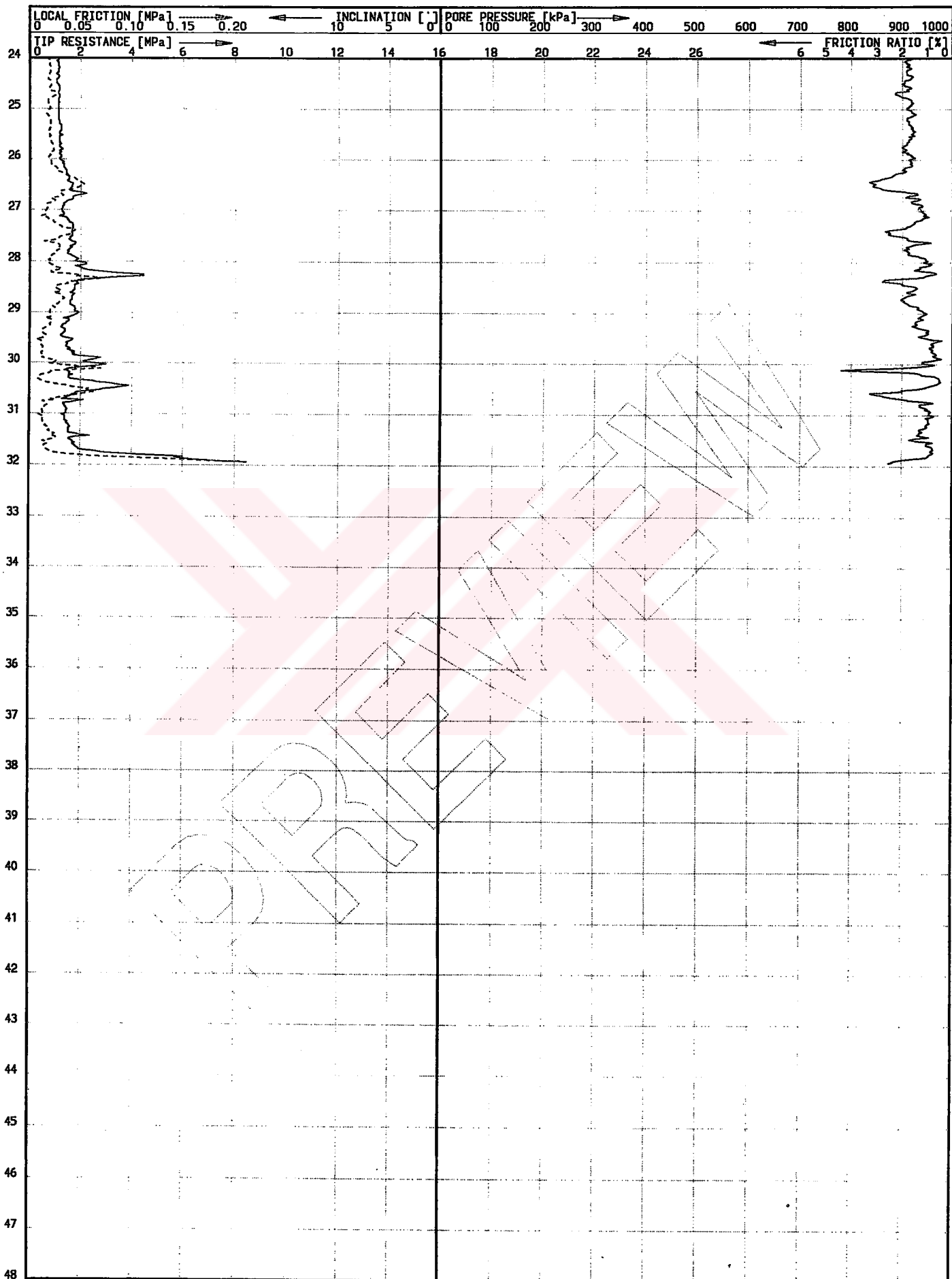
CPT Logs of KEDAŞ Building



JOB NUMBER : ITU
TEST NUMBER : 1
DATE : 4-5-2002 TIME : 10:57
CLIENT : KORFEZ ELEKTRIK
DESCRIPTION : ZEMIN ETUDU

OPERATOR :
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CONE TYPE : CF Nr.: 020305
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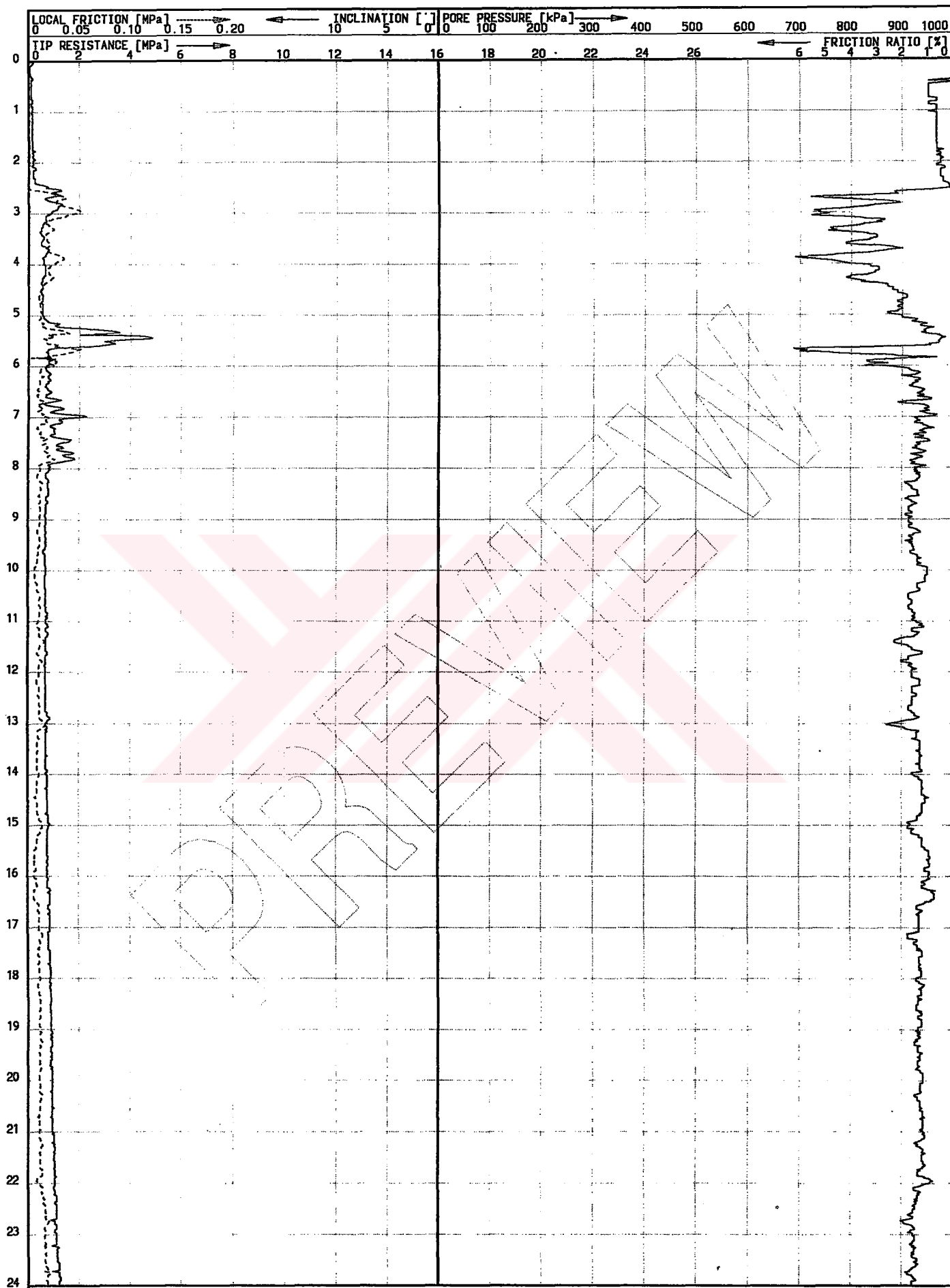
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LOCAL FRICTION	0.000	0.001		[MPa]
PORE PRESSURE	0	0		[kPa]
INCLINATION	0.0	0.0		[°]



JOB NUMBER : ITU
TEST NUMBER : 1
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OPERATOR :
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CONE TYPE : CF Nr.: 020305
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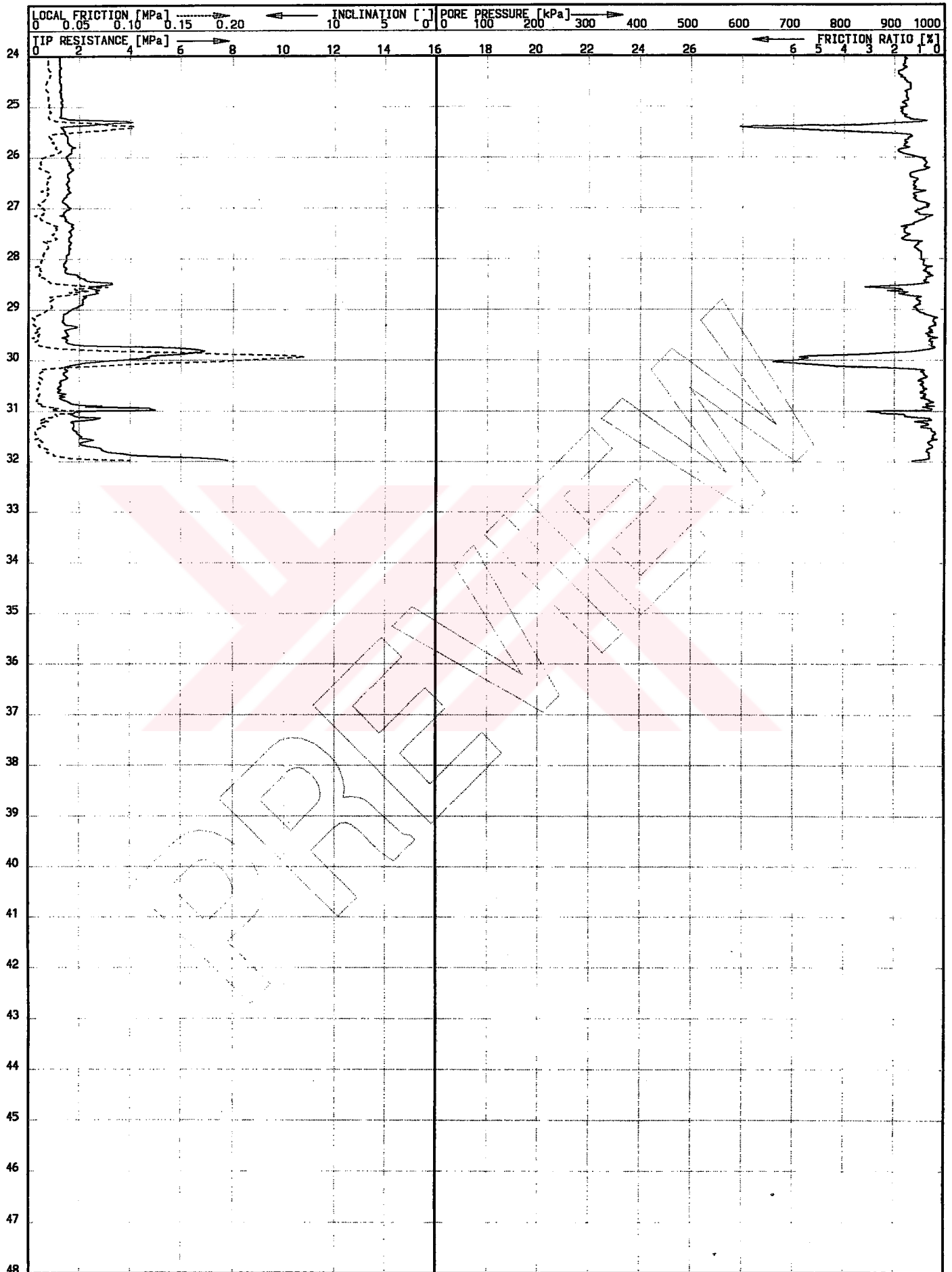
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TIP RESISTANCE	0.6	0.5	[MPa]
LOCAL FRICTION	0.000	0.001	[MPa]
PORE PRESSURE	0	0	[kPa]
	0	0	[kPa]



