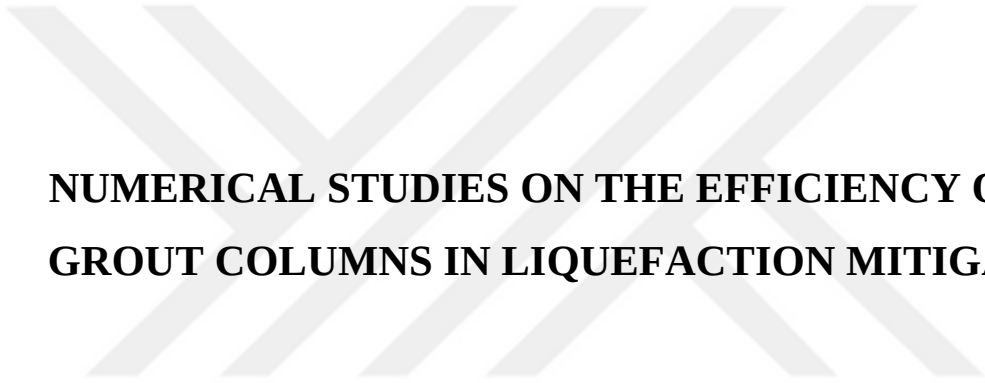


**REPUBLIC OF TURKEY
MUĞLA SITKI KOÇMAN UNIVERSITY
GRADUATE SCHOOL OF
NATURAL AND APPLIED SCIENCES**

DEPARTMENT OF CIVIL ENGINEERING



**NUMERICAL STUDIES ON THE EFFICIENCY OF JET
GROUT COLUMNS IN LIQUEFACTION MITIGATION**

MASTER OF SCIENCE

GİZEM ŞAHİN

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MUĞLA SITKI KOÇMAN UNIVERSITY
Graduate School of Natural and Applied Sciences

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ABSTRACT
**NUMERICAL STUDIES ON THE EFFICIENCY OF JET GROUT
COLUMNS IN LIQUEFACTION MITIGATION**

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Master of Science (M.Sc.)

Graduate School of Natural and Applied Sciences

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Liquefaction, one of the main research topics of geotechnical engineering, is one of the major causes of structural damage caused by dynamic loads. In this study, the contribution of jet grout columns to bearing capacity, settlement problems and liquefaction was tried to be determined by numerical investigations. The results of the ground studies conducted in the field where the Transformer Assembly-Dismantling Building is located in Kocaeli were evaluated. The local soil consists of silty sand. The bearing capacity, settlement and liquefaction potential were evaluated using different methods based on the data of the drillings, the index properties of the research pits and soils. The liquefaction risk of the ground in the work isn't the acceptable value. In the study area where bearing capacity, settlement and liquefaction problems were determined according to the determined soil properties, it was investigated by analytical and numerical methods whether the problems in question disappeared after the production of the jet grout columns selected as an improvement method. To investigate the effectiveness of jet grout columns in preventing liquefaction through a field study, MIDAS GTS NX performed numerical analyses using the UBCSAND constitutive model. One of the most important features of the model is that it is an elastoplastic model that can examine the pore water pressure depending on time. The shear stresses, shear deformations, and changes in excess pore pressure ratios in the liquefied soil during the earthquake were examined by considering different column strength ratios and area ratios. The results were evaluated and interpreted. As a result, it has been shown that although jet-grout columns are more rigid and subjected to greater shear stress, they do not significantly reduce shear deformations and excess pore water pressures in liquefiable soil.

Keywords: Jet Grout Column, Bearing Capacity, Settlement, Liquefaction, Finite Element Model, UBCSAND Constitutive Model.

ÖZET
JET-GROUT KOLONLARININ SIVILAŞMA AZALTIMINDA ETKİNLİĞİ
ÜZERİNE SAYISAL ÇALIŞMALAR

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Geoteknik mühendisliğinin ana araştırma konularından biri olan sıvılaşma, dinamik yüklerin neden olduğu yapısal hasarların önemli nedenlerinden biridir. Bu çalışmada, jet grout kolonlarının taşıma ve oturma problemleri ile sıvılaşmaya karşı katkısı, sayısal araştırmalar ile belirlenmeye çalışılmıştır. Kocaeli İli Alstom Grid Enerji Endüstrisi A.Ş. bölgesinde bulunan Trafo Montaj-Demontaj binasının bulunduğu sahada yapılan zemin etüd çalışmalarının sonuçları değerlendirilmiştir. Zemin siltli kumdan oluşmaktadır. Sondajlara ait veriler, araştırma çukurları ve zeminlerin indeks özellikleri temel alınarak, taşıma kapasitesi, oturma ve sıvılaşma potansiyeli farklı yöntemlerle değerlendirilmiştir. Çalışma alanındaki zeminin sıvılaşmaya karşı güvenlik katsayısı kabul edilebilir değer altındadır. Belirlenen zemin özelliklerine göre taşıma kapasitesi, oturma, sıvılaşma problemleri tespit edilen çalışma alanında, iyileştirme yöntemi olarak seçilen jet grout kolonlarının imalatından sonrası için söz konusu problemlerin ortadan kalkıp kalkmadığı analitik ve nümerik yöntemlerle araştırılmıştır. Bir saha çalışması yoluyla sıvılaşmayı önlemede jet grout kolonlarının etkinliğini araştırmak için MIDAS GTS NX, UBCSAND bünye modeli kullanılarak nümerik analizler gerçekleştirmiştir. Modelin en önemli özelliklerinden biri de zamana bağlı boşluk suyu basıncı da inceleyebilen elasto plastik bir model olmasıdır. Farklı kolon dayanım oranları ve alan oranları dikkate alınarak deprem sırasında sıvılaşan zeminde oluşan kayma gerilmeleri, kayma deformasyonları ve aşırı boşluk suyu basıncı oranları değişimleri incelenmiş, sonuçlar değerlendirilerek yorumlanmıştır. Sonuç olarak, jet-grout kolonlarının daha rijit olmasına ve daha fazla kayma gerilmesine maruz kalmasına rağmen, sıvılaşabilir zeminde kayma deformasyonlarını ve aşırı boşluk suyu basınçlarını, önemli ölçüde azaltmadığını göstermiştir.

Anahtar Kelimeler: Jet Grout Kolon, Taşıma Kapasitesi, Oturma, Sıvılaşma, Sonlu Elemanlar Modeli, UBCSAND Bünye Modeli.



To my family,

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LIST OF SYMBOLS AND ABBREVIATIONS

ν	Poisson Ratio
g	Acceleration of Gravity
w	Water Content
ϕ	Internal Friction Angle
c	Cohesion
E_s	Elasticity Modulus
P_a	Atmosphere Pressure
U_c	Coefficient of Uniformity
D_r	Relative Density
V_s	Shear Wave Velocity
V_p	Primary (Compressional) Wave Velocity
τ_{cyc}	Uniform Cyclic Shear Stress Amplitude of the Earthquake
σ_{vo}	Total Vertical Stress
σ'_{vo}	Vertical Effective Stress
a_{max}	Maximum Ground Acceleration
r_d	Stress Reduction Factor
z	Depth of the Ground Level
N	Number of SPT blow counts
C_N	Correction Factor of According to Vertical Geological Stress
C_R	Correction Factor of Stem Bar Length
C_S	Correction Factor of According to the Method of Sampling
C_B	Correction Factor of Diameter of the Drilling Pit
C_E	Correction Factor of the Energy Rate of the Rammer
$N_{1,60}$	Corrected SPT Blow Count
M_W	Moment Magnitude of the Earthquake
q_{c1N}	Corrected CPT Tip Resistance
G_s	Shear Modulus
C_M	Moment Magnitude Correction Coefficient
τ_R	Liquefaction Resistance
$\tau_{earthquake}$	Average Repetitive Shear Stress in Soil Caused by an Earthquake

σ'_{v0}	Effective Stress
σ_{v0}	Total Stress
S_{DS}	Short Period Design Spectral Acceleration Value
σ'_v	Effective Vertical Stress
$\varepsilon_v\%$	Post Liquefaction Volumetric Strain
u_e	Excess Pore Pressure
r_u	Excess Pore Pressure Ratio
r'_{3c}	Consolidation Stress (for Triaxial Pressure Test)
r'_{vc}	Consolidation Stress (for Free Soil)
k_G^e	Elastic Shear Modulus Number
k_G^p	Plastic Shear Modulus Number
n_e	Elastic Shear Modulus Exponent
n_p	Plastic Shear Modulus Exponent
Φ_{cv}	Constant Volume Friction Angle
Φ_p	Peak Friction Angle
R_f	Failure Ratio
f_{dens}	Soil Densification Calibration
f_{EPOST}	Post Liquefaction Calibration
Δt	Time Interval
σ_c	Unconfined Compressive Strength
σ_t	Tensile Strength
t_0	28-Day Shear Strength
q_u	Compressive Strength
B	Diameter
S	Distance
K_G	Shear Stress Ratio
t_s	Ratio of Redunreduced stress,
t	Unreduced Shear Stress
G_r	Shear Modulus Ratio
A_r	Displacement Ratio
A	Foundation Area
GWT	Groundwater Level
FC	Fine Content
SM	Non-plastic Silty Sand

SPT	Standard Penetration Test
CPT	Cone Penetration Tests
DMT	Dilatometer Index
CSR	Cyclic Shear Stress Ratio
CRR	Cyclic Resistance Ratio
FS	Safety Coefficient
LL	Liquid Limit
PI	Plasticity Index
MSF	Magnitude Scaling Parameter
PGA	Peak Ground Acceleration
ML	Local Size
UCS	28 Days Free Compressive Strength

1. INTRODUCTION

1.1. General Overview

The energy released during earthquakes can cause many problems. Soil improvement methods are used to increase the ground strength, especially on soils that are sensitive to problems such as bearing capacity, settlement, liquefaction originating from earthquake-related problems. Jet grout columns, also known as high modulus columns as a soil improvement method, are widely preferred as bearing capacity increasing, settlement reducing and liquefaction prevention methods (Durgunoğlu, 2004).

Fact that this method is widely used today, the reliability of the jet grout method has made it necessary to compare and examine the field tests during and after the manufacturing phase with various methods and methods. Jet grout columns have become a general engineering practice that is used as a preventive measure for such problems, although their efficiency has not been sufficiently examined by case analysis or numerical studies.

