SOIL IMPROVEMENT CASE STUDIES USING JET GROUTS

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İZMİR

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The author hopes that jet-grouting methods, calculation of operating parameters, and a case study on the performance of jet-grout columns during earthquakes will be useful for professional engineers and researchers.

ABSTRACT

Jet grouting technology has been found widespread application in recent years in Turkey. Necessary technology is locally available in most places. Applications can be completed in a relatively less time and the cost is not very high when compared with other alternatives. Besides jet grout columns can also function as load carrying deep foundations. This aspect of the technology has triggered some debates especially over the lateral load carrying capacity of jet grout columns since they are not manufactured with traditional reinforcing steel bars. Reported successful performance of jet grout applications during the 1999 Marmara Earthquake were not enough to avoid suspicions of practicing engineers. Main reason for this is thought to be the insufficient information regarding load transfer mechanism from the superstructure to the column head and response of the columns to kinematical loads. This dissertation has targeted both aspects of seismic lateral loads within the context of a jet grouting application case study in Bostanli district of the city of Izmir. Simple analytical and numerical methods that are commonly available were used. It was found that capping layer can be quite effective in reducing transferred inertial loads and inertial loads can be much more smaller with the positive contribution of soil-structure interaction effects to column response. Further studies on this subject are considered as feasible. Kinematical effects, on the other hand, were found to be negligible for the studied soil profile and earthquake loading. Calculated bending moments were several magnitudes smaller than those for the reinforced concrete pile and it is thought that flexible soilcrete material is the main cause for this positive behavior.

ÖZET

Jet grout teknolijisi son yıllarda Türkiye'de geniş uygulama alanı bulmuştur. Gerekli olan teknolojiye bir çok yerde ulaşılabilmektedir. Uygulamalar diğer zemin iyileştirme yöntemleri ile kıyaslandığında daha kısa zamanda tamamlanabilmektedir ve maliyetler çok yüksek değildir. Ayrıca jet grout kolonlar yük taşıyan derin temeller olarak da davranabilmektedir. Yöntemin bu yanı, özellikle kolonların yanal yük taşıma kapasitesi üzerinde tartışmaların doğmasına neden olmuştur. Zira kolonlar geleneksel çelik donatılar ile üretilmemektedir. Jet grout uygulamalarının 1999 Marmara Depremi sırasındaki başarılı performansı da uygulamadaki mühendislerin kuşkularını gidermekte yeterli olmamıştır. Bunun başlıca nedeninin üst yapıdan kolonlara yük aktarma mekanizması ve kolonların kinematik yüklere tepkisi hakkındaki yetersiz bilgi olduğu düşünülmektedir. Bu tez, sismik yanal yüklerin bu iki yönünü İzmir ili Bostanlı bölgesindeki bir jet grout uygulaması kapsamında ele almayı hedeflemiştir. Rahatça ulaşılabilen basit analitik ve sayısal yöntemler kullanılmıştır. Jet grout ile radye temel arasına yerleştirilen granüler tabakanın kolonlara aktarılan üst yapı yüklerini azaltmakta oldukça etkili olabileceği bulunmuştur. Zemin-yapı etkileşimi etkilerinin olumlu katkılarıyla atalet yükleri çok daha küçük olabilecektir. Bu konu hakkında daha fazla çalışma yapılmasında yarar vardır. Diğer yandan kinematik etkiler ele alınan zemin profili ve deprem yükü için ihmal edilebilir bulunmuştur. Hesaplanan eğilme momentleri betonarme kazık için hesaplananlar ile karşılaştırıldığında bir çok mertebe daha düşüktür ve bu olumlu davranışa jet grout kolonun esnek malzeme yapısının neden olduğu düşünülmektedir.

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CHAPTER ONE
INTRODUCTION

Soil improvement applications have gained acceleration following 1999 Marmara Earthquake in Turkey. Both popular comments on the influence of local soil conditions and good performance of some critical structures on improved grounds became effective on this trend. Jet-grouting technology among other available method has found more opportunities due to faster construction rate and relative ease during production. Three successful case histories of jet grout columns during the Marmara Earthquake were attracted attention of both researchers and practicing engineers. All cases were reported to be located in saturated sedimentary soil deposits as explained in the fifth chapter of this dissertation. No signs of liquefaction were observed in the treated areas although nearby soils between improved zones liquefied in the earthquake.

Despite the above-mentioned case histories some designers have approached to jet-grouting method as a foundation element with suspicion since they were not confident about the lateral load carrying capacity of the soilcrete columns. It is known that lateral loads mostly govern the design of deep foundations especially during large seismic events and this aspect of the design should be taken into consideration if projected jet grout columns are thought to function as load carrying deep foundations.

This dissertation specifically targeted study of performance of soilcrete columns under lateral seismic loads. There were two aspects considered to be most influential on the response of jet grout columns during a seismic event: inertial loads and kinematical loads. These two aspects were studied separately. Attempt has been made to observe influence of capping layer on the transfer of base shear forces to columns.

Nonlinear pseudo-static p-y curve method was employed during the analyses of inertial response whereas linear closed form solution method was the preferred approach for the study of kinematical loading case.

First two chapters of the dissertation are devoted to the general review of soil improvement methods and jet grouting technology. The fourth chapter is spared for calculation methods of jet grout operation parameters. General aspects of a simple computer algorithm for the estimation of certain parameters are also included in this chapter. The fifth chapter contains case histories of improved ground with jet grouting, inertial and kinematical load analyses. Analyses were made within the context of a jet grout column case study in Bostanli district of the city of Izmir. Final conclusions and future recommendations are given in the sixth chapter.

CHAPTER TWO
GENERAL REVIEW OF SOIL IMPROVEMENT METHODS

Soil improvement methods are developed to stabilize and thereby improve weak or problematic soils that would otherwise impose negative effects on constructed facilities. On the basis of the mechanism by which they improve engineering properties of soil deposits, soil improvement methods can be commonly divided into three major categories: densification, reinforcement, and grouting.

2.1. DENSIFICATION

The stiffness and strength of the soil are higher if the particles are packed in a denser configuration. Besides, the tendency to generate positive pore water pressure under seismic loading conditions is less in denser soils. Therefore densification is one of the most effective and commonly used means of modification of soil characteristics in cohessionless soils for mitigation of seismic hazards.

2.1.1. Vibroflotation

The vibroflot, a special apparatus attached to a crane, is used to densify soil deposits of clean granular soils with fines and clay contents being less than 20% and 3%, respectively.

Usually 10 to 16 ft long cylindrical vibrating apparatus of which diameter is 12 to 18 inch is mounted eccentrically on a central shaft driven by electric or hydraulic power. Vibroflotation has been used successfully to densify soils to a depth of 35 m.

The most significant limitations of vibroflotation method are the need for special equipments and its inapplicability in soils containing large size particles (i.e. cobbles and boulders).

2.1.2. Vibroflot

A vibratory driving hammer is used to vibrate a long probe in soil layers. The probe is then withdrawn while still being vibrated to densify the soil. Suitable soil types to this method are sands, silty sands, gravelly sands and soils with fine content fraction less than 20%. Grid spacing in vibro rod application is usually smaller than that in vibroflotation. Vibro rod systems are most effective in soils with similar characteristics to those mentioned for vibroflotation method. Effective depth in vibro rod technique is usually said to be around 30 m (Harder, 1984 and Dobson, 1987).

Effects of modification are observed in various parameters such as relative density (D_r), standard penetration test blow count (SPT-N), and cone penetration test (CPT) tip resistance (q_c). It has been found that D_r may exceed 80%; q_c may increase upto 10~12 MPa and SPT-N may be equal to 25 or higher following vibroflot application.

2.1.3. Deep Dynamic Compaction

Deep dynamic compaction is applied by dropping a heavy weight on the ground surface. The drops follow a grid pattern. Weight of the drop mass ranges from 6 to 30 tons. Drop height is generally between 10 and 30 m. The number of drops varies between 3 and 8 depending on soil type and target density. The method is more suitable for sands and silty sands. Influence of modification on soil parameters is quite similar to vibroflot application. The effective application depth is 10 m in deep dynamic compaction method.

Deep dynamic compaction is a low cost and relatively simple improvement method. However, the effective depth is limited, wide space is needed during application, and vibrations induced during compaction may cause negative impacts on neighboring structures.

2.2. REINFORCEMENT

In some cases the stiffness and strength of an existing soil deposit may be improved by installing reinforcing elements in the soil. These elements may be structural materials, such as steel, concrete, or timber column. In some cases geotextiles can be used. Geomaterials such as stone columns may also be utilized.

2.2.1. Stone Columns

Soil improvement using stone column technique can be carried out by the installation of dense columns of gravel. This method is generally used in both fine and coarse grained soils. As discussed previously, stone columns may be constructed by introducing gravel during the process of vibroflotation (Brown, 1977). In Franki method, however, a steel casing initially closed at the bottom by a gravel plug is driven to the desired depth by an internal hammer. At that depth, part of the plug is driven beyond the bottom of the casing to form a bulb of gravel. Additional gravel is then added and compacted as the casing is withdrawn (Broms, 1991). In fine grained soils, stone columns are usually used to increase shear strength beneath structures and embankments by accelerating consolidation and forming columns of stronger material. In coarse grained soils, stone column technique is commonly used to increase relative density and permeability of liquefiable soil deposits in order to reduce liquefaction risk. There are four basic mechanisms that play a role in stone column applications:

1. Stone columns provide strength and stiffness to soil deposits as they reinforce them.
2. Highly permeable drainage boundaries are provided to avoid excess pore water pressure development during dynamic loading.

3. The soil is densified during stone column installation.
4. The lateral stiffness of the soil is also increased.

Due to above mentioned advantages, stone columns have been in use and quite popular among practicing engineers. Case histories of seismic hazard mitigation by stone columns have been presented by Hayden and Welch (1991), Priebe (1991), and Mitchell and Wentz, (1991).

2.2.2. Compaction Piles

Compaction piles improve seismic performance of a soil deposit by three different mechanisms:

1. The flexural strength of the piles provides resistance to soil movement.
2. Vibrations and displacements produced by pile installation cause densification.
3. The installation process increases lateral stresses in the soil surrounding piles.

2.3. GROUTING

Grouting is the injection of materials under pressure into rock or soil through drilled holes to change the physical characteristics of the formation. The results are sealing of voids, cracks, seams and fissures in the existing rock or soil, and making them less permeable and stronger. Grouting is often viewed as a versatile method of ground improvement for application in difficult soil and rock conditions. A large portion of information on grouting evolves from experiences in-situ. Conferences at national and international level have been important sources of information and also milestones in development of grouting practice (British National Society of International Society of Soil Mechanics and Foundation Engineering, 1963; Baker,1982; Jessberger, 1982). This chapter will cover only soil grouting and will exclude the treatment of rock.

Grouting also proves especially effective in the following foundation construction cases:

1. When the foundation has to be constructed below the groundwater table. The deeper the foundation, the longer the time needed for construction; therefore, there is more benefit gained from grouting as compared with dewatering.
2. When there is difficult access to the foundation level. This is very often the case in city work, in tunnel shafts, sewers, and subway construction.
3. When the geometric dimensions of the foundation are complicated and involve many boundaries and contact zones.
4. When the adjacent structure requires that the soil of the foundation strata should not be excavated (extension of existing foundation into deeper layers).

Grouting is a process in which grout in liquid form is pumped into the voids of the soil and then hardens. As a result, the soil is densified and its permeability is decreased or waterproofed. Some of the applications of grouting are listed below (Mitchell, 1981):

1. Waterproofing a certain volume of ground below or around a structure in order to achieve permanent or temporary cutoff zone.
2. Densification of foundation soils to increase shear strength and reduce compressibility.
3. Void filling to prevent excessive settlement.
4. Ground strengthening under existing structures to prevent movement during adjacent excavation or pile driving.
5. Ground movement control during tunneling
6. Soil strengthening to reduce lateral support requirements
7. Soil strengthening to increase lateral load resistance of piles
8. Stabilization of loose sands against liquefaction
9. Foundation underpinning
10. Slope stabilization
11. Volume change control of expansive soils.

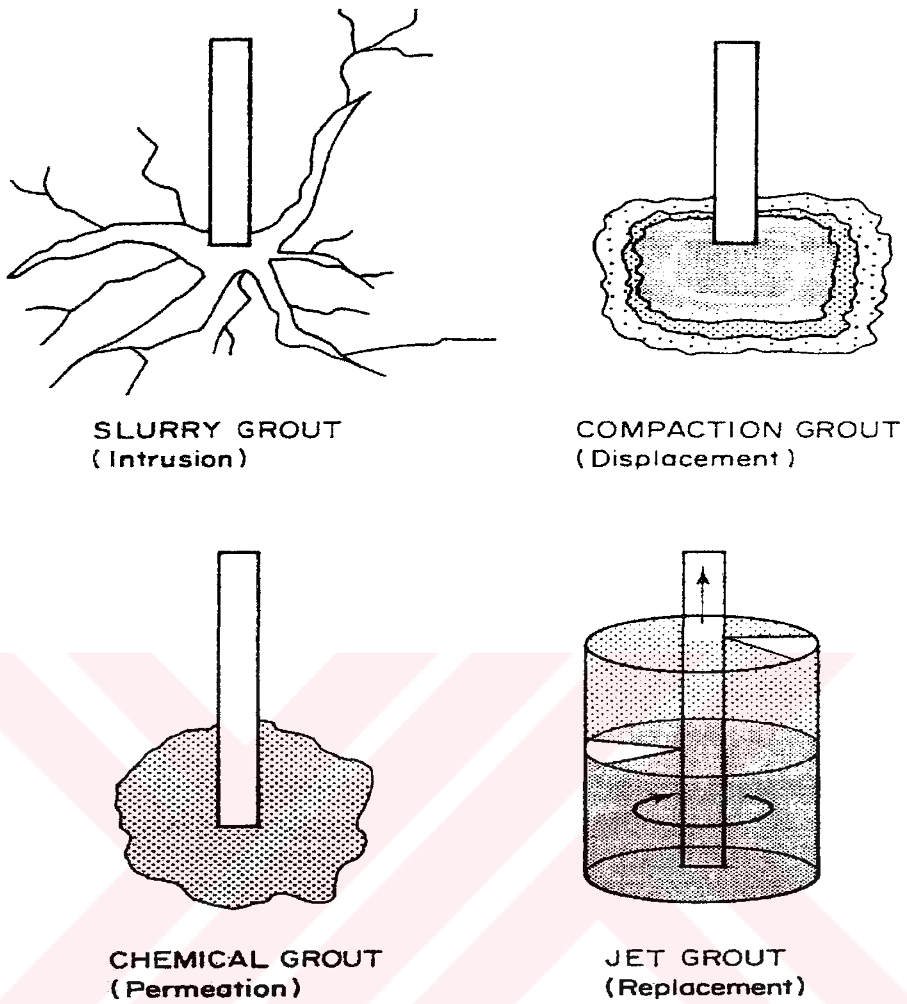

Figure 2.1 Grouting methods (After Welsh, 1986)

Currently there are four types of grouting methods in use; compaction (displacement), chemical (permeation), slurry (intrusion), and jet (replacement) grouting (Welsh, 1986). Figure 2.1 illustrates these grouting techniques. Each one serves a different purpose and uses different equipment. Chemical and slurry grouting are low-pressure methods (~20 kPa), while jet and compaction grouting are high-pressure applications. Jet grouting uses grouting pressures up to 69 MPa and pressure in compaction grouting is about 2.7 MPa (Koerner et al., 1985).

2.3.1 Compaction Grouting

Compaction grouting was developed in United States. It has been used in several soil improvement applications ranging from remediation to densification of soils prior to construction work. This method has also been utilized in preventing ground settlement while tunneling through soft ground. Ground mix consistency should be under control for successful applications. A low slump very stiff soil-cement mortar is injected to displace and compact the soil. Mortars with high water content would behave similar to slurry grout and fracture through the soil skeleton. In general, compaction grout consists of silty sands (10 to 30% passing No.200 sieve), cement or fly-ash, additives (fluidifiers, accelerators) and water.

This technique was used in Bolton Hill subway of Baltimore project to prevent settlement (Baker et al., 1983) and in densification of a liquefiable stratum in West Pinpolis Dam site in South Carolina (Salley et al., 1987).

2.3.2 Chemical Grouting

Chemical Grouting is the injection of properly formulated chemicals into sandy soils. The fraction of the soil passing No.200 sieve should be less than 20 percent. The resulting product is often sandstone-like material with unconfined compressive strength over 4.1 MPa. Chemical grouts are often used for water control purposes owing to their favorable characteristics such as low viscosity and good control of set time. Chemical grouts are generally based on sodium aluminate and sodium bicarbonate. Acrylates (AC-400) and sodium silicate (GEOLOC-4) are commonly used as grout materials.

2.3.3 Slurry Grouting

Slurry grouting is the intrusion of viscous particulate grouts into voids and cracks underground. Slurry or particulate grouting is frequently utilized in United States primarily to reduce permeability of rock beneath new dams. The grout materials are cement, clay (bentonite), sand, additives, micro fine cement, fly-ash, lime, and water. These grouts cannot be injected into soils finer than medium to coarse sands.

In recent years a new class of particulate grouts have evolved with following properties:

1. Improved penetrability under low pressure in sandy-gravelly soils;
2. Lower water loss and, therefore, greater volume of voids filled with the same volume of grout; and
3. The possibility of filling all the voids, permeating medium-coarse sands with refined products and minimizing any hydrofracturing effects.

Typical application of slurry grouting are treatment of foundation rocks of dams, rock cut-off curtains, pressure injected anchors and stabilization of gravelly soils. Expansive soils are also good candidates for stabilization with lime slurry grouting. An axample to slurry grouting is the restoration of failed highway embankments successfully improved using lime/fly ash injection. (Blacklock and Wright, 1986).

2.3.4 Jet Grouting

Jet grouting is another way of improving problematic soils by means of mixing in-situ soils with high pressure cement grout. In this technique, injection equipment is taken down to the target depth, which is the bottom elevation of jet-grout columns, by drilling the soil using pressurized water and hydraulic load applied on ground surface on the drilling equipment. Injection process, which is made by injecting the grout through nozzles on rotating pipes that are being withdrawn to the ground surface at a constant speed, commences once the target depth is reached.

This method is advantagous in forming columns at relatively controllable dimensions and strength characteristics. Homogeneously improved soil zones, either massively or partially, can be produced using jet-grouting technology. Massive treatment can be achieved by generating tangential columns whereas partial treatment is done by producing columns at predetermined intervals. It should be emphasized that jet-grout columns could also behave as concrete piles with certain vertical load-carrying capacities. Detailed information on this method, is given in the third chapter.

2.4 INFLUENCE OF SOIL TYPE ON SOIL IMPROVEMENT METHODS

Soil type determines applicability of a certain soil improvement method. One should consider soil type while deciding for suitable improvement methods in the initial design stage. Application range of various methods (many of them are not covered here) depending on soil type is given in Figure 2.2.

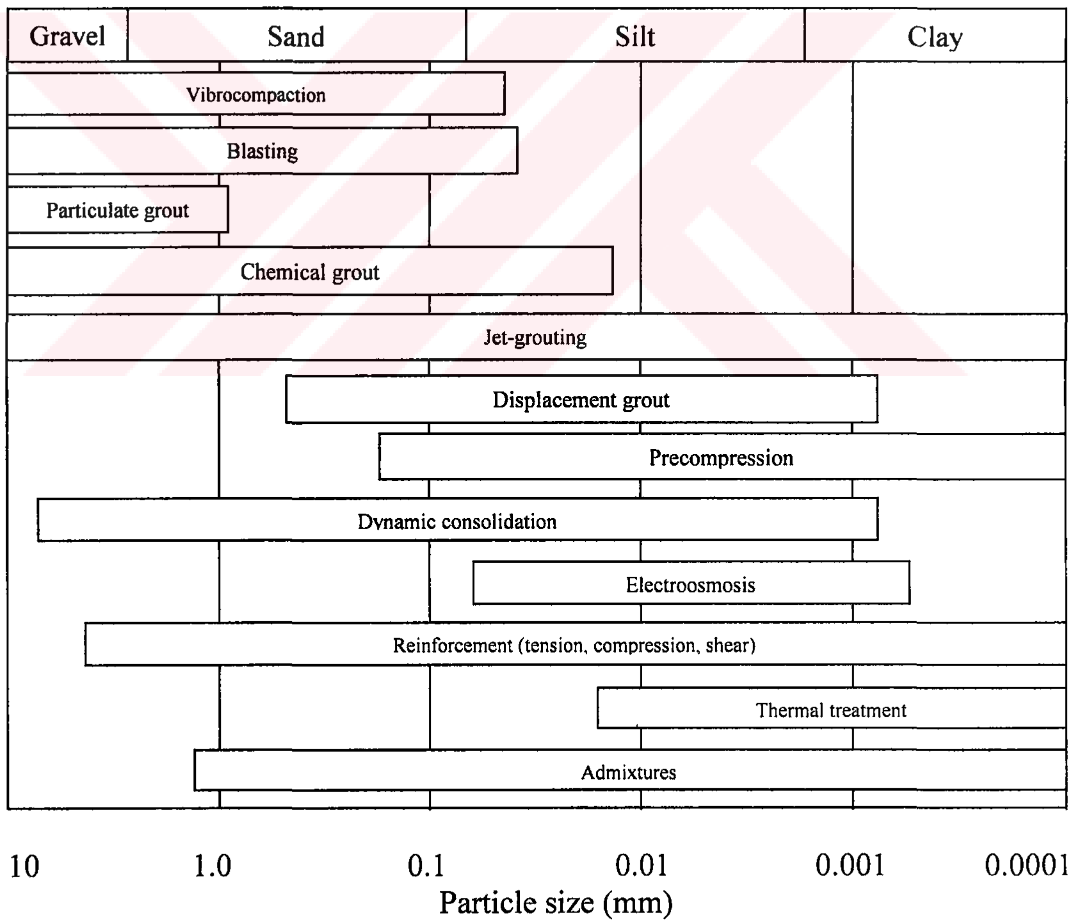

Figure 2.2 Soil particle sizes for grouting and other ground improvement techniques

2.5 DESIGN PROCEDURES FOR GROUND IMPROVEMENT METHODS

First consideration in the design of soil improvement projects is to satisfy static design criteria such as settlement, bearing capacity and slope stability. Settlement of building foundations under static loading conditions are encountered in highly compressible soils. Normally consolidated clays and organic soils are in general main sources of settlement problem. Besides, horizontal discontinuities in the soil profile cause differential settlement problem. Bearing capacity can be satisfied for most soils by choosing appropriate foundation type. However, in some cases enough factor of safety value may not be satisfied. Slope stability problem, on the other hand, can be observed in various soil types and profiles. Providing a satisfactory factor of safety against slope failure may well become a tedious and difficult task especially in critical projects. Soil improvement methods, in general, increase stiffness and strength of problematic soil deposits thereby eliminating settlement, bearing capacity and slope failure problems. Design principles regarding soil improvement projects under static loading conditions shall not be very complex since well-established traditional engineering and soil mechanics principles apply to this problem. In this study, we will give more weight to design principles of soil improvement under seismic loading conditions.

Improvement of a problematic soil layer can result in a very satisfactory performance under static loading conditions. However, this may not mean that same soil would behave well under seismic loading conditions. Unforeseen effects may be generated on both shallow and deep foundations following improvement of foundation soils.

In addition to modification of ground acceleration with respect to original soil profile, displacements both on the ground surface and along the treated soil profile would take unexpected values and create intolerable stresses on surface and deep foundations. Design response spectrum as suggested in the local design code may not be satisfied and spectral accelerations for a particular structure may be exceeded. Settlements may still be a problem after seismic loading. Treated soil zone underneath foundation of a structure may become an improved soil region in the middle of a problematic soil zone. Any failures such as liquefaction, consecutive permanent deformations, lateral spreading may cause stresses and strains in the improved soil profile. Excess pore water pressure generated in the failed soil zone during seismic loading may dissipate towards the improved region following seismic activity. This may cause softening of the treated zone. These aspects of seismic loading case should be handled using available design methods and engineering judgment.