JOB NUMBER : ITU
TEST NUMBER : 002
DATE : 4-5-2002 TIME : 10.57
CLIENT : KORFEZ ELEKTRIK
DESCRIPTION : ZEMIN ETUDU

OPERATOR :
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INCLINOMETER :
REM :

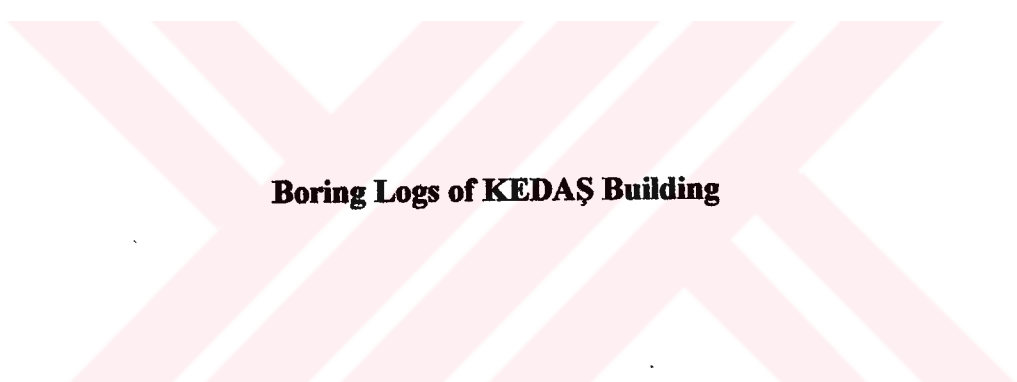
	ZERO OFFSET	BEGIN	END
TIP RESISTANCE	0.5	0.4	[MPa]
LOCAL FRICTION	-0.001	0.001	[MPa]
PORE PRESSURE	0	0	[kPa]
INCLINATION	0.0	0.0	[°]



JOB NUMBER : ITU
TEST NUMBER : 002
DATE : 4-5-2002 TIME : 10.57

OPERATOR :
REFERENCE LEVEL : 0.0 to
CONE TYPE : CF Nr.: 020305
INCLINOMETER :

	ZERO OFFSET	BEGIN	END	
TIP RESISTANCE	0.5	0.4		[MPa]
LOCAL FRICTION	-0.001	0.001		[MPa]
PORE PRESSURE	0	0		[kPa]



Boring Logs of KEDAŞ Building

KUYU NO

ZEMİN KOTU 1.89 m.

E: 4.514.423

12.07.2000

SONDA 9

KURUPAN DAN PERTURAN

SEHİRİCİ/İZMİT	OFFSET :	KUYU NO : S1/S53
N : GMS-300		ZEMİN KOTU : 2.50 m.

YÖNTEMİ : ROTARY : 0.00-30.45m arası.	KOORD. : N : 494.877 E : 451.431
API : 0.00-6.00m arası - 90mm. 6.00-30.45m arası - 76mm.	BAŞLANGIÇ : 30.08.2000 BİTİŞ : 30.08.2000

VE	DENEY.	S.P.T. darbe sayısı	Muh. Dm. (m)	Y.A.S. Dm. (m)	TCR %	RQD %	SCR %	Dm. (m)	ZEMİN CİNSİ	KOT (m)	LEJAND
TİP	15	7.5	7.5	7.5							
50	D1	50/1	-	-							
51											
20	D2	4	7	11				3.20	Doğu zemin	-0.70	
45			N = 18								
50	D3	3	3	3				4.30	Cok katı, yeşil-kahverengli, siltli KIL	-1.80	
35			N = 6								
30	U1										
50											
50	D4	1	2	3							
85			N = 5								
00	D5	2	3	2							
45			N = 5								
50	D6	2	2	2							
85			N = 4								
00	D7	2	2	2				12.50	Orta katı, gri renkli, kavkılı siltli KIL	-10.00	
45			N = 4								
50	D8		1	1							
85			N = 2								
00	D9		1	1				16.00	Cok yumusak, gri renkli, kavkılı siltli KIL	-13.50	
45			N = 2								
50	D10	2	3	3							
85			N = 5								
00											
45											
50	C12	5	1	2							
85			N = 8								