In this project, the contribution of jet grout columns against transport and settlement problems and liquefaction was tried to be determined with a case study. Alstom Grid Energy Industry Inc. The field studies report carried out in the project area received from the authorized person was examined in detail, and using the soil parameters determined as a result of the field tests, the bearing capacity, settlement and liquefaction were analyzed. The soil improvement project has been designed using jet grout columns for the site where bearing capacity, settlement and liquefaction problems are detected. After the production of the designed jet grout columns, the site improved soil parameters were determined by analytical methods, and the post-improvement bearing capacity, settlement and liquefaction calculations were repeated using these parameters. In addition, numerical liquefaction analysis was performed with two-dimensional finite element programs. The bearing capacity in

the subject, whether settlement and liquefaction problems have been eliminated has been determined by analytical and numerical analysis.

1.2. Objective of the Study

The results have been evaluated of the ground surveys carried out in the area where the Transformer Assembly-Dismantling Building is located in the Kocaeli Province Alstom Grid Energy Industry Inc. Bearing capacity, settlement and liquefaction problems were determined in the study area. It was investigated by analytical and MIDAS GTS NX performed numerical analyzes using the UBCSAND constituent model whether these problems disappear after the production of the jet grout columns selected as an improvement method. The shear stresses, shear deformations, and changes in excess pore pressure ratios in the liquefied soil during the earthquake were examined by considering different column strength ratios and area ratios. The results were evaluated and interpreted.

1.3. Outline of the Thesis

This study is organized into five chapters. A summary of the thesis structure is given as below:

- Chapter 1 gives brief information about the liquefaction of the ground and the jet grout method, one of the ground improvement methods. The existing gaps and needs for the present study are described. The objectives and scope of work are presented briefly.
- Chapter 2 provides a detailed literature review. The liquefaction mechanism, what are the factors affecting liquefaction and how the liquefaction risk is explained. Constitutive models used in liquefaction analysis and mitigation measures are explained.
- Chapter 3 details site properties and seismicity of the region in the site explained. Liquefaction Risk by using analytical method and MIDAS GTS-NX evaluated. In addition, other founding models are also mentioned and

analyses made with the UBCSAND model without soil improvement are included.

- In Chapter 4, findings Jet grout properties as a soil improvement method and findings obtained jet grout columns study are presented. Using the MIDAS GTS NX UBCSAND model, Jet grout analyzes were repeated statically and dynamically for each different spacing between adjacent columns. Jet grout columns in reducing the liquefaction potential in the field, numerical analyzes (nonlinear stress-time analysis) were made for different strength and area ratios. These parametric studies were evaluated in terms of shear stress-shear deformation distribution and the effect of Jet-Grout columns on the formation of excess pore water pressure.
- Chapter 5 presents the summary and conclusion of the dissertation. Concluding jet grout columns performance of in liquefiable soil are outlined. Recommendations for further investigations are provided.

2. LITERATURE REVIEW

Liquefaction can be defined as its transformation into a liquid state by losing its effective stress for a certain period as a result of the increase in pore water pressure caused by repeated shear stresses in undrained loading conditions in water-saturated cohesionless soils (Mogami and Kubo, 1953; Terzaghi, 1956).

Liquefaction and lateral flow behaviours occur on loose sandy soils saturated with water, which are the most important factor that causes loss of bearing capacity on the ground during earthquakes, are among the important research topics in recent years. First, liquefaction and related structural damages were observed as a result of the 1906 San Francisco earthquake (Youd and Hoose, 1976). However, it was started to be investigated due to the problem that occurred on the soil due to the effect of two big earthquakes (Alaska, 1964 and Niigata, 1964) in the 1960s, and research on the subject has continued since then. Two examples in which the detrimental effects of liquefaction have been observed in recent history are the 1995 Kobe and 1999 Gölcük earthquakes. The liquefaction caused by the Gölcük earthquake has affected a significant portion of approximately 55,000 buildings with mild, moderate and severe intensity.

By the reason of these negativities, several effective prevention strategies have been developed to reduce the risk of liquefaction, including building safety (Idriss and Boulanger, 2008). The choice of liquefaction prevention method is directly related to risk management or risk probability assessment. Mitchell (2008) categorized the improvement methods according to their applicability in different particle sizes. The effectiveness of the soil improvement method depends on the grain size distribution. Depending on the grain size, vibration methods, deep dynamic compaction, compression grouting, soil mixing, drainage and jet grouting are the most common examples of soil improvement methods.

According to Moseley and Kirch (2004), jet injection is a multi-purpose soil improvement method that involves both mixing and partial displacement of the soil

with cement. In terms of liquefaction prevention, the effectiveness of jet injection has been studied with many studies based on various case analyses (Hausler and Sitar, 2001; Martin and et al., 2004; Spagnoli, 2008). Apart from these case studies, Adalier et al. (2003) determined that stone columns prevent liquefaction in non-plastic silty sands with their centrifuge experiments. Similar to the stone column approach, Özsoy and Durgunoğlu (2003) showed that the effectiveness of jet grout columns is theoretically dependent on the ratio of jet grout column shear modulus and soil shear modulus. In addition to these studies, the contribution of jet grout columns in liquefaction prevention was confirmed by a few numerical analyzes. While some of these analyses are performed using the simplified approach, which is also named the total stress approach (Martin & Olgun, 2007; Olgun & Martin, 2008; Unutmaz, 2012; Callisto et al., 2013; Cristovao et al., 2015), some are based on the effective stress approach depending on the pore water pressure ratio (Almani et al. 2012; Almani et al., 2013; Keepa et al., 2014). According to the aforementioned studies, it has been determined that the effectiveness of jet grout columns in preventing liquefaction will increase due to the difference between jet grout and soil shear strength.

Liquefaction and related uncertainties have not yet been resolved, despite the serious progress achieved through many studies (Hausler & Sitar, 2001). Studies are needed to more clearly determine the effect of soil behaviour and liquefaction mechanism on increasing the liquefaction resistance of jet grout columns. For this reason, besides case analyses, numerical analyzes using finite element and finite difference methods should be preferred mostly (Unutmaz, 2012). To determine the behaviour of soils during liquefaction, the numerical model to be developed should define the earthquake movement that will cause the movement of the soil, the response of the soil to this effect and the internal effects that will occur in the soil during this process (pore water pressure increase, distribution and damping) in a way that best suits the conditions in the site. For this reason, a numerical solution created using the UBCSAND constitutive model in case of insufficient site data is very important, especially for situations that require calculating more than one size such as liquefaction analysis, and post-earthquake observations are necessary to obtain site data.

This chapter aims to give detailed information about liquefaction. Firstly, the liquefaction phenomenon and its mechanism during an earthquake is explained in general and exemplified by cases in Turkey and worldwide. Then, determination of liquefaction risk using the simplified analytical method, laboratory tests and other numerical analyses. Finally, mitigation measures against liquefaction are discussed with both numerical and field studies in the literature.

2.1. Overview of Liquefaction

Seismic risks can be classified as tsunamis, landslides, ground shaking, liquefaction, morphological risk, retaining structure failures (Muduli, 2013). Liquefaction of soil is an essential creator of both losses of life and destruction of structures and infrastructure systems. Liquefaction is one of the most quirky, complicated, important issues discussed in the geotechnical earthquake engineering branch (Kramer, 1996). In general expression, liquefaction is the loss of strength of fully saturated, cohesionless soils due to an increase in pore water pressure during an earthquake (Rauch, 1997).

Firstly, Casagrande (1936) was able to show the behaviour of saturated sand under undrained conditions without using the liquefaction term. It is the first study that shows the occurrence of quick strength loss of fully saturated loose sand deposits flow like viscous fluid even under not fully saturated (Terzaghi and Peck, 1948). There are many definitions and studies about liquefaction. The term liquefaction can be expressed as a partially or completely loss of shear strength in saturated soils due to high pore water pressure generation during sudden, strong ground motions such as an earthquake (Seed and Lee, 1966). During liquefaction, the soil may behave similar to a viscous fluid, so that they may flow significant distances. The resulting settlement may cause destruction of structures conducting towards both casualties and economical loss.

The pore water pressure increases due to cyclic shear stress induced by the spread of shear waves (Ishihara, 1985). The earthquake occurs suddenly in a very short time interval and pore water pressure increases continuously during the earthquake. For cohesionless soils in undrained conditions, water inside the soil matrix tries to move

due to high pore water pressure and flows to the ground surface. Contact between soil particles disappears due to the movement of liquid particles (water). This event results that effective stress becomes zero and soil losing its capability of bearing shear load. The phenomenon is called liquefaction.

Liquefaction, one of the most dangerous geotechnical problems, can be seen during and/or after the earthquake and the results are highly damaging (Tonaroğlu, 2006). The effects of liquefaction were firstly observed in the San Francisco earthquake in the 1900s. Thereafter, liquefaction was observed in Loma Prieta (1989), Los Angeles Northridge (1994) and Kobe (1995), Kocaeli (1999) earthquakes. In the 1906 San Fransisco earthquake, several bridges, roadways, pipelines and buildings were damaged ground failures associated with liquefaction (Youd and Hoose, 1976). In the 1995 Kobe earthquake, Port of Kobe, ranked 6th among container ports all around the world, was heavily damaged and shut down for 2 years. The economic extend of the event cause loss of more than US\$ 100 billion (Chang, 2000). Daniell et al. (2012) reported that 1906 San Fransisco, 1989 Loma Prieta, 1994 Northridge, 1995 Kobe and 1999 Kocaeli earthquakes caused approximately more than 6bn. US\$, 2.5bn. US\$, 23.5bn. US\$, 6.5bn. US\$, 3.5bn. US\$, respectively. The economic loss along with the loss of many lives may externalize the effects of liquefaction after earthquakes.

The first studies on liquefaction were reported after the Niigata (Japan, 1964) and Alaska (USA, 1964) earthquakes in the 1960s (Anonymous, 1997). These two major earthquakes have caused terrible damage. As a result of these events, liquefaction has become important and started to be investigated. The first soil liquefaction observed in Turkey was in the 1992 Erzincan earthquake. After that, there were liquefaction events in Dinar (1995), Adana-Ceyhan (1998), Kocaeli (1999) and Düzce (1999) earthquakes (Orhan et al. 2013). Especially, the 1999 Kocaeli earthquake showed devastating effects of liquefaction after rupturing of the western part of the North Anatolian Fault (Ulusay et al. 2001). In summary, soil liquefaction is a very damaging phenomenon and needs special attention.