Seismic design of soil improvement projects should be both safe and economic. State of practice in this area is still not established in depth. However, below procedure as suggested by Mitchell et al (1998) may be followed to determine if there is a need for soil improvement:

a. **Project performance requirements should be defined**: Expectations from the projected facility under seismic loading conditions should be defined clearly since this directly affects soil improvement budget. Also project requirements have an influence on the type of technology to be applied. Requirements may be defined for design and maximum credible earthquakes separately. During design earthquake the facility should serve without loosing its functions while it might face some repairable damage during maximum credible earthquake. It should be noted that total collapse of the improved soil zone must be avoided in any case.

b. **Analysis of seismic hazards and ground motions**: Maximum values for the earthquake magnitude and acceleration must be estimated. In some projects an input ground motion specific to local seismic characteristics should be specified. This, however, is not an easy task and involves many uncertainties.

c. **Site characterization:** Studies are performed for evaluation of the soil profile and properties. This step should be performed by competent engineers to provide reliable data.

d. **Liquefaction potential assessment:** Liquefaction risk should be determined using one of the available methods such as SPT or CPT based methods (Youd and Idriss, 1998).

e. **Estimates of settlement and lateral spreading magnitudes and directions:** Estimation of settlement and lateral spreading shall be made provided that residual strength and excess pore water pressure immediately after the earthquake are known.

If the above given five-step analysis procedure reveals a need for soil improvement, remediation project may be prepared following below given outline (Mitchell et al, 1998):

1. Establishing the level of modification required: The extent of modification depends on structural characteristics such as tolerable settlements and seismic demand.

2. Principal approaches in the remediation project should be set.

3. On the basis of second item appropriate methods should be chosen.

4. Target soil properties, size, location and shape of the region to be improved should be established.

5. Specifications shall be prepared for quality control and quality assurance purposes.

6. Field and laboratory tests shall be conducted to check the quality of the improvement..

2.6 DESIGN PROBLEMS IN GROUND IMPROVEMENT

Several factors influence the stability and deformation of improved ground zones and supported structures during and after an earthquake. These factors are illustrated schematically in Figure 2.3 (Mitchell et al, 1998). They can be summarized in seven major categories.

a. Size, location, and type of treated zone.
b. Pore pressure migration: During and after an earthquake differential excess pore water pressures will exist in an area where improved and unimproved soils are adjacent. For instance, excess pore water pressure in a liquefied or softened will dissipate towards improved soil region (Figure 2.3a).
c. Ground motion amplification: Ground motion at the base rock may be transferred faster to ground surface through improved soil zone loosing less energy and resulting in higher ground surface acceleration. This may cause negative effects in the superstructure (Figure 2.3b).
d. Inertial force phasing: Phase difference between responses of the structure and improved soil region may cause unexpected forces both on the structure and in the soil (Figure 2.3c).
e. Dynamic fluid pressures: Since improved soil zone acts as an isolated region in an area of untreated soils, additional dynamic unbalanced water pressures will be generated on the boundaries of the treated soil region (Figure 2.3d).
f. Influence of structure: Soil-structure interaction will inevitably take place during seismic activity causing changes in ground motion and soil characteristics. Depending on the interaction properties the design may be conservative or vice versa.
g. Lateral spreading forces: In cases where liquefied soils exhibit significant permanent displacements on sloping ground huge forces may act on the boundaries of the improved soil region (Figure 2.3e).

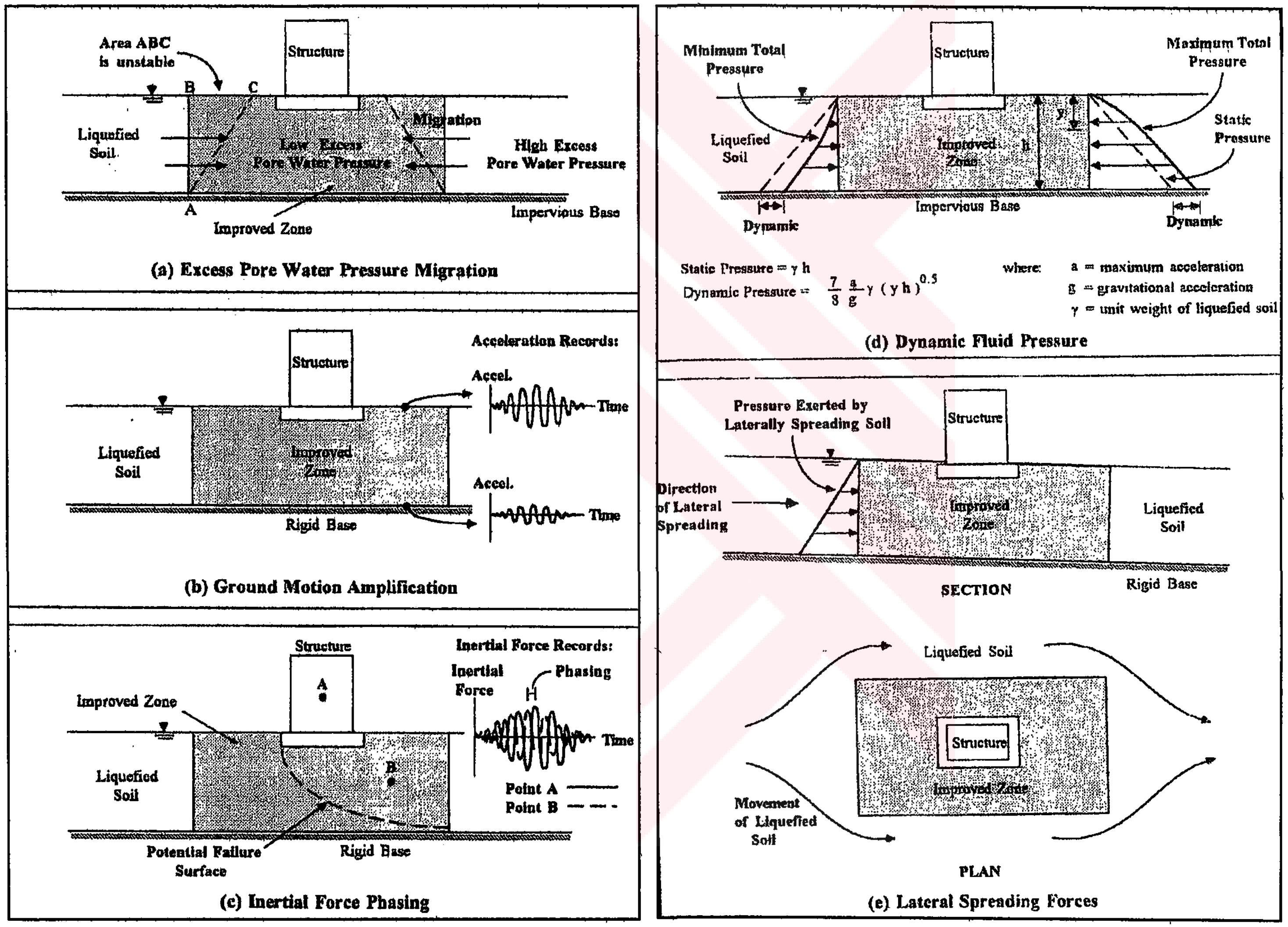

Figure 2.3 Factors affecting improved ground performance (Mitchell et all, 1998)

The issues related to design problems in improved soils are complex and mostly beyond capabilities of practicing engineers. Some of them, however, may be worked out without great difficulties. For instance, dynamic site response analyses to estimate ground acceleration, soil stresses and relative displacements can be made by specialized geotechnical engineers. Excess pore water pressure computation, on the other hand, is not an industry standard project item yet. Permanent displacement and lateral spreading calculations are also among tasks that cannot be guaranteed by design offices. This does not mean that simple and proven design methodologies such as pseudo-static approach for stability analysis of improved soils are not available. Engineer can confidently analyze general stability and overall expected displacements of the treated zone.

CHAPTER THREE
OVERVIEW OF JET-GROUTING METHODS

3.1 INTRODUCTION

Jet grouting was introduced in Japan in early 1970s (Fang, H. Y., 1991). The method excavates a cavity by the action of horizontally applied high-pressure air/water jet in the soil, from a grout pipe that is rotated at a controlled rate. The cavity, which is being formed by erosion of soil, is simultaneously filled using a certain amount of grout. The dosage and other characteristics of the grout depend on project requirements. In general, injected grout is a mixture of cement/bentonite and water. In fact, the resultant product is a column composed of grout material and soil that are mixed with a high speed during rotational high-pressurized injection of grout material. Therefore, this composite column can also be called as soilcrete.

Jet-grouting is a contemporary soil improvement method by which quite successful applications are made to achieve goals of an improvement project explained in the second chapter. The most significant advantage of this method over others is that continuous soilcrete elements with pre-determined characteristics could be produced using high-speed injection of grouting material coming out of powerful pumps (up to 400 to 600 bar). Due to this advantage, jet-grout columns or soilcrete elements are applicable in almost any kind of soil including clays and cobbles. Quite innovative and versatile soil improvement projects can be realized as shown in Figure 3.1. Potential applications include use of jet-grout columns as reinforcing elements to increase vertical and horizontal bearing capacity of soil layers (Figure 3.1d and 3.1e). Jet-grout columns can also effectively reduce permeability of the soil and be utilized as cut-off elements (Figure 3.1d). It is also possible to use jet-grout columns in foundation improvement work as underpinning elements (Figure 3.1a and Figure 3.2).

Some advantages of jet grouting are summarized below:

a. A more predictable degree of soil improvement.
b. A high level of permeability control.
c. Geometric flexibility.
d. Improved cost forecasting over alternative process.
e. Minimal vibration and noise.
f. Ability to operate in headroom as low as 3.0m.
g. Minimum site disturbance and limited working space required.
h. Only environmentally acceptable grouts are used in the process.
i. Installation beneath foundation from outside building.

The disadvantages of the method are that many variables need to be controlled (grout mix, jet nozzle diameter and number, pressure, grout flow rate, pipe withdrawal rate, rotation rate), special equipment is necessary and application cost is relatively high. Besides, cross-sectional discontinuities are not uncommon and difficult to notice. Strength and stiffness of the column may change abruptly in soil profiles with contrasting soil layers. Column strength and diameter are usually smaller than target values in highly plastic and organic soils. Such anomalies cannot be easily detected even by means of advanced non-destructive testing equipments.

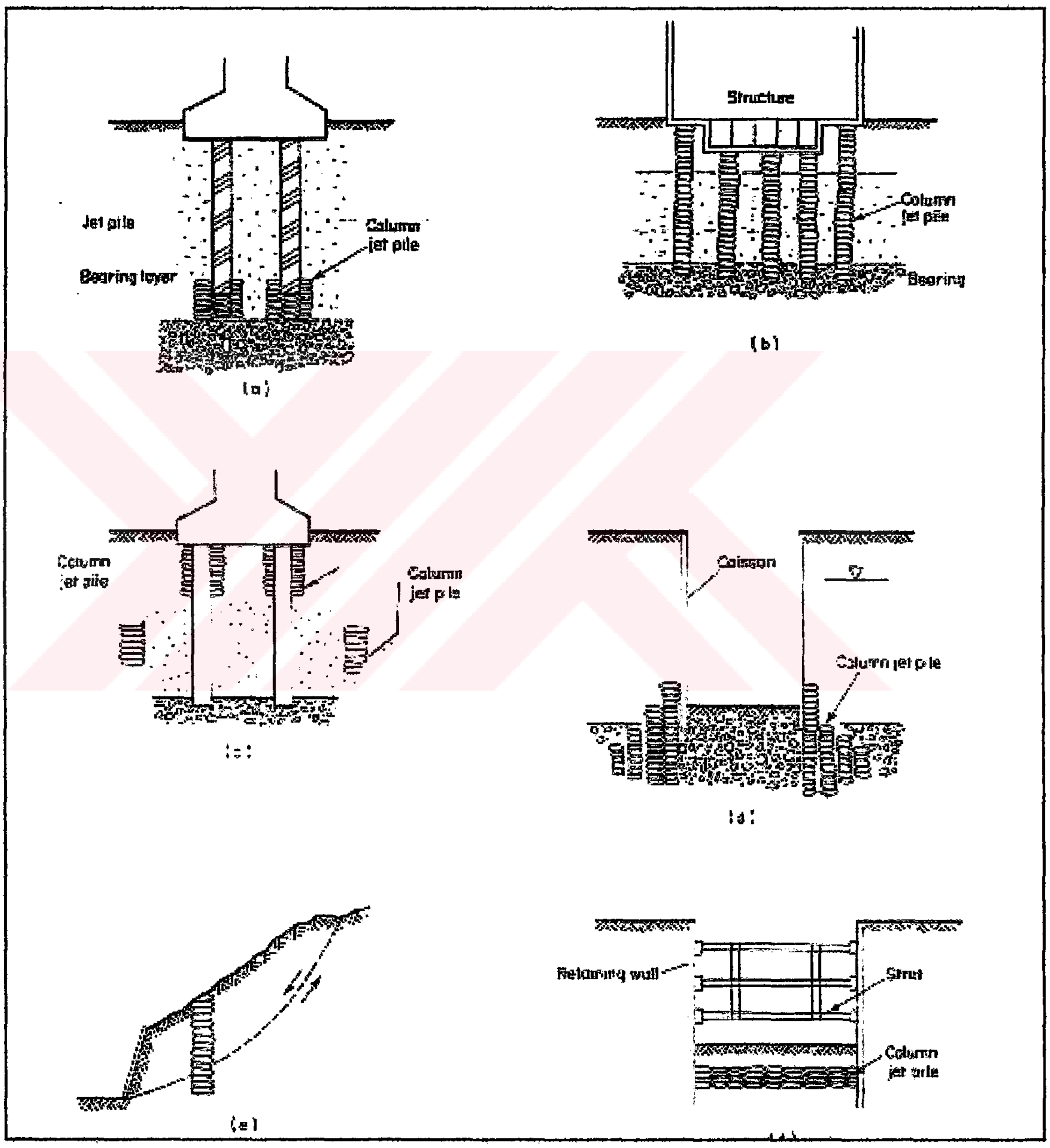

Figure 3.1 Potential applications of jet-grouting

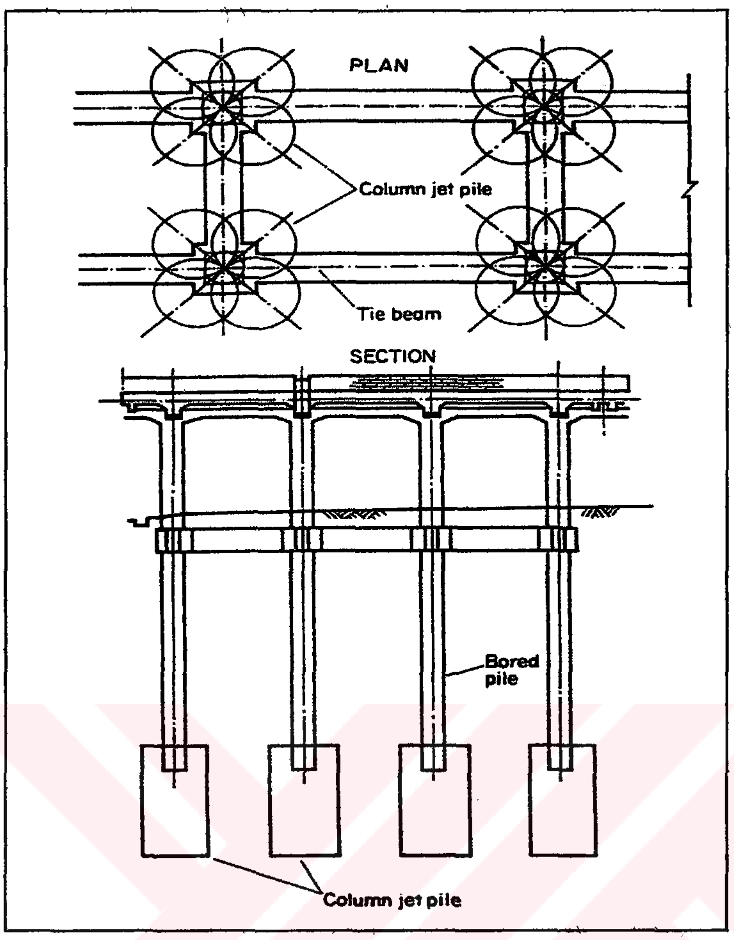

Figure 3.2 Use of jet-grout in pile underpinning

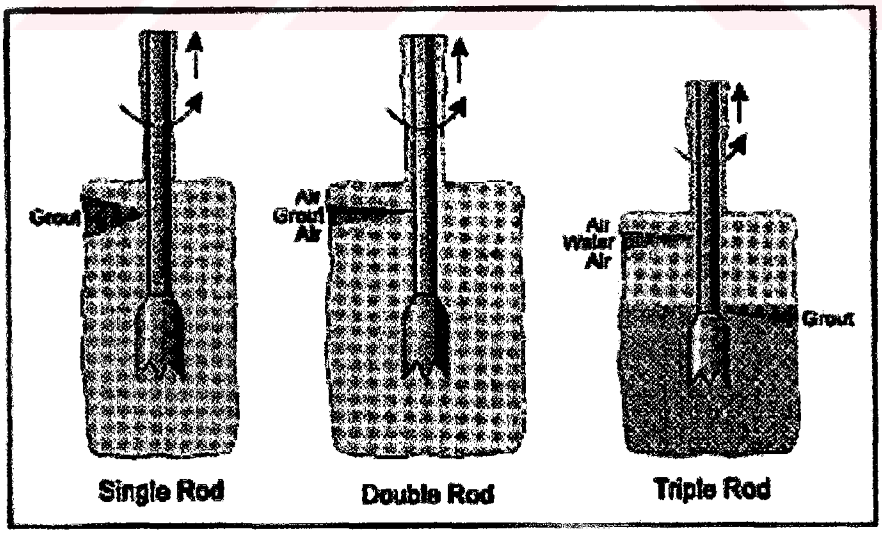

Figure 3.3 Jet-grouting techniques

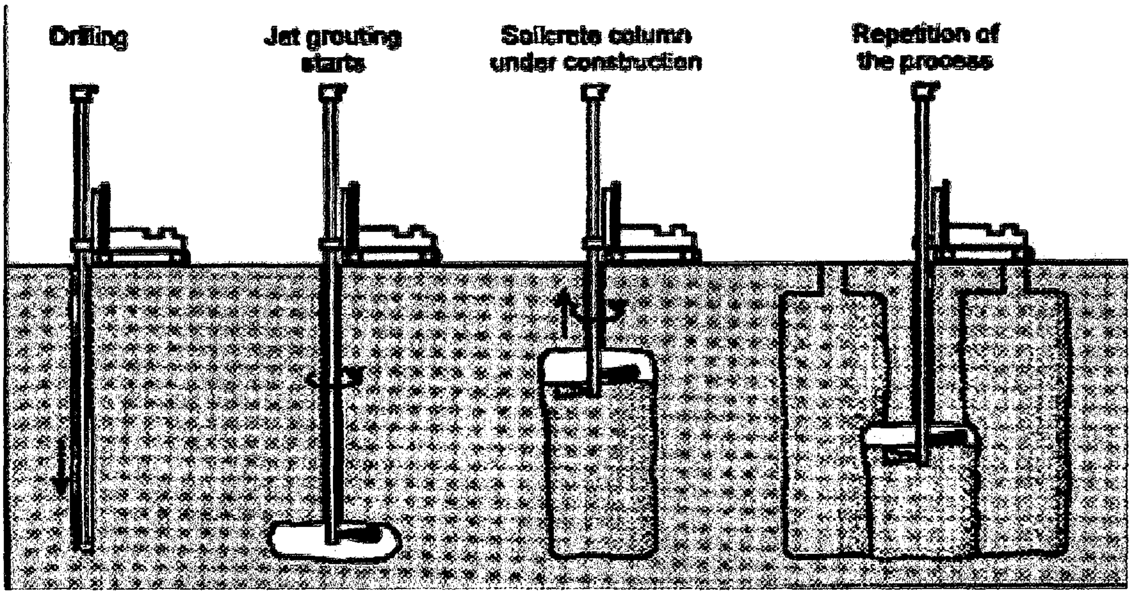

Figure 3.4 Jet-grouting process

Different types of soilcrete columns can be produced by changing rotational and lift-up (withdrawal) speed of the injection rod. There are three major jet-grouting techniques that greatly affect diameter and strength of the jet-grout column. These techniques are known as mono fluid (Jet-1), double fluid (Jet-2) and triple fluid (Jet-3) systems as shown in Figure 3.3. In single fluid system only injected mixture is used as eroding fluid whereas air and water are added as additional high pressure fluid in double and triple fluids systems, respectively. In double and triple systems air and water precedes the injected material to help erode finer soils such as silt and clay. When these systems are used larger size columns can be obtained but strength of the column can decrease. Trapped air in the column and higher water content of the soilcrete generate this effect.

Mono-fluid system is the originally developed method in Japan. Therefore, it is the most common and widespread jet-grouting technique. Since only injection material is used to erode the soil, its use is in general limited to cohesionless soils (Croce and Flora, 2000). The diameter of the jet-grout column in mono-fluid applications varies between 0.5m and 1.0m for cohesionless soils. For cohesive soils, on the other hand, the diameter range is said to be approximately 0.4m to 0.6m.

Triple-fluid technique was developed to overcome difficulties encountered for mono-fluid systems in fine-grained soils. The fundamental difference between the mono-fluid and the triple-fluid system is that a high-energy jet of water rather than the binder is used to break up the soil surrounding the drill string; the passage of the jet of water through the soil is accompanied by pressurized air concentric about the jet. 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 bore hole wall and the drill rods, to the surface. At the same time the binder is injected into the soil/water mix at approximately 5 MPa through a second nozzle positioned just below the air/water nozzle. Jet-grout columns up to 2m in diameter can be produced using triple-fluid method. This method is more complex and expensive, requires more equipment and large volumes of discharged material to the ground surface may become difficult to haul.

The mono-fluid system, however, is more versatile. It can be applied at any inclination and consequently can even be used in tunnel applications and in-building foundation remediation applications. There is no danger of explosion of air and water jets through ground surface as it is in triple-fluid system.

The most recent system is the double-fluid technique, which was developed as an intermediate system. This system is based upon the principles of the mono-fluid system but uses an aureole of compressed air concentric about the jet of binder to enhance its radius of influence. The diameter of a column of soil treated using double-fluid technique is approximately 60 to 80% larger than that of a column produced by means of mono-fluid method.

Application, planning and design of jet-grouting method are still highly empirical and require engineering judgment. Experience is necessary for successful jet-grout applications.

3.2 STAGES IN JET GROUT APPLICATION

3.2.1 Drilling

First step in jet-grouting application is drilling down to the project elevation of jet-grout column tip. This process is carried out with traditional rotary or rotary-percussive drilling machines having some special attachments. Tricones or clay bits are used in soft/loose soils. Rods used during drilling are also utilized during injection. For this reason drilling rods are manufactured to resist high pressure (up to 500~600 bar). Their junctions are also sealed to avoid leakage during injection. Water, air, bentonite slurry or cement grout can be utilized as drilling fluid. A close view of the drilling system at ground surface elevation just before the drilling stage can be seen in the picture, shot during an application in Bostanlı-İzmir region, is given in Figure 3.9.

3.2.2 Injection

After reaching to the design depth, the injection rod is plugged by dropping a steel sphere down the pipe or closing an automatic valve, located at the bottom of the rod. Once the nozzles for drilling fluid are closed the injection mix is forced to rush out of the injection nozzles placed right above the drilling nozzles. High kinetic energy of the mix breaks the bonds between soil particles; forms columnar elements as it erodes and scans the soil during 360° rotation of the injection equipment as it is lifted toward ground surface at a certain rate. This stage may also be called as jetting.

Backflow of the injection material from the top of the drilled hole is a good indication of normal pressure. In the case of overpressure the injected material and soil mix can flow toward ground surface through the gap between the rods and the soil. This may result in sectional discontinuities in the jet-grout column and may harm nearby in-ground structures. This problem is more common in cohesive soils, and can be eliminated by reducing flow rate, increasing injection pressure or applying double fluid technique instead of mono fluid.

A sketch of concentric drilling and injection rods are given in Figure 3.5.