Revizyon 0C/
Revizyon 0B/
Revizyon 0A/

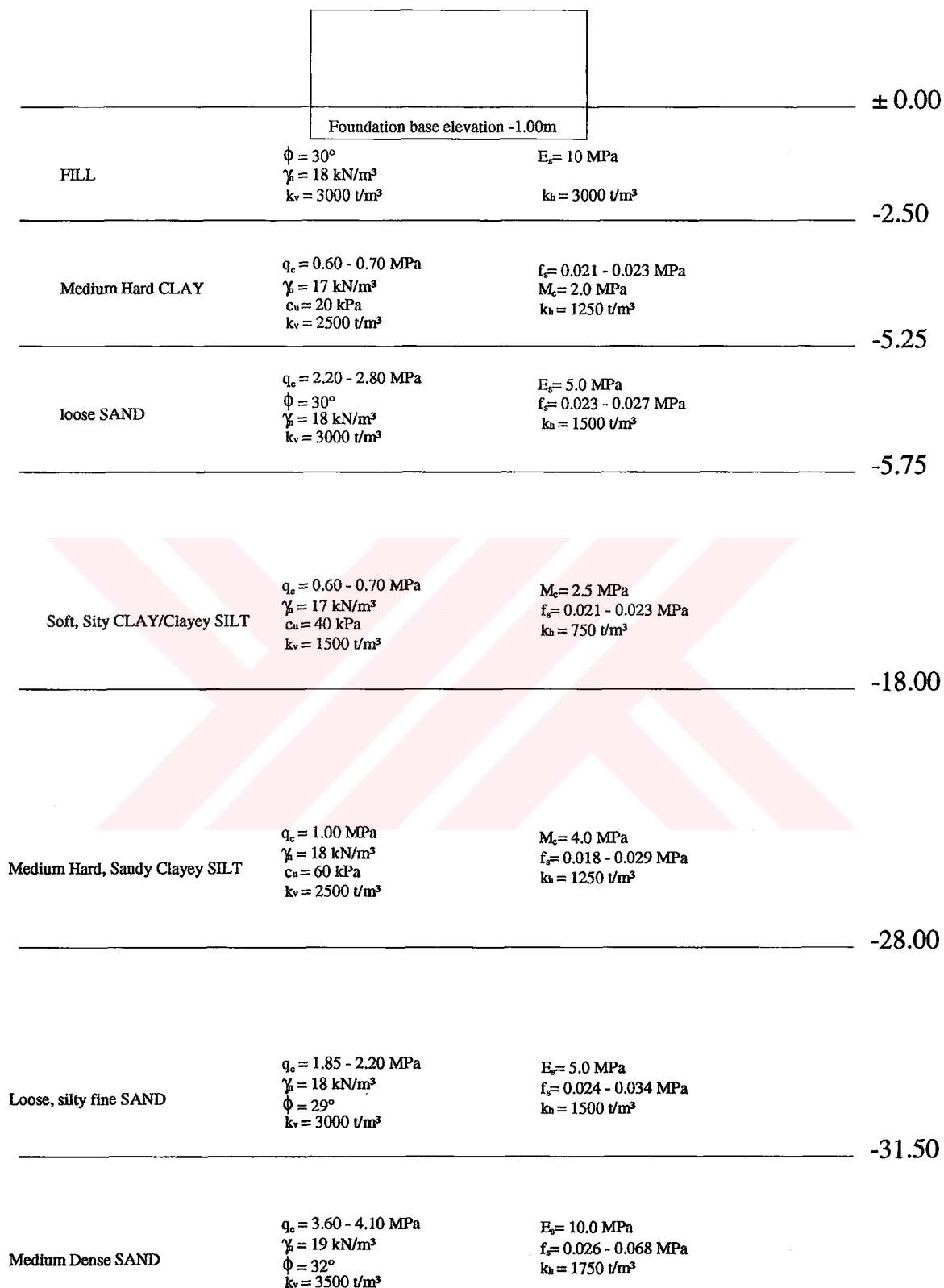
R : C.GOZCU

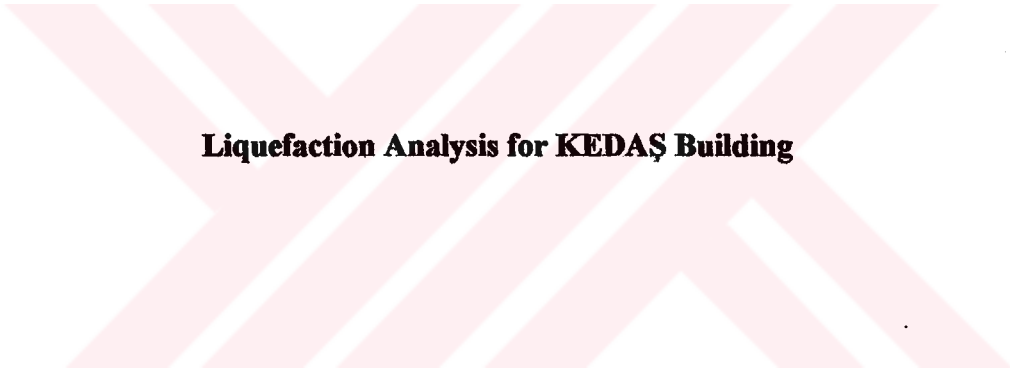
LOGU HAZIRLAYAN : A.AKKAS

KONTROL EDEN : A.ERTURAN



Idealized Soil Profile for KEDAŞ Building





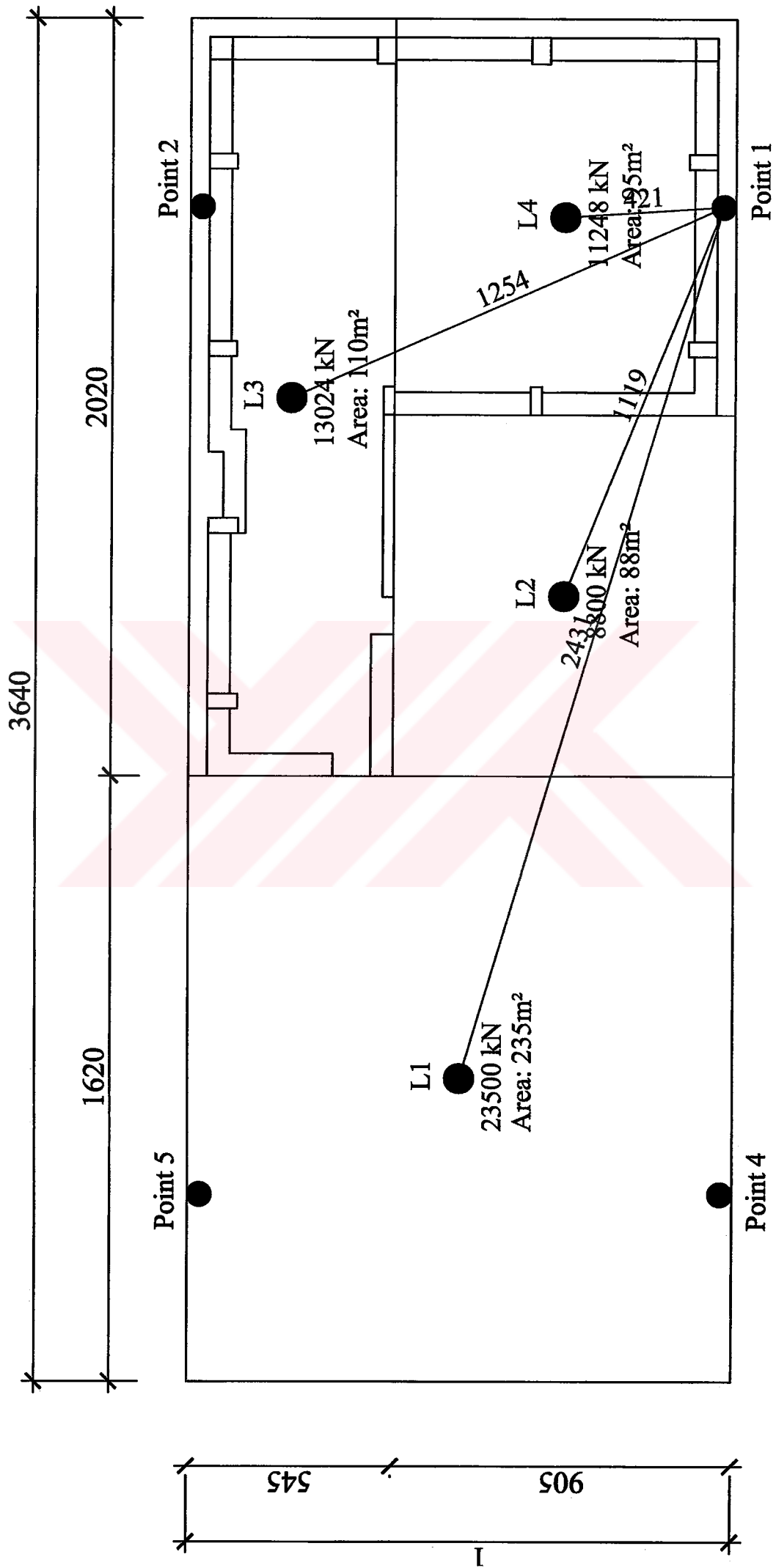
Liquefaction Analysis for KEDAŞ Building

Liquefaction Analysis

Layer	Soil Type:	γ_n (kN/m ³):	GWT(m)	Depth:	σ (kN/m ²)	σ' (kN/m ²)	Fine Content:	N ₃₀	N'	Earthquake Magnitude (M)
1	CLAY	17,00	3	2,50	42,50	42,50	51	2		7
2	CLAY	17,00		4,00	68,00	58,19	51	2		7
3	SAND	18,00		5,25	94,50	72,43	10	4	5	7
4	SAND	18,00		5,75	103,50	76,52	10	2	2	7
5	CLAY	17,00		6,50	110,50	76,17	51	5		7
6	CLAY	17,00		9,00	153,00	94,14	51	5		7
7	CLAY	17,00		12,00	204,00	115,71	51	5		7
8	CLAY	17,00		15,00	255,00	137,28	51	5		7
9	CLAY	17,00		18,00	306,00	158,85	51	5		7
10	SILT	18,00		20,00	360,00	193,23	51	5		7
Layer	Soil Type:	MSF: magnitude scale factor	$\alpha_{(7,5)}$:	α :	τ_s :	a_{max}/g :	r_d :	τ_0 :	τ_s/τ_0 :	End Condition:
1	CLAY	1,08	0,05	0,05	2,13	0,4	0,981	10,84	>1	no liquefaction
2	CLAY	1,08	0,05	0,05	2,91	0,4	0,969	17,13	>1	no liquefaction
3	SAND	1,08	0,06	0,06	4,35	0,4	0,960	23,59	0,18	liquefaction
4	SAND	1,08	0,05	0,05	3,83	0,4	0,956	25,73	0,15	liquefaction
5	CLAY	1,08	0,07	0,08	6,09	0,4	0,950	27,29	>1	no liquefaction
6	CLAY	1,08	0,07	0,08	7,53	0,4	0,931	37,04	>1	no liquefaction
7	CLAY	1,08	0,07	0,08	9,26	0,4	0,854	45,30	>1	no liquefaction
8	CLAY	1,08	0,06	0,06	8,24	0,4	0,774	51,32	>1	no liquefaction
9	CLAY	1,08	0,06	0,06	9,53	0,4	0,693	55,14	>1	no liquefaction
10	SILT	1,08	0,06	0,06	11,59	0,4	0,640	59,90	>1	no liquefaction



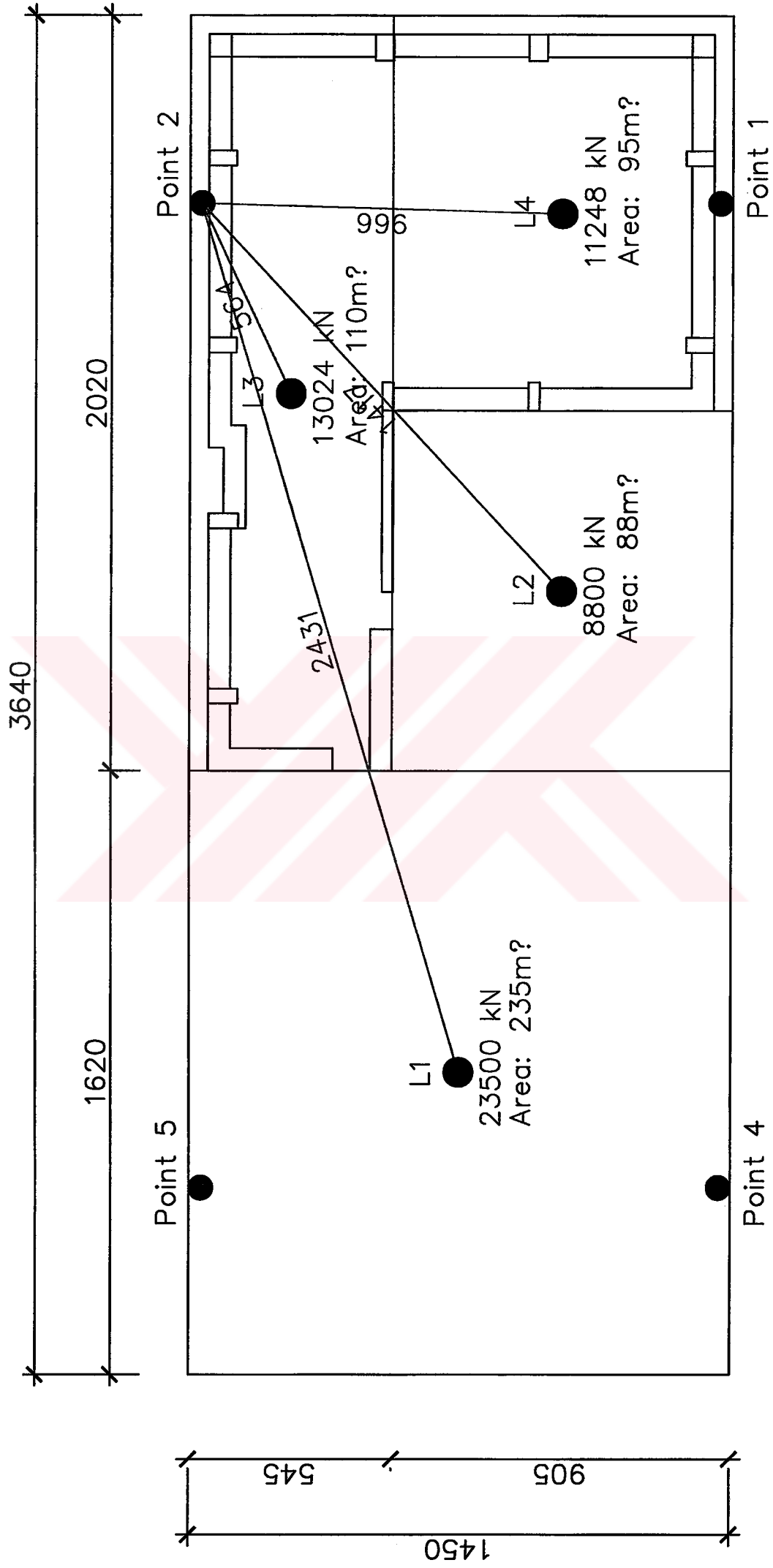
Settlement Analyses after Westergaard's Point Loading Formula



Soil Type	Layer Location	H (m)	Layer Center Depth; z (m)	Unit Weight (kN/m ³)	r _{L1} (m)	r _{L2} (m)	r _{L3} (m)	r _{L4} (m)
Fill	0,00 - 1,50	1,50	0,75	19	24,31	11,19	12,54	4,21
Fill	1,50 - 2,50	1,00	1,75	19	24,31	11,19	12,54	4,21
Soft - Medium Stiff Clay	2,50 - 3,50	1,00	3,00	17	24,31	11,19	12,54	4,21
Soft - Medium Stiff Clay	3,50 - 4,50	1,00	4,00	17	24,31	11,19	12,54	4,21
Soft - Medium Stiff Clay	4,50 - 5,25	0,75	4,88	17	24,31	11,19	12,54	4,21
Loose Sand	5,25 - 5,75	0,50	5,50	18	24,31	11,19	12,54	4,21
Soft - Medium Stiff Silty Clay - Clayey Silt	5,75 - 7,00	1,25	7,35	17	24,31	11,19	12,54	4,21
Soft - Medium Stiff Silty Clay - Clayey Silt	7,00 - 8,00	1,00	7,50	17	24,31	11,19	12,54	4,21
Soft - Medium Stiff Silty Clay - Clayey Silt	8,00 - 9,00	1,00	8,50	17	24,31	11,19	12,54	4,21
Soft - Medium Stiff Silty Clay - Clayey Silt	9,00 - 10,00	1,00	9,50	17	24,31	11,19	12,54	4,21
Soft - Medium Stiff Silty Clay - Clayey Silt	10,00 - 12,00	2,00	11,50	17	24,31	11,19	12,54	4,21
Soft - Medium Stiff Silty Clay - Clayey Silt	12,00 - 14,00	2,00	13,00	17	24,31	11,19	12,54	4,21
Soft - Medium Stiff Silty Clay - Clayey Silt	14,00 - 16,00	2,00	15,00	17	24,31	11,19	12,54	4,21
Soft - Medium Stiff Silty Clay - Clayey Silt	16,00 - 18,00	2,00	17,00	17	24,31	11,19	12,54	4,21
Stiff Clayey Silt with Sand	18,00 - 20,00	2,00	19,00	18	24,31	11,19	12,54	4,21
Stiff Clayey Silt with Sand	20,00 - 22,00	2,00	21,00	18	24,31	11,19	12,54	4,21
Stiff Clayey Silt with Sand	22,00 - 24,00	2,00	23,00	18	24,31	11,19	12,54	4,21
Stiff Clayey Silt with Sand	24,00 - 26,00	2,00	25,00	18	24,31	11,19	12,54	4,21
Stiff Clayey Silt with Sand	26,00 - 28,00	2,00	27,00	18	24,31	11,19	12,54	4,21
Medium Dense Sand	28,00 - 29,25	1,25	28,65	18	24,31	11,19	12,54	4,21
Medium Dense Sand	29,25 - 31,50	2,25	30,85	18	24,31	11,19	12,54	4,21