2.2. Mechanism of Liquefaction

As aforementioned in Chapter 2.1, liquefaction phenomena may be summarized as a sudden loss of shear strength in soil due to a decrease in effective stress. Pore water pressure increases under repetitive stresses (dynamic loadings such as an earthquake) for saturated, cohesionless sand or sandy silt. Knowing effective stress is mainly dependent on pore water pressure, shear strength of the soil decreases or even completely lost. Then, soil behaves like a liquid (Youd, 1984). This is the main description of the liquefaction process.

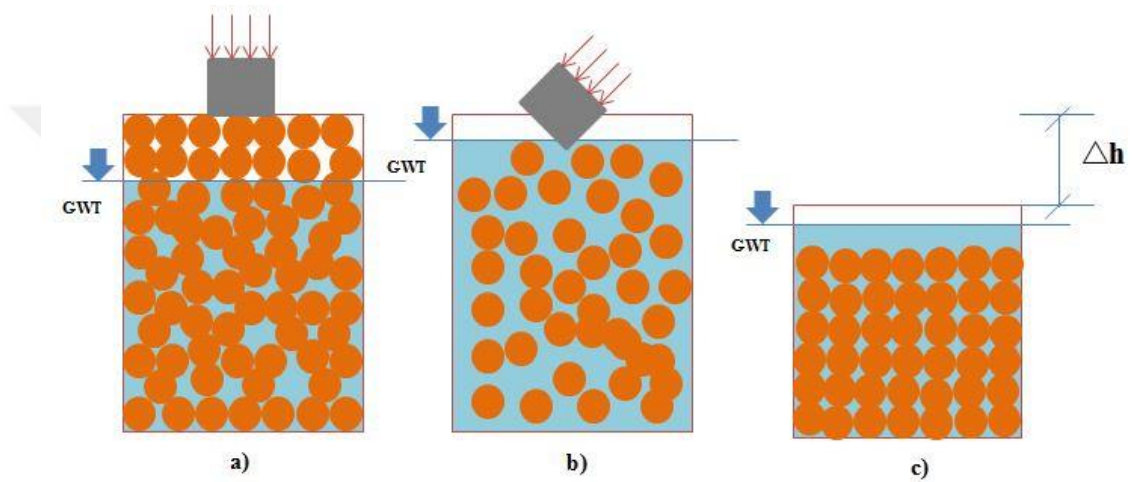


Figure 2.1. Liquefaction process a) before b) during c) after

The phases of the liquefaction process are illustrated in Figure 2.1. Initially, the soil strata below the ground water table have both water and solid particles. The contact between solid particles implies the existence of effective stress. In the case of liquefaction, the loading increases rapidly and causes excess pore pressure. These results decrease in contact between solid particles and decrease in effective stress. After the liquefaction process is completed, the soil becomes in a denser form as shown in Figure 2.1.c.

Soil liquefaction can be divided into two categories as flow liquefaction and cyclic liquefaction (Kramer, 1996). Cyclic mobility occurs more frequently, but the effects of flow liquefaction are more destructive. The mechanism and the characteristics of flow liquefaction and cyclic mobility are discussed in Chapter 2.2.1 and Chapter 2.2.2 in detail.

2.2.1. Flow liquefaction

Flow liquefaction can take place when the shear stress necessary for static equilibrium of a soil mass (the static shear stress) is greater than the shear strength of the soil in its liquefied condition (Kramer, 1996). Once large deformations are generated by flow liquefaction, the soil mass is driven by static shear stresses. In cases where static stresses as produce as flow failure reduce the ground strength, cyclic stresses can easily make the ground unstable. Static loading, for instance, can be affected by new constructions on a slope that applies additional forces on the soil beneath the foundations (Johansson, 2000). Pile driving, blasting, seismic waves etc. are all cases of dynamic loads that could occur flow liquefaction. After triggered, the strength of a soil precision to flow liquefaction is no more sufficient to resist the static stresses that act on the soil before the disturbance. Flow liquefaction is characterized by sudden occurrence, rapid development and generally, movement of liquefied material at great distances (Kramer, 1996). For example, a sudden decrease in bearing capacity caused 80° tilting of Kawagishi-cho apartments due to liquefaction in the 1964 Niigata Earthquake (Figure 2.2.) having a magnitude of 7.5 (Liu, 2009).

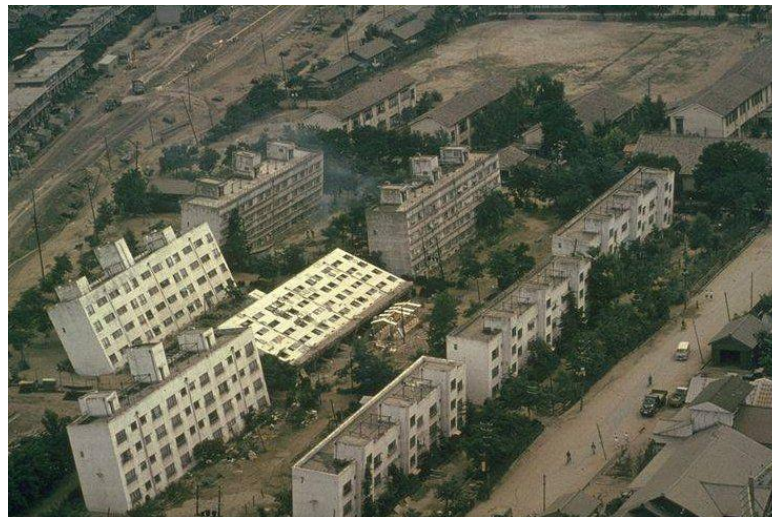


Figure 2.2. Tilting of Kawagishi-cho apartments in 1964 Niigata Earthquake (Liu, 2009)

Flow liquefaction was observed also in Alaska Earthquake on March 27, 1964 (Taylor, 2014). The moment magnitude was 9.2 which was the second most powerful earthquake ever recorded. The initial seismic wave and following underwater

landslides caused a wide range of tsunamis so, coastal towns were damaged to cities of Valdez, Whittier, Seward, and Kodiak (Figure 2.3.).



Figure 2.3. Alaska Earthquake in 1964 (Taylor, 2014)

Valdez is located on the seaward edge of a large outwash delta containing saturated silty sand comprising some gravel (Coulter and Migliaccio, 1966). This liquefaction, its accompaniment waves and other conditions were responsible for the loss of 30 lives and damages to many buildings. The reason was that loose, saturated sand suffered a loss of bearing capacity when subjected to earthquake (Terzaghi et al., 1948). The geologic environment at Valdez provided ideal conditions for flow liquefaction and many similarities to other liquefaction flow sites (Terzaghi, 1956).

2.2.2. Cyclic mobility

Cyclic mobility is another type of liquefaction that causes large and permanent deformations during earthquake shaking (Kramer, 1996). In comparison to flow liquefaction, cyclic mobility occurs when the shear strength of the liquefied soil is more than the static shear stress. The deformations caused by cyclic mobility gradually form during an earthquake shaking. The cause of deformations due to cyclic mobility is the gradually decreasing of the strength due to cyclic stresses. Lateral spreading, a prevalent result of cyclic mobility, can occur on mildly sloping and on flat ground near lakes and rivers (Johansson, 2000). An example of cyclic

mobility is demonstrated in Figure 2.4. 1976 Guatemala Earthquake caused lateral spreading along the Motagua River.



Figure 2.4. Motagua Riverside ground cracks in 1976 Guatemala Earthquake

2.3. Factors Affecting Liquefaction

2.3.1. Effect of source

Earthquakes are required to liquefy soil because the most overall cause of liquefaction is the seismic energy released during the earthquake (Day, 2002). In addition to earthquakes, situations that produce strong vibrations in the soil, such as caused by pile driving, train traffic, and underground explosions can cause liquefaction. However, the most important factor causing liquefaction is the earthquake. Depending on the earthquake, numerous factors direct the liquefaction process of the ground. For example, earthquake duration, magnitude, peak ground acceleration (PGA) and seismic history of this region.

With relation to the effect of the source Seed and Idriss (1971), The susceptibility of the soil in certain conditions to the liquefiable can be explained by depending on the magnitude of an earthquake and with stresses and deformations by earthquake forces.

Ground motion characteristics such as acceleration and vibration time are important factors that determine shear stresses and residual pore water pressure (Day, 2002). The liquefaction potential increases as the the intensity of the earthquake and shake

duration increase. As the earthquake magnitude increases, ground acceleration and earthquake duration increase.

Until the 20th century, the size of earthquake was some vehicles with quantitative measurements were based on qualitative observations before they were discovered (Kramer, 1996).

Acceleration is often used to describe the magnitude of the earthquake. Anonymous (1985) and Ishihara (1985) for the formation of liquefaction peak ground acceleration of 0.1g and local size (ML) is less than 5 so liquefaction analysis may not be required for the site.

2.3.2. Properties of soil

2.3.2.1. Geological process

Generally, Holocene older delta and stream sediments are more liquefiable soils to older sediments (Youd and Perkins 1978). Particularly sensitive to liquefaction are those formed in lacustrine, alluvial and marine deposition soil (Day, 2002). The underwater fillings are liquefiable soil due to the fact that the soil grains are in a loose and decomposed soil structure also the natural soil sediments formed in the lakes, rivers and seas that are prone to form this structure are also tend to liquefaction.

One of the conditions for liquefaction is that soil is saturated. So, groundwater level (GWT) from the surface depth is important for liquefaction (Karaca, 2001). Liquefaction develops rarely at the groundwater level where it is deeper than 10 m from the surface. Generally, it doesn't occur in environments where the groundwater level is deeper than 20 m (Youd, 1984). The liquefaction events haven't been reported at depths exceeding 15 m when the liquefaction events to date have been examined (Derinöz, 2004). However, in critical structures such as the dam, in some cases, it may be necessary to perform a liquefaction analysis for more than 15m deep soils (Wang and Law, 1994).

It also affects the sensitivity of the soil to liquefaction, the intensity and durations of the stresses that it is exposed to during the geological process (Seed, 1976). Under

stresses, the grains of the soil are better subsided and cementing may occur depending on the geological process. This increases the resistance of the soil to liquefaction.

In addition, both laboratory studies and field observations show that in the past, liquefied soil is the bigger possibility to liquefaction with new earthquakes (Ansal and Ogunc, 1981). For this reason, the seismic history of the soil is preliminary information of the danger of liquefaction. The maps produced by the micro-zoning method are quite suitable the way in this regard.