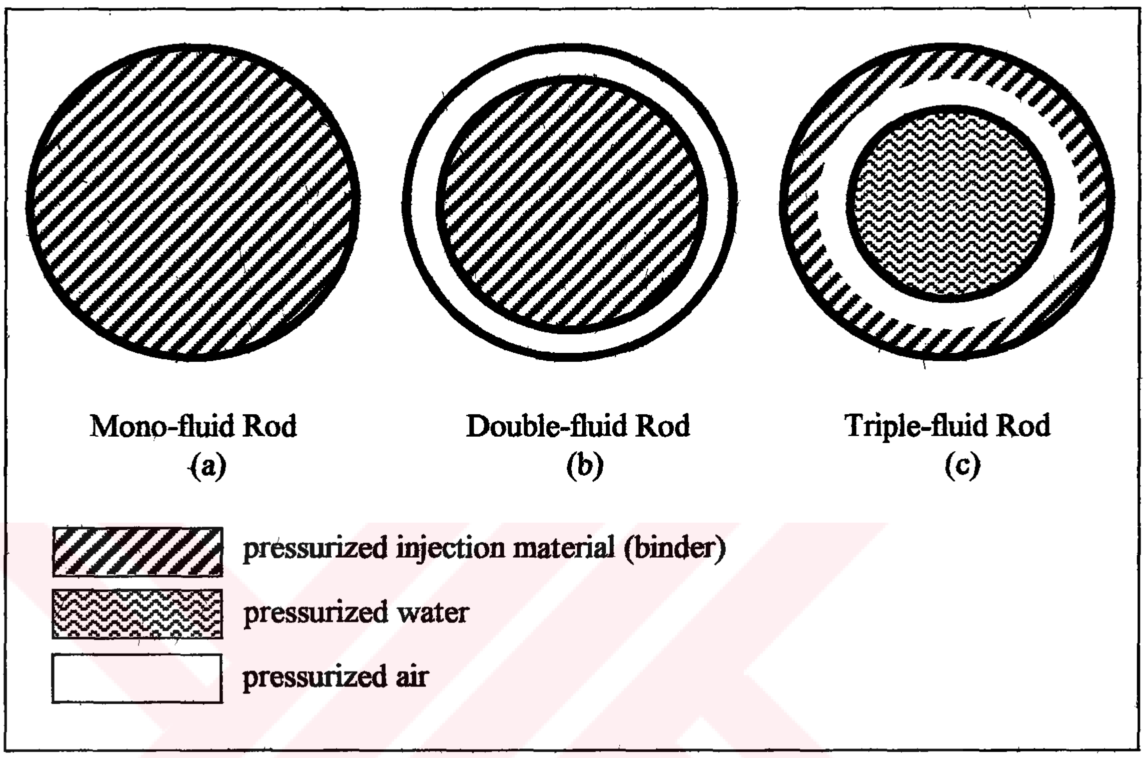

Figure 3.5 Cross-sectional view jet-grouting rods

3.3 JET GROUTING EQUIPMENT

Equipments common to most jet-grouting methods are given in Figure 3.6. As seen in the figure, there are six major units in a system. Additional units can be used depending on the particular application.

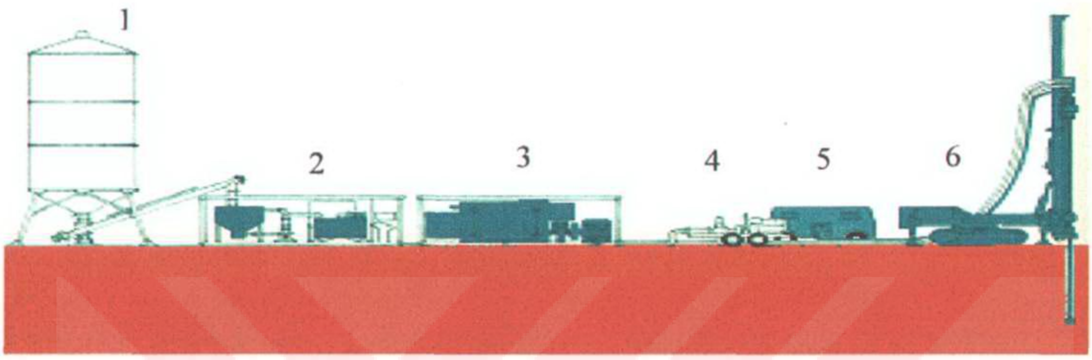

Figure 3.6 General jet-grouting equipment

In the above figures numbered units are explained below:

1.Silos
2. Batching plant
3. High-pressure jet pump
4. Blinder pump
5. Air compressor
6. Drilling rig with jet timer

Equipment characteristics may change depending on the technique to be used. Therefore, it would be more appropriate to give equipment properties for mono, triple and double systems separately in the following paragraphs.

a. Mono-fluid (Jet-1)

The main equipment required for this system is:

- A high pressure, high flow pump with a pressure and flow capacities of 70 MPa, and 300 l/min, respectively (see Figure 3.7).
- A drilling rig fitted with a special drill string (see Figure 3.8) and a suitable timer with which step-raising of the drill string can be accurately controlled.
- An efficient batching plant with sufficient capacity

b. Triple-fluid (Jet-3)

The main equipment required for this system is:

- A high pressure, high flow pump for the water jet (70 MPa, 300 l/min).
- A low pressure pump for the binder (7 MPa, 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.

c. Double-fluid (Jet-2)

Equipment for the application of this system is the same as for the monofluid system with the exception of the drill string, which has to be made up of coaxial 2-way rods. The injection system is given in Figure 3.10. Note that twisted pipes for air and injection material pumping are marked in the figure.

Figure 3.7 High pressure pump (Bostanlı Vilayet Evi Project, 2001)

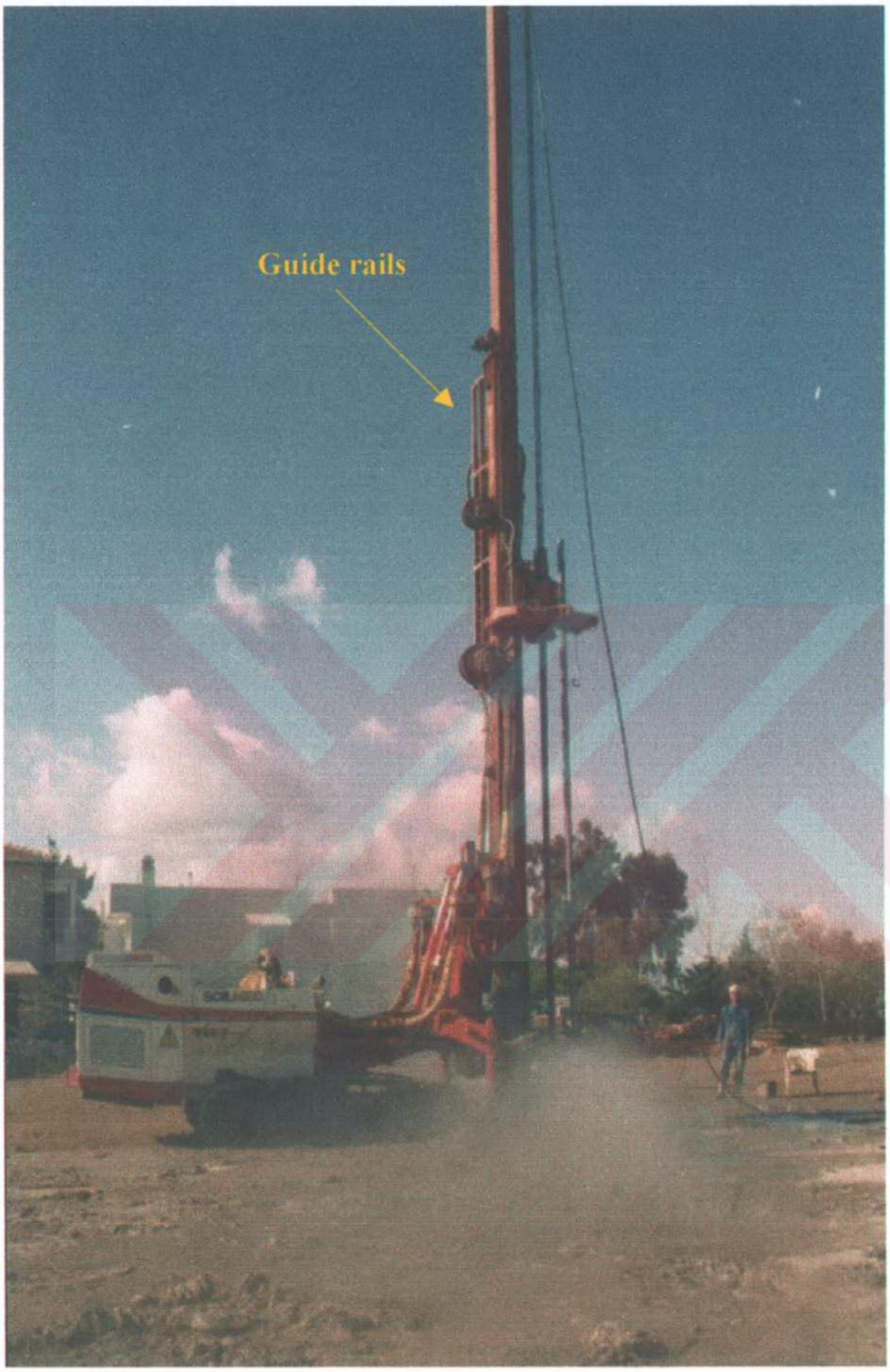

Figure 3.8 General view mono-fluid drill rig and clean-up of injection nozzles (Bostanlı Vilayet Evi Project, 2001)

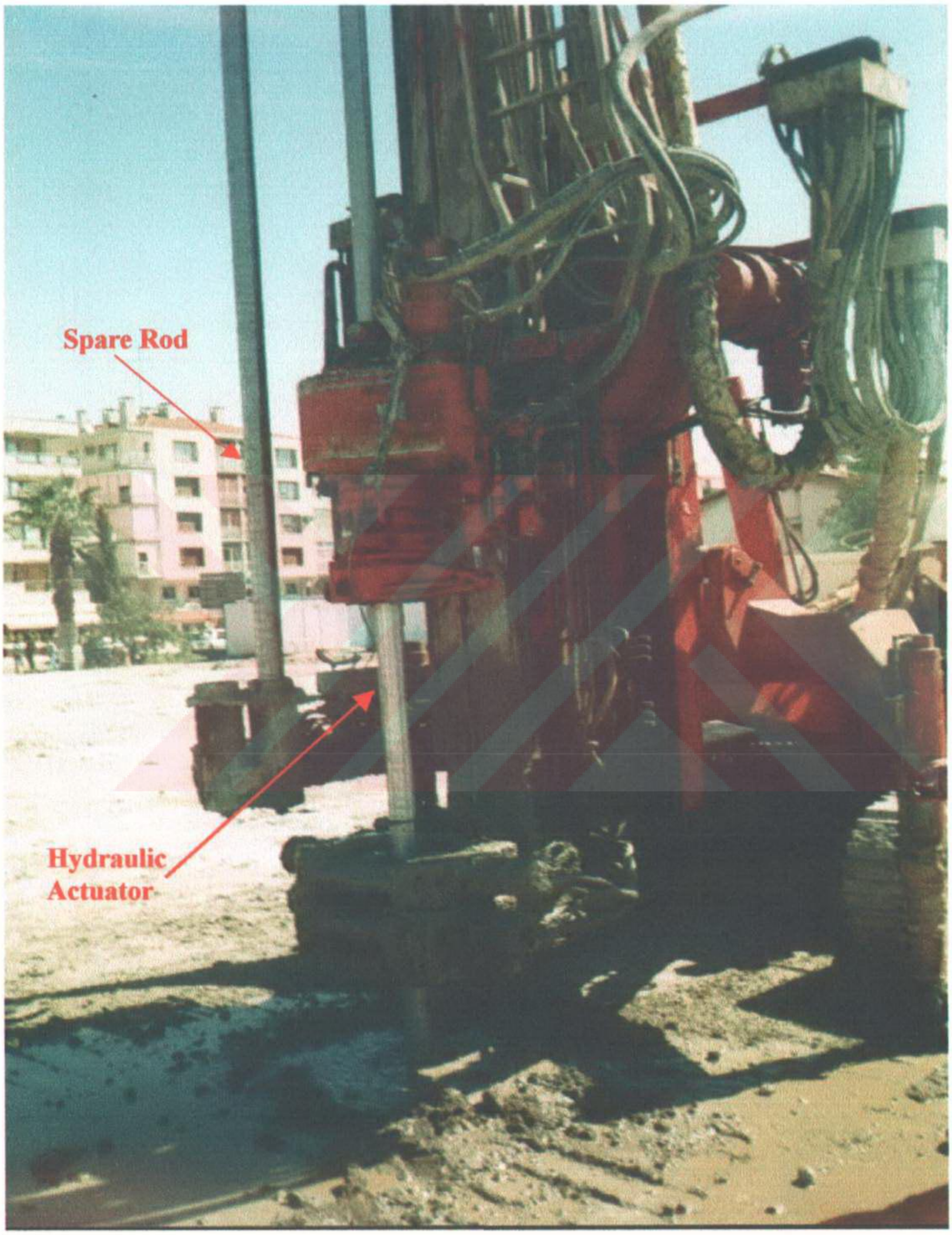

Figure 3.9 Close view of drilling rigs under operation (Bostanlı Vilayet Evi Project, 2001)

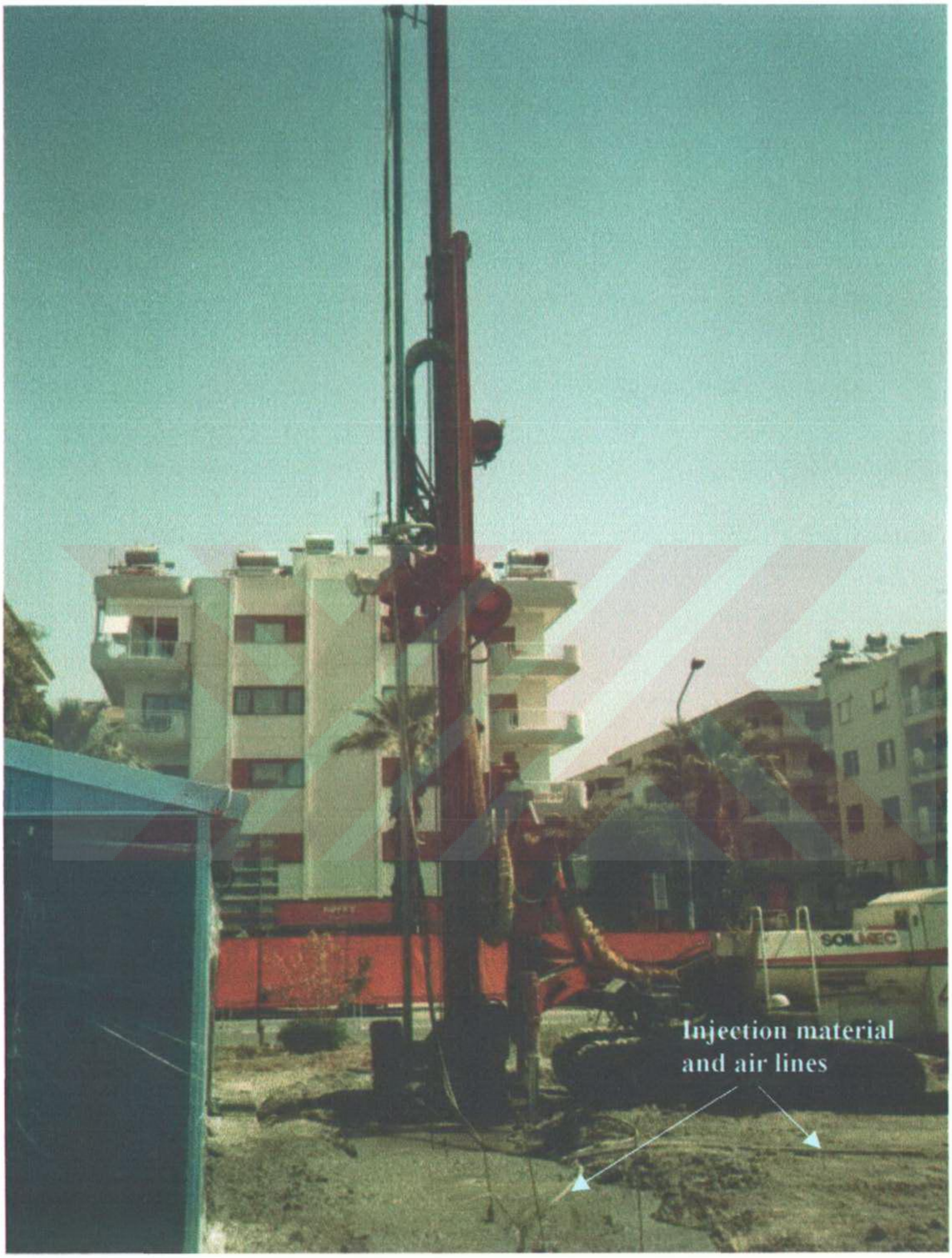

Figure 3 10 Double-fluid grouting application during injection. (Bostanlı Vilayet Evi Project, 2001)

3.3.1 FLUID PIPES, RODS AND NOZZLES

The mixed solution or water is pumped into the soil through high pressure resistant flexible pipes. Diameter of a pipe may change but it was approximately 8.255 cm (3.25″) in the field application, which was monitored during this thesis study. Details of the jet-grout column project of the field application are given in the next chapter. The rods are connected to the flexible pipes by means of special attachments. The rods are guided using a long rail system (the tower like portion in Figures 3.8 and 3.10). The pipes are hydraulically pushed into the soil using the actuator shown in Figure 3.9. The length of each rod is usually around 3~4 m. The special attachment, where nozzles are located takes place at the tip of the final rod in the soil. The diameter and number of nozzles vary depending on the project requirements as explained in the following section. A view of the nozzle attachment can be seen in Figure 3.11.

Figure 3.11 Double-tube nozzle attachment

3.4 OPERATING PARAMETERS

Operating parameters are governed by target column strength, soil type, diameter and selected jetting method. Major operating parameters are pressure, rotation and withdrawal (lifting) speed and dosage. In the following, these parameters are briefly explained. An application example regarding calculation and selection of operating parameters is given in Chapter 4.

3.4.1 Pressure

Variation of injection pressure with respect to adopted jetting method is given in the following table (Table 3.1). Note that water pressure is applied as pre-washing (drilling) in mono-fluid (Jet 1) and double-fluid (Jet 2) methods. It functions as eroding fluid under higher-pressure in triple-fluid (Jet 3) method. Although injection pressure greatly affects the diameter of the column, there is a myth that there is direct connection between injection pressure and column diameter. Instead, rotation and withdrawal speeds are considered more effective in reaching larger size and homogeneous jet-grouting columns. In other words the diameter of a jet-grout column is mostly a function of time.

3.4.2 Rotation and Withdrawal Speed

Rotation and withdrawal speeds should be adjusted in such a manner that homogeneous and continued columns are produced. This, in general, requires slow speeds. The rotation speed usually varies between 10 and 20 rpm and seldom reaches 30 rpm. Lifting time depends on the type of soil and on the quantity of binder to be injected. Cohesive soils require longer time. Jetting method is also another factor on withdrawal time (see Table 3.2)

3.4.3 Dosage

Two parameters are regarded as a measure of dosage: Water/Cement (w/c) ratio and amount of injected material per cubic meter of treated soil. Water/Cement ratio is usually set as 1.0. It is seldom taken as 0.7. Amount of cement per cubic meter of treated soil, on the other hand, varies between 350 and 700 kg/m^3

Softer and more organic soils usually require higher cement dosage. Relationship between compressive strength and cement quantity is given in Figure 3.12 for various soils. It can be seen that compressive strength of the column quickly increases with grain size.

For organic soil, the dosage of cement per m^3 of ground can be increased from 450 to 600~700 kg. This is because a part of the employed cement is used to neutralize the humic acids of peat, which reduces the binder action of the cement (Melegari, 1997).

The average values of the compressive strength of various treated soils (soilcrete) for a water cement ratio, w/c, of 1 and standard dosage of 450 kg of cement per m^3 of soil are given in Table 3.3.

If the same quantity of cement and previous ratios w/c are used in Jet2, the compressive strengths of the soilcrete columns are lowered by 10-15% with respect to Jet1 method. This is due to the presence of air in the treated soil. The strengths of the columns of treated soils are much lower in Jet3 method than those in Jet1 method if the same dosages are applied in both cases.

Table 3.1 Typical range of jet-grouting operating pressures (Melegari, 1997)

Injection pressure	Unit	Mono-fluid	Double-fluid	Triple-fluid
Water	bar	PW* (200-300)	PW (200-300)	300-500
Grout	bar	300-600	300-600	40-60
Compressed air	bar	not used	8-12	8-12

* Pre-washing (drilling) stage

Table 3.2 Parameters for jet-grouting (Recon, 2002)

Parameters for Jet Grouting	Mono-Fluid		Double-Fluid		Triple-Fluid	
	Min	Max	Min	Max	Min	Max
Binder injection pressure (MPa)	20	60	30	60	3	7
Binder flow (1/min)	40	120	70	150	70	150
Air Pressure (MPa)	-	-	0.6	1.2	0.6	1.2
Air flow (1/min)	-	-	2000	6000	2000	6000
Water injection pressure (MPa)	-	-	-	-	20	50
Water flow (1/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

Table 3.3 Average compressive strength of soilcrete (Melegari, 1997)

Soil Type	Compressive Strength, q_u (kg/cm^2)
Organic soil	3
Clay	18 - 30
Silt	30 - 45
Sand	60 - 90
Gravel	100

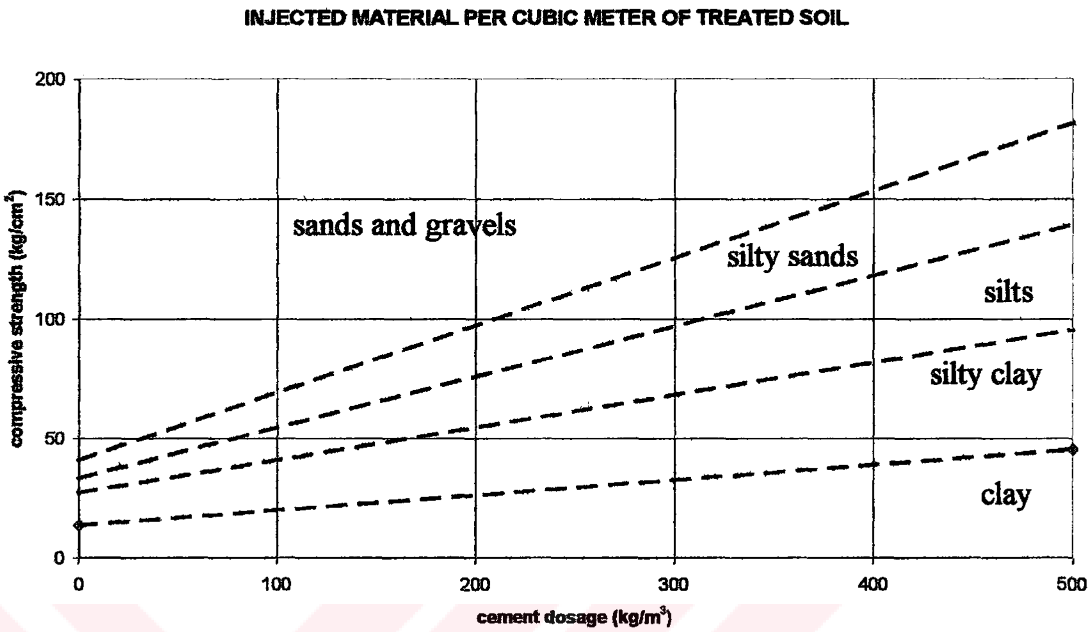

Figure 3.12 Correlation between strength and the quantity of cement (Recon, 2002)

In addition to above-mentioned parameters nozzle number and diameter should also be selected properly for each grouting method and soil type. Less and smaller diameter nozzles are used for finer soils. Smaller number of nozzles (at most 2) prevents energy loss during injection (Melegari, 1997). Nozzle diameter is kept small (approximately 1.6 to 2 mm) in clay soils to obtain homogenous columns. For cohesionless soils nozzle diameter becomes 2.5~3.0 mm. Jetting method is also effective on nozzle diameter. Variation ranges for water, air and binder (injection material) nozzles are given in Table 3.2. Optimum selection of nozzle number and diameter greatly depends on experience of the practicing engineer. Above given values should be considered as broad recommendations.