Soil Type	Layer Location	Δp_1 (kPa)	Δp_2 (kPa)	Δp_3 (kPa)	Δp_4 (kPa)	$\Sigma \Delta p$ (kPa)	E_s / M_c (MPa)	S (m)
Fill	0,00 - 1,50	0,00	0,01	0,01	1,58	1,60	10	0,00
Fill	1,50 - 2,50	0,01	0,12	0,10	14,61	14,84	10	0,00
Soft - Medium Stiff Clay	2,50 - 3,50	0,03	0,54	0,47	39,28	40,32	2	0,02
Soft - Medium Stiff Clay	3,50 - 4,50	0,08	1,13	1,01	52,04	54,26	2	0,03
Soft - Medium Stiff Clay	4,50 - 5,25	0,14	1,80	1,64	56,12	59,70	2	0,02
Loose Sand	5,25 - 5,75	0,19	2,32	2,15	56,05	60,71	5	0,01
Soft - Medium Stiff Silty Clay - Clayey Silt	5,75 - 7,00	0,42	3,88	3,81	48,91	57,01	5	0,01
Soft - Medium Stiff Silty Clay - Clayey Silt	7,00 - 8,00	0,44	4,00	3,94	48,14	56,52	2,5	0,02
Soft - Medium Stiff Silty Clay - Clayey Silt	8,00 - 9,00	0,61	4,71	4,79	42,95	53,06	2,5	0,02
Soft - Medium Stiff Silty Clay - Clayey Silt	9,00 - 10,00	0,79	5,29	5,53	38,01	49,62	2,5	0,02
Soft - Medium Stiff Silty Clay - Clayey Silt	10,00 - 12,00	1,21	6,01	6,63	29,65	43,51	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	12,00 - 14,00	1,55	6,22	7,11	24,77	39,64	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	14,00 - 16,00	1,99	6,18	7,35	19,75	35,26	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	16,00 - 18,00	2,40	5,91	7,26	16,01	31,59	2,5	0,03
Stiff Clayey Silt with Sand	18,00 - 20,00	2,75	5,53	6,98	13,20	28,45	4	0,01
Stiff Clayey Silt with Sand	20,00 - 22,00	3,04	5,10	6,58	11,04	25,75	4	0,01
Stiff Clayey Silt with Sand	22,00 - 24,00	3,25	4,67	6,13	9,35	23,40	4	0,01
Stiff Clayey Silt with Sand	24,00 - 26,00	3,40	4,26	5,68	8,01	21,35	4	0,01
Stiff Clayey Silt with Sand	26,00 - 28,00	3,49	3,88	5,23	6,94	19,54	4	0,01
Medium Dense Sand	28,00 - 29,25	3,52	3,59	4,89	6,20	18,20	10	0,00
Medium Dense Sand	29,25 - 31,50	3,52	3,24	4,46	5,39	16,61	10	0,00
Total Settlement (m) ==>								0,34

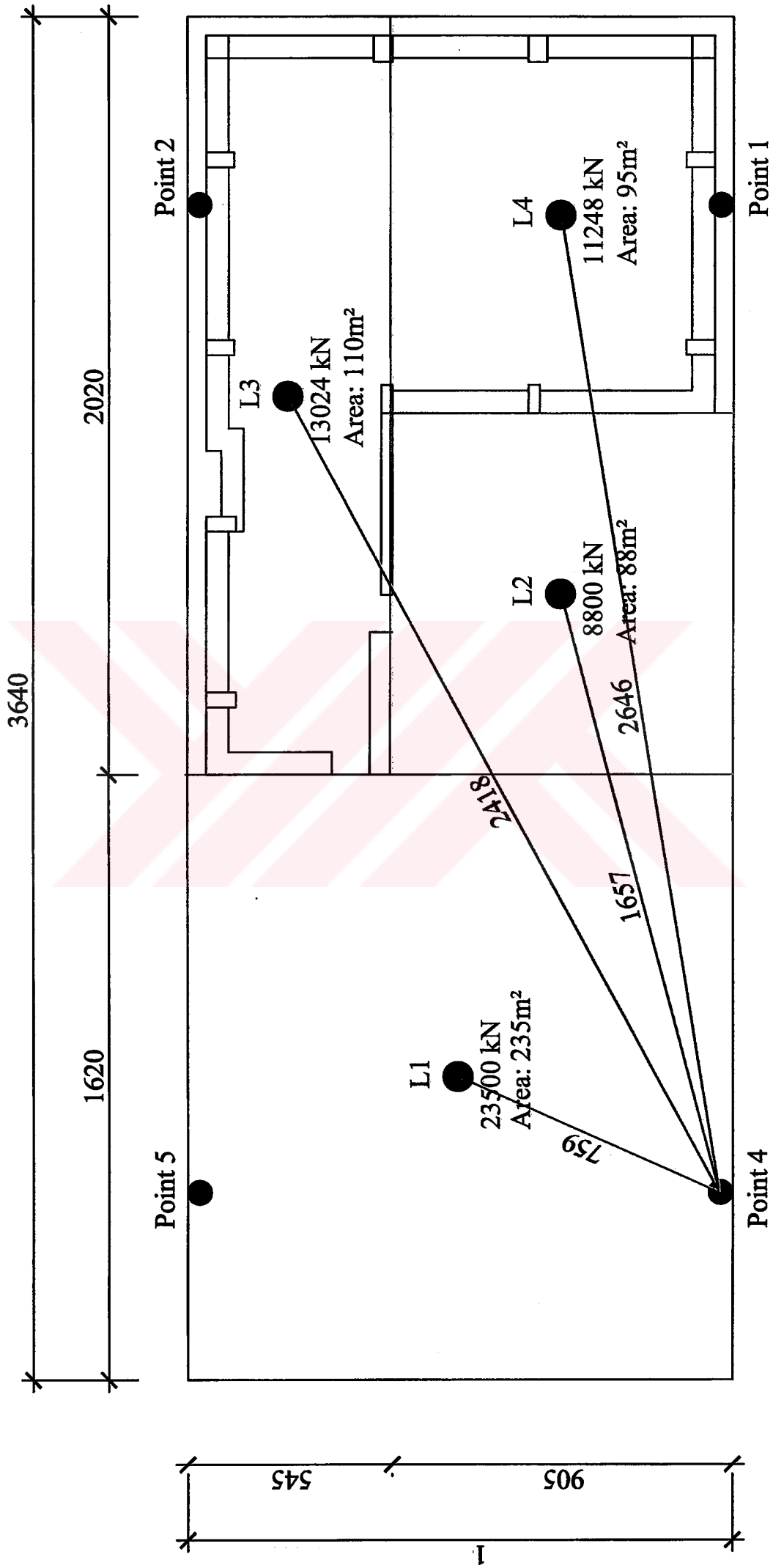
$$\Delta p = Q / ((z^2 \cdot \pi) (1/(1+2(r/z)^{2,3/2}) (Westergaard's Stress Distribution for point load)$$



Soil Type	Layer Location	H (m)	Layer Center Depth; z (m)	Unit Weight (kN/m ³)	r _{L1} (m)	r _{L2} (m)	r _{L3} (m)	r _{L4} (m)
Fill	0,00 - 1,50	1,50	0,75	19	24,31	14,17	5,64	9,66
Fill	1,50 - 2,50	1,00	1,75	19	24,31	14,17	5,64	9,66
Soft - Medium Stiff Clay	2,50 - 3,50	1,00	3,00	17	24,31	14,17	5,64	9,66
Soft - Medium Stiff Clay	3,50 - 4,50	1,00	4,00	17	24,31	14,17	5,64	9,66
Soft - Medium Stiff Clay	4,50 - 5,25	0,75	4,88	17	24,31	14,17	5,64	9,66
Loose Sand	5,25 - 5,75	0,50	5,50	18	24,31	14,17	5,64	9,66
Soft - Medium Stiff Silty Clay - Clayey Silt	5,75 - 7,00	1,25	7,35	17	24,31	14,17	5,64	9,66
Soft - Medium Stiff Silty Clay - Clayey Silt	7,00 - 8,00	1,00	7,50	17	24,31	14,17	5,64	9,66
Soft - Medium Stiff Silty Clay - Clayey Silt	8,00 - 9,00	1,00	8,50	17	24,31	14,17	5,64	9,66
Soft - Medium Stiff Silty Clay - Clayey Silt	9,00 - 10,00	1,00	9,50	17	24,31	14,17	5,64	9,66
Soft - Medium Stiff Silty Clay - Clayey Silt	10,00 - 12,00	2,00	11,50	17	24,31	14,17	5,64	9,66
Soft - Medium Stiff Silty Clay - Clayey Silt	12,00 - 14,00	2,00	13,00	17	24,31	14,17	5,64	9,66
Soft - Medium Stiff Silty Clay - Clayey Silt	14,00 - 16,00	2,00	15,00	17	24,31	14,17	5,64	9,66
Soft - Medium Stiff Silty Clay - Clayey Silt	16,00 - 18,00	2,00	17,00	17	24,31	14,17	5,64	9,66
Stiff Clayey Silt with Sand	18,00 - 20,00	2,00	19,00	18	24,31	14,17	5,64	9,66
Stiff Clayey Silt with Sand	20,00 - 22,00	2,00	21,00	18	24,31	14,17	5,64	9,66
Stiff Clayey Silt with Sand	22,00 - 24,00	2,00	23,00	18	24,31	14,17	5,64	9,66
Stiff Clayey Silt with Sand	24,00 - 26,00	2,00	25,00	18	24,31	14,17	5,64	9,66
Stiff Clayey Silt with Sand	26,00 - 28,00	2,00	27,00	18	24,31	14,17	5,64	9,66
Medium Dense Sand	28,00 - 29,25	1,25	28,65	18	24,31	14,17	5,64	9,66
Medium Dense Sand	29,25 - 31,50	2,25	30,85	18	24,31	14,17	5,64	9,66