2.3.2.2. Soil type

Soil type for liquefaction is in sediments consisting of fine, medium sands and fine grains with low plasticity sands (Ishihara, 1985). Also, it has been reported that liquefaction occasional occurs in gravelly soils. Youd et al. (1985) were able to indicate sandy soils containing significant amounts of gravel (~ 50%) can be liquefied. If water-saturated gravelly soil levels are overlapped by less permeable levels, an increase in pore water pressure isn't able to absorb and the soil can liquefy (Aslan, 2010). The best example of this is gravelly ground that liquefied during the Alaska earthquake of 1964 (Obermeier, 1996). In addition, non-plastic silt has also been reported to be liquefiable (Dobry and Alvarez, 1967). Generally, agreeable that there is no liquefaction on clayey soil (Ozaydin, 2007). That means the increased clay content is a factor that reduces the risk of liquefaction on non-cohesive soils (Kramer, 1996). Consequently, it can be said that the types of soils prone to liquefaction are generally cohesionless soils.

The fact that the fine-grained material is silty or clay and more importantly shows plastic or non-plastic properties, has been accepted by many researchers the fact that there are consistent differences in ground cyclic strength (Polito, 1999). In the majority of the studies conducted on this subject, it is reported that the liquefaction strength of the soil is increased if there is a plastic thin grain on the soil. The widely used method for the liquefaction of low plasticity, fine-grained soils is the modified Chinese method four criteria (Wang, 1979);

Ratio finer than 0,005 mm $\leq$ %15

Liquid limit, $LL \leq \%35$

Natural water content $\geq 0.9LL$

Liquidity index ≤ 0.75

Andrews and Martin (2000) modified the Chinese criterion taking consider the 0.002 mm value for fine-grain content (Table 2.1). Kocaeli earthquake, the test results obtained from Adapazarı indicate that the liquefied soils are not included in the Chinese criteria and according to Andrews and Martin (2000) are not considered to be liquefaction (Dülger, 2015). Based on this data Seed et al. (2003) defined new relationships to determine the liquefaction performance of fine-grained soils (Figure 2.5).

Table 2.1. Liquefaction criteria of non-plastic fine-grained soils (Andrews and Martin, 2000)

	Liquid Limit (LL) $< \%32$	Liquid Limit (LL) $> \%32$
Content of Clay $< \%10$	Liquifiable soil	Need research
Content of Clay $> \%10$	Need research	Not liquifiable soil

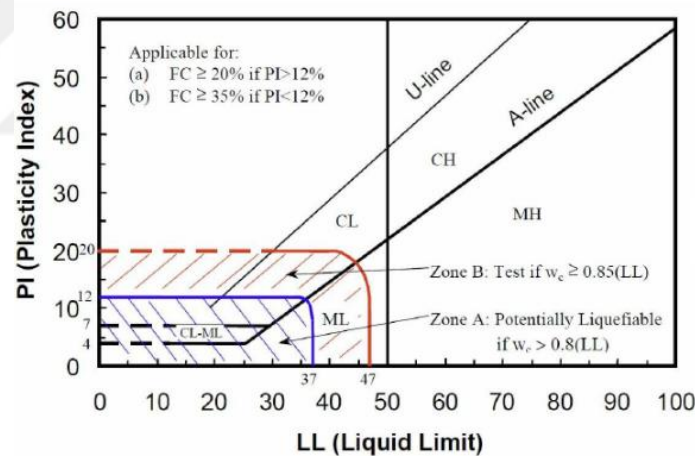


Figure 2.5. Liquefaction susceptibility criteria proposed by Seed et al. (2003)

In addition, these properties, grain size distribution, grain shape and relative density are also important in the interpretation of liquefaction. Nonuniformly status, cohesionless soils are more susceptible to liquefaction as they tend to form more unstable grain patterns than well-graded soils. The grain size distribution ranges of the well-liquefied soils are given in Figure 2.6.

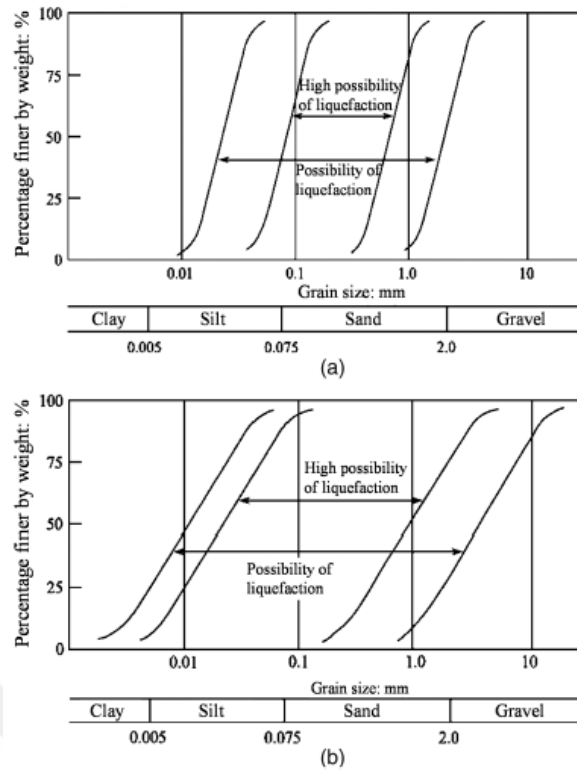


Figure 2.6. (a) soil with a low coefficient of uniformity $U_c < 3.5$; (b) soil with a high coefficient of uniformity $U_c \geq 3.5$. Grain size distribution curves with the ranges of the possibility of liquefaction marked (Anonymous, 2007)

In non-cohesive loose soils, shrinkage occurs at the level of residual cavity pressure during the earthquake shake (Day, 2002). When initial liquefaction occurs, a sudden and striking increase in slip displacement occurs for loose sands. More dense grained soil compare to low density grained soil is higher resistance to liquefaction so relative density (D_r) increases resistance to liquefaction increases (Aslan, 2010). Silver and Seed (1971) was determined that the sands having a relative density (D_r) of less than 60 percent showed a much more volumetric change. Then, Tezcan and Teri (1996) said If relative density (D_r) is less than 47% soils are in a looser position and they would be more susceptible to liquefaction.

The round grained soils tend to be more easily together than the soil containing angular grains so their liquefaction potential is higher (Kramer, 1996). Round grained soils are observed in stream environments where loose saturated soils deposit.

2.4 Determination of Liquefaction Risk

Liquefaction risk analysis required performed by a possible earthquake prediction in regions with high liquefaction potential (Dülger, 2015). The first stage of soil liquefaction calculations is to determine the probability of liquefaction risk using numerical methods.

Liquefaction can be investigated in the laboratory or site, the natural conditions of the soil (Öz, 2015). It is possible to determine the stress-strain behaviour of soils during and after the earthquake by various experimental systems in the laboratory. Particularly, the water-saturated sandy soils performance under repetitive loads and after their liquefaction behaviour can be analysed by dynamic test systems such as dynamic simple shear, dynamic triaxial, dynamic torsion shaking table test in the laboratory. Laboratory tests are difficult to taking a sample and occurring disturbance as a result of a consolidation of the sample due to stresses in the field (Dülger, 2015).

In field tests, a continuous definition can be made at the desired depth in a larger volume in the natural environment of the ground, but it is difficult to show seismic activity in the field in the experiments so, in experiments conducted to measure liquefaction potential, the soil parameters directly associated with liquefaction resistance cannot be reached (Öz, 2015). Four different site test methods are used to determine the liquefaction potential. These tests are; measurement of land shear wave velocity (V_s) standard penetration tests (SPT), Becker penetration tests and cone penetration tests (CPT) (Dülger, 2015).

2.4.1. Simplified method

Seed and Idriss (1971) proposed a method called simplified procedure which evaluates the liquefaction resistance of soils. This method was later developed and finally in January 1996; Youd and Idriss combined the results of the last 10 years and the simplified procedure 20 experts and finalized them (Youd and Idriss, 2001). Simplified method field, data from a laboratory experiment, site observations and site experiments consist of empirical equations using the data. Caused by the earthquake

and require for liquefaction is cyclic shear stress ratio (CSR) calculates with the following Eq. 2.1 for an earthquake of 7,5.

$$CSR = \tau_{cyc} / \sigma'_{vo} = 0,65 \times a_{max} / g \times \sigma_{vo} / \sigma'_{vo} \times r_d \quad (2.1)$$

τ_{cyc} = Uniform cyclic shear stress amplitude of the earthquake (lb/ft² or kPa)

σ_{vo} = Total vertical stress (lb/ft² or kPa)

σ'_{vo} = Vertical effective stress (lb/ft² or kPa)

a_{max} = Maximum ground acceleration (ft/s² or m/s²)

g = Acceleration of gravity (32.2 ft/s² or 9.81 m/s²)

r_d = Stress reduction factor (dimensionless)

The stress reduction factor (r_d) is depending on the depth of the ground level (z) being examined and indicates the flexibility of the ground profile. For general applications or projects the average value of r_d is given below (Eq. 2.2, 2.3) (Liao and Whitman, 1986).

$$r_d = 1,0 - 0,00765z \quad z \leq 9,15m \quad (2.2)$$

$$r_d = 1,174 - 0,0267z \quad 9,15 \leq z \leq 23m \quad (2.3)$$

z = Depth of ground level (meter)

The second parameter used in the simplified method is the cyclic resistance ratio (CRR), For calculate CRR, Standard Penetration Test (SPT) blow count can be used by making corrections (Eq. 2.4). SPT, shear wave velocity (V_s) and CPT measurements were used to investigate the liquefaction areas by many researchers. Depending on the correction factors based on the Standard Penetration Test (SPT), the following calculation method is performed.

$$N_{1,60} = N C_N C_R C_S C_B C_E \quad (2.4)$$

N = Number of SPT blow counts

C_N = Correction factor of according to vertical geological stress

C_R = Correction factor of stem bar length

C_S = Correction factor of according to the method of sampling

C_B = Correction factor of the diameter of the drilling pit

C_E = Correction factor of the energy rate of the rammer

C_N value, should not be less than 0.5 and not exceed 1.7 (Liao and Whitman, 1986) (Eq. 2.5).

$$C_N = \sqrt[3]{(P_a / \sigma'_{v0})} \quad (2.5)$$

$P_a \cong 100\text{kPa}$ (Atmosphere pressure)

Table 2.2. Correction factors used to determine $N_{1,60}$ (Robertson and Wride 1998; Youd et al. 2001)

Correction Factors	System	Correction
C_N	-	$(P_a / \sigma'_{v0})^{0.5} ** C_N \leq 2$
C_E	Safety pile driver *Donut pile driver	0.60-1.17 *0.45-1.00
C_B	65-115 mm 150 mm 200 mm	1.00 1.05 1.15
C_R	3-4 m 4-6 m 6-10 m 10-30 m >30 m,	0.75 0.85 0.95 1.0 <1.0
C_S	Standard lined	1.0 1.15-1.30

* The equipment used and its value in Turkey

** Liao and Whitman (1986)

When the soil contains fine grain, correction to the N value should be made to allow for the effect of these particles.

$$(N_{1,60})_{cs} = \alpha + \beta N_{1,60} \quad (\text{Youd and Idriss, 1997}) \quad (2.6)$$

Percentage of fine grain $\leq 5\%$ for $\alpha=0$ and $\beta=1.0$

$5\% < \text{Percentage of fine grain} < 35\%$ for $\alpha=e^{(1.76-(190/\text{fine grain}^2))}$

$$\beta = (0.99 - (\text{fine grain}^{1.5}/1000))$$

Percentage of fine-grain $\geq 35\%$ for $\alpha=5.0$ and $\beta=1.2$

$(N_{1,60})_{cs}$ is used in Figure 2.7 to find the CRR under magnitude 7.5 earthquakes

$N_{1,60}$ = Corrected SPT blow count

α and β = Coefficients determined from the above relationships

Calculated a value is used to find the seismic resistance ($CRR_{7.5}$) by the following chart (Figure 2.7).