CHAPTER FOUR
CALCULATION OF OPERATION PARAMETERS

4.1 INTRODUCTION

There are approaches for the calculation of operation parameters in jet-grouting method. Two of these are quite similar in nature. One needs to estimate bearing strength of a single jet-grout column depending on the type of the treated soil and then calculates necessary parameters utilizing phase relationships and amount of kinetic energy at nozzles. Such two methods are given by Recon (2002) and Melegari (1997) in the literature,. The third method has been developed in this study as a computer algorithm to combine Standard Penetration Test blow counts, undrained shear strength and above-mentioned methods to calculate operation parameters. While doing this, correlations by Bell (1993), Botto (1985), Miki and Nakashi (1984) and Tornaghi (1989) were used as presented in the paper by Croce and Flora (2000).

In the below paragraphs Recon approach is presented first. Then Melegari approach, which may be assumed as formulated version of the Recon method is given. Since both methods yield similar results, an application example for Bostanli Vilayet Evi Project is given with Melegari method.

4.2 Recon Method

The calculation steps of jet grouting process as suggested in the Recon Company website are listed in the following:

1 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 (Correlation between strength and quantity of cement to be injected) determine the amount of cement. which should be injected into each cubic meter of treated soil. This step becomes easier and more accurate with increased local experience. When binders other than cement are used, local experience from laboratory or site tests must be developed to determine this parameter.

2 Pick the required diameter of the soilcrete column and calculate the amount of cement, which should be injected per meter of column.

3 Choose the composition of the binder. The principal characteristic of the binder is that it must be pumpable. In the state of a water/cement mix, the properties 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. The other opinions that the mix composition must be based are; the nature of the soil, its grain size distribution, 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 varies between 1.0 and 1.5. The ideal mix however must be determined from in-situ tests.

4 Using results of the first, second and third steps, calculate the amount of binder to be injected per meter of treated column.

5 Select the injection pressure (commonly between 40 and 50Mpa).

6 Select the size and number of nozzles to be fitted to drill string and from Figure 4.1 determine the flow of binder. First select number of nozzles and nozzle diameter and then follow curves on the chart to come up with injection flow.

7 Using the results of the fourth and sixth steps, calculate the injection time per meter of column.

8 Select the withdrawal step size and calculate the time required to inject the predetermined amount of binder at each level.

9 Select the speed of rotation of the drill string. It should rotate one or two complete turns per step.

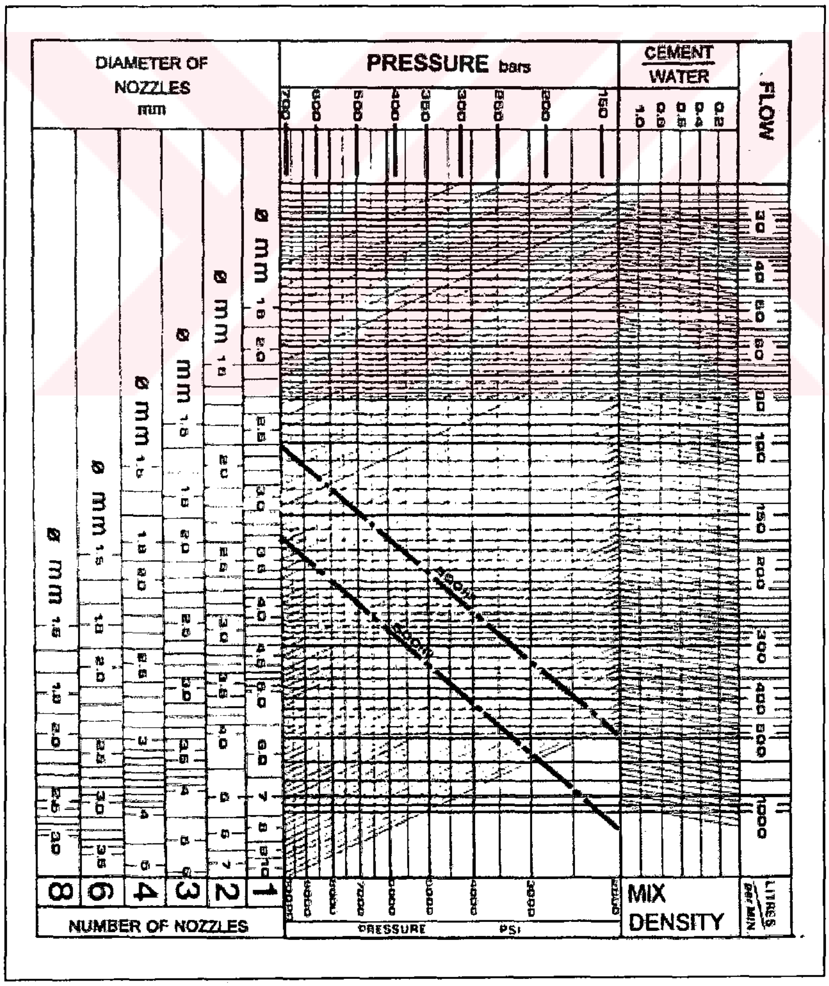

Figure 4.1 Operation parameter selection chart

4.3 Melegari Method

This method simply utilizes phase relationships and released kinetic energy to calculate operation parameters such as amount of injected grout, process time per column, withdrawal speed and rotation speed. Water/cement ratio, injection pressure, nozzle diameter and number of nozzles are input as assumed parameters. Column diameter is governed by the axial load, cement dosage and type of treated soil.

Calculation of operation parameters using this method is given for a jet-grout improvement application in Bostanlı Vilayet Evi Project. Details of this project can be seen in the fifth chapter. Soil profile, SPT-N, undrained shear strength and angle of internal friction values are given in Table 4.1. Other project parameters are given where necessary.

4.4 Application Example for Recon and Melegari Methods

Basic parameters in Bostanlı Vilayet Evi Jet-Grout Application Project are given below:

Employed technique	: JET-1
Selected column diameter	: 600 mm (clay layer governs, see *a*)
Cement dosage	: 450 kg/m^3
Water/cement ratio, w/c	: 1.0
Number of nozzles	: 2
Diameter of nozzles	: 2 mm
Injection pressure	: 500 bar
Axial force per column	: 441.74 kN (service load)
Factor of safety for axial load	: 2.0
Column length	: 17.0 m
Type of cement	: PÇ-42.5 (Portland Cement Type-I)
Specific gravity of cement	: 3.15
Number of rotations per step	: 1.5

a. Calculation of column diameter considering axial load and soil type

For sand layer, the compressive strength value can be determined as 160 kg/cm^2 for a dosage of 450 kg/m^3 (check Figure 4.2). On the other hand, the compressive strength for the clay layer is found as 30 kg/cm^2.

Column diameter may be calculated as below:

$$D_i = \sqrt{\frac{\mathrm{FS} \cdot \mathrm{F_A}}{\mathrm{q_u}}} \qquad (4.1)$$

where FS: factor of safety; F_A: axial load and q_u: compressive strength of jet-grout column.

For clay layer:

$$D_i = \sqrt{\frac{2*45.03*9.81*4}{33*98.1*\pi}} = 0.588 \cong 0.6\,m$$

For sand layer:

$$D_i = \sqrt{\frac{2*45.03*9.81*4}{160*98.1*\pi}} = 0.27 \cong 0.3m$$

$\Rightarrow$ Clay layer governs the diameter. $\Rightarrow$ D = 0.6 m

b. Volume of treated soil per meter

$\pi \cdot (0.6^2)/4 = 0.283\ m^3/m$

⇒ Quantity of cement per meter of treated soil, Q_c= 450·0.28 =127.17 kg/m

⇒ Quantity of cement, $Q_T = L \cdot Q_c$ =17·127.17= 2161.89 kg (L=column length)

Volume of the injection mixture can be written as the sum of water and cement components (Equation 4.2):

$$V_m = V_w + V_c \qquad (4.2)$$

Water/cement ratio can be shown with r_g, and this ratio is written as in Equation 4.3:

$$r_g = \frac{W_w}{W_c} = \frac{V_w \cdot \gamma_w}{V_c \cdot \gamma_c} \qquad (4.3)$$

Total mixture volume can be assumed to be equal to 1lt. Therefore Equation 4.3 can be rewritten as:

$$r_g = \frac{1}{G_c} \cdot \frac{(1-V_c)}{V_c} \qquad (4.3a)$$

Unit weight of the mixture can be calculated for 1 lt of volume as follows:

$$W_c = \gamma_c V_c \text{ and } W_w = (1-V_c)\gamma_w \qquad (4.4)$$

$$V_c \cdot \gamma_c = \frac{1}{r_g}(1-V_c) \cdot \gamma_w$$

$$V_c = \frac{1}{r_g} \cdot (1 - V_c) \cdot \frac{1}{G_c} \Rightarrow V_c = 0.241 lt \text{ and } V_w = (1 - V_c) = 0.759 lt$$

$$\gamma_m = \frac{W_c + W_w}{V_{total}} = \frac{\gamma_c . V_c + \gamma_w . V_w}{1} = \frac{0.241 \cdot 3.15 + 0.759}{1} = 1.518\, kg / lt$$

Using Equation 4.3a volume of cement can be expressed as below:

The weight of the mixture for r_g=w/c=1.0 becomes 2 kg for 1 kg of cement. Therefore, the volume of the mixture is V_m = 1.318 lt/$kg_{(cement)}$.

$\Rightarrow$ Total volume of the injected grout per meter of column:

$V_t = V_m \cdot Q_c$=127.17·1.318 = 167.54 lt/m

c. Injection flow

Jet output speed at the nozzles is calculated using Equation 4.5:

$$V=(2 \cdot g \cdot h)^{0.5} \qquad (4.5)$$

In the above equation, h is the equivalent hydrostatic height (10m = 1bar). Note that the unit weight of the mixture is calculated as 1.518 kg/dm^3. Equivalent height at 500 bar is:

h = 5000/1.518=3294 m $\Rightarrow$

V=(2·9.81·3294)$^{0.5}$=254.23 m/s

$\Rightarrow$ Flow of grout per meter is found as

$$Q = V \cdot A \qquad (4.6)$$

where V is jet output speed and A is total cross-sectional area of the nozzles.

$$A = n \cdot \pi \cdot d^2 / 4 \qquad (4.7)$$

where

n: number of nozzles

d: nozzle diameter

$A = 2 \cdot \pi \cdot (0.02)^2/4 = 0.000628\ dm^2$

$Q = 2542.3 \cdot 0.000628 \cong 1.60$ lit/sec

d. Process and withdrawal time, rotation speed

⇒ Select the withdrawal step size (usually between 3~8 cm) and calculate the time required to inject the predetermined amount at each step:

⇒ Process time per column:

$T = V_t / Q = 167.54 / 1.60 = 104.7$ sec

⇒ About 105 seconds is necessary to develop a meter of jet-grout column.
Considering a withdrawal step of 4cm, 25 steps per meter are necessary.

⇒ Process time per step is then calculated as:

$T_{step} = T / steps = 105 / 25$ steps $= 4.2$ sec per step

⇒ Total elapsed time for the column is:

Ttotal = L·T = 17 · 104.7 = 1779.9 sec ≡ 30 min

⇒ Rotation speed can be calculated using Equation 4.8:

$$\omega = N_{step} \cdot 60 / T_{step} \qquad (4.8)$$

In the above equation N_{step} is number of rotations per step (usually 1~2), T_{step} is process time per step and ω is the rotation speed. The time unit for the rotation speed is 1 minute (60 seconds). The number of rotation is selected as 1.5 somehow arbitrarily. The rotation speed is found as:

$\omega = 1.5 \cdot 60/4.2 = 21.43$ rpm ≈ 22 rpm

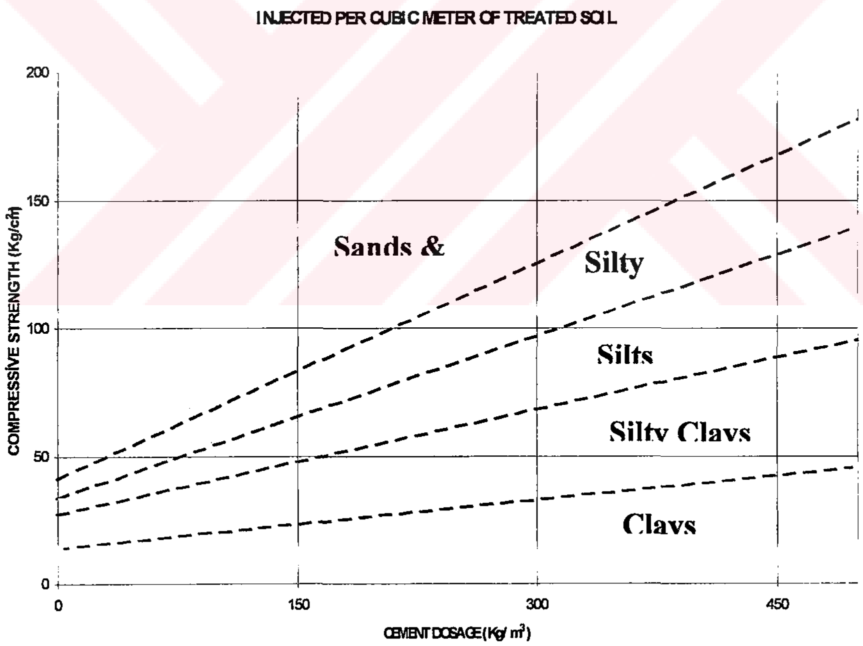

Figure 4.2 Compressive strength of jet grout column as a function of dosage and soil type

Table 4.1 Idealized soil profile for Bostanlı Vilayet Evi Project

Depth (m)	N'_{60}	Description	ϕ	c_{uavr} (kPa)	γ_n kN/m^3	Soil Profile
1.5	48	medium-very dense	48		20	Fill
2.9	6	Coarse -very loose	29		15.55	
4.4	4	Coarse - very loose	30		13.5	
6	17	Corse-medium	34		17.65	
7.5	22	Coarse-medium	35		18.5	
9	11	medium-medium	33		17.65	
10.5	15	medium-medium	33		17.72	Clayey Sand
12	20	Fine-dense	33		17.65	
13.5	13	fine-medium	32		17.75	
15	8	coarse-loose	31		17.55	
15.5	7	Medium-loose	31		17.95	
16.5	5	Medium-medium	30		15.5	
17.5	6	Medium-medium	30		15.95	
18	9	Medium(NC)		55.16	17.6	
18.5	9	Medium(NC)		55.16	17.85	
19.7	8	Medium(NC)		49.04	17.95	Clay
20	8	Medium(NC)		49.04	17.9	
21	10	Medium(NC)		61.29	17.95	
21.4	10	Fine-medium	31		18.35	
22.5	11	medium-medium	33		17.65	
23	12	medium-medium	32		17.7	
24	14	coarse-medium	33		18.5	
24.3	15	coarse-medium	34		18.65	Gravelly Clayey Sand
25.5	14	coarse-medium	33		18.6	
26	14	coarse-medium	33		18.7	
27	12	medium-medium	33		18.2	
27.5	12	medium-coarse	33		18.55	
28.5	14	medium-coarse	33		18.8	
29.5	13	medium-coarse	33		18.75	
30	23	fine-dense		140.98	19.1	
31.65	23	medium-dense		140.98	19.1	
32.8	16	fine-dense		98.07	18.9	
34.4	13	fine-dense		79.68	18.8	Gravelly Clay
36	10	medium-medium		61.29	17.68	
37.3	15	fine-medium		91.94	17.72	
38.5	15	medium-medium		91.94	18.85	
40.4	15	fine-medium		91.94	17.9	

4.5 COMPUTER ALGORITHM FOR OPERATION PARAMETERS

A simple computer algorithm named as JETv1 has been developed for calculation of certain operation parameters. Main objective in the development of the algorithm was to utilize available correlations between Standard Penetration Test blow counts, undrained shear strength, soil type and column diameter in a reliable and quick manner. The program specifically works for mono fluid system (Jet-1) since utilized correlations were developed for this system (Bell,1993; Botto, 1985; Miki and Nakashi, 1984 and Tornaghi, 1989). All necessary graphs are digitized in the program; calculated diameters are plotted on the correlation graphs, summary of the operation parameters can be printed and saved in the disk. MATLAB programming language, which is a Windows based scientific programming language has been used. Flowchart of the algorithm is presented in Figure 4.3.

Stages of a sample run using JETv1 are shown in Figures 4.4 and 4.5 for cohesionless and cohesive soils, respectively. Output of calculated parameters for clay soil layer are given in Figure 4.6. The run was made for Bostanlı Vilayet Evi Project site soils. All other parameters such as injection pressure and number of nozzles are same as the ones used in the previous example. One should note that calculated diameter for Jet-I method appears to be at the edge of utilized correlations for cohesive soils (Undrained shear strength is 55 kPa, SPT-N′ = 8). It was even not possible to use Miki-Nakashi correlation for cohesive soils since SPT-N values above 5 are discarded in that method.

As can be noticed in the figures clay layer dominates the design in this method as well. Although larger diameter column could be produced (~0.8m) and smaller size columns appear to be satisfactory in terms of compressive strength for axial load (~0.25m) in cohesionless layers of Bostanlı Vilayet Evi Project, one should use column diameters that can be realized in the clay layer (~0.4m in terms of the diameter that can be produced and ~0.6m in terms of compressive strength for axial load).

Silty soils are considered as granular, whereas peat and silty clay are assumed as cohesive in JETv1. The program enables the user to input data interactively; allows repeated calculation of operation parameters for various soil and jetting data, reduces the time for optimization analyses. One can use field and laboratory soil data to decide for optimum column diameter in Jet-I method. Unfortunately, no such correlations were encountered in the scanned literature for Jet-II and Jet-III methods. The algorithm is not a complete design tool for the jet grout method, currently. However, it is open for further development.

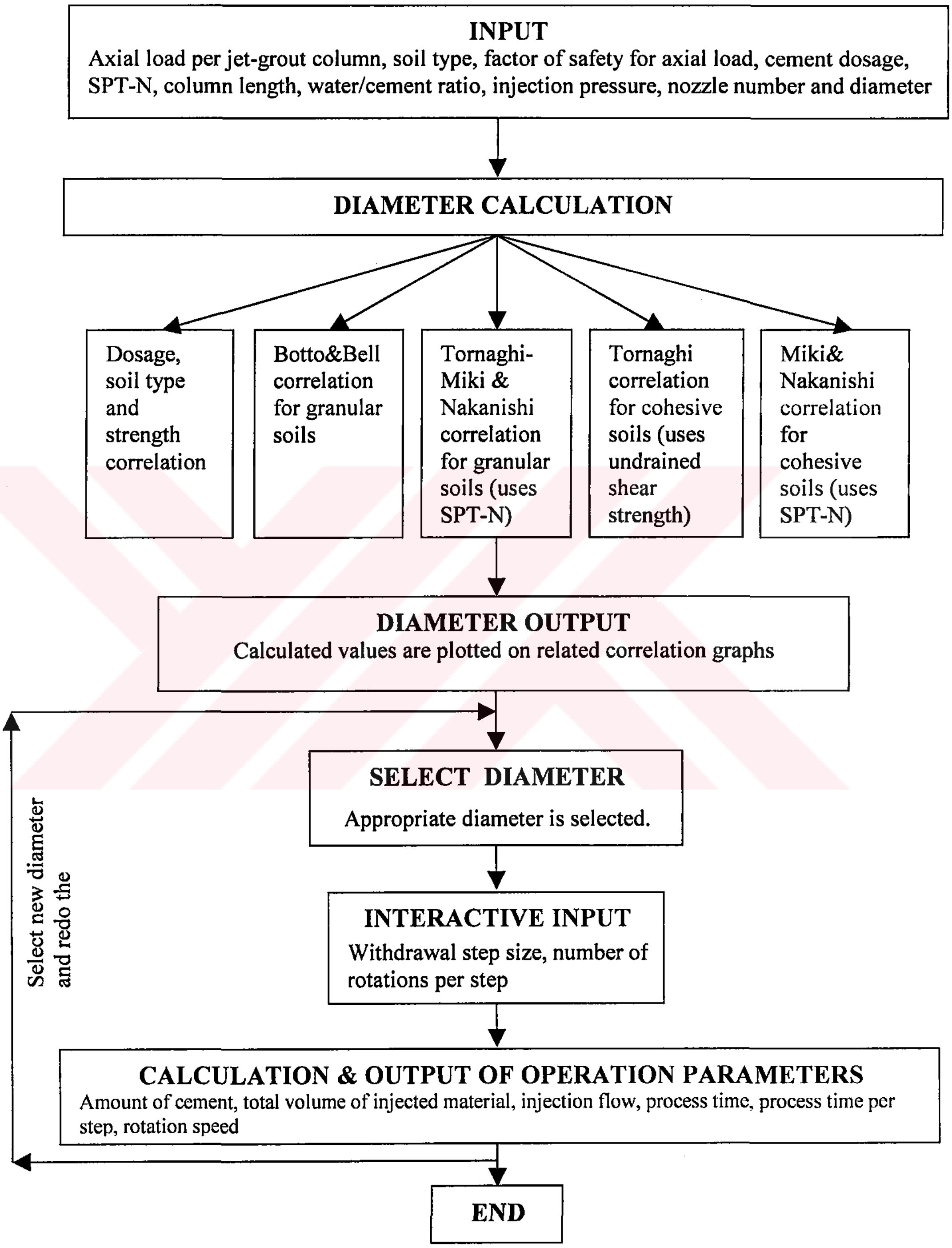

Figure 4.3 Flowchart of JETv1 algorithm

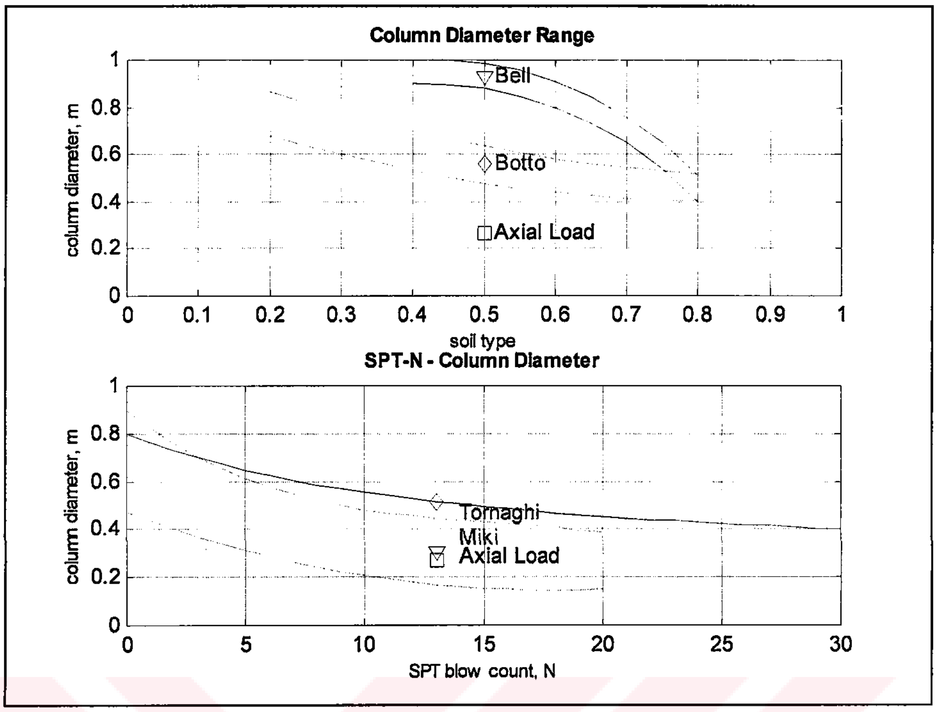

Figure 4.4 Calculation of column diameter for granular soil layer in Bostanlı Vilayet Evi Project (SPT-N=13)