Soil Type	Layer Location	Δp_1 (kPa)	Δp_2 (kPa)	Δp_3 (kPa)	Δp_4 (kPa)	$\Sigma \Delta p$ (kPa)	E_s / M_c (MPa)	S (m)
Fill	0,00 - 1,50	0,00	0,00	0,44	0,03	0,47	10	0,00
Fill	1,50 - 2,50	0,01	0,04	4,64	0,32	5,00	10	0,00
Soft - Medium Stiff Clay	2,50 - 3,50	0,03	0,18	15,78	1,37	17,36	2	0,01
Soft - Medium Stiff Clay	3,50 - 4,50	0,08	0,39	25,18	2,75	28,40	2	0,01
Soft - Medium Stiff Clay	4,50 - 5,25	0,14	0,65	31,32	4,20	36,31	2	0,01
Loose Sand	5,25 - 5,75	0,19	0,86	34,10	5,26	40,42	5	0,00
Soft - Medium Stiff Silty Clay - Clayey Silt	5,75 - 7,00	0,42	1,61	36,18	8,09	46,30	5	0,01
Soft - Medium Stiff Silty Clay - Clayey Silt	7,00 - 8,00	0,44	1,67	36,05	8,28	46,45	2,5	0,02
Soft - Medium Stiff Silty Clay - Clayey Silt	8,00 - 9,00	0,61	2,09	34,57	9,35	46,63	2,5	0,02
Soft - Medium Stiff Silty Clay - Clayey Silt	9,00 - 10,00	0,79	2,49	32,39	10,09	45,76	2,5	0,02
Soft - Medium Stiff Silty Clay - Clayey Silt	10,00 - 12,00	1,21	3,16	27,43	10,69	42,49	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	12,00 - 14,00	1,55	3,51	23,91	10,59	39,55	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	14,00 - 16,00	1,99	3,79	19,86	10,03	35,67	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	16,00 - 18,00	2,40	3,89	16,57	9,23	32,09	2,5	0,03
Stiff Clayey Silt with Sand	18,00 - 20,00	2,75	3,85	13,95	8,37	28,93	4	0,01
Stiff Clayey Silt with Sand	20,00 - 22,00	3,04	3,73	11,85	7,54	26,15	4	0,01
Stiff Clayey Silt with Sand	22,00 - 24,00	3,25	3,55	10,16	6,76	23,73	4	0,01
Stiff Clayey Silt with Sand	24,00 - 26,00	3,40	3,35	8,79	6,07	21,61	4	0,01
Stiff Clayey Silt with Sand	26,00 - 28,00	3,49	3,14	7,67	5,45	19,74	4	0,01
Medium Dense Sand	28,00 - 29,25	3,52	2,96	6,89	5,00	18,37	10	0,00
Medium Dense Sand	29,25 - 31,50	3,52	2,74	6,02	4,47	16,74	10	0,00
Total Settlement (m) ==>								0,29

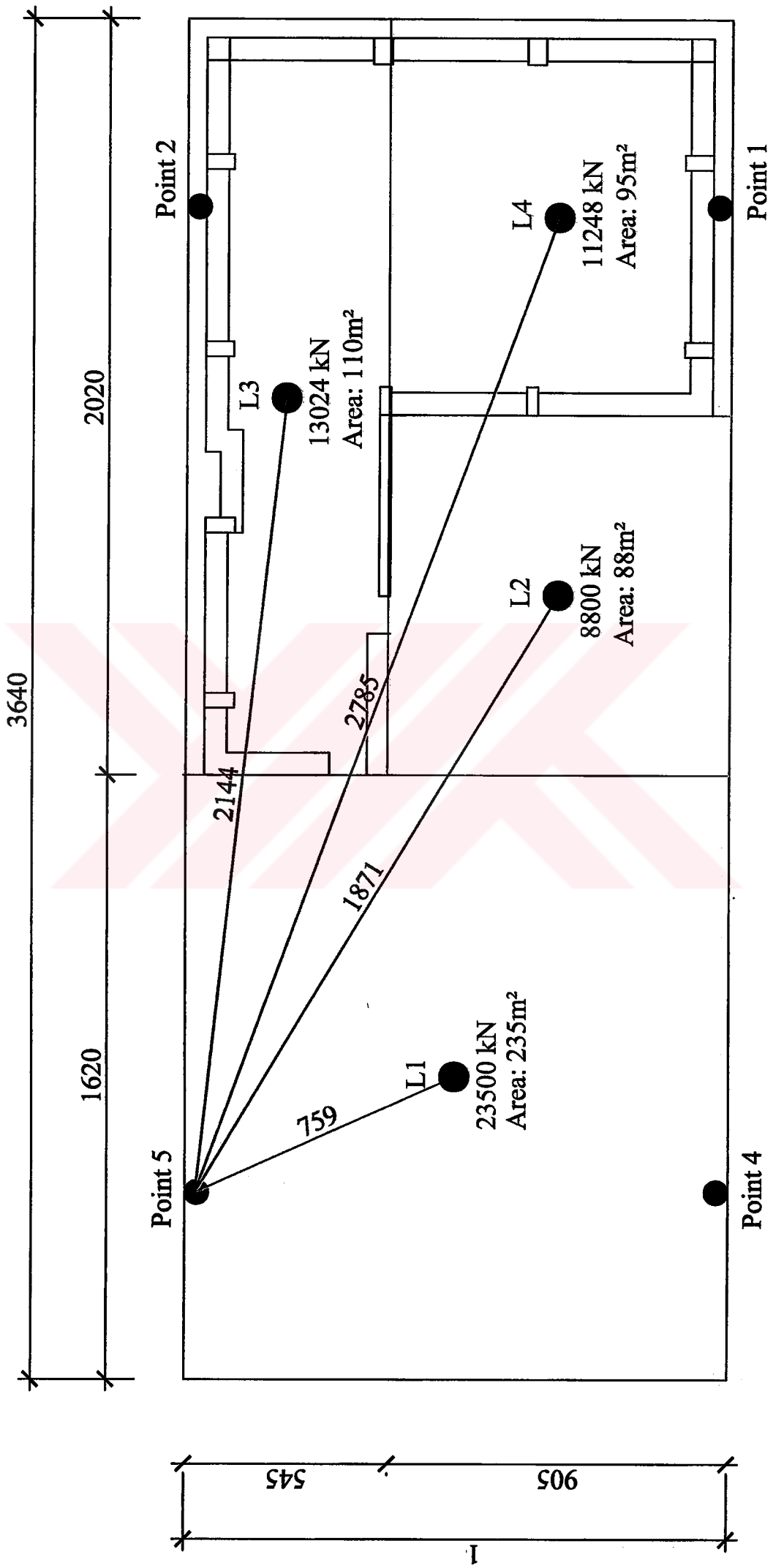
$$\Delta p = Q / ((z^2 \pi) (1/(1+2(\pi/z)^{2,3/2}) (Westergaard's Stress Distribution for point load)$$



Soil Type	Layer Location	H (m)	Layer Center Depth; z (m)	Unit Weight (kN/m ³)	r _{L1} (m)	r _{L2} (m)	r _{L3} (m)	r _{L4} (m)
Fill	0,00 - 1,50	1,50	0,75	19	7,59	16,57	24,18	26,46
Fill	1,50 - 2,50	1,00	1,75	19	7,59	16,57	24,18	26,46
Soft - Medium Stiff Clay	2,50 - 3,50	1,00	3,00	17	7,59	16,57	24,18	26,46
Soft - Medium Stiff Clay	3,50 - 4,50	1,00	4,00	17	7,59	16,57	24,18	26,46
Soft - Medium Stiff Clay	4,50 - 5,25	0,75	4,88	17	7,59	16,57	24,18	26,46
Loose Sand	5,25 - 5,75	0,50	5,50	18	7,59	16,57	24,18	26,46
Soft - Medium Stiff Silty Clay - Clayey Silt	5,75 - 7,00	1,25	7,35	17	7,59	16,57	24,18	26,46
Soft - Medium Stiff Silty Clay - Clayey Silt	7,00 - 8,00	1,00	7,50	17	7,59	16,57	24,18	26,46
Soft - Medium Stiff Silty Clay - Clayey Silt	8,00 - 9,00	1,00	8,50	17	7,59	16,57	24,18	26,46
Soft - Medium Stiff Silty Clay - Clayey Silt	9,00 - 10,00	1,00	9,50	17	7,59	16,57	24,18	26,46
Soft - Medium Stiff Silty Clay - Clayey Silt	10,00 - 12,00	2,00	11,50	17	7,59	16,57	24,18	26,46
Soft - Medium Stiff Silty Clay - Clayey Silt	12,00 - 14,00	2,00	13,00	17	7,59	16,57	24,18	26,46
Soft - Medium Stiff Silty Clay - Clayey Silt	14,00 - 16,00	2,00	15,00	17	7,59	16,57	24,18	26,46
Soft - Medium Stiff Silty Clay - Clayey Silt	16,00 - 18,00	2,00	17,00	17	7,59	16,57	24,18	26,46
Stiff Clayey Silt with Sand	18,00 - 20,00	2,00	19,00	18	7,59	16,57	24,18	26,46
Stiff Clayey Silt with Sand	20,00 - 22,00	2,00	21,00	18	7,59	16,57	24,18	26,46
Stiff Clayey Silt with Sand	22,00 - 24,00	2,00	23,00	18	7,59	16,57	24,18	26,46
Stiff Clayey Silt with Sand	24,00 - 26,00	2,00	25,00	18	7,59	16,57	24,18	26,46
Stiff Clayey Silt with Sand	26,00 - 28,00	2,00	27,00	18	7,59	16,57	24,18	26,46
Medium Dense Sand	28,00 - 29,25	1,25	28,65	18	7,59	16,57	24,18	26,46
Medium Dense Sand	29,25 - 31,50	2,25	30,85	18	7,59	16,57	24,18	26,46

Soil Type	Layer Location	Δp_1 (kPa)	Δp_2 (kPa)	Δp_3 (kPa)	Δp_4 (kPa)	$\Sigma \Delta p$ (kPa)	E_s / M_c (Mpa)	S (m)
Fill	0,00 - 1,50	0,18	0,00	0,00	0,00	0,19	10	0,00
Fill	1,50 - 2,50	2,10	0,02	0,00	0,00	2,12	10	0,00
Soft - Medium Stiff Clay	2,50 - 3,50	8,37	0,08	0,02	0,01	8,48	2	0,00
Soft - Medium Stiff Clay	3,50 - 4,50	15,45	0,19	0,05	0,03	15,71	2	0,01
Soft - Medium Stiff Clay	4,50 - 5,25	21,80	0,32	0,08	0,04	22,24	2	0,01
Loose Sand	5,25 - 5,75	25,80	0,43	0,11	0,06	26,40	5	0,00
Soft - Medium Stiff Silty Clay - Clayey Silt	5,75 - 7,00	33,84	0,85	0,24	0,14	35,07	5	0,01
Soft - Medium Stiff Silty Clay - Clayey Silt	7,00 - 8,00	34,22	0,89	0,25	0,14	35,51	2,5	0,01
Soft - Medium Stiff Silty Clay - Clayey Silt	8,00 - 9,00	35,86	1,15	0,35	0,20	37,55	2,5	0,02
Soft - Medium Stiff Silty Clay - Clayey Silt	9,00 - 10,00	36,19	1,42	0,45	0,26	38,32	2,5	0,02
Soft - Medium Stiff Silty Clay - Clayey Silt	10,00 - 12,00	34,36	1,91	0,69	0,41	37,37	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	12,00 - 14,00	31,89	2,23	0,88	0,53	35,52	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	14,00 - 16,00	28,20	2,54	1,13	0,70	32,57	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	16,00 - 18,00	24,65	2,74	1,35	0,86	29,60	2,5	0,02
Stiff Clayey Silt with Sand	18,00 - 20,00	21,47	2,83	1,55	1,00	26,85	4	0,01
Stiff Clayey Silt with Sand	20,00 - 22,00	18,72	2,84	1,71	1,13	24,40	4	0,01
Stiff Clayey Silt with Sand	22,00 - 24,00	16,38	2,79	1,83	1,23	22,23	4	0,01
Stiff Clayey Silt with Sand	24,00 - 26,00	14,40	2,70	1,91	1,31	20,33	4	0,01
Stiff Clayey Silt with Sand	26,00 - 28,00	12,73	2,59	1,96	1,37	18,64	4	0,01
Medium Dense Sand	28,00 - 29,25	11,54	2,49	1,97	1,40	17,40	10	0,00
Medium Dense Sand	29,25 - 31,50	10,18	2,34	1,97	1,42	15,92	10	0,00
Total Settlement (m) ==>								0,25