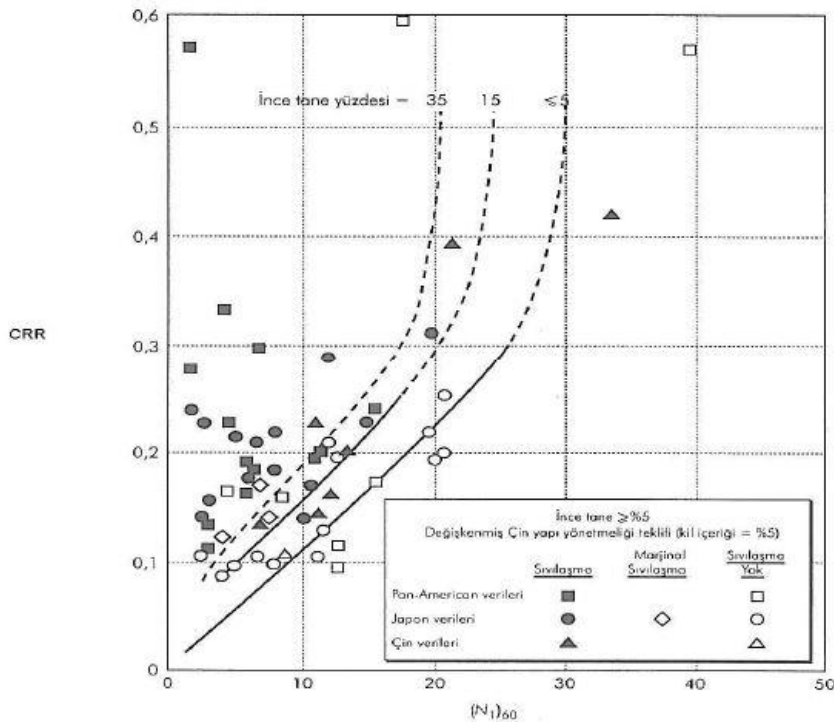


Figure 2.7. Relationship with liquefaction resistance-standard penetration pulse count (Seed et al. 1975)

The seismic resistance ratio found in the previous step is a value that should be taken for an earthquake with a moment magnitude of 7.5. The seismic resistance ratio must be corrected using the magnitude scaling parameter (MSF) for the possible earthquake magnitude. Youd et al. (2001) has made an offer for this correction (Eq. 2.7).

$$MSF = 10^{2.24/M_W - 2.56} \quad (2.7)$$

M_W = Moment Magnitude of the Earthquake

The number of safety against liquefaction is calculated (Eq. 2.8).

$$FS = (CRR_{7.5}/CSR) \times MSF \quad (2.8)$$

It is agreeable that, liquefaction will occur for values less than and equal to 1 of the safety coefficient (FS) against liquefaction and it will not occur for values greater than 1 (Seed and Idriss 1982).

Another method is the calculation method based on Cone Penetration Test (CPT). Figure 2.8 includes collected liquefaction data from the events of history. It shows CPT clean sand base curve and separates regions into liquefaction and non-liquefaction regions. In addition, the curve recommended calculation of CRR from CPT's corrected tip resistance (q_{c1N}).

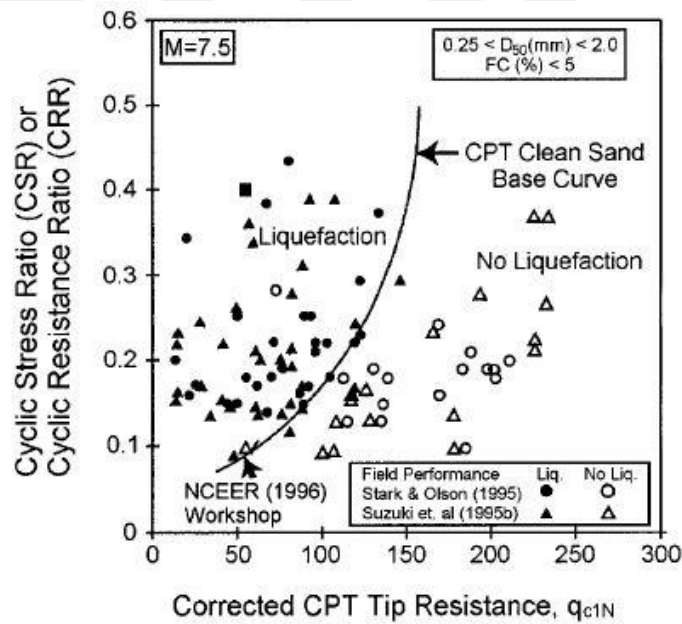


Figure 2.8. Collected liquefaction data from the events of history and curve recommended calculation of CRR from CPT's corrected tip resistance (q_{c1N}) (Robertson and Wride 1998)

2.4.2. Experimental studies

We use various field and laboratory experiments to examine the liquefaction potential of soil layers. Experiments conducted to investigate the liquefaction potential of soils in the laboratory;

- Dynamic triaxial pressure tests
- Dynamic simple shear experiments
- Torsional shear experiments
- Laboratory model experiments (shake table experiments, centrifuge experiments) (Özaydın, 2007).

Even if the disturbance effect is considered negligible when using laboratory test methods, laboratory test results may not reflect the properties of the entire layer, since the samples were taken to represent a very small area of the soil layer. The results can better represent the soil layer as field tests are applied over a larger area and stress conditions are available. Field tests are frequently used to eliminate particles the difficulty of sampling in split-grained soils and the problems of sample disturbance in fine-grained soils (Sivrikaya and Toğrol, 2007).

Experiments conducted to investigate the liquefaction potential of soils in situ;

- Standard Penetration Resistance (SPT)
- Cone Penetration Resistance (CPT)
- Shear Wave (V_s) Velocity
- Dilatometer Index (DMT)

In geotechnical investigations, Standard Penetration Test (SPT) resistance is by far the most widely used field test parameter for the characterization of liquefaction resistance (Kramer, 1996).

2.5. Constitutive Models Used in Liquefaction Analysis

Pore water pressure should be taken into consideration to determine the rigidity and strength loss that may occur in the soil and possible settlements (Özener, 2007). Also, the physical aspect of the problem should be considered in addition to a mechanically based approach in the calculation of large displacements occurring in sandy soils (Özener, 2007). In all methods used for stress-strain analysis in soil, to model material behaviour stress-strain relationships, or in other words, constitutive models are needed (Özener, 2007).

A numerical model to be developed to calculate the liquefaction potential of soils under the effect of an earthquake and their behaviour during/after liquefaction, defining the effect that will cause the movement of the ground (earthquake motion), the response of the ground to this effect (stress-deformation behaviour) and the internal effects that will occur during this process (damping of the movement, pore pressure increase, distribution and damping) should be defined in a way that best suits the conditions in the site (Tonaroğlu, 2006).

The choice of a constitutive model for a geotechnical practice depends on the mechanical properties of the soil (i.e., strength, stiffness and permeability), history of the soil and the stress changes that will happen in future (Wood and Muir, 1990). In finite element analysis, dependable forecasts can be achieved by using an appropriate constitutive model for a particular geotechnical problem (Khalid, 2013).

There are constitutive models based on elasticity and elastoplasticity theories to express the behaviour of the soil under dynamic loads, and incremental elastoplastic constituent models constituted to explain the behaviour of sand under dynamic loads (Özener, 2007).

There are several ways to include the liquefaction behaviour in numerical methods, ranging from total-stress based empirical schemes to estimate liquefaction conditions such as the UBCTOT model (Beaty and Byrne, 2000), to simple effective-stress shear-volume coupling schemes partial combined effective stress-based models such as the TARA-3 (Finn and Kumar, 1989) and the "Roth" model (Dawson et al., 2001), to more comprehensive constitutive models "full combined effective stress-based models" such as the UBCSAND model (Beaty and Byrne, 2011); PM+Sand model

(Boulanger and Ziotopoulou, 2012); NTUA-SAND Model (Papadimitriou et al., 2001); METUSAND (Oral, 2014) and Wang Model (Wang, 1990) bounding surface model (Oral, 2014).

Choosing the constitutive model is very important to obtain reliable results. An advanced structural soil model is required to perform effective dynamic analysis. Inelastic behaviours under cyclic loading can be accurately defined using elastic-plastic constructive models (Khalid, 2013). Initially, one of the best options for dynamic analysis can be considered from the structural models found in Plaxis, the hardening soil model (HSsmall) with small strain values. However, this model does not include the gradual softening behaviour under cyclic loading conditions and liquefaction behaviour under dynamic loading conditions (Brinkgreve, 2011). Therefore, the hardening soil model (HSsmall) is not suitable for liquefaction analysis. The UBCSAND model is especially one of the models developed for the prediction of liquefaction. UBCSAND model, Byrne et al. (1995) is based on the elastic-plastic stress-strain model proposed. Model, Puebla et al. (1997 and 1999) and Beaty and Byrne (1998), the model was implemented as a user-defined soil model in the finite element program Plaxis (Petalas ve Galavi, 2013). The UBCSAND model has also been implemented in MIDAS GTS NX. One of the most important features of the model is that it is an elastoplastic model that can examine the pore water pressure depending on time. In the model, the hysterical response of sand under cyclic loading is captured in a simplified way, and therefore the model can be used to more accurately predict unload-reload cycles. Since loading and unloading is one of the features of dynamic loading, this model can be used in the case of dynamic loading such as earthquake loads (Puebla, 1999).