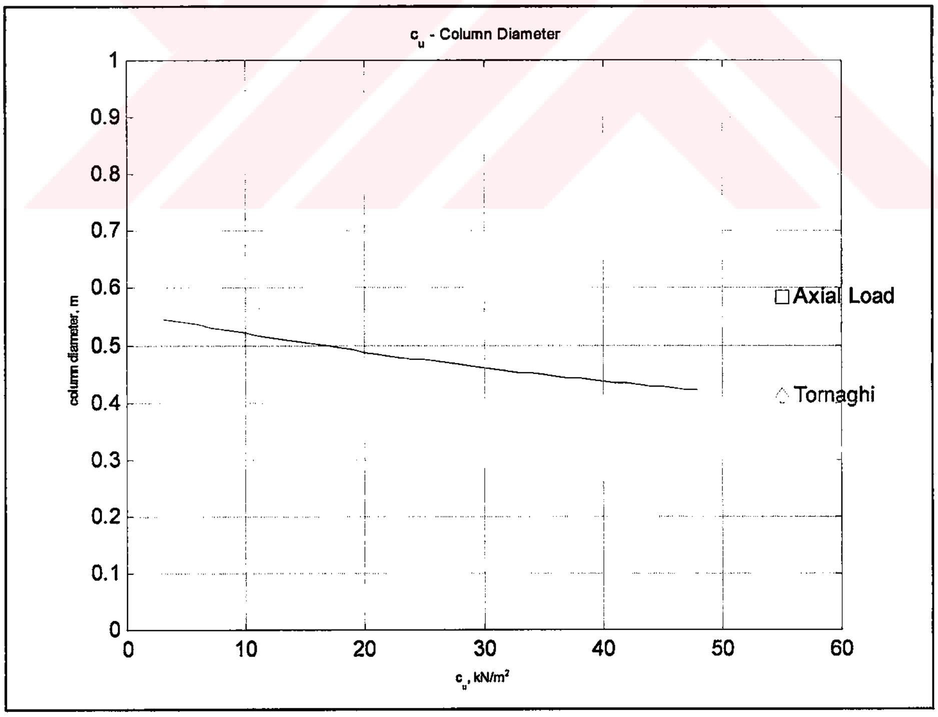

Figure 4.5 Calculation of column diameter for cohesive soil layer in Bostanlı Vilayet Evi Project (SPT-N′ = 8, c_u = 55 kPa)

```
PREPARED BY ALPER ASKAY
        JET GROUT CALCULATION PROGRAM
        INPUTS:

Force per jet-grout column (kN)? 441.74
Soil type ? G/Gravel; S/Sand; Sg/Sandy Gravel; SM/Silty Sand; M/Silt; Sc/Silty Clay; C/Clay; Pt/Peat C
Factor of Safety for Axial Stress ? 2
Cement dosage (kg/m^3)? 450
SPT-N ? 8
Undrained shear strength (kPa)? 55
Select diameter? 0.6
Length of soilcrete column? 17
water/cement ratio of grout? 1
Specific gravity of cement? 3.15
Injection Pressure (bar)? 500
Number of nozzle(s)? 2
Diameter of nozzle(mm)? 2

spec_grout =
   1.5181

volume_per_weight_grout =
   1.3175

volume_grout_col_meter =
  167.5414

time_process =
  104.8948

Decided process time (sn)? 105
Withdrawal step (cm)? 4
Number of complete turns per step? 1.5

rot_speed =

   21.4286

total_time =
  1.7832e+003
```

Figure 4.6 Sample output for clay soils in Bostanlı Vilayet Evi Project

CHAPTER FIVE
A CASE STUDY ON THE PERFORMANCE OF JET-GROUT COLUMNS DURING EARTHQUAKES

5.1 JET-GROUT COLUMNS IN EARTHQUAKE RISK REGIONS

Jet-grout column technique is a contemporary soil improvement method. Like other methods main goal is the improvement of certain soil parameters. Increase in the bearing capacity and liquefaction resistance as well as reduction of both differential and total settlement potentials of a foundation system are targeted by the production of jet-grout columns.

Following 1999 Marmara Earthquake, jet-grout column technique has been widely applied to improve dynamic soil behavior in earthquake prone regions of Turkey. This trend can be partially attributed to the superior performances of foundation systems of certain critical structures during the Marmara Earthquake. The third chapter of the comprehensive reconnaissance report of the Marmara Earthquake (Scawthorn, 2000) has been devoted to case histories of improved ground and authored by James Mitchell of Virginia Polytechnic Institute. Performances of three industrial facilities on liquefiable soil deposits treated using jet-grouting technique are included in the report: Carrefour Shopping Center (under construction at the time of earthquake), İpekkağıt Tissue Factory and Ford Plant. All of these sites were located close to the epicenter of the earthquake. Ford Plant was even on the fault rupture line in Gölcük. In all three cases liquefaction has been reported to be confined in the untreated areas between improved soil zones. There have been no signs of liquefaction or destructive lateral spreading in the improved ground. It was even argued that diversion of surface fault rupture in Ford Plant was due to the presence of heavily improved ground containing stone and jet-grout columns. Despite these reported cases, however, there have been an ongoing debate over reliability of jet-grout columns under seismic loads. Reasons for this argument can be listed as in the following:

a. Analysis methods for the performance of jet-grout columns under lateral seismic loads are not well-established and sound.

b. Although jet-grout columns improve certain characteristics of a problematic soil profile, they also function as vertical foundation elements and transfer structural loads to underlying stiffer soil layers. Therefore, geotechnical engineer should ensure that enough factor of safety for axial loads always exist under all conditions. Nevertheless, grout columns are not reinforced elements and this provokes fears about overall collapse of the foundation system if some of the columns fail under lateral seismic loading due to their poor bending moment capacity.

The subject deserves more attention and should be handled in a systematic manner. In this chapter, performance of jet-grout columns under lateral seismic loads as foundation elements will be evaluated considering lateral loads that are expected to act on. There are two basic mechanisms, which will be considered within the context of evaluation of a jet-grout application in Bostanli-İzmir region as a case study. These mechanisms are listed below and explained in the subsections of the case study.

1. Performance under inertial lateral loads
2. Performance under kinematical lateral loads

5.2 A CASE STUDY ON JET-GROUT COLUMN APPLICATION: BOSTANLI VİLAYET EVİ PROJECT

A three story building has been planned and projected by Vilayetler Hizmet Birliği in Bostanlı district of Karşıyaka county in the city of İzmir. Geotechnical site investigation and foundation analyses have resulted in high liquefaction risk and necessity for soil improvement to avoid liquefaction (ZETAŞ, 2000). Jet-grouting technology has been considered as the most appropriate method for this site since it was locally available, required less time and more economic compared with pile foundation alternative. In the following sections, data regarding idealized soil profile and structural loads will be given. Design details of jet-grout column system for axial loads will be excluded. Only column disposition and calculation sheets of the original geotechnical evaluation report by ZETAŞ company will be provided. Emphasis will be given to the analysis of the jet-grout column system under lateral seismic loads.

5.2.1 Idealized Soil Profile

Undrained shear strength, c_u, angle of internal friction, ϕ, unit weight and corrected standard penetration blow count number, N'_{60}, of idealized soil profile in the project site have been already tabulated in Table 4.1 of previous chapter. There are five major soil layers in the idealized soil profile. Average geotechnical parameters pertaining to lateral inertial load analysis of jet-grout columns are given in Figure 5.1. Ground water table is quite close to surface as can be seen in the figure. The soil profile is an example to the Old Gediz River Delta soils, which were deposited in Bostanlı region by the Gediz River in the Quaternary Era. Turkish Code of Hazards classifies this profile as Z4 and requires assessment of liquefaction potential. It should be mentioned that idealized soil profile has been derived using data acquired in three boreholes (SK1, SK2, SK3). Locations and logs of these boreholes and soil cross-section can be seen in Appendix-A.

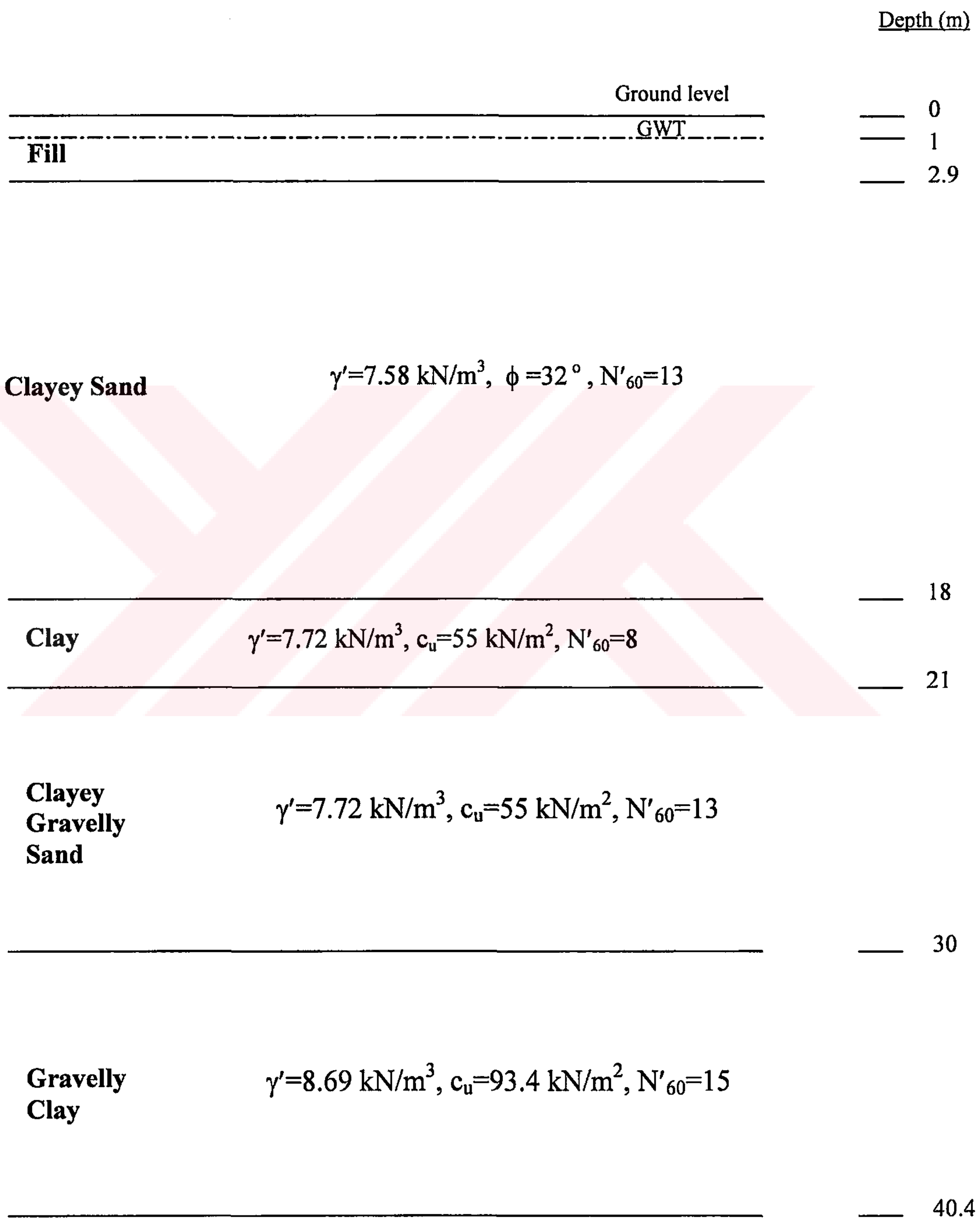

Figure 5.1 Average geotechnical parameters of the idealized soil profile

5.2.2 Structural Loads

Total structural load excluding weight of the foundation can be approximately taken as 95,000 kN. The foundation of the structure was designed as a mat foundation with a total area of 1850 m^2 and a thickness of 0.8 m at an elevation of 4.0 m below ground surface. The contact pressure including foundation weight has been reported as 71 kN/m^2. Since the projected structure was planned with one basement floor below ground surface there would be no significant net pressure at the foundation level.. Therefore, one should not expect settlement problem under static loads and bearing capacity should be adequate due mat foundation and large foundation depth.

Total lateral shear force at foundation level has been calculated as 33,250 kN using equivalent earthquake load method as suggested in Turkish Code of Hazards. Details are given in Table 5.1.

Table 5.1 Calculation of base shear force

Structural coefficient, I	1.4
Effective ground surface acceleration (g), A_0	0.4
Total height of the structure (m), H_n	~11.0
Fundamental period of the structure (s), $T_0 = C_t.H_n^{3/4}$	0.42 ($C_t = 0.07$)
Spectral coefficient, $S(T_0)$	2.5
Spectral Acceleration (g), $A(t)=A_o*I*S(T)$	1.4
Coefficient of flexibility, R	4
Load reduction factor, $R_a(T_0)$	4
Base shear force (kN), V_t $V_t=W*A(T_0)/R_a(T_0)>=0.10\ A_o\ I.\ W$	33,250

5.2.3 Jet-Grout Column Axial Load Capacity and Disposition

Analyses of liquefaction and settlement potential following an earthquake that would trigger liquefaction in the site have necessitated soil improvement and jet-grouting technology was chosen as the optimum method as explained above. It has been stated in the geotechnical evaluation report for the project (ZETAŞ, 2000) earthquake related settlements would be on the order of 30 cm exceeding settlement criteria. Besides, there would be differential settlements due to horizontal discontinuities as shown in the soil-cross section given in Appendix-A.

Jet-grout column diameter and length have been set as 0.6 m and 17.0 m, respectively, yielding an allowable load carrying capacity of 688 kN by ZETAŞ (2000). Related sheet for the calculation of load carrying capacity is provided in Appendix-B. Maximum vertical load to act on a single column has been given as 420 kN. It has been understood that allowable capacity of a jet-grout column was assumed as 420 kN to stay on the safe side. Axial load distribution has been recalculated in this study and maximum vertical load has been found as 445 kN. The difference may be due to calculation details and is considered as insignificant. Details of this calculation are presented in Appendix-B for outer columns. Number of columns 0.6 m in diameter was determined as 293 (ZETAŞ, 2001). Disposition of jet-grout columns is given in Appendix-B. It can be seen in the disposition plan that column spacing is not constant varying between 2.15 m and 3.50 m. A 0.3 m thick capping layer has been suggested between jet-grout columns and mat foundation (ZETAŞ, 2000). One should keep in mind that these parameters were taken from geotechnical evaluation reports. They might have been altered somehow in final design project.

5.2.4 Inertial Lateral Load Analyses

Response of jet columns to inertial lateral loading can be computed similar to vertical pile foundations using equivalent nonlinear p-y methodology for single piles. Nonlinear soil-pile-soil interaction (i.e. group effect) is not considered since column spacing is larger than three column diameter (i.e. S>3B). Load-deformation relationships to express nonlinear soil-pile interaction under inertial loads can be established using American Petroleum Institute standards (API, 1980).

Nonlinear p-y curves were established based on the idealized soil profile given in Figure 5.1. They are presented in Figures 5.2 and 5.3 for sand and clay layers, respectively. Note that Imperial units were used in the figures since computational software (COM62) requires input to be provided in these units.

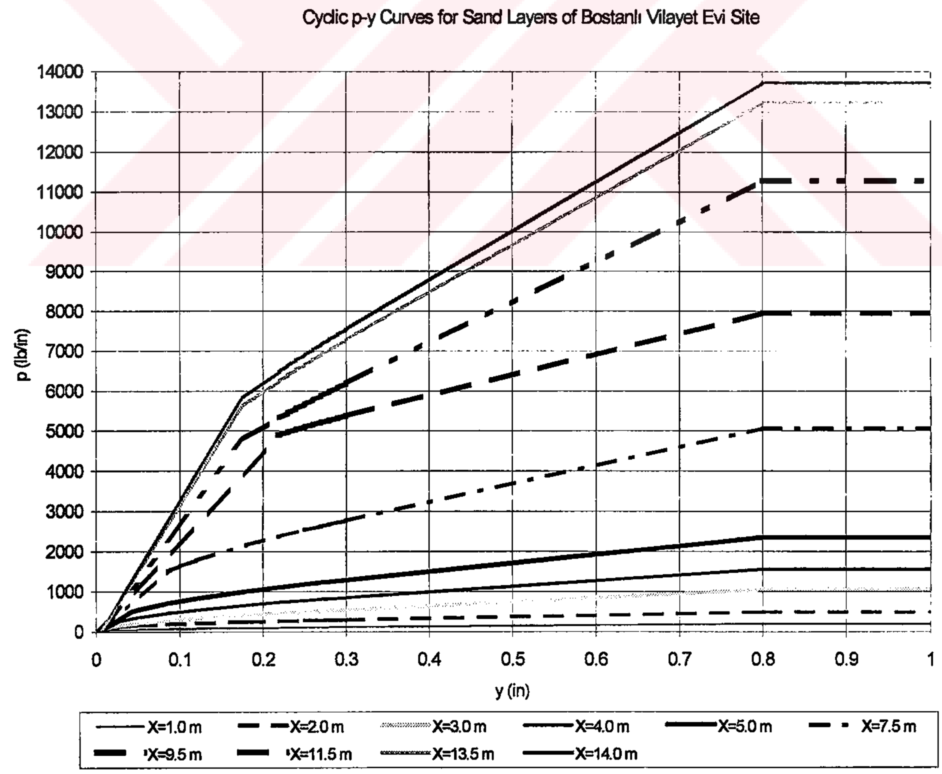

Figure 5.2 Nonlinear p-y curves for sand layer in idealized soil profile

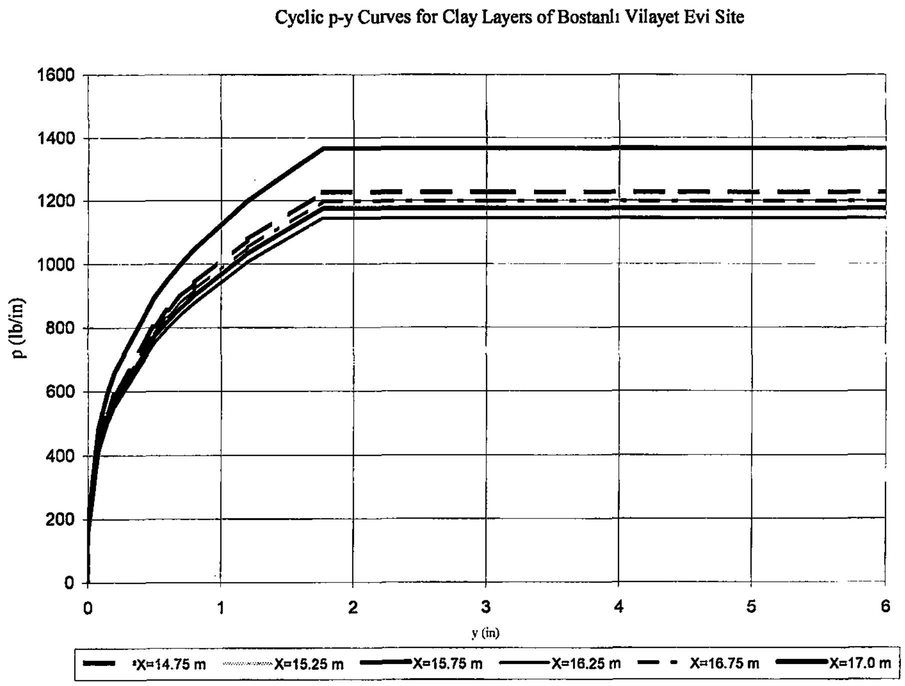

Figure 5.3 Nonlinear p-y curves for clay layer in idealized soil profile

Lateral load analyses have been performed for various load combinations. Minimum and maximum axial loads were considered separately. Horizontal load per column was calculated by assuming even distribution of total base shear force to jet-grout columns. Therefore, horizontal load per column has been found as 112 kN. Elasticity modulus, E_{jg}, of jet-grout columns was taken as 10^6 kN/m^2. Analyses have also been performed by varying column head shear between 50% and 100% of the maximum value with increments of 10%. The purpose behind these analyses was to observe change of factor of safety for bending stresses and to calculate column head shear at which columns can resist generated lateral stresses safely.

Variation of bending moment, column shear and deformation along column length are illustrated in Figures 5.4, 5.5 and 5.6. Note that cases only envelopes (i.e 50% and 100% of maximum pile head shear at minimum and maximum axial loads) are included in the graphs. However, variation of factor of safety, maximum bending moment and column shear with column head shear force are given for all increment steps are presented in Figures 5.7 to 5.9.