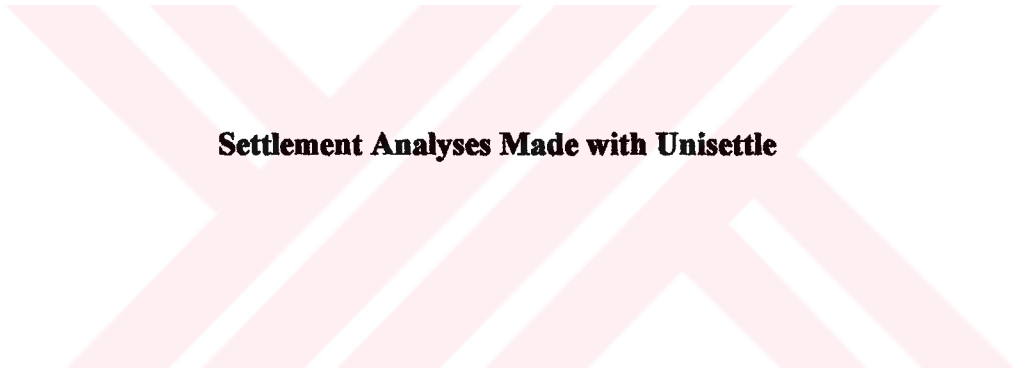
$$\Delta p = Q / ((z^2 \cdot \pi) (1/(1+2(r/z)^{2,3/2}) \text{ (Westergaard's Stress Distribution for point load)})$$



Soil Type	Layer Location	H (m)	Layer Center Depth; z (m)	Unit Weight (kN/m ³)	r _{L1} (m)	r _{L2} (m)	r _{L3} (m)	r _{L4} (m)
Fill	0,00 - 1,50	1,50	0,75	19	7,59	18,71	21,44	27,85
Fill	1,50 - 2,50	1,00	1,75	19	7,59	18,71	21,44	27,85
Soft - Medium Stiff Clay	2,50 - 3,50	1,00	3,00	17	7,59	18,71	21,44	27,85
Soft - Medium Stiff Clay	3,50 - 4,50	1,00	4,00	17	7,59	18,71	21,44	27,85
Soft - Medium Stiff Clay	4,50 - 5,25	0,75	4,88	17	7,59	18,71	21,44	27,85
Loose Sand	5,25 - 5,75	0,50	5,50	18	7,59	18,71	21,44	27,85
Soft - Medium Stiff Silty Clay - Clayey Silt	5,75 - 7,00	1,25	7,35	17	7,59	18,71	21,44	27,85
Soft - Medium Stiff Silty Clay - Clayey Silt	7,00 - 8,00	1,00	7,50	17	7,59	18,71	21,44	27,85
Soft - Medium Stiff Silty Clay - Clayey Silt	8,00 - 9,00	1,00	8,50	17	7,59	18,71	21,44	27,85
Soft - Medium Stiff Silty Clay - Clayey Silt	9,00 - 10,00	1,00	9,50	17	7,59	18,71	21,44	27,85
Soft - Medium Stiff Silty Clay - Clayey Silt	10,00 - 12,00	2,00	11,50	17	7,59	18,71	21,44	27,85
Soft - Medium Stiff Silty Clay - Clayey Silt	12,00 - 14,00	2,00	13,00	17	7,59	18,71	21,44	27,85
Soft - Medium Stiff Silty Clay - Clayey Silt	14,00 - 16,00	2,00	15,00	17	7,59	18,71	21,44	27,85
Soft - Medium Stiff Silty Clay - Clayey Silt	16,00 - 18,00	2,00	17,00	17	7,59	18,71	21,44	27,85
Stiff Clayey Silt with Sand	18,00 - 20,00	2,00	19,00	18	7,59	18,71	21,44	27,85
Stiff Clayey Silt with Sand	20,00 - 22,00	2,00	21,00	18	7,59	18,71	21,44	27,85
Stiff Clayey Silt with Sand	22,00 - 24,00	2,00	23,00	18	7,59	18,71	21,44	27,85
Stiff Clayey Silt with Sand	24,00 - 26,00	2,00	25,00	18	7,59	18,71	21,44	27,85
Stiff Clayey Silt with Sand	26,00 - 28,00	2,00	27,00	18	7,59	18,71	21,44	27,85
Medium Dense Sand	28,00 - 29,25	1,25	28,65	18	7,59	18,71	21,44	27,85
Medium Dense Sand	29,25 - 31,50	2,25	30,85	18	7,59	18,71	21,44	27,85

Soil Type	Layer Location	Δp_1 (kPa)	Δp_2 (kPa)	Δp_3 (kPa)	Δp_4 (kPa)	$\Sigma \Delta p$ (kPa)	E_s / M_c (Mpa)	S (m)
Fill	0,00 - 1,50	0,18	0,00	0,00	0,00	0,18	10	0,00
Fill	1,50 - 2,50	2,10	0,01	0,01	0,00	2,12	10	0,00
Soft - Medium Stiff Clay	2,50 - 3,50	8,37	0,05	0,04	0,01	8,46	2	0,00
Soft - Medium Stiff Clay	3,50 - 4,50	15,45	0,10	0,08	0,02	15,65	2	0,01
Soft - Medium Stiff Clay	4,50 - 5,25	21,80	0,18	0,14	0,03	22,15	2	0,01
Loose Sand	5,25 - 5,75	25,80	0,25	0,19	0,05	26,29	5	0,00
Soft - Medium Stiff Silty Clay - Clayey Silt	5,75 - 7,00	33,84	0,51	0,41	0,11	34,87	5	0,01
Soft - Medium Stiff Silty Clay - Clayey Silt	7,00 - 8,00	34,22	0,53	0,43	0,11	35,30	2,5	0,01
Soft - Medium Stiff Silty Clay - Clayey Silt	8,00 - 9,00	35,86	0,70	0,59	0,16	37,31	2,5	0,01
Soft - Medium Stiff Silty Clay - Clayey Silt	9,00 - 10,00	36,19	0,89	0,75	0,21	38,03	2,5	0,02
Soft - Medium Stiff Silty Clay - Clayey Silt	10,00 - 12,00	34,36	1,25	1,11	0,33	37,05	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	12,00 - 14,00	31,89	1,50	1,38	0,43	35,20	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	14,00 - 16,00	28,20	1,79	1,71	0,57	32,28	2,5	0,03
Soft - Medium Stiff Silty Clay - Clayey Silt	16,00 - 18,00	24,65	2,00	1,99	0,71	29,35	2,5	0,02
Stiff Clayey Silt with Sand	18,00 - 20,00	21,47	2,14	2,21	0,85	26,66	4	0,01
Stiff Clayey Silt with Sand	20,00 - 22,00	18,72	2,21	2,37	0,96	24,26	4	0,01
Stiff Clayey Silt with Sand	22,00 - 24,00	16,38	2,23	2,46	1,06	22,14	4	0,01
Stiff Clayey Silt with Sand	24,00 - 26,00	14,40	2,21	2,51	1,14	20,26	4	0,01
Stiff Clayey Silt with Sand	26,00 - 28,00	12,73	2,16	2,51	1,20	18,60	4	0,01
Medium Dense Sand	28,00 - 29,25	11,54	2,11	2,49	1,24	17,38	10	0,00
Medium Dense Sand	29,25 - 31,50	10,18	2,02	2,44	1,27	15,91	10	0,00
Total Settlement (m) ==>								0,25

$$\Delta p = Q / ((z^2 \pi) (1/(1+2(r/z)^{2/3}))^{3/2} \text{ (Westergaard's Stress Distribution for point load)})$$



Settlement Analyses Made with Unisettle

Soil Data

Layer	:	1	2	3	4	5
Type	:	Dolguorta	kati KIL	gevsek KUM	Killi SILT-siltli KIL	Kumlu Killi SI
Thick. (m)	:	1,50	2,75	0,50	12,25	10,00
Depth (m)	:	1,50	4,25	4,75	17,00	27,00
Density (kg/m ³)	:	1800,00	1700,00	1800,00	1800,00	1800,00
Exp. j	:	0,50	0,00	0,50	0,00	0,00
m(top)	:	100,00	20,00	50,00	25,00	40,00
m(bot)	:	100,00	20,00	50,00	25,00	40,00
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	:	gevsek, siltli KUM	Morta SIKI	Siltli KUM		
Thick. (m)	:	3,50	1,50	0,00	0,00	0,00
Depth (m)	:	30,50	32,00	32,00	32,00	32,00
Density (kg/m ³)	:	1800,00	1900,00	0,00	0,00	0,00
Exp. j	:	0,50	0,50	0,00	0,00	0,00
m(top)	:	50,00	100,00	0,00	0,00	0,00
m(bot)	:	50,00	100,00	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

Loading Data...

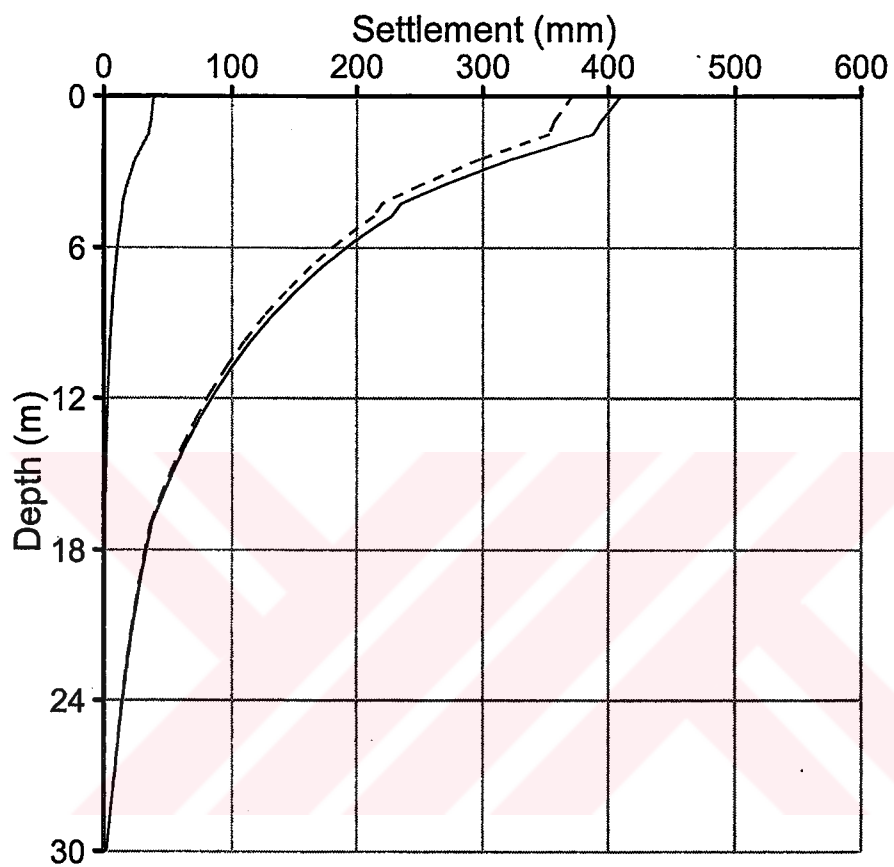
Load 1	X	Y	Load	Stress
Load type : Polygon	(m)	(m)	(kN)	(kPa)
Appl.:Fin	0,00	0,00	46575,00	100,00
Stress Distr.: Integration	0,00	34,50		
Precision(m)= 1,00	13,50	34,50		
Depth (m): 0,00	13,50	0,00		
Elev. (m): 0,00				
Load 2	X	Y	Load	Stress
Load type : Polygon	(m)	(m)	(kN)	(kPa)
Appl.:Fin	0,00	0,00	3450,60	28,40
Stress Distr.: Integration	0,00	9,00		
Precision(m)= 1,00	13,50	9,00		
Depth (m): 0,00	13,50	0,00		
Elev. (m): 0,00				
Load 3	X	Y	Load	Stress
Load type : Polygon	(m)	(m)	(kN)	(kPa)
Appl.:Fin	10,00	9,00	447,30	28,40
Stress Distr.: Integration	10,00	13,50		
Precision(m)= 1,00	13,50	13,50		
Depth (m): 0,00	13,50	9,00		
Elev. (m): 0,00				