3. SITE

3.1. Site Properties

Our site of work is within limits within Alstom Grid Energy Industry Inc. (Figure 3.1). As shown in Figure 3.2, our field of study is the area planned as a Transformer Assembly-Disassembly building with an area of 1119.73 m² with in Alstom Grid Energy Industry Inc. The stress transferred to soil by the structure to be built was calculated as 150 kPa.

Alluvium form, which is formed by the accumulation of pebble, sand and clay units, is dominant in the surface area. As a result of the drilling work, it was determined that the ground profile consisted of very deep silty sand with clay bands after the first 1 m fill layer on the surface. A groundwater layer was observed that started at 3 m levels during ground surveys.



Figure 3.1. Work site location

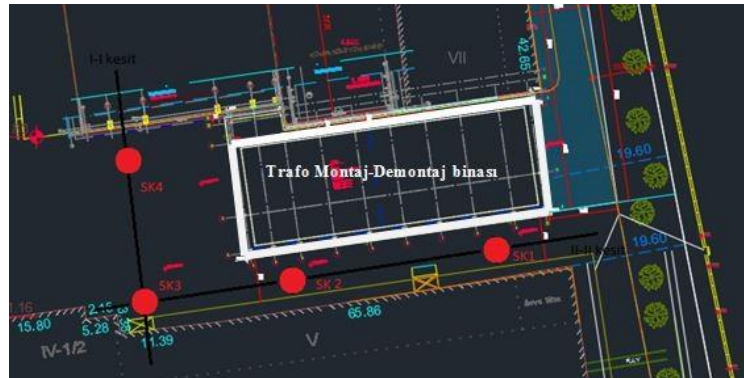


Figure 3.2. Area planned as Transformer Assembly-Disassembly building and drilled drilling pits

4 base boreholes were drilled in the study area (Figure 3.2). The physical and strength properties of the soil profile were calculated with the help of experiments performed on samples taken at various depths from these drillings and various correlations were used depending on the properties of the samples and SPT-N values. The entire soil profile is non-plastic silty sand (SM) consisting of 10% gravel, 58% sand and 32% silt and clay, and the average water content (w) is 21.5% (Figure 3.3).

By examining the Standard Penetration Test (SPT) results (Figure 3.3), the SPT- $N'_{1,60}$ value was taken as 13 on average. Using the average SPT-N value, the elasticity modulus (E_s) value and the internal friction angle (ϕ) value were calculated as 14 MPa and 25° , respectively. The shear wave velocities for the Z3-C group and the soil profile with silty sand were determined as $V_s=230$ m/s and $V_p=506$ m/s. The Poison ratio (ν) was calculated as 0.37 depending on the V_p and V_s , the relative stiffness (D_r) 50% depending on the soil class, the shear modulus (G_s) as 5100 kPa depending on the Poison ratio and the modulus of elasticity. Soil idealized profile and features are given in Figure 3.3. In the study area, the effective ground acceleration was determined as $PGA_{475} = 0.546g$.

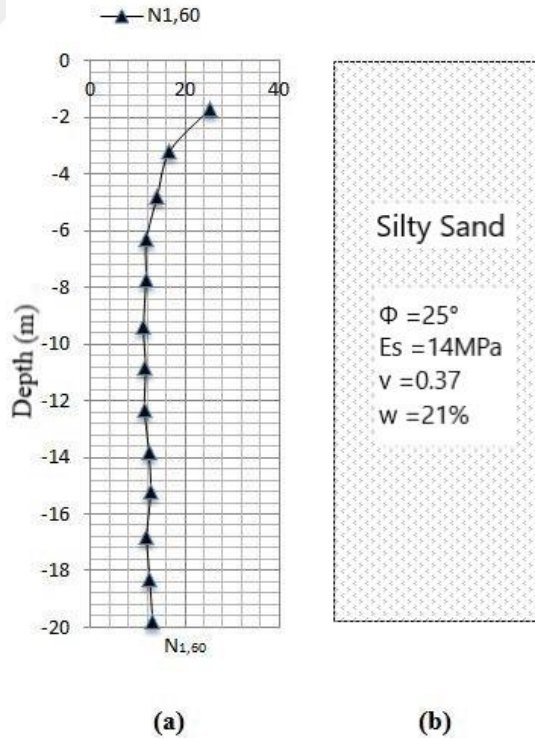


Figure 3.3. (a) Variation of SPT $N_{1,60}$ with the depth (b) soil profile and properties

3.2. Seismicity of the Region

When the earthquakes in the region are examined; There were 16 earthquakes with intensities of 6 and greater around Izmit, until 1900. The distribution of $M \geq 4.0$ earthquakes that took place between 1900-2000 by years is given in Figure 3.4 (Anonymous, 2003). Before the 17 August 1999 earthquake, there was an earthquake activity in the region, and an increase in earthquake activity was observed with large earthquakes that occurred in 1912, 1953, 1957, 1967, 1975, 1983 (Figure 3.4).

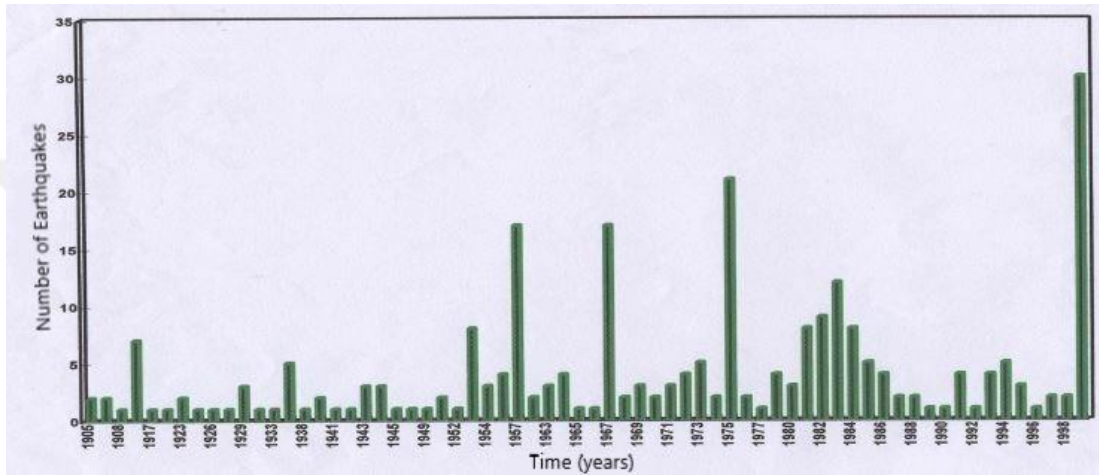


Figure 3.4. Distribution of $M \geq 4.0$ earthquakes that took place between 1900-2000 by years (Anonymous, 2003)

This activity reached its maximum value with the Izmit earthquake that occurred in 1999 (Figure 3.5). The Izmit Bay earthquake with a magnitude of $M_w = 7.6$, whose epicentre was Gölcük on August 17, 1999, and caused great destruction and loss of life in our country, occurred on the North Anatolian Fault Line.



Figure 3.5. 17 August 1999 earthquake rupture and fault segments (Demirtaş et al. 1999)

The high seismicity of the study area and the destructive effect of past earthquakes in this region, the groundwater being close to the ground level, and the non-plastic silty sand (SP-SM) type of loose material constituting the general ground profile showed the necessity of careful and detailed investigations on the soil of the building.

3.3. Assessment of Liquefaction Risk by Using Analytical Method

3.3.1. Liquefaction analysis based on SPT

Silty sand layers found in the study area have the potential to cause liquefaction below the groundwater level due to their situation of compactness. In force as of 2019, according to "the Turkey Earthquake Building Regulations" are liquefaction analysis has to be done on such soils. For this reason, liquefaction analyzes were carried out in 4 borehole locations as summarized below. Liquefaction risk is between 3.0-20.0 m from the ground surface. The risk of liquefaction below 20.0 m from the surface is not taken into account in the current analysis method, and the effect of possible liquefaction below this level on shallow-based structures on the surface remains negligible. In the study area, the effective ground acceleration was determined as $PGA_{475} = 0.546g$ and calculations were made based on the 0.546g acceleration value.

The method followed in liquefaction analysis: 1996 ve 1998 NCEER Liquefaction Resistance Workshop Recommendations (Youd and Idriss, 2001).

Earthquake magnitude, M_w : 7.6

Peak ground acceleration, $PGA_{475} = 0.546g$

Ground water table: 3.0 m

$$(N_{1,60})_{cs} = \alpha + \beta(N_{1,60})_{av} \quad FC = \%25 \quad (3.1)$$

$$\alpha = \exp [1.76 - 190/(FC)^2] \quad (3.2)$$

$$\alpha = \exp [1.76 - 190/(25)^2] = 4.4289$$

$$\beta = [0.99 + (FC)^{1.5}/1000] \quad (3.3)$$

$$\beta = [0.99 + (25)^{1.5}/1000] = 1.115$$

$$CRR = \frac{1}{34 - (N_{1,60})_{cs}} + \frac{(N_{1,60})_{cs}}{135} + \frac{50}{[10x(N_{1,60})_{cs} + 45]^2} - \frac{1}{200} \quad (3.4)$$

$$C_M = \frac{10^{2.24}}{M_W^{2.56}} \quad (3.5)$$

$$\tau_R = CRR_{M7.5} \times C_M \times \sigma'_{v0} \quad (3.6)$$

$$\tau_{\text{earthquake}} = 0.65 \times \sigma_{v0} \times 0.4 \times S_{DS} \times r_d \quad (3.7)$$

$$r_d = 1 - 0.00765z \quad z \leq 9.15 \text{ m} \quad (3.8)$$

$$r_d = 1.174 - 0.0267z \quad 9.15 \text{ m} < z \leq 23 \text{ m} \quad (3.9)$$

$$F.S. = \frac{\tau_R}{\tau_{\text{earthquake}}} \quad (3.10)$$

C_M = Moment magnitude correction coefficient

τ_R = Liquefaction resistance

$\tau_{\text{earthquake}}$ = Average repetitive shear stress in soil caused by an earthquake

σ'_{v0} = Effective stress

σ_{v0} = Total stress

S_{DS} = Short period design spectral acceleration value

According to TBDY 2018, the factor of safety (F.S.) should be a minimum 1.1.

$$F.S. = 0.745 \leq 1.1$$

There is a risk of liquefaction. In the analysis, against liquefaction, the safety factors were found $F.S. \leq 1.1$ in the range of 3.0-20.0 m in the silty sand (SM) unit. Liquefaction potential is quite high in this range.