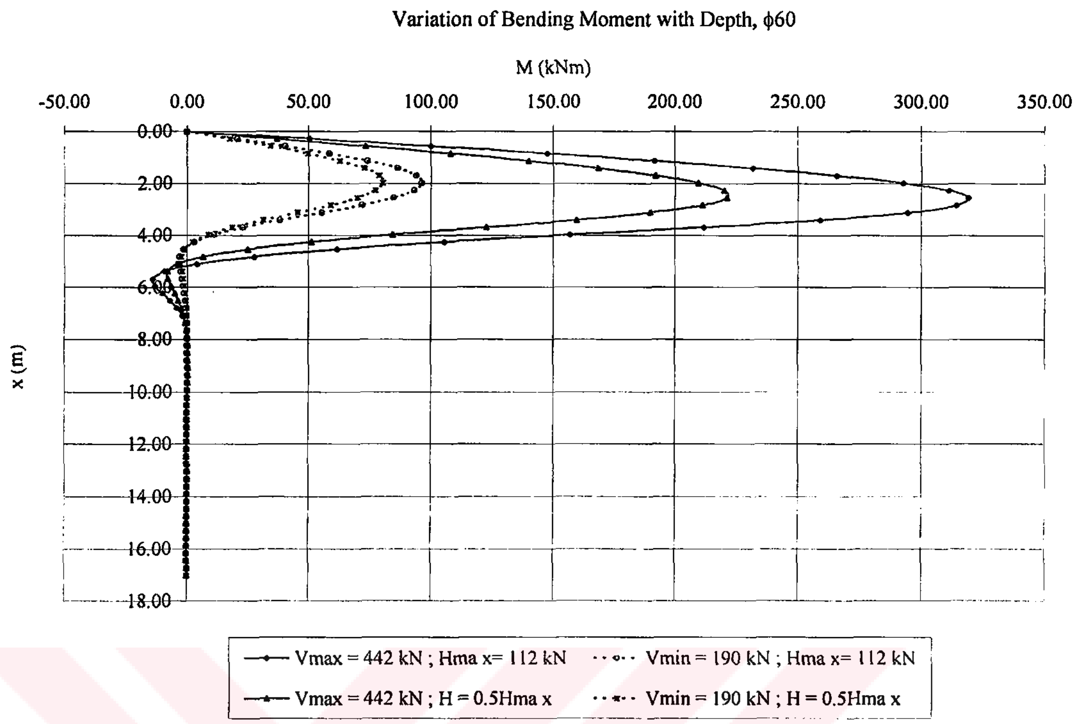

Figure 5.4 Variation of bending moment along column length

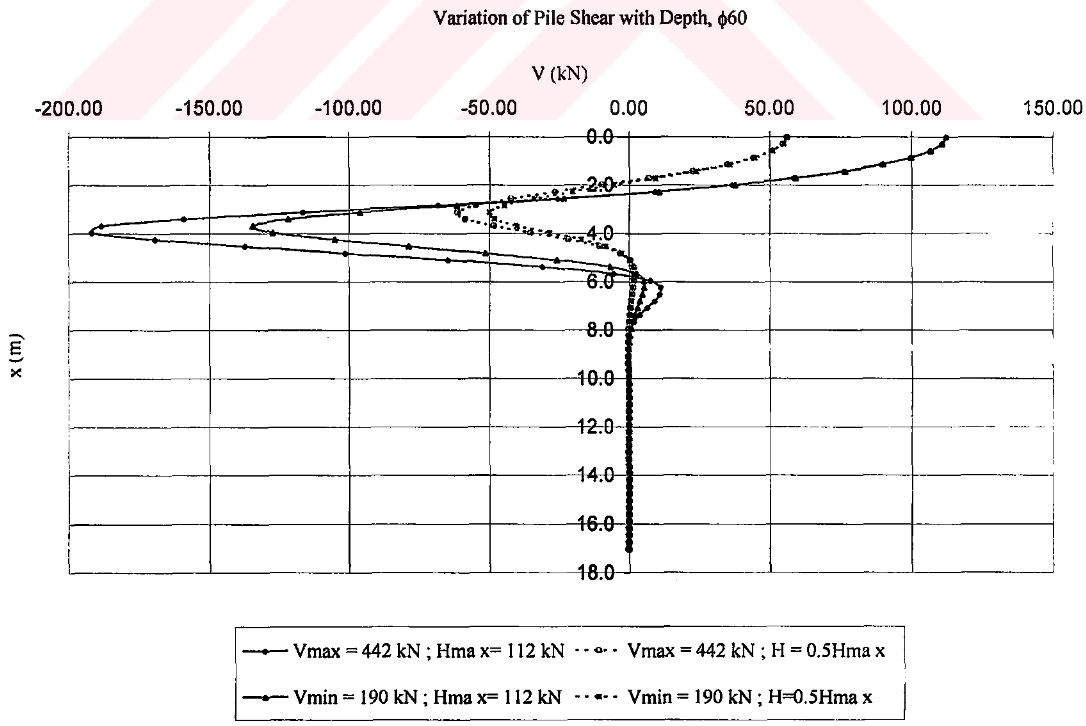

Figure 5.5 Variation of pile shear along column length

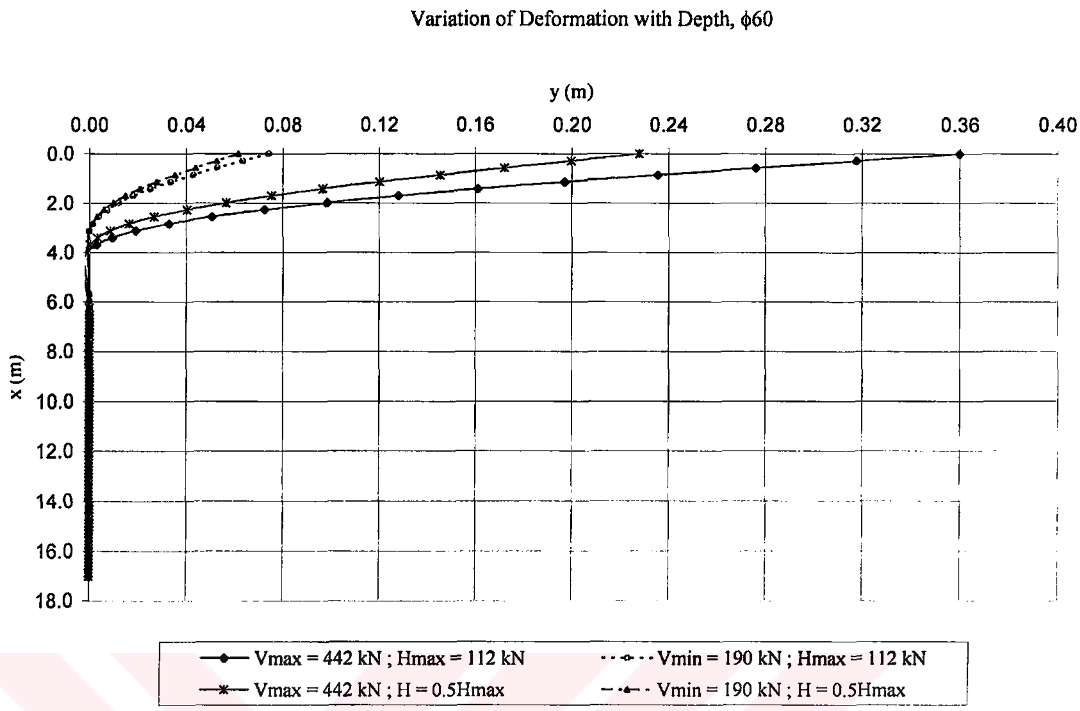

Figure 5.6 Variation of column deformation along column length

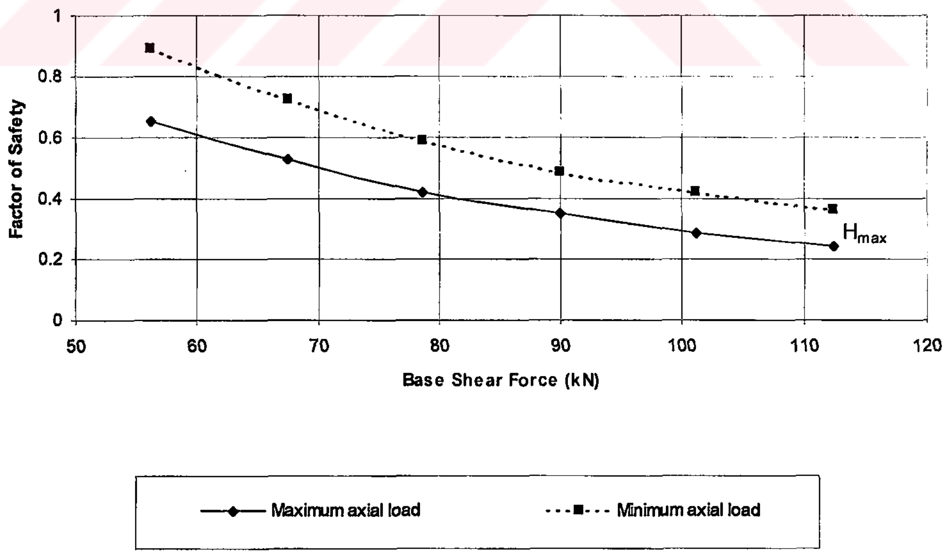

Figure 5.7 Variation of factor of safety for bending stress with base shear

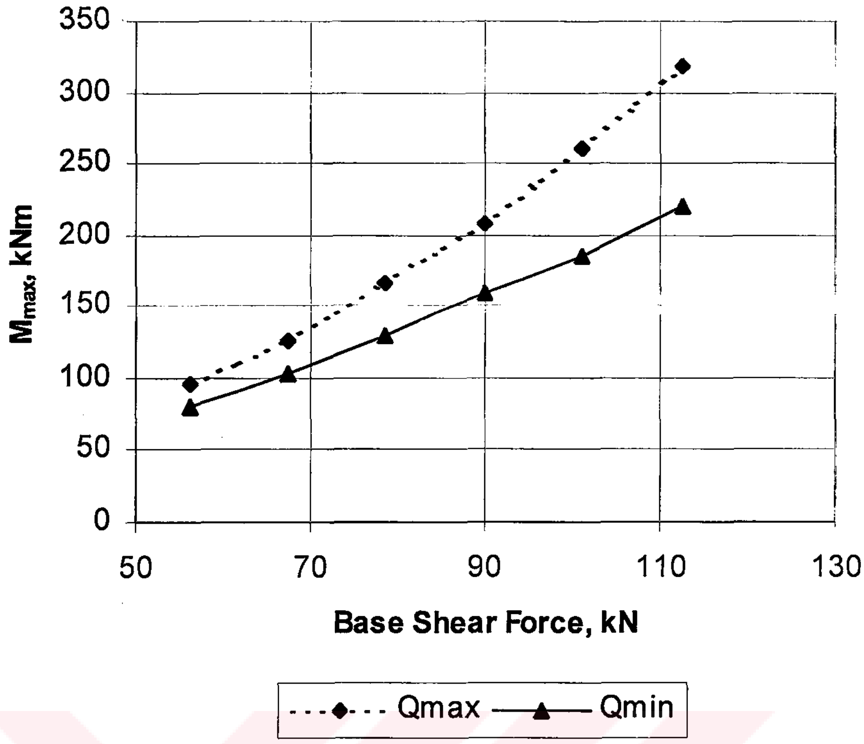

Figure 5.8 Variation of maximum bending moment, M_{max}, with base shear

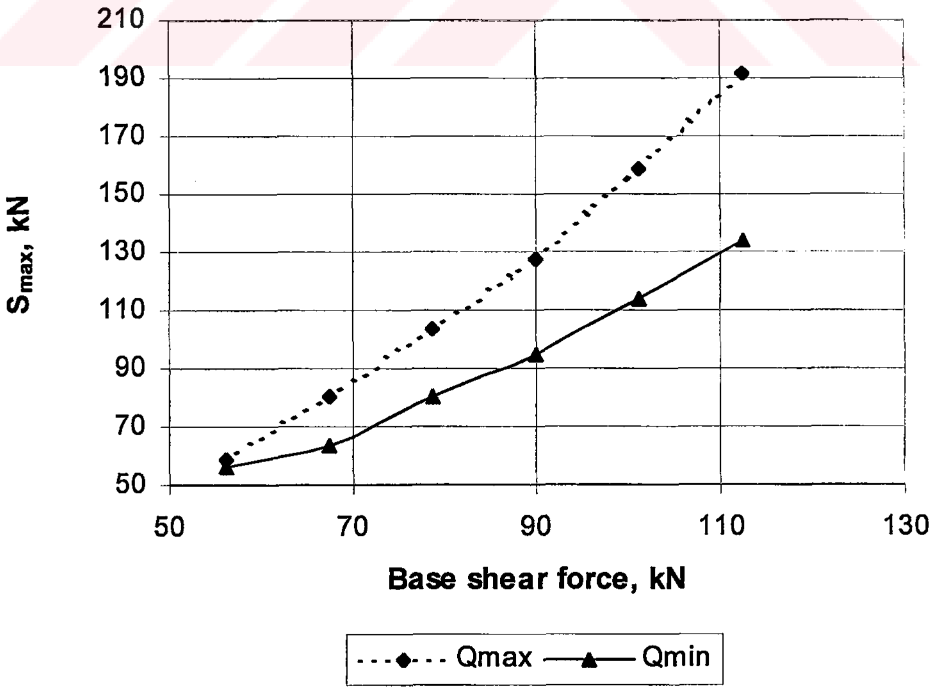

Figure 5.9 Variation of maximum column shear, S_{max}, with base shear

Study of Figures 5.4 to 5.9 reveals following findings of inertial lateral load analyses:

a. As expected bending moment decreases with decreasing base shear.
b. Axial load is effective on lateral column response unlike to reinforced concrete piles. It can be seen that maximum bending moments at maximum axial loads are almost 50% larger than those at minimum axial loads reminding that p-Δ effects are considerable in jet-grout columns.
c. It is found that column deformation attains very large values at maximum base shear which is an indication of exceedence of column's lateral load carrying capacity. It is a traditional check for deep foundations that head deformation stays on the order of 4.0 cm. This criterion is not satisfied for all cases. However, column deformation decreases with decreasing base shear and axial load.
d. Computed column deformation, shear and bending moment are negligible below approximately 15B. This depth is characteristic for most deep foundations.
e. Factor of safety for bending stresses are less than unity for all base shear values. This means that analyzed ϕ60 columns are not capable to withstand inertial loads. However, it can be seen in Figure 5.7 that factor of safety value approaches to one as base shear decreases. Further investigation of this finding is worthwhile since base shear at the bottom of the capping layer may be considerably less than the maximum base shear at the bottom of mat foundation.
f. Variations of maximum bending moment and column shear with respect to base shear are shown in Figures 5.8 and 5.9. Influence of axial load on lateral column response becomes more significant as base shear increases.

Inertial analyses have shown that jet-grout columns cannot withstand project loads as calculated at the botttom of the mat foundation. Nevertheless, it would be quite over design if lateral loads are considered in this manner. It would be expected that major portion of base shear at mat foundation level be damped out by capping layer and soil-structure interaction taking place along basement walls and foundation area (i.e. radiation damping). It should be remembered that calculation of base shear to form a basis for lateral analysis of jet-grout columns may be a very conservative approximation. Building codes require design of superstructures with heavy penalties to avoid total collapse. However, soil improvement projects are special design studies and neither suggestions nor requirements exist in Turkish Code of Hazards. Study of soil-structure effects is beyond the scope of this dissertation. On the other hand, an attempt has been made to investigate influence of capping layer on the damping of base shear and to pave the road for further soil-structure analyses in this area.

5.2.4.1 Effect of Capping Layer on Horizontal Force at Column Head

Influence of capping layer on the base shear that is transferred to jet-grout columns has been studied by varying its thickness and relative density. Dynamic site response analyses have been performed to compute maximum acceleration at the bottom of capping layer. EERA software was used during the analyses. Detailed information about this sofware were given by Eker (2002). Dynamic soil properties such as shear modulus reduction and damping curves and maximum shear moduli were assessed for all soil layers including the capping layer. Thickness of the capping layer increased from 0.3 m to 2.0 m. Intermediate values were selected as 0.7 m and 1.0 m. Influence of relative density was studied at very loose and very dense conditions (i.e. 30% and 90%).

Shear modulus reduction and damping curves were generated using the relationships suggested by Ishibashi and Zhang (1993). Details of these relationships may be found elsewhere (Kramer, 1996). Generated curves are given in Figures 5.10, 5.11 and 5.12 for sand, clay and capping layers, respectively.

Maximum shear moduli have been estimated using empirical relationships. Shear moduli were computed for sand layer taking average of three values obtained using different methods. The methods by Seed et al (1986) and Imai and Tonouchi (1982) utilize SPT blowcount numbers. They are respectively given in Equations 5.1 and

5.2. The third relationship employed for sand layer was the well known Seed and Idriss (1970) method that accounts for mean effective stress and relative density of the sand layer. This relationship is given in Equation 5.3 and Table 5.2. Maximum shear moduli for clay layers were estimated employing the method by Mayne and Rix (1993) that utilizes correlation with CPT tip resistance value. This relationship is given in Equation 5.4.

$$G_{\max} = 20000 \cdot (N')^{0.333} \cdot (\sigma'_m)^{0.5} \qquad (G_{max} \text{ and } \sigma'_m \text{ in } lb/ft^2) \qquad (5.1)$$

$$G_{\max} = 325' \cdot N'^{0.68} \qquad (G_{max} \text{ in } kips/ft^2) \qquad (5.2)$$

$$G_{\max} = 1000 \cdot K_{2,\max} \cdot (\sigma'_m)^{0.5} \qquad (\sigma'_m \text{ in } lb/ft^2) \qquad (5.3)$$

In the above equations σ'_m is mean effective stress and N′ is corrected SPT blowcount number. $K_{2,max}$ values are given in Table 5.2 as a function of void ratio and relative density.

Table 5.2 Variation of $K_{2,max}$

e	$K_{2,max}$	D_r (%)	$K_{2,max}$
0.4	70	30	34
0.5	60	40	40
0.6	51	45	43
0.7	44	60	52
0.8	39	75	59
0.9	34	90	70

$$G_{\max} = 406 \cdot (q_c)^{0.695} \cdot e^{-1.130} \qquad (G_{max} \text{ and } q_c \text{ in } kN/m^2) \qquad (5.4)$$

Maximum shear moduli values that were calculated using above given relationships are given for each soil layer in Table 5.3. Note that shear modulus of the capping layer in this figure belongs to 90% relative density. The modulus for 30% relative density has been calculated as 51 MPa.

Table 5.3 Soil profile data for site response analyses

Fundamental period (s) = 2.33
Average shear wave velocity (m/sec) = 205.78
Total number of sublayers = 85

	Layer Number	Soil Material Type	Number of sublayers in layer	Thickness of layer (m)	Maximum shear modulus G_{max} (MPa)	Initial critical damping ratio (%)	Total unit weight (kN/m^3)	Shear wave velocity (m/sec)	Location and type of earthquake input motion	Location of water table
Surface	1	2	*4*	1.0	95.25		23.00	*201.56*	Outcrop	*W*
	2	2	*4*	0.4	30.47		14.53	*143.44*		
	3	2	*4*	3.1	57.61		16.50	*185.08*		
	4	2	*4*	3.0	69.57		17.96	*194.94*		
	5	2	*4*	3.0	82.45		17.71	*213.71*		
	6	2	*4*	2.0	78.94		17.75	*208.88*		
	7	2	*4*	2.0	72.10		16.47	*207.23*		
	8	1	*4*	1.0	99.37		17.13	*238.55*		
	9	1	*4*	1.2	54.96		17.90	*173.55*		
	10	1	*4*	1.3	102.22		17.93	*236.49*		
	11	2	*4*	3.0	103.45		18.03	*237.25*		
	12	2	*4*	3.0	115.13		18.53	*246.88*		
	13	2	*4*	3.0	124.66		18.65	*256.07*		
	14	1	*4*	2.8	42.59		19.03	*148.18*		
	15	1	*4*	3.2	61.79		18.46	*181.21*		
	16	1	*4*	4.4	77.00		18.04	*204.63*		
	17	1	*20*	*82.6*	98.53		23.00	*205*		
Bedrock	18	0			28782.63	1	24.00	*3430*		

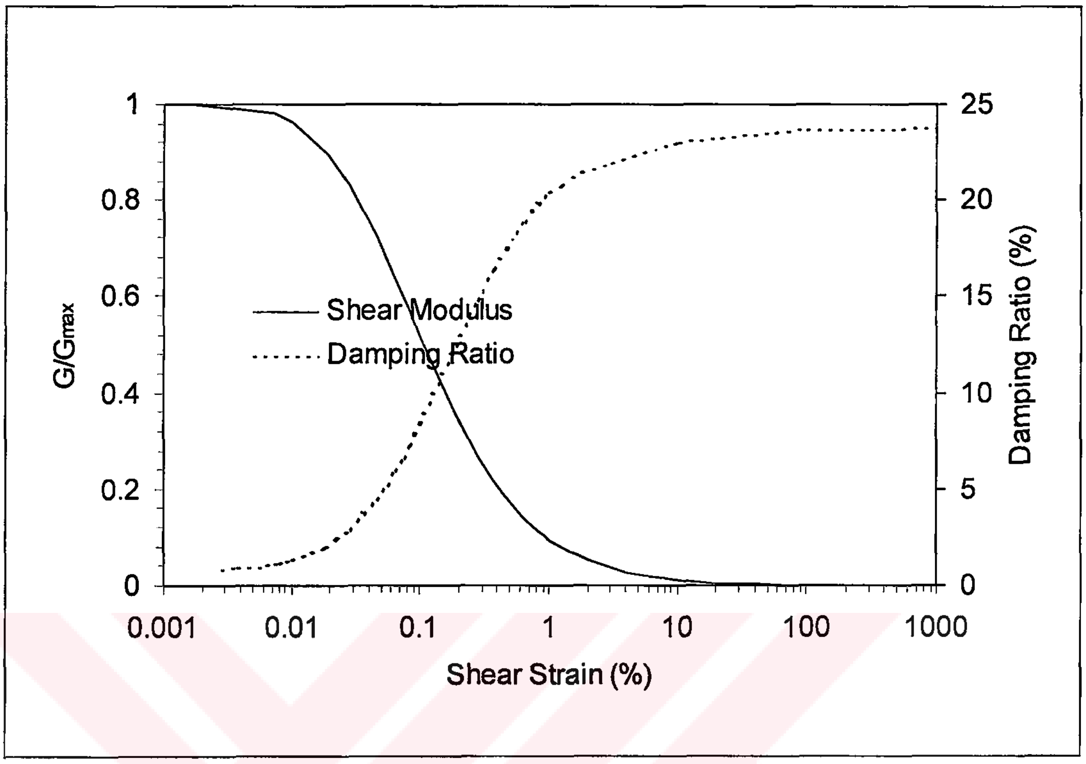

Figure 5.10 Shear modulus reduction and damping curves for sand layer

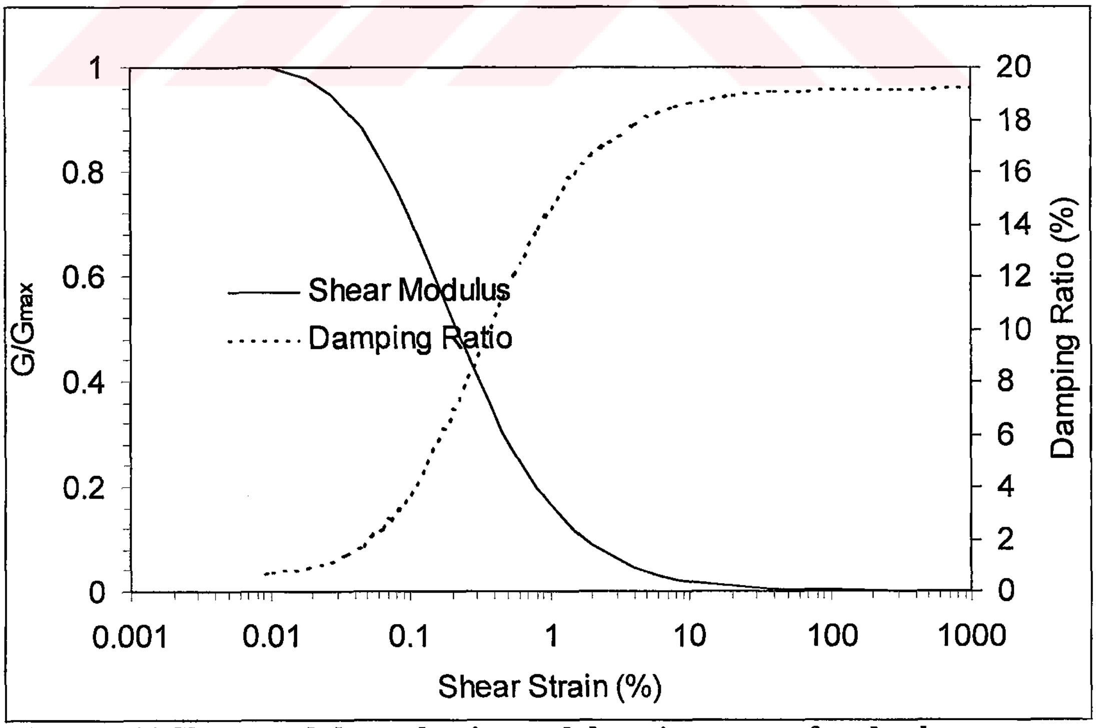

Figure 5.11 Shear modulus reduction and damping curves for clay layer

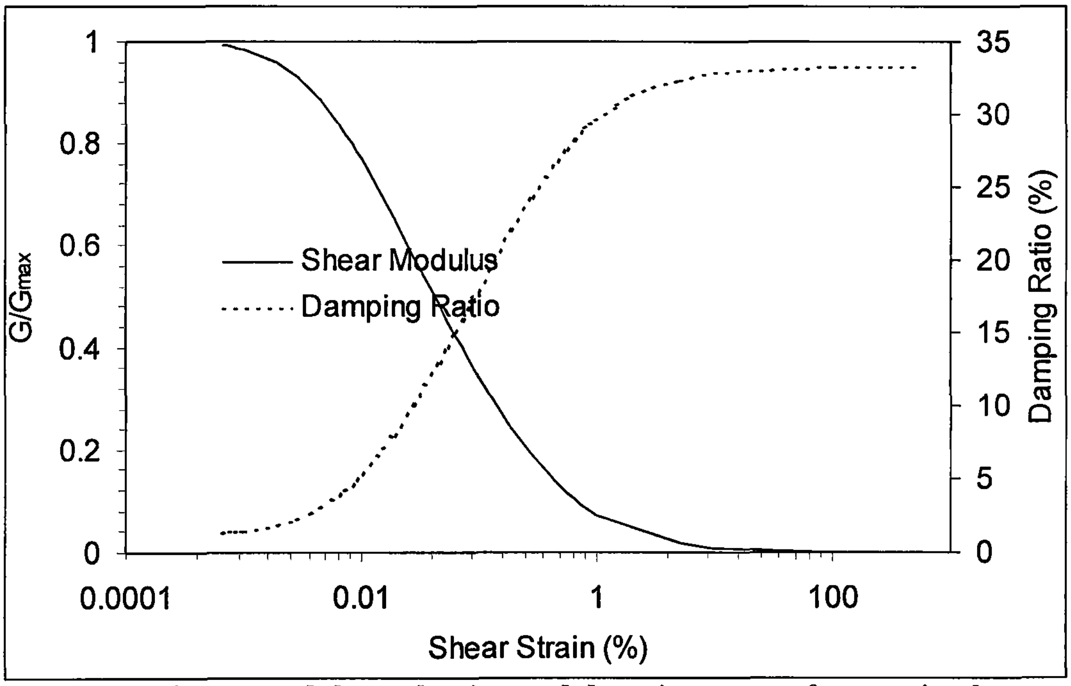

Figure 5.12 Shear modulus reduction and damping curves for capping layer

Site response analyses have been conducted using acceleration record of 1977 İzmir Earthquake. The maximum acceleration at bedrock has been scaled to 0.18 s at Bostanlı in order to account for attenuation effects. Time history of the earthquake is given in Figure 5.13.