Differential Settlement Analysis - Boussinesq

----- (0,,4,5) (13,5,4,5) -----					
Depth	Incr.	Total	Incr.	Total	Diff.
(m)	S	S	S	S	S
(mm)	(mm)	(mm)	(mm)	(mm)	(mm)

Layer 1	Dolgu	1800 kg/m ³	j=0,5		
0,00	14,36	370,99	16,46	410,35	39,36
1,00	4,38	356,63	6,16	393,89	37,26
1,50	0,00	352,25	0,00	387,73	35,48
Layer 2	o rta kati KIL	1700 kg/m ³	j=0,		
GWT 1,50	54,40	352,25	64,81	387,73	35,48
2,50	46,80	297,84	53,33	322,92	25,07
3,50	30,92	251,04	34,55	269,59	18,54
4,25	0,00	220,12	0,00	235,04	14,92
Layer 3	gevsek KUM	1800 kg/m ³	j=0,5		
4,25	6,34	220,12	7,17	235,04	14,92
4,75	0,00	213,78	0,00	227,87	14,09
Layer 4	Killi SIL T-siltli KIL	1800 kg/m ³	j=0,		
4,75	26,86	213,78	29,53	227,87	14,09
5,75	23,24	186,92	25,35	198,34	11,42
6,75	20,25	163,68	21,95	172,99	9,31
7,75	17,76	143,43	19,14	151,04	7,61
8,75	15,66	125,67	16,79	131,90	6,23
9,75	13,89	110,01	14,82	115,11	5,10
10,75	12,37	96,12	13,14	100,29	4,17
11,75	11,06	83,76	11,70	87,16	3,40
12,75	9,93	72,70	10,46	75,46	2,76
13,75	8,94	62,77	9,38	65,00	2,23
14,75	8,08	53,83	8,45	55,62	1,78
15,75	7,32	45,76	7,63	47,17	1,41
16,75	1,72	38,44	1,79	39,54	1,10
17,00	0,00	36,72	0,00	37,74	1,03
Layer 5	Kuml u Killi SILT	1800 kg/m ³	j=0,		
17,00	4,06	36,72	4,22	37,74	1,03
18,00	3,70	32,66	3,83	33,53	0,87
19,00	3,38	28,96	3,49	29,69	0,73
20,00	3,09	25,58	3,19	26,20	0,62
21,00	2,84	22,49	2,92	23,01	0,52
22,00	2,61	19,65	2,68	20,08	0,44
23,00	2,40	17,04	2,47	17,40	0,36
24,00	2,22	14,63	2,27	14,93	0,30
25,00	2,05	12,41	2,10	12,66	0,25
26,00	1,90	10,36	1,94	10,56	0,20
27,00	0,00	8,46	0,00	8,62	0,16
Layer 6	gevsek , siltli KUM	1800 kg/m ³	j=0,5		
27,00	2,19	8,46	2,23	8,62	0,16
28,00	2,06	6,27	2,10	6,39	0,11
29,00	1,95	4,21	1,99	4,28	0,07
30,00	0,93	2,26	0,95	2,30	0,04
30,50	0,00	1,33	0,00	1,35	0,02
Layer 7	orta SIK I Siltli KUM	1900 kg/m ³	j=0,5		
30,50	0,90	1,33	0,91	1,35	0,02
31,50	0,43	0,43	0,44	0,44	0,01
32,00	0,00	0,00	0,00	0,00	0,00

Differential Settlement

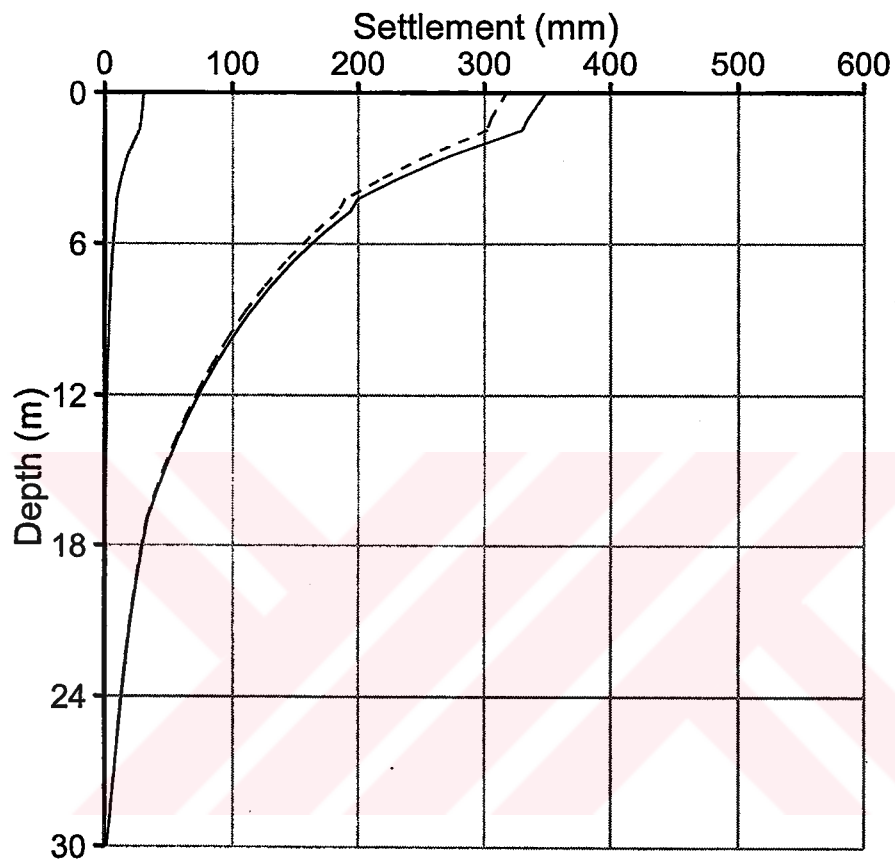


Differential Settlement Analysis - Boussinesq

4. nolu nokta					
(0,,30,5)					
Depth	Incr.	Total			
(m)	S	S	Incr.	Total	Diff.
	(mm)	(mm)	S	S	S
(m)	(mm)	(mm)	(mm)	(mm)	(mm)

	Layer 1	Dolgu	1800 kg/m^3	j=0,5	
	0,00	12,18	317,90	14,01	349,02
	1,00	3,61	305,72	5,12	335,01
	1,50	0,00	302,11	0,00	329,89
	Layer 2	o rta kati KIL	1700 kg/m^3	j=0,	
GWT	1,50	46,50	302,11	55,90	329,89
	2,50	39,75	255,61	45,33	273,99
	3,50	26,19	215,86	29,10	228,66
	4,25	0,00	189,67	0,00	199,56
	Layer 3	gevsek KUM	1800 kg/m^3	j=0,5	
	4,25	5,20	189,67	5,80	199,56
	4,75	0,00	184,46	0,00	193,75
	Layer 4	Killi SIL T-siltli KIL	1800 kg/m^3	j=0,	
	4,75	22,71	184,46	24,67	193,75
	5,75	19,67	161,75	21,12	169,09
	6,75	17,17	142,09	18,29	147,97
	7,75	15,10	124,92	15,98	129,68
	8,75	13,36	109,82	14,06	113,70
	9,75	11,88	96,46	12,45	99,64
	10,75	10,62	84,58	11,09	87,19
	11,75	9,54	73,96	9,92	76,10
	12,75	8,60	64,42	8,92	66,17
	13,75	7,77	55,83	8,04	57,25
	14,75	7,05	48,05	7,28	49,21
	15,75	6,42	41,00	6,61	41,93
	16,75	1,51	34,58	1,56	35,31
	17,00	0,00	33,07	0,00	33,76
	Layer 5	Kuml u Killi SILT	1800 kg/m^3	j=0,	
	17,00	3,58	33,07	3,68	33,76
	18,00	3,27	29,49	3,36	30,08
	19,00	3,00	26,22	3,08	26,72
	20,00	2,76	23,21	2,83	23,64
	21,00	2,54	20,45	2,60	20,82
	22,00	2,35	17,91	2,40	18,22
	23,00	2,17	15,56	2,21	15,82
	24,00	2,01	13,39	2,05	13,61
	25,00	1,87	11,38	1,90	11,56
	26,00	1,73	9,51	1,76	9,66
	27,00	0,00	7,78	0,00	7,89
	Layer 6	gevsek , siltli KUM	1800 kg/m^3	j=0,5	
	27,00	2,00	7,78	2,03	7,89
	28,00	1,89	5,78	1,92	5,86
	29,00	1,79	3,88	1,82	3,94
	30,00	0,86	2,09	0,87	2,12
	30,50	0,00	1,23	0,00	1,24
	Layer 7	orta SIK I Siltli KUM	1900 kg/m^3	j=0,5	
	30,50	0,83	1,23	0,84	1,24
	31,50	0,40	0,40	0,40	0,40
	32,00	0,00	0,00	0,00	0,00

Differential Settlement





Settlement Analyses Made with Unisettle after Preloading

Soil Data

Layer	:	1	2	3	4	5
Type	:	Dolguorta	kati KIL	gevsek KUM	Killi Silt-siltli KIL	Kumlu Killi SI
Thick. (m)	:	1,50	2,75	0,50	12,25	10,00
Depth (m)	:	1,50	4,25	4,75	17,00	27,00
Density (kg/m ³)	:	1800,00	1700,00	1800,00	1800,00	1800,00
Exp. j	:	0,50	0,00	0,50	0,00	0,00
m(top)	:	100,00	20,00	50,00	25,00	40,00
m(bot)	:	100,00	20,00	50,00	25,00	40,00
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	:	gevsek,	siltli KUM	orta SIKI	Siltli KUM	
Thick. (m)	:	3,50	3,50	0,00	0,00	0,00
Depth (m)	:	30,50	34,00	34,00	34,00	34,00
Density (kg/m ³)	:	1800,00	1900,00	0,00	0,00	0,00
Exp. j	:	0,50	0,50	0,00	0,00	0,00
m(top)	:	50,00	100,00	0,00	0,00	0,00
m(bot)	:	50,00	100,00	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

Loading Data...