3.3.2. Liquefaction analysis based on Vs

$V_s=230\text{m/s}$ with correction $V_s=V_{s1}$

$$V_{s1}=V_s (\text{Pa}/(\sigma'_v)_{av})^{0.25} \quad (3.11)$$

using Eq. 3.11, $V_{s1}=216\text{m/s}$

FC=25% using Figure 3.8, CSR=0.33

$$\text{CRR}=a(V_{s1}/100)^2+b ((1/(V_s- V_{s1})) - (1/V_s)) \quad (3.12)$$

$M=7.5$ ($a=0.03$, $b=0.9$); $\text{CRR}=0.20$

$$\text{FS}=\text{CRR}/\text{CSR}=0.20/0.33=0.61 < 1 \quad (3.13)$$

σ'_v = Effective vertical stress

There is a risk of liquefaction.

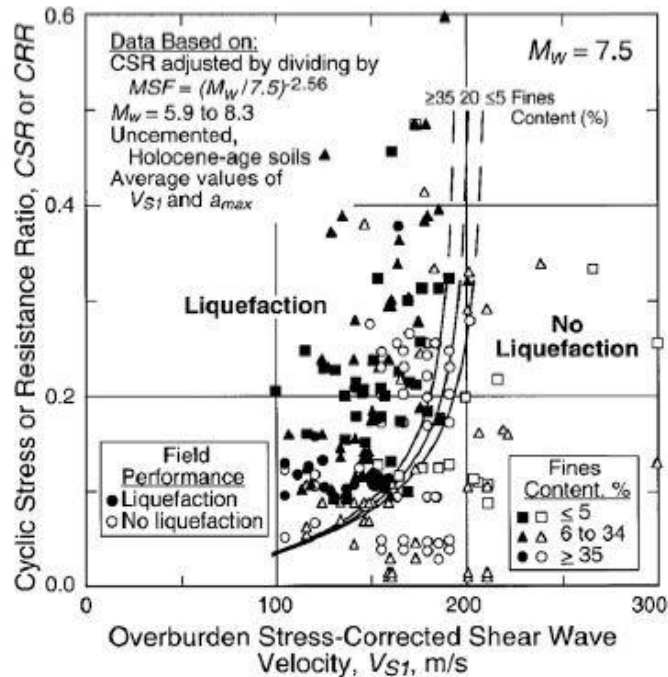


Figure 3.6. Liquefaction graph based on 225 land shear wave velocity (V_{s1}) records for clay, silt and gravel ground (Andrus and Stokoe, 2000)

3.3.3. Settlement due to liquefaction

$$[(N'_{1,60})_{CS}]_{av} = \alpha + \beta(N'_{60})_{av}, \quad (\alpha=4.289 \quad \beta=1.115) \quad (3.14)$$

$$[(N'_{1,60})_{CS}]_{av} = 4.289 + 1.115 \times 13 = 19$$

$$[(N'_{1,90})_{CS}]_{av} = 19 \times 60 / 90 = 13, \quad (FS)_{av} = 0.58$$

$$\epsilon_v = 3\% \text{ equals } 0.03$$

$$\Delta H = H \times \epsilon_v \quad (3.15)$$

$\epsilon_v\%$ = Post liquefaction volumetric strain

When liquefaction analysis is made for the soil profile depending on the SPT-N and Vs values, it has been determined that the first 20 m of the soil is liquefiable. In addition, the settlement amount after liquefaction is calculated as approximately 60 cm. Considering the ground properties and settlement amount, the jet grout method was considered the most suitable method and jet grout columns were designed for the study area.

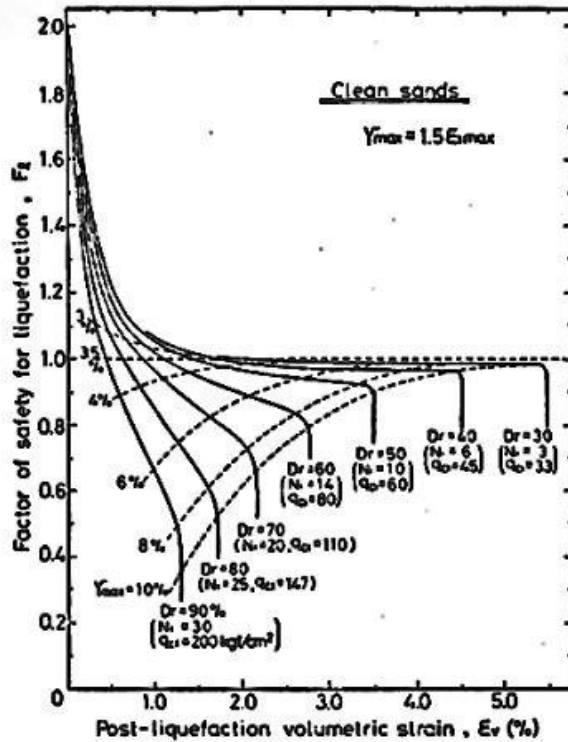


Figure 3.7. $N_1=(N_1)_{90}$, Relationship between volumetric strain after liquefaction and safety factor (Ishihara and Yoshimine, 1992)

3.4. Assessment of Liquefaction Risk by Using MIDAS GTS-NX

3.4.1. Constitutive model

UBCSAND model is an elastoplastic constitutive model is presented that simulates the liquefaction reactions of sands in a comparatively uncomplicated method. The model, UBCSAND, is based on the representative conduct of the soil skeleton as observed in laboratory element tests and it is shown to reasonably represent the soil reaction as observed in element tests. It is believed that a model which seizes the characteristic behaviour of granular soil can be a worthwhile tool in evaluating and understanding its performance under static and cyclic loading. The model has a few essential properties, including a hyperbolic relationship between stress ratio and plastic shear strain, a flow rule for estimating plastic volumetric strain from plastic shear strain, and the ability to handle anisotropy (Beaty ve Byrne, 1998).

UBCSAND is an effective stress plasticity model for use in advanced stress-deformation analysis of geotechnical structures. This model is used to observe the liquefaction behaviour of granular soils in dynamic loading statuses. The model computes the stresses for each loading cycle rely on the mobilized stress ratio. Using the mobilized stress ratio provides dropping the location of the yield surface. Correspondingly change of the yield surface, plastic volumetric strains are generated by using flow rule. Additionally, the pore water pressures are computed in synchronized with others (Byrne and Beaty 2011; Gürbüz, 2019).

To evaluate the liquefaction risk of soil, 2-dimensional numerical analysis was carried out using MIDAS GTS NX finite element program.

Although the method based on total stress is simple and practical in analysis, it does not take into account the pore water pressure. As stated earlier, liquefaction is thought to begin when the excess pore pressure equals the initial vertical effective stress (Seed and Lee, 1966). In other words, the ratio of dynamically induced excess pore pressure (u_e) to initial vertical effective stress (σ'_v) should be 100% as a prerequisite for liquefaction initiation. As the effective stress approaches zero due to the positive pore pressure, significant shear strength loss occurs according to the Mohr-Coulomb soil model (or theory).

In this study, the soil has been evaluated according to the change in excess pore pressure ratio (r_u) values.

First, by performing static analysis, geostatic conditions were obtained for a homogeneous soil profile that could liquefy under free field conditions. Then, it was investigated whether the soil liquefied or not under dynamic load, under dynamic load, which forms a part of free field analysis, according to r_u values. The simplified finite element model of the studied field is shown in Figure 3.8.

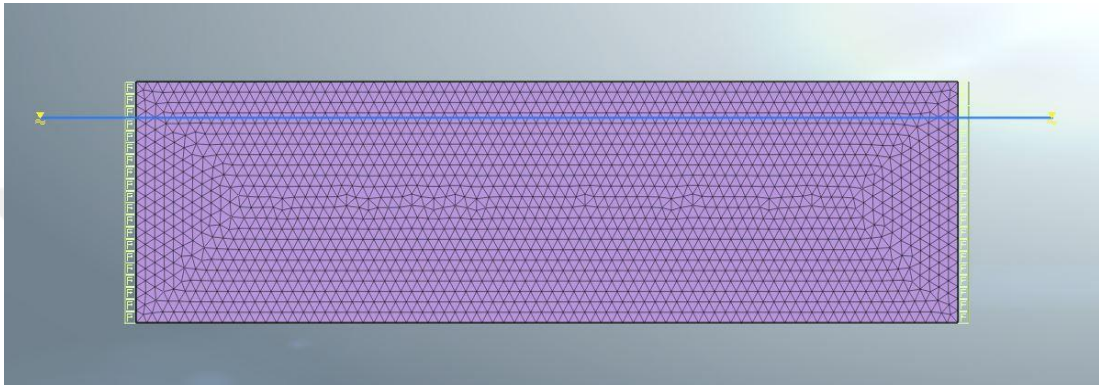


Figure 3.8. Finite Element Model (MIDAS-GTS NX)

The mathematical expression of r_u can be given by the following equation (Idriss & Boulanger, 2006):

$$r_u = u_{\text{excess}} / r'_{3c} \text{ (for triaxial pressure test)} \quad (3.16)$$

$$r_u = u_{\text{excess}} / r'_{vc} \text{ (for free soil)} \quad (3.17)$$

u_{excess} = Excessive pore water pressure

r'_{3c} = Consolidation stress

r'_{vc} = Consolidation stress

The state of the project area before improvement was carried out with the UBC-SAND constitutive model in using effective stress based finite element analyses MIDAS-GTS NX software program. The UBC-SAND model is a nonlinear elastic-plastic model that can determine the seismic liquefaction behaviour of sands and silty sands. (Beaty and Byrne, 2011).

Standard earthquake boundaries are determined. While the boundaries are determined in MIDAS-GTS NX, the "free field" property is defined to the model and "absorbent boundary" is used. In this way, a realistic perception is created as if our boundaries are far away and our model is that big. The purpose here is to ensure that earthquake waves are not affected by reflections in boundary conditions. Ground motion recording from the IZT recording station, about 8 km from the site, was used as an entry move in the base of the model. UBCSAND model non-linear parameters for the liquefiable soil layer are summarized in Table 3.1.