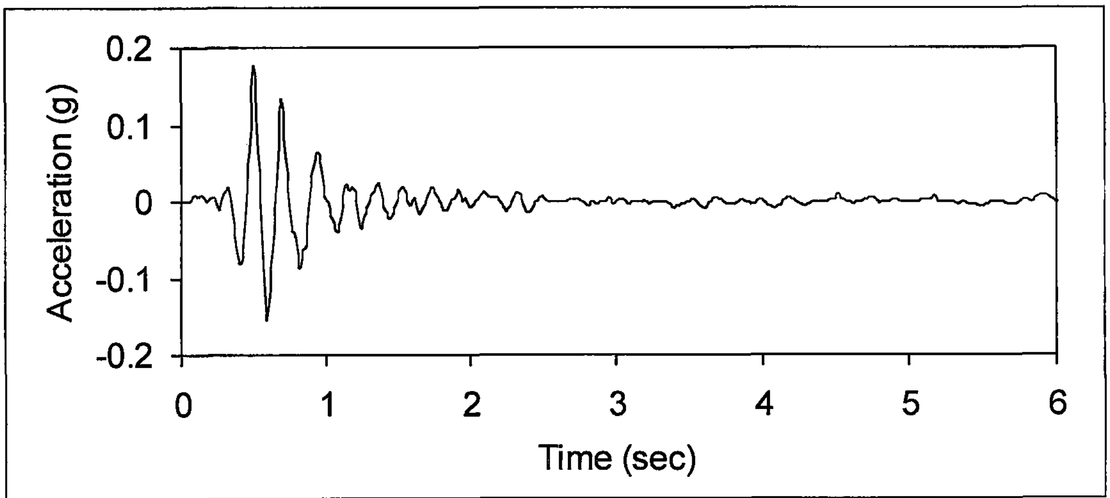

Figure 5.13 Scaled acceleration record of 1977 İzmir Earthquake

Acceleration values in the free field, at the bottom of capping layer were obtained for various combinations of capping layer thickness and relative density. In the first stage of analyses acceleration time history was obtained in the free field without capping layer (i.e. natural site conditions without influence of foundations and structures). Then computed free field time history was defined at the top of capping layer to observe influence of thickness and density on the acceleration taking place at the bottom of the capping. In this manner soil-structure interaction (SSI) effects were not taken into consideration. However, an approximation was made to account for SSI by assuming that capping layer behaves in combination with the structure and computing spectral accelerations at the bottom of the capping layer. This approximation was also helpful to observe differences between forces that would likely act on the structure during the earthquake and code suggestions. Computed maximum and spectral accelerations are presented and compared with code requirements in Table 5.4. Horizontal forces at column heads were estimated using acceleration values given in Table 5.5. Maximum horizontal forces in Table 5.5 were computed without taking flexibility coefficient into account. Similarly, spectral base shear forces were found. It was thought that flexibility coefficient suggested by the code should not apply to the calculated values since only one earthquake record was employed in the analyses. Results are shown in Table 5.5. Findings of analyses can be summarized as follows:

a. Dynamic site response analyses have resulted in smaller maximum ground surface accelerations than the value (0.4g) suggested in Turkish Code of Hazards for the studied earthquake (i.e. 1977 İzmir Earthquake) both in the free field and at the bottom of capping layer. One should keep in mind that higher values could be computed for different accelaration time histories.

b. Influence of capping layer is more pronounced for layers that are thicker than 0.3 m. It can be seen in Table 5.4 that maximum acceleration is reduced by 7.43% and 5.74 for 30% and 90% relative densities for 2.0 m thickness case. In reality these accelerations should attain lower values due to the positive contribution of SSI.

c. A smooth correlation has been found between the capping layer thickness and maximum acceleration at the bottom of this layer as shown in Figure 5.14. Similar correlations may be searched for in SSI analyses.

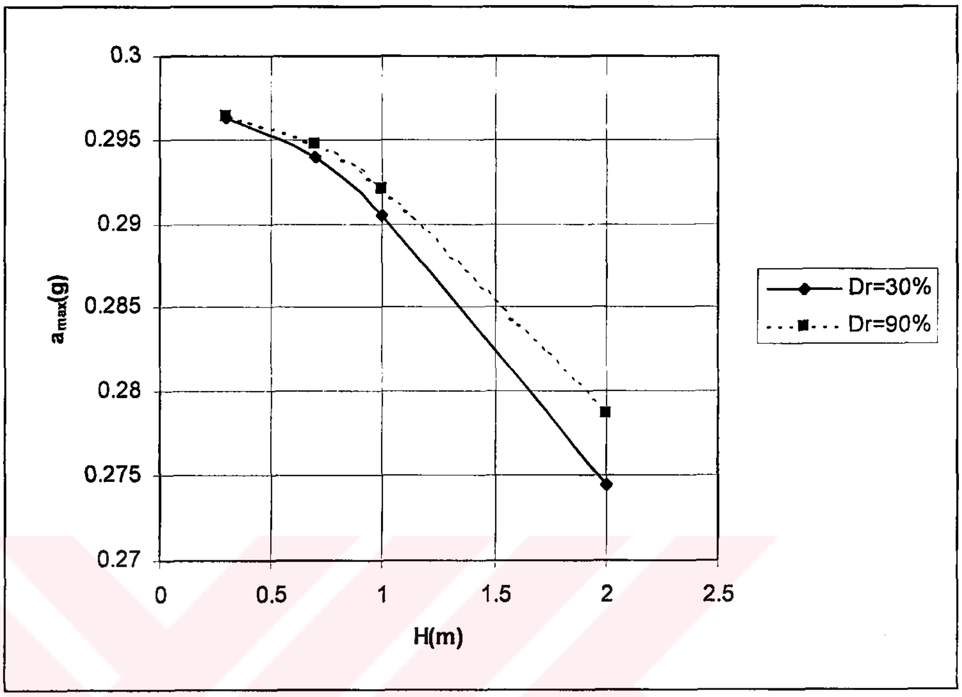

Figure 5.14 Correlation between capping thickness and acceleration

d. Computed spectral accelarations and consequent horizontal forces are considerably smaller than those found by the code requirements. This is due to the inherent conservative approach of the code to avoid total structural collapse. This approach may not apply to soil improvement projects. It is wortwhile to develop specific suggestions. Note that inertial lateral loads may not pose any risk to jet-grout columns when SSI effects are considered in conjunction with the influence of capping layer.

e. Further analyses are justified to come up with optimum capping layer thickness and relative density and to develop specific analyses methodologies for soil improvement projects using jet-grout technology.

Table 5.4 Computed maximum and spectral accelerations for capping layer

Capping Layer Soil Parameters				Maximum Acceleration						Spectral Acceleration at T_0					
				a_{max} (g)	a_{max} (g) (at the bottom of capping layer)				a_{max} (g) (Code)	$a_{spectral}$ (g)	$a_{spectral}$ (g) (at the bottom of capping layer)				$a_{spectral}$ (g) (Code)
					Capping Layer Thickness (m)						Capping Layer Thickness (m)				
D_r (%)	G_{max} (MPa)	V_s (m/s)	γ (kN/m^3)	free field	0.3	0.7	1.0	2.0	free field	free field	0.3	0.7	1.0	2.0	free field
30	50.7	176.4	16	0.296	0.296	0.294	0.291	0.274	0.4	0.19	0.19	0.19	0.19	0.18	1.4
90	95.2	201.6	23	0.296	0.296	0.295	0.292	0.279	0.4	0.19	0.19	0.19	0.19	0.18	1.4

Table 5.5 Computed maximum and spectral horizontal forces for capping layer

Capping Layer Soil Parameters				Maximum Horizontal Force at Column Head						Spectral Horizontal Force at T_0					
				V_{max} (ton)	V_{max} (ton) (at the bottom of capping layer)				F_{max} (ton) (Code)	$F_{spectral}$ (ton)	$F_{spectral}$ (ton) (at the bottom of capping layer)				$F_{spectral}$ (ton) (Code)
					Capping Layer Thickness (m)						Capping Layer Thickness (m)				
D_r (%)	G_{max} (MPa)	V_s (m/s)	γ (kN/m^3)	free field	0.3	0.7	1.0	2.0	free field	free field	0.3	0.7	1.0	2.0	free field
30	50.7	176.4	16	9.597	9.597	9.532	9.435	8.884	12.97	6.16	6.16	6.16	6.16	5.84	11.35
90	95.2	201.6	23	9.597	9.597	9.565	9.468	9.046	12.97	6.16	6.16	6.16	6.16	5.84	11.35

5.2.5 Kinematical Lateral Load Analyses

Kinematical interaction between surrounding soil layers and jet-grout column take place during passage of seismic waves that generate vibratory deformations on the column, and result in bending moments along the column length. These moments especially become significant at layer interfaces where stiffness and strength characteristics suddenly change. Incompatible motion of the column and surrounding soils is the main reason for induced moments. Stiffness difference between column material and soil is the cause for incompatibility. Therefore, one should expect higher bending moments for the stiffer one of the two columnar elements having same diameter.

Kinematical bending moments may be calculated approximately using simple analytical method of Nikolaou and Gazetas (1997). Suggested relationships are given in Equations 5.5 and 5.6.

$$\tau_c \approx a_s \cdot \rho_1 \cdot h_1 \qquad (5.5)$$

$$M \cong 0.042 \cdot \tau_c \cdot d^3 \left(\frac{L}{d}\right)^{0.30} \cdot \left(\frac{E_p}{E_1}\right)^{0.65} \left(\frac{V_2}{V_1}\right)^{0.50} \qquad (5.6)$$

where,

τ_c: proportional to the actual shear stress that is likely to develop at the interface, as a function of the free-field acceleration at the soil surface, a_s

M : Kinematical bending moment

d : Pile diameter

L : Pile length

E_p : Pile modulus

E_1 : Average soil modulus of layers above interface

V_2 : Average shear wave velocity of layers between interface and approximately 10d below interface

V_1 : Average shear wave velocity of layers above interface

a_s : Ground surface acceleration

ρ_1 : Average density of layers above interface

h_1 : Distance between ground surface and interface

Above given relationships enable calculation of bending moments under steady state conditions. Therefore, corrections need to be made for transient loading. Following correction factors were suggested for resonant and non-resonant conditions (i.e. correction factors are different whether fundamental frequencies of bedrock motion and the soil profile coincide or not).

$\eta \approx 0.04N_c + 0.23$ (resonant conditions) (5.7)

$\eta \approx 0.015N_c + 0.17 \approx 0.2$ (non-resonant conditions) (5.8)

where,

η: moment reduction factor

N_c: effective number of cycles in the acceleration record

The soil profile in the case study site does not exhibit very sharp change in terms of shear wave velocities of alternating layers. One may conclude that a relatively mild variation exists at about 11.5 m depth below ground surface based on the dynamic soil profile given in Table 5.3. The method has been applied at this elevation to check significance of kinematical interaction in this profile. Bending moment has also been calculated for a reinforced concrete pile with same diameter in order to compare two columnar elements having same dimensions but different stiffness characteristics. Results of this analysis are shown in Table 5.6. Pertinent parameters are also given in the same table. Shear wave velocities, soil densities were all calculated based on Table 5.3. Poisson's ratio has been set to 0.4.

Table 5.6 Results of kinematical interaction analyses

<table>
<tr><th colspan="2" rowspan="3">Analyses parameters</th><th colspan="4">Kinematical Bending Moments (kNm)</th></tr>
<tr><th colspan="2">Jet-Grout Column</th><th colspan="2">RC Pile</th></tr>
<tr><th>resonant</th><th>non-resonant</th><th>resonant</th><th>non-resonant</th></tr>
<tr><td>h_1 (m)</td><td>11.5</td><td rowspan="10">1.63</td><td rowspan="10">1.0</td><td rowspan="10">14.5</td><td rowspan="10">8.6</td></tr>
<tr><td>V_1 (m/sn)</td><td>188.9</td></tr>
<tr><td>V_2 (m/sn)</td><td>206.4</td></tr>
<tr><td>E_{jg} (kN/m^2)</td><td>1x10^6</td></tr>
<tr><td>E_{pile} (kN/m^2)</td><td>2.9x10^7</td></tr>
<tr><td>E_1 (kN/m^2)</td><td>188,392</td></tr>
<tr><td>a_s (m/s^2)</td><td>2.9</td></tr>
<tr><td>ρ_1 (Mg/m^3)</td><td>1.64</td></tr>
<tr><td>L (m)</td><td>17</td></tr>
<tr><td>d (m)</td><td>0.6</td></tr>
</table>

It can be seen in the above table that kinematical bending moments are quite negligible for both jet-grout column and RC pile cases. It is noticeable that moments for the RC pile are several times larger than those for the jet-grout column. This is expected since jet-grout column can follow vibration of soil layers more closely than the RC pile due to its low stiffness. It can be concluded that kinematical effects are not harmful to the jet-grout columns for the analyzed soil profile, column dimensions and the considered earthquake.

CHAPTER SIX
CONCLUSIONS

The goal of this study was to study jet-grouting technology within the context of a case study with an emphasis on the analysis of jet-grout columns under lateral seismic loads. Conclusions of the dissertation are given below:

1. Jet-grouting technology is a contemporary soil improvement method commonly used to increase bearing capacity and liquefaction resistance of problematic soils.
2. Jet-grouting method gives best results in cohesionless soils. Its advanced versions, however, can still be used in cohesive soils.
3. Greatest advantage of jet grouting is the predetermination of the content and amount of binder to be injected. Besides, the method can be applied fast and quietly compared with most other improvement methods.
4. Most apparent disadvantage of jet grouting, on the other hand, is the difficulties in quality control and assurance of the resultant product, which is a soilcrete column in most cases.
5. Soilcrete columns produced during the injection process function similar to deep foundations under structural loads. Therefore, estimation of axial load carrying capacity of a jet-grouting column is made using pile foundation design principles.
6. Principles of estimation of lateral load carrying capacity of soilcrete columns have not been established with confidence yet. This is especially true for seismic lateral loads.
7. Turkish Code of Hazards does not suggest any specific regulations for the design and analysis of special soil improvement projects such as jet-grout applications.

8. Both inertial loads and kinematical loads should be taken into account in the lateral load carrying capacity analysis of jet grout columns. The greatest uncertainty in this process is the estimation of load magnitudes properly. This requires understanding of load transfer mechanism. Debates over the lateral load carrying capacity of soilcrete columns originate due to lack of soundly established load estimation methods.
9. Good performance of jet grout columns in the 1999 Marmara Earthquake was documented in referenced reconnaissance report. This is largely in contradiction with the analyses results if they are conducted with the traditional approach (i.e. inertial loads are calculated with the equivalent method suggested in the code and assumed to be fully transferred as base shear at the column heads).
10. It was found by simple dynamic soil response analysis that capping layer placed between the mat foundation and jet grout columns can effectively reduce inertial loads being transferred to column heads during a seismic event. Capping thickness and relative density were the studied parameters. Calculated spectral accelerations using 1977 Izmir Earthquake record were found considerably smaller that those of traditional code requirements suggested for structural design. Results have shown that further studies on the subject are justified. Soil-structure interaction analyses are necessary for the understanding of influence of various parameters such as foundation type, capping layer characteristics and jet grout properties. It is recommended that future numerical analyses be made to come up with simple methods that can be used in daily design work.
11. Response of jet grout columns to kinematical loads has been found superior to RC piles having same dimensions for the same profile and earthquake loading. This is expected since soilcrete columns are more flexible than RC piles and can better follow soil deformations.

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APPENDICES

APPENDIX A

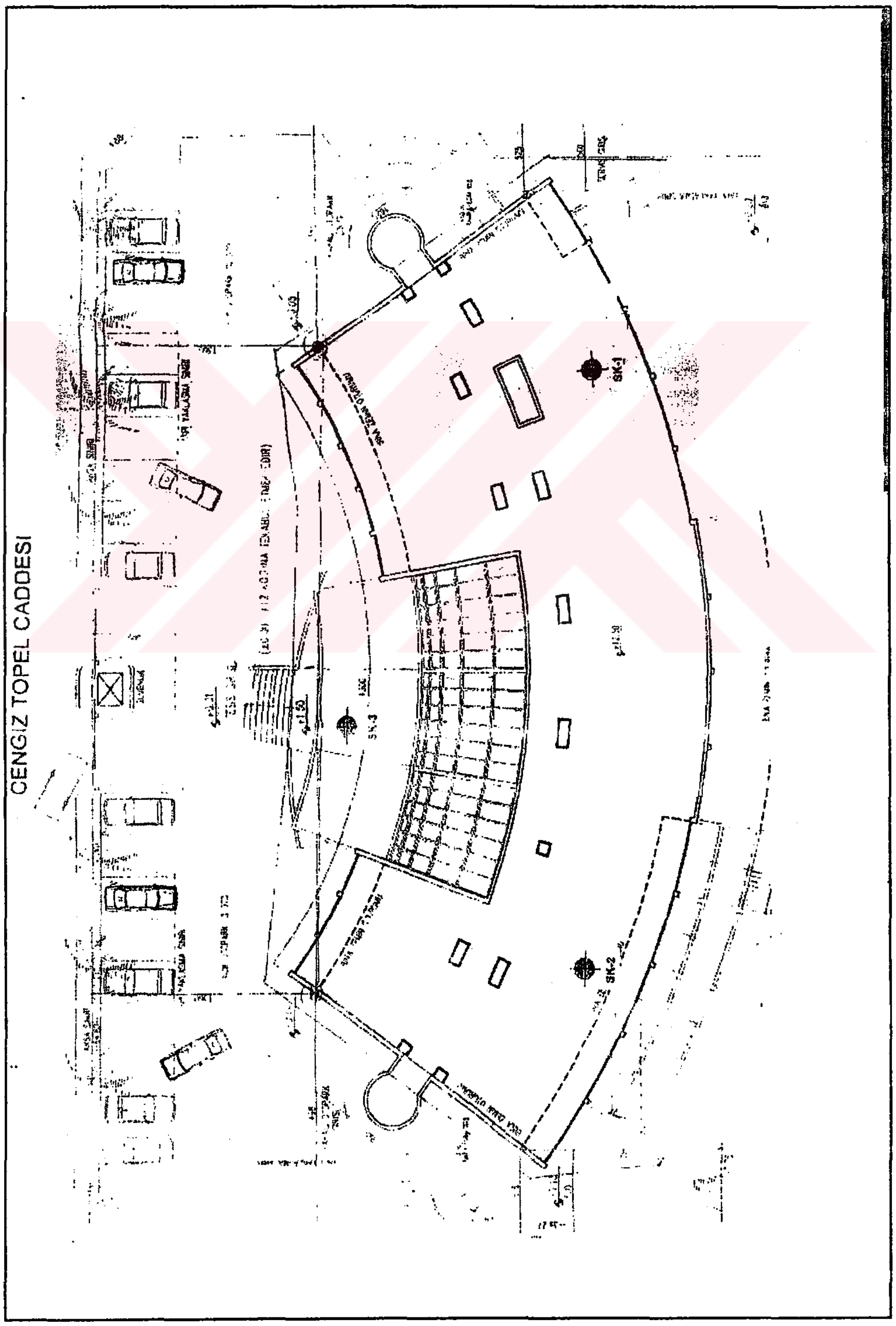


Figure A.1 Borehole locations

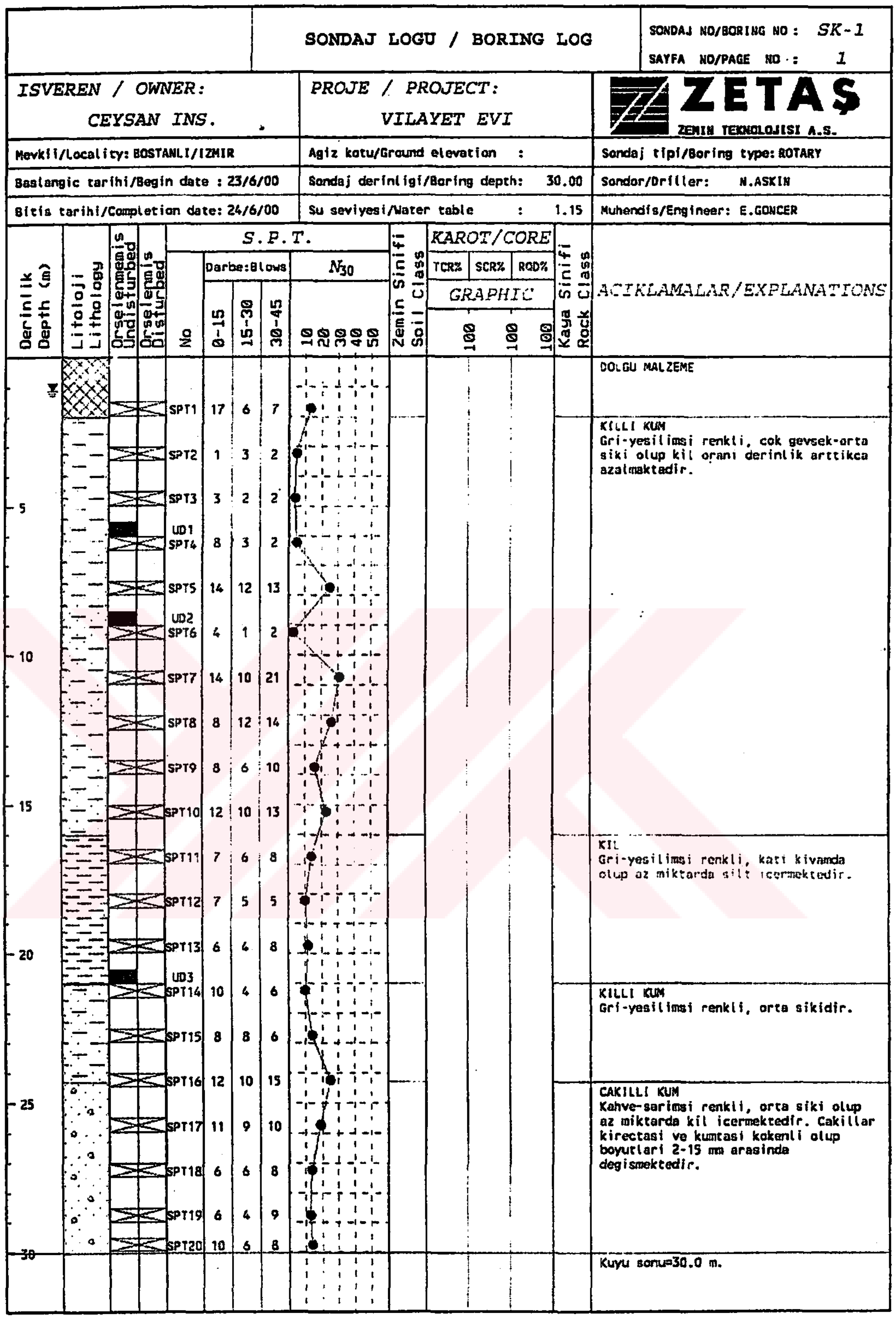


Figure A.2 Borehole log of SK1

SONDAJ LOGU / BORING LOG

SONDAJ NO/BORING NO :	SK-2
SAYFA NO/PAGE NO :	1

ZETAŞ

ISVEREN / OWNER: [illegible]	PROJE / PROJECT: [illegible]	
Mevkii/Locality: BOSTANLI/IZMIR	Agiz kotu/Ground elevation :	Sondaj tipi/Boring type: ROTARY
Baslangic tarihi/Begin date : 25/6/00	Sondaj derinligi/Boring depth: 41.00	Sondor/Driller: N.ASKIN
Bitis tarihi/Completion date: 27/6/00	Su seviyesi/Water table : 1.00	Muhendis/Engineer: E.GONCER

Derinlik Depth (m)	Litoloji Lithology	Orselenmemis Undisturbed	Orselenmis Disturbed	No	S.P.T. Darbe:Blows 0-15	15-30	30-45	N_{30} (10 20 30 40 50)	Zemin Sinifi Soil Class	KAROT / CORE TCR%	SCR%	RQD%	Kaya Sinifi Rock Class	ACIKLAMALAR/EXPLANATIONS
				SPT1	50/5		-							
				SPT2	2	1	1							KILLI KUM Gri-yesilimsi renkli, cok gevsek sıkı durumdadır. Kil miktarı duzensizdir.
5				SPT3	3	1	2							
				SPT4	8	6	12							
				SPT5	6	6	9							
				SPT6	4	3	3							
10				SPT7	2	1	1							
				SPT8	8	18	22							
				SPT9	7	17	14							
15				SPT10	2	1	4							
				SPT11	2	2	4							
				UD1 SPT12	6	9	11							KIL Gri-yesilimsi renkli, kat-cok kati kivamda olup az miktarda silt icerikli ve yuksek plastisitelidir.
20				SPT13	6	6	7							
				SPT14	5	8	8							
				SPT15	8	8	17							CAKILLI KUM Kahve-sarimsi [illegible] pek az kil ve duzensiz cakil icermektedir. Cakil boyutlari 4-10 mm. arasinda degismektedir.
				SPT16	8	17	14							
25				SPT17	9	15	17							
				SPT18	13	16	18							
				SPT19	17	35	24							
30				SPT20	33	28	24							
				SPT21	15	25	25							
				SPT22	18	20	14							
35				SPT23	13	15	15							CAKILLI KIL Kahve-sarimsi renkli, cok kati-sert kivamda az kum icerikli.
				SPT24	12	9	14							
				SPT25	10	8	13							
				SPT26	9	16	28							
40				SPT27	5	7	11							Kuyu sonu=41.00 m.