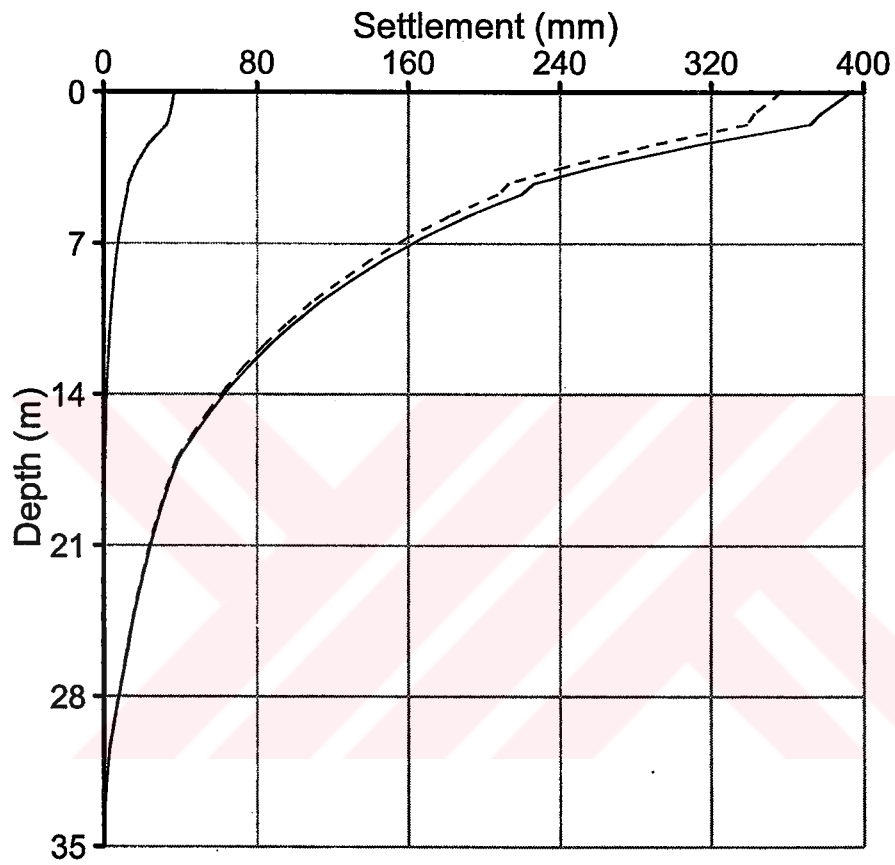
Load 1	X	Y	Load	Stress
Load type : Polygon	(m)	(m)	(kN)	(kPa)
Appl.:Fin	0,00	0,00	46575,00	100,00
Stress Distr.: Integration	0,00	34,50		
Precision(m)= 1,00	13,50	34,50		
Depth (m): 0,00	13,50	0,00		
Elev. (m): 0,00				
Load 2	X	Y	Load	Stress
Load type : Polygon	(m)	(m)	(kN)	(kPa)
Appl.:Fin	0,00	26,50	4320,00	40,00
Stress Distr.: Integration	0,00	34,50		
Precision(m)= 1,00	13,50	34,50		
Depth (m): 0,00	13,50	26,50		
Elev. (m): 0,00				
Load 3	X	Y	Load	Stress
Load type : Polygon	(m)	(m)	(kN)	(kPa)
Appl.:Fin	0,00	20,00	1300,00	40,00
Stress Distr.: Integration	0,00	26,50		
Precision(m)= 1,00	5,00	26,50		
Depth (m): 0,00	5,00	20,00		
Elev. (m): 0,00				
Load 4	X	Y	Load	Stress
Load type : Polygon	(m)	(m)	(kN)	(kPa)
Appl.:Fin	0,00	0,00	2235,60	18,40
Stress Distr.: Integration	0,00	9,00		
Precision(m)= 1,00	13,50	9,00		
Depth (m): 0,00	13,50	0,00		
Elev. (m): 0,00				
Load 5	X	Y	Load	Stress
Load type : Polygon	(m)	(m)	(kN)	(kPa)
Appl.:Fin	10,00	9,00	289,80	18,40
Stress Distr.: Integration	10,00	13,50		
Precision(m)= 1,00	13,50	13,50		
Depth (m): 0,00	13,50	9,00		
Elev. (m): 0,00				

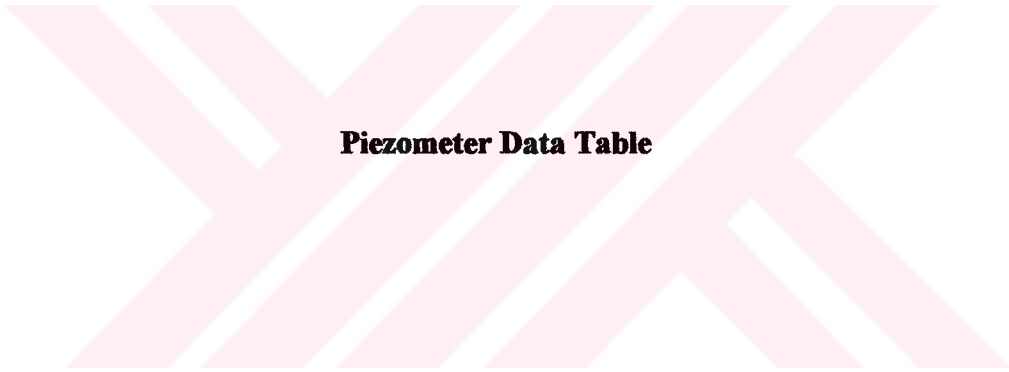
Differential Settlement Analysis - Boussinesq

1000 nokta 2 nokta						
(0,,4,5) (13,5,4,5)						
Depth	Incr.	Total	Incr.	Total	Diff.	
(m)	S	S	S	S	S	

Layer 1		Dolgu	1800 kg/m ³	j=0,5		
0,00	13,62	356,46	15,63	393,28	36,82	
1,00	4,12	342,85	5,81	377,66	34,81	
1,50	0,00	338,73	0,00	371,85	33,12	
GWT	Layer 2 orta kati KIL		1700 kg/m ³	j=0,		
	1,50	51,76	338,73	61,89	371,85	33,12
	2,50	44,44	286,97	50,72	309,96	23,00
	3,50	29,33	242,53	32,78	259,24	16,71
	4,25	0,00	213,20	0,00	226,45	13,26
Layer 3		gevsek KUM	1800 kg/m ³	j=0,5		
4,25	5,95	213,20	6,72	226,45	13,26	
4,75	0,00	207,24	0,00	219,73	12,49	
Layer 4		Killi Siltli KIL	1800 kg/m ³	j=0,		
4,75	25,46	207,24	27,96	219,73	12,49	
5,75	22,05	181,78	23,99	191,77	9,99	
6,75	19,23	159,73	20,77	167,78	8,05	
7,75	16,89	140,50	18,13	147,01	6,51	
8,75	14,93	123,61	15,93	128,88	5,27	
9,75	13,26	108,68	14,08	112,95	4,27	
10,75	11,84	95,42	12,51	98,88	3,46	
11,75	10,62	83,58	11,17	86,37	2,79	
12,75	9,56	72,96	10,01	75,20	2,24	
13,75	8,64	63,40	9,01	65,19	1,79	
14,75	7,83	54,76	8,14	56,18	1,42	
15,75	7,12	46,93	7,37	48,05	1,11	
16,75	1,68	39,82	1,73	40,67	0,86	
17,00	0,00	38,14	0,00	38,94	0,80	
Layer 5		Kumlu Killi Silt	1800 kg/m ³	j=0,		
17,00	3,96	38,14	4,09	38,94	0,80	
18,00	3,62	34,18	3,73	34,85	0,67	
19,00	3,32	30,55	3,41	31,12	0,57	
20,00	3,05	27,23	3,13	27,71	0,48	
21,00	2,81	24,18	2,87	24,58	0,40	
22,00	2,59	21,37	2,65	21,70	0,33	
23,00	2,39	18,78	2,44	19,06	0,28	
24,00	2,22	16,38	2,26	16,62	0,23	
25,00	2,06	14,17	2,09	14,36	0,19	
26,00	1,91	12,11	1,94	12,27	0,16	
27,00	0,00	10,20	0,00	10,33	0,13	
Layer 6		gevsek , siltli KUM	1800 kg/m ³	j=0,5		
27,00	2,20	10,20	2,24	10,33	0,13	
28,00	2,09	8,00	2,11	8,09	0,10	
29,00	1,98	5,91	2,00	5,98	0,07	
30,00	0,95	3,94	0,96	3,98	0,04	
30,50	0,00	2,99	0,00	3,02	0,03	
Layer 7		orta SIK I Siltli KUM	1900 kg/m ³	j=0,5		
30,50	0,91	2,99	0,92	3,02	0,03	
31,50	0,86	2,08	0,87	2,10	0,02	
32,50	0,82	1,21	0,83	1,22	0,01	

Differential Settlement





Piezometer Data Table

Encountered Pore Water Pressure Data via Piezometers

Reading Date	Piezometer 1 - 8.0m	Piezometer 1 - 14.0m	Piezometer 2 - 8.0m	Piezometer 2 - 14.0m
12.07.2002	57,07	118,01	67,92	123,98
19.07.2002	46,22	175,98	64,71	156,2
26.07.2002	49,69	175,4	63,25	157,04
02.08.2002	51,74	175,77	64,1	154,24
09.08.2002	58,74	175,4	64,17	151,59
16.08.2002	63,72	173,91	64,71	148,51
23.08.2002	62,69	174,81	64,55	151,06
29.08.2002	51,74	173,86	63,64	152,88
06.09.2002	63,72	173,44	65,17	150,78
13.09.2002	68,15	173,33	66,01	150,71
20.09.2002	82,65	173,28	68,38	155,8
27.09.2002	75,71	173,01	66,7	154,12
04.10.2002	83,13	172,8	68,38	157,16
11.10.2002	79,65	172,91	68	156,32
18.10.2002	82,17	172,33	67,92	156,71
25.10.2002	85,65	172,22	67,85	157,54
01.11.2002	89,12	170,68	68,69	158,38
08.11.2002	84,69	170,31	67,85	157,54
15.11.2002	88,64	170,68	68,69	158,83
22.11.2002	91,16	170,1	68,15	158,76
29.11.2002	85,17	169,83	67,85	158,91
06.12.2002	90,69	169,14	68,15	159,22
13.12.2002	104,57	168,24	69,68	161,2
23.12.2002	112,94	171,38	71,37	164,24
27.12.2002	182,37	279,36	156,73	270,51
03.01.2003	159,4	246,02	141,2	251,05
10.01.2003	129,35	226,37	110,3	235,44



Jet Grout Application Plan for KEDAŞ Building

ÖZGEÇMİŞ

Hüsnü Korhan Özalp 01.08.1979 tarihinde Ankara'da doğdu. İlk ve orta öğrenimini İsviçre'nin Zürih kentinde tamamladıktan sonra lise öğrenimini tamamlamak üzere Ankara'ya döndü ve Ankara Anadolu Lisesi'nden mezun oldu. 1997 yılında başladığı Lisans eğitimini Dokuz Eylül Üniversitesi Mühendislik Fakültesi İnşaat Mühendisliği Bölümü'nde 2001 yılında tamamlayan Hüsnü Korhan Özalp, aynı yıl, halen bünyesinde bulunduğu İstanbul Teknik Üniversitesi İnşaat Fakültesine, Geoteknik Ana Bilim Dalı'nda Yüksek Lisans eğitimi almak isteğiyle başvurdu ve kabul oldu.

CIRRUCULUM VITAE

Hüsnü Korhan Özalp was borne in Ankara on 01.08.1979. He had finalized his primary and secondary education in Zurich, Switzerland. Afterwards he went to the Ankara Anatolian High School. He had obtained his bachelor's degree from the Civil Engineering Department of the Engineering Faculty of Dokuz Eylül University in 2001. After graduation, he applied for the Master Program of Istanbul Technical University, Civil Engineering Faculty, Geotechnical Department and was accepted.