Figure A.3 Borehole log of SK2

SONDAJ LOGU / BORING LOG

SAYFA NO/PAGE NO : 1

ISVEREN / OWNER: CEYSAN INS.	PROJE / PROJECT: VILAYET EVI	ZETAŞ ZEMIN TEKNOLOJISI A.S.
Mevkii/Locality: BOSTANLI/IZMIR	Agiz kotu/Ground elevation :	Sondaj tipi/Boring type: ROTARY
Baslangic tarihi/Begin date : 28/6/00	Sondaj derinligi/Boring depth: 30.00	Sondor/Driller: N.ASKIN
Bitis tarihi/Completion date: 29/6/00	Su seviyesi/Water table : 1.10	Muhendis/Engineer: E.GONCER

Derinlik Depth (m)	Orselenmemis Undisturbed	No	S.P.T. Darbe:Blows 0-15	15-30	30-45
		SPT1	10	8	8
		SPT2	3	2	2
5		SPT3	4	1	1
		[illegible]	[illegible]	[illegible]	[illegible]
		SPT5	8	12	16
10	UD1	SPT6	9	12	18
		SPT7	10	8	11
		SPT8	3	3	5
15		SPT9	3	4	3
		SPT10	4	5	2
	UD2	SPT11	2	1	1
		SPT12	6	4	6
20		SPT13	6	6	8
		SPT14	8	13	13
		[illegible]	[illegible]	[illegible]	[illegible]
25		SPT16	12	15	14
		SPT17	15	18	16
		SPT18	12	10	13
30		SPT19	14	11	10

Column headings: Derinlik / Depth (m); Litoloji / Lithology; Orselenmemis / Undisturbed; Orselenmis / Disturbed; S.P.T. No; Darbe:Blows 0-15, 15-30, 30-45; N_{30} 10 20 30 40 50; Zemin Sinifi / Soil Class; KAROT/CORE TCR% SCR% RQD% GRAPHIC 100 100 100; Kaya Sinifi / Rock Class; ACIKLAMALAR/EXPLANATIONS

ACIKLAMALAR/EXPLANATIONS

DOLGU MALZEME

KILLI KUM
Gri-yesilimsi renkli, cok gevsek-orta siki olup pek az silt icermektedir.

KIL
Gri yesilimsi renkli, yumusak orta kati kivamda olup az miktarda silt icermektedir.

KILLI KUM
Kohve-yesilimsi renkli, orta siki, pek az ince -orta cakil icermektedir.

CAKILLI KUM
Kahve-sarimsi renkli, orta siki olup az miktarda kil icermektedir.

Kuyu sonu=30.0 m.

Figure A.4 Borehole log of SK3

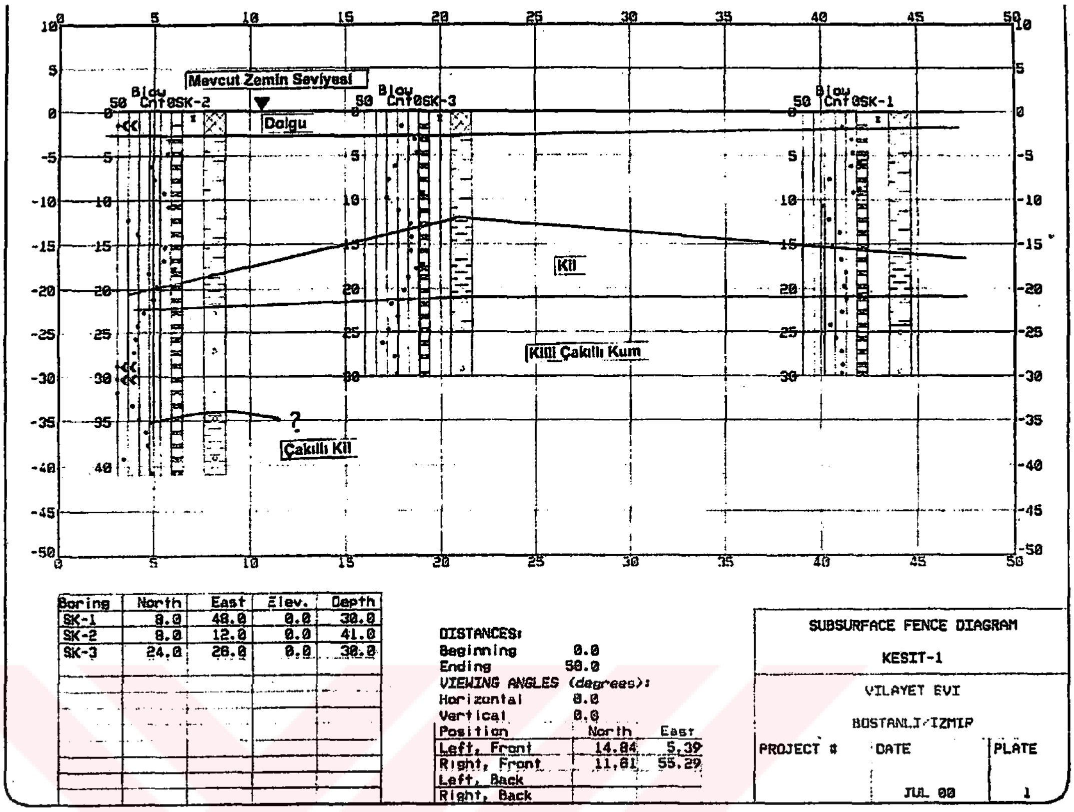


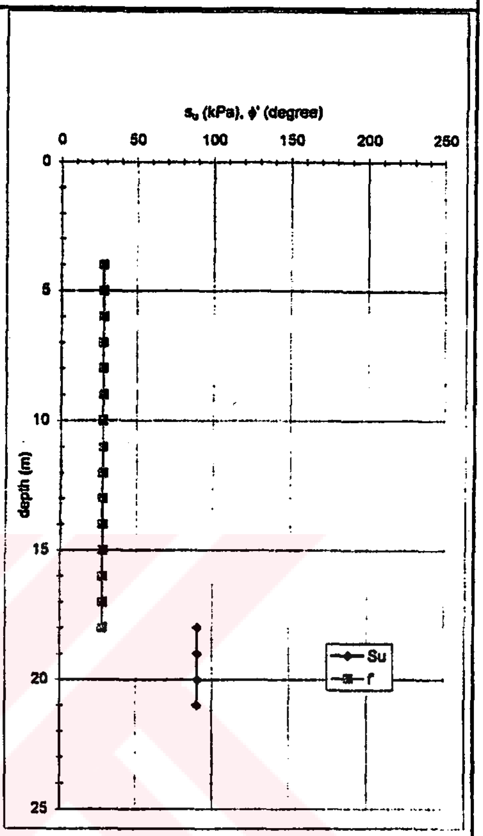
Figure A.5 Soil profile cross-section in project site

APPENDIX B

VILAYET EVI PROJESI
JET GROUT KOLONU KAPASİTESİ HESAPLARI
JET GROUT COLUMN CAPACITY ESTIMATION
BASED ON s_u †

s_u vs depth

depth (m)	s_u (kPa)	ϕ' (degree)
0		
1		
2		
3		
4		28
5		28
6		28
7		28
8		28
9		28
10		28
11		28
12		28
13		28
14		28
15		28
16		28
17		28
18	90	28
19	90	
20	90	
21	90	



Dcr=10B (loose sand)
Dcr (m) = 6.0
YASS/GWT (m) = 1.0
Nq:
(K=1.0 / δ=ϕ)

jet grout column

diameter	length	tip area	skin area/1.0m length
ϕ 60 cm	L = 17.0 m	A = 0.28 m2	S = 1.88 m2

tip resistance

$q_{tip} = N_c \cdot s_u \cdot A \;/\; \sigma vo' \cdot Nq \cdot A \cdot Rb$ (Rb=Reduction factor due to pile dimension)
= (Rb=1)

q_{tip} all = 0 kN (for FS = 2.5)

skin friction

$q_s = f_s \cdot S$ where $f_s = \alpha . s_u \;/\; K.\sigma vo'.\tan\delta$
= 1375 kN

q_s all = 688 kN (for FS = 2.0)

total allowable jet grout column capacity

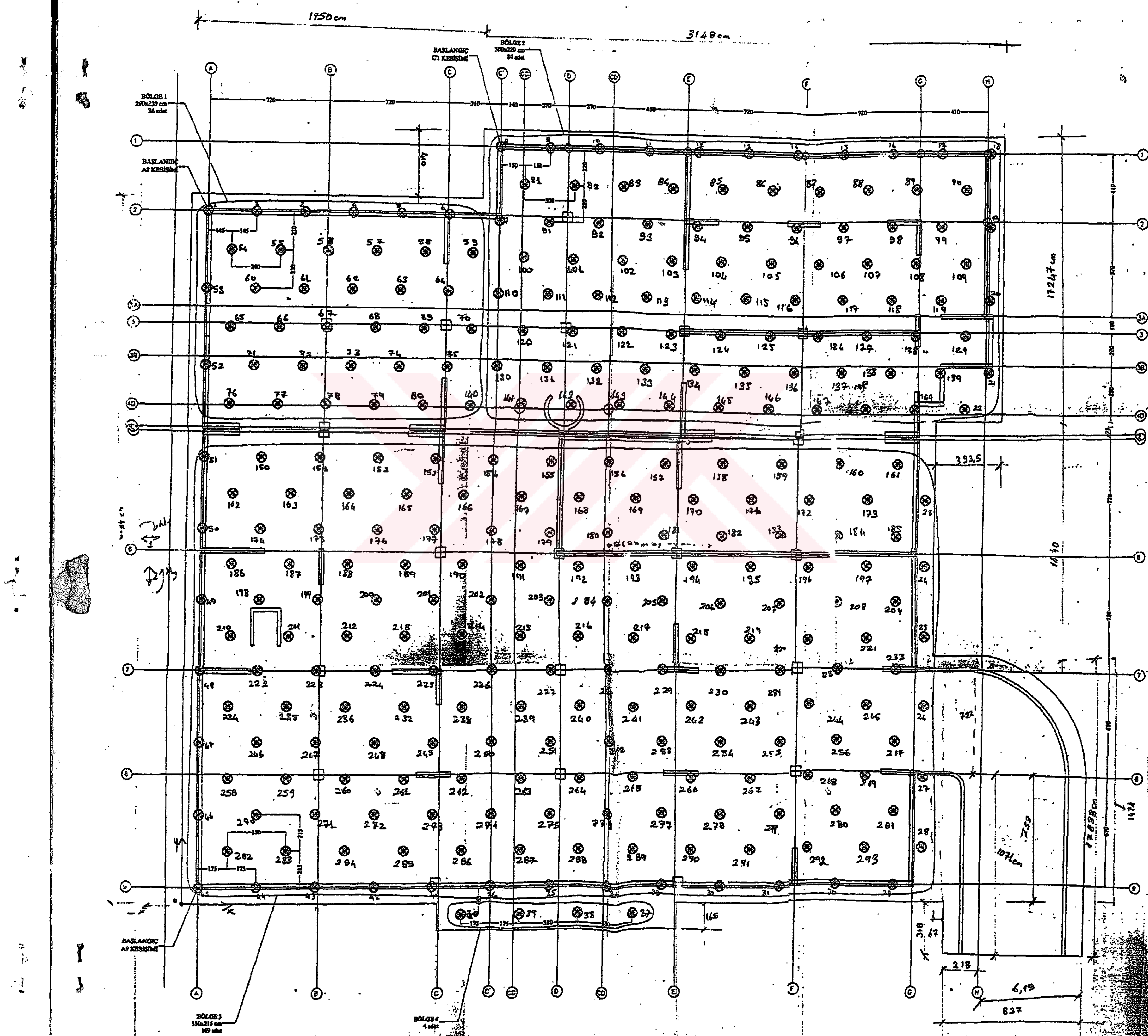
$q_a = q_{tip}$all + $q_s$all = 688 kN ≈ 69 tonnes > maximum capacity check: $\boxed{\text{42 tonnes}}$

q_a = 42 tonnes

† The method is based on Tomlinson (1971) .

Figure B.1 Jet-grout column axial capacity calculation sheet

B-2



1750 cm
3148 cm
BÖLGE 1
290x230 cm
36 adet
BASLANGIÇ
A2 KESİŞİMİ
BASLANGIÇ
C1 KESİŞİMİ
BÖLGE 2
300x220 cm
84 adet
BASLANGIÇ
A9 KESİŞİMİ
BÖLGE 3
350x215 cm
169 adet
BÖLGE 4
4 adet
1724,7 cm
1670
17,898 cm
1071 cm
393,5
6,19
837
218

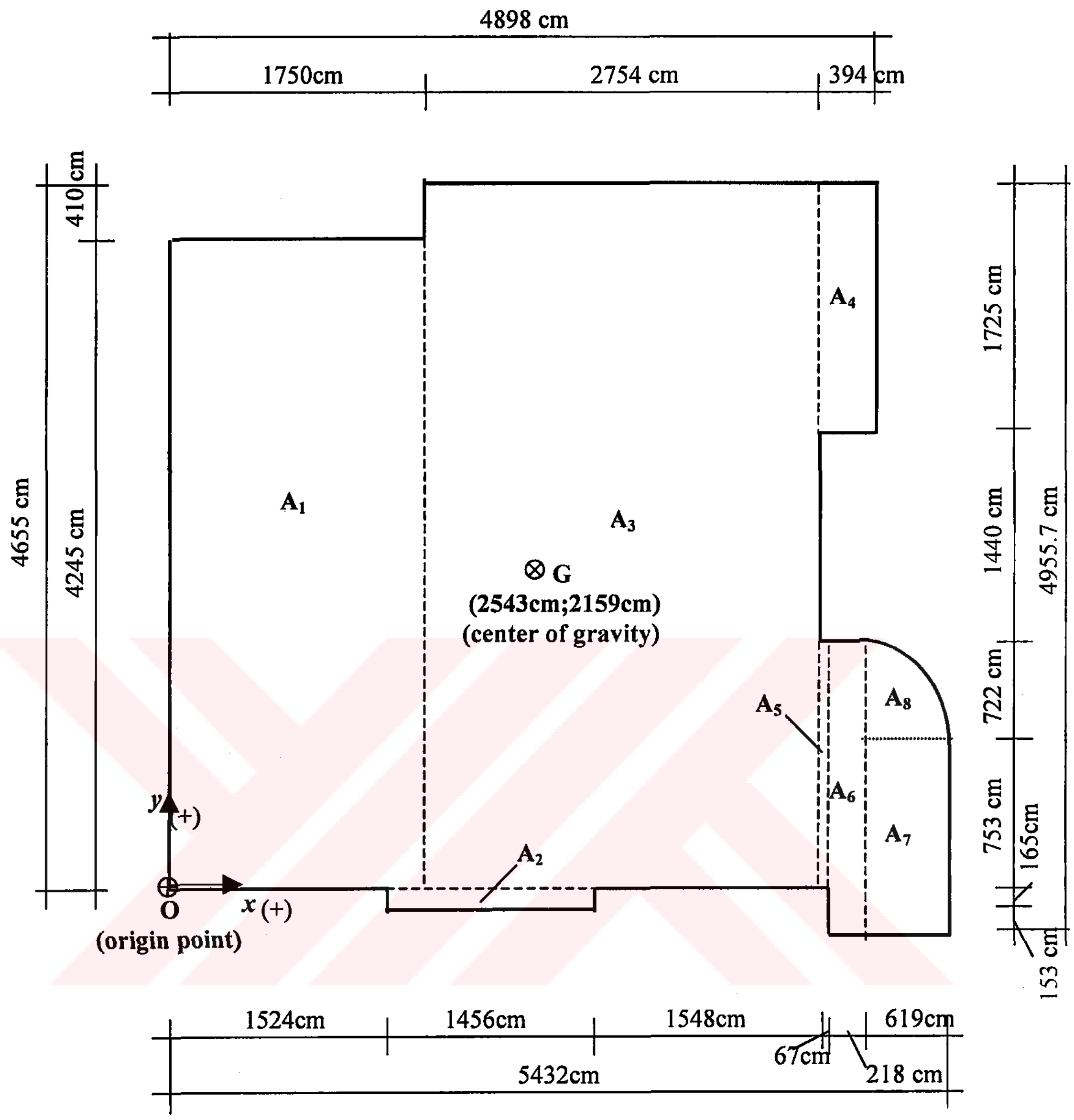


Figure B.2 Calculation of the center of gravity of the foundation area of Bostanlı Vilayet Evi Project

Table B.1 Calculation of the center of gravity

Portion	Area, A (m^2)	Center of gravity coordinates of portion		Area moment with respect to origin	
		$\bar{x}$ (m)	$\bar{y}$ (m)	$\bar{x} \cdot A$ (m^3)	$\bar{y} \cdot A$ (m^3)
A_1	742.87	8.75	21,23	6500,113	15767,42
A_2	24.02	24.78	-1,65	595,3147	-39,64
A_3	1281.99	31.27	23,19	40087,73	29733,77
A_4	67.97	47.01	37,76	3195,035	2566,60
A_5	9.88	45.38	7,37	448,1144	72,78
A_6	38.87	46.8	5,78	1819,116	224,67
A_7	66.29	51.00	2,18	3380,459	144,18
A_8	29.79	50.21	11,86	1495,756	353,31
Total ($\sum$)	2261.67			57521,64	48823,08

Calculation of coordinates of center of the gravity :

$$\bar{x}_G = \frac{\sum \bar{x} \cdot A}{\sum A} = \frac{57521.64}{2261.67} = 25.43m \qquad \bar{y}_G = \frac{\sum \bar{y} \cdot A}{\sum A} = \frac{48823.08}{2261.67} = 21.59m$$

Table B.2 Calculation of base structural bending moment

Calculation of base structural bending moment	Effective Load for a floor, q_1	Number of floor, N	Total height of the structure, H	Calculated total area of mat foundation, A_{TC}	Calculated Total axial load, $Q_c=q_1 \cdot N \cdot A_{TC}$	Base Shear Force, V_t (code)	Base Structural bending moment, $M_t=2/3 \cdot H \cdot V_t$
	kN/m^2	-	(m)	(m^2)	kN	kN	kNm
	15	3		2261.67	101.745	33,250	243,833

Table B.3 Calculation of jet-grout column axial capacity

Outer column #	Center of gravity coordinates of pile: scaler distance from origin point		$\sum \bar{x}^2$ (m^2)	$\sum y^2$ (m^2)	M (M_x or M_y) kNm	When M acts on x direction $Q_{max,min} = Q/A \pm \frac{M_x}{\sum y^2} y$		When M acts on y direction $Q_{max,min} = Q/A \pm \frac{M_y}{\sum x^2} x$	
	$\bar{x}$ (m)	$\bar{y}$ (m)				Q_{max} (kN)	Q_{min} (kN)	Q_{max} (kN)	Q_{min} (kN)
1	24,61	19,76				404,38	231,77	445,16	190,98
2	21,43	19,76				404,38	231,77	428,74	207,41
3	18,53	19,76				404,38	231,77	413,76	222,38
4	15,63	19,76				404,38	231,77	398,79	237,35
5	12,73	19,76				404,38	231,77	383,82	252,33
6	9,83	19,76				404,38	231,77	368,84	267,30
7	6,93	19,76				404,38	231,77	353,87	282,27
8	6,93	19,76				404,38	231,77	353,87	282,27
9	3,93	23,96				422,72	213,42	338,38	297,76
10	0,93	23,96				422,72	213,42	322,89	313,25
11	1,50	23,96				422,72	213,42	325,82	310,33
12	4,50	23,96	46327.1	54761	243833.3	422,72	213,42	341,31	294,84
13	7,50	23,96				422,72	213,42	356,80	279,35
14	10,50	23,96				422,72	213,42	372,29	263,86
15	13,50	23,96				422,72	213,42	387,78	248,37
16	16,50	23,96				422,72	213,42	403,27	232,88
17	19,50	23,96				422,72	213,42	418,76	217,39
18	22,50	23,96				422,72	213,42	434,25	201,90
19	22,50	19,76				404,38	231,77	434,25	201,90
20	22,50	15,56				386,03	250,11	434,25	201,90
21	22,50	11,36				367,68	268,46	434,25	201,90
22	21,00	9,06				357,64	278,51	426,50	209,64
23	19,00	3,54				333,53	302,62	416,17	219,97
24	19,00	1,25				323,53	312,61	416,17	219,97

25	19,00	5,43				341,79	294,35	416,17	219,97
26	19,00	9,61				360,05	276,09	416,17	219,97
27	19,00	13,79				378,31	257,84	416,17	219,97
28	19,00	17,97				396,57	239,58	416,17	219,97
29	17,73	20,12				405,96	230,19	409,62	226,53
30	14,23	20,12				405,96	230,19	391,55	244,60
31	10,73	20,12				405,96	230,19	373,47	262,67
32	7,23	20,12				405,96	230,19	355,40	280,74
33	3,73	20,12				405,96	230,19	337,33	298,81
34	0,23	20,12				405,96	230,19	319,26	316,88
35	3,27	20,12				405,96	230,19	334,96	301,19
36	6,77	20,12				405,96	230,19	353,03	283,12
37	1,65	22,27				415,35	220,79	326,59	309,55
38	1,86	22,27				415,35	220,79	327,68	308,47
39	5,36	22,27				415,35	220,79	345,75	290,40
40	8,86	22,27				415,35	220,79	363,82	272,32
41	10,61	20,12				405,96	230,19	372,85	263,29
42	14,11	20,12				405,96	230,19	390,93	245,22
43	17,61	20,12				405,96	230,19	409,00	227,15
44	21,11	20,12				405,96	230,19	427,07	209,07
45	24,61	20,12				405,96	230,19	445,14	191,00
46	24,61	18,03				396,83	239,32	445,14	191,00
47	24,61	13,73				378,05	258,10	445,14	191,00
48	24,61	9,43				359,26	276,88	445,14	191,00
49	24,61	5,13				340,48	295,66	445,14	191,00
50	24,61	0,83				321,70	314,45	445,14	191,00
51	24,61	5,13				340,48	295,66	445,14	191,00
52	24,61	10,03				361,88	274,26	445,14	191,00
53	24,61	14,63				381,98	254,17	445,14	191,00

Result : The maximum and minimum values obtained as Qmax: 445.16 kN and Qmin=191 kN. (for outer rows)