Table 3.1. Required nonlinear parameters for UBCSAND

Parameters	Definition	Value
$N_{1,60}$	SPT value	13
k_G^e	Elastic Shear Modulus Number	766
k_G^p	Plastic Shear Modulus Number	490
n_e	Elastic Shear Modulus Exponent	0.5
n_p	Plastic Shear Modulus Exponent	0.4
Φ_{cv}	Constant Volume Friction Angle	33°
Φ_p	Peak Friction Angle	34.3°
c	Cohesion	5 kN/m ²
R_f	Failure Ratio	0.75
f_{dens}	Soil Densification Calibration	1
f_{Epost}	Post Liquefaction Calibration	0.7

Constant volume friction angle (Φ_{cv}), peak friction angle (Φ_p) were evaluated from SPT test results in the field. Elastic shear modulus number (k_G^e), plastic shear modulus number (k_G^p) and failure ratio (R_f) are obtained by the following equations recommended by Beaty and Byrne, 2011.

$$k_G^e = 21.7A(N_1)_{60}^{0.333} \quad (3.18)$$

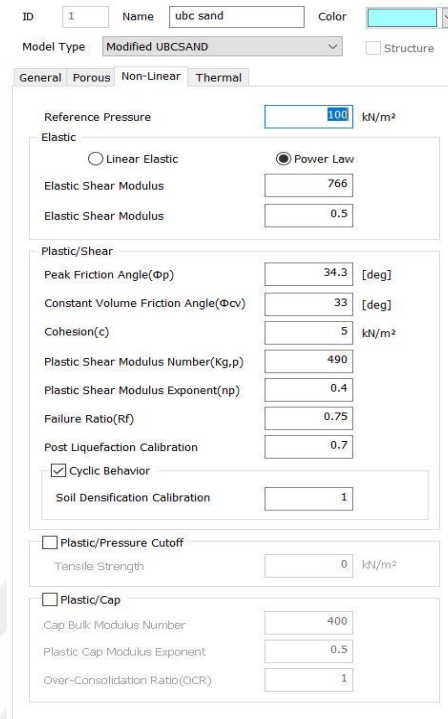
$$k_G^p = k_G^e((N_1)_{60})^2 0.003 + 100 \quad (3.19)$$

$$R_f = 1.1(N_1)_{60}^{-0.15} \quad (3.20)$$

where $(N_1)_{60}$ is the number of SPT-N corrected for effective stress and energy ratio, A and is constant in the range 15-20. n_e is constantly equal to 0.5, and n_p is another constant equal to 0.4 (Figure 3.9.).

The refraction rate (R_f) value can be taken in the range of 0.5 to 0.98. The R_f value was obtained by curve fitting directly from the cut-off test results (Byrne et al., 1987; Duncan et al., 1980).

The soil densification calibration (f_{dens}) is taken as 1. Post liquefaction calibration value can be taken between 0.2-1. (Petelas and Galavi, 2013).



The screenshot shows the 'Non-Linear' tab of the MIDAS GTS NX software interface for the UBCSAND constitutive model. The parameters are as follows:

Parameter	Value	Unit
Reference Pressure	100	kN/m ²
Elastic Model	Power Law	
Elastic Shear Modulus	766	
Elastic Shear Modulus	0.5	
Peak Friction Angle(Φ_p)	34.3	[deg]
Constant Volume Friction Angle(Φ_{cv})	33	[deg]
Cohesion(c)	5	kN/m ²
Plastic Shear Modulus Number($K_{g,p}$)	490	
Plastic Shear Modulus Exponent(n_p)	0.4	
Failure Ratio(R_f)	0.75	
Post Liquefaction Calibration	0.7	
Cyclic Behavior	☒	
Soil Densification Calibration	1	
Plastic/Pressure Cutoff	☐	
Tensile Strength	0	kN/m ²
Plastic/Cap	☐	
Cap Bulk Modulus Number	400	
Plastic Cap Modulus Exponent	0.5	
Over-Consolidation Ratio(OCR)	1	

Figure 3.9. Non-Linear soil parameters of UBCSAND constitutive model in MIDAS GTS NX

3.4.2. Analysis

After constituting the constitutive model and entering the necessary UBCSAND parameters into the MIDAS GTS NX program, we performed eigenvalue analysis before on to dynamic analysis.

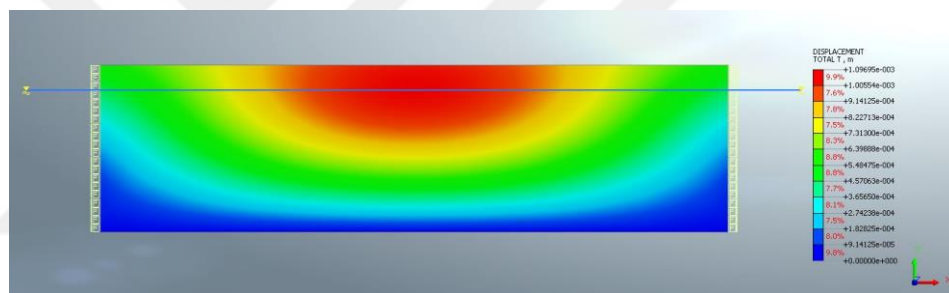
To determine the damping values to be used in time-dependent earthquake analysis, first of all, the vibration modes of the system will be determined by performing eigenvalue analysis. Thus, the damping values in earthquake analysis can be determined based on the periods in critical modes.

In the MIDAS GTS NX program, we can view the displacement status for all modes. But, looking at the percentage modal effective mass, the 1st and the 13th modes are more likely to occur. The probability of mode 1 occurrence is 65.25%, while the probability of mod 13 is 9.65% (Figure 3.10.).

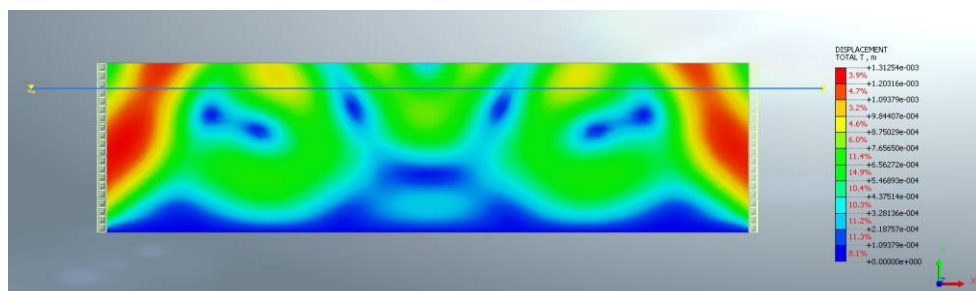
MODE NUMBER	PERCENTAGE MODAL EFFECTIVE MASS					
	T1	T2	T3	R1	R2	R3
1	65.25%	0.00%	0.00%	0.00%	0.00%	1.61%
2	0.00%	0.09%	0.00%	0.00%	0.00%	0.00%
3	2.42%	0.00%	0.00%	0.00%	0.00%	0.20%
4	0.00%	0.07%	0.00%	0.00%	0.00%	0.00%
5	0.41%	0.00%	0.00%	0.00%	0.00%	0.27%
6	0.37%	0.00%	0.00%	0.00%	0.00%	28.29%
7	0.00%	0.97%	0.00%	0.00%	0.00%	0.00%
8	0.00%	0.33%	0.00%	0.00%	0.00%	0.00%
9	4.90%	0.00%	0.00%	0.00%	0.00%	0.91%
10	0.76%	0.00%	0.00%	0.00%	0.00%	0.54%
11	0.00%	3.61%	0.00%	0.00%	0.00%	0.00%
12	0.00%	3.83%	0.00%	0.00%	0.00%	0.00%
13	9.65%	0.00%	0.00%	0.00%	0.00%	14.59%
14	0.02%	16.48%	0.00%	0.00%	0.00%	0.06%
15	1.29%	0.13%	0.00%	0.00%	0.00%	4.34%
16	1.37%	0.02%	0.00%	0.00%	0.00%	4.13%
17	0.00%	1.03%	0.00%	0.00%	0.00%	0.02%
18	5.18%	0.00%	0.00%	0.00%	0.00%	0.39%
19	0.00%	0.53%	0.00%	0.00%	0.00%	0.00%
20	0.00%	0.01%	0.00%	0.00%	0.00%	0.24%

Figure 3.10. Percentage modal effective mass

Figure 3.11. shows total translation for mod 1 and mod 13. The percentage of unexpected behaviour will be less.



a)



b)

Figure 3.11. Total translation a) mode 1 b) mode 13

In this case, we will foresee our dampings between the recommended period value for the 1st mode and the recommended period value for the 13th mode. Thus, we will analyze our dampings in the correct periods (Figure 3.12.).

MODE NUMBER	REAL EIGENVALUES					
	EIGENVALUE	RADIANS	CYCLES	PERIOD	GENERALIZE D	GENERALIZE D
1	1.380888e+0	1.175112e+0	1.870249e+0	5.346882e-001	1.000000e+0	1.380888e+0
2	2.662555e+0	1.631734e+0	2.596985e+0	3.850619e-001	1.000000e+0	2.662555e+0
3	3.515692e+0	1.875018e+0	2.984184e+0	3.351000e-001	1.000000e+0	3.515692e+0
4	4.425298e+0	2.103639e+0	3.348046e+0	2.986817e-001	1.000000e+0	4.425298e+0
5	5.783485e+0	2.404888e+0	3.827498e+0	2.612673e-001	1.000000e+0	5.783485e+0
6	5.908269e+0	2.430693e+0	3.868568e+0	2.584936e-001	1.000000e+0	5.908269e+0
7	6.091422e+0	2.468081e+0	3.928072e+0	2.545778e-001	1.000000e+0	6.091422e+0
8	7.650364e+0	2.765929e+0	4.402113e+0	2.271636e-001	1.000000e+0	7.650364e+0
9	8.357327e+0	2.890904e+0	4.601017e+0	2.173433e-001	1.000000e+0	8.357327e+0
10	1.004249e+0	3.168988e+0	5.043601e+0	1.982710e-001	1.000000e+0	1.004249e+0
11	1.134688e+0	3.368513e+0	5.361154e+0	1.865270e-001	1.000000e+0	1.134688e+0
12	1.281887e+0	3.580344e+0	5.698295e+0	1.754911e-001	1.000000e+0	1.281887e+0
13	1.377669e+0	3.711696e+0	5.907348e+0	1.692807e-001	1.000000e+0	1.377669e+0
14	1.493531e+0	3.864623e+0	6.150739e+0	1.625821e-001	1.000000e+0	1.493531e+0
15	1.539718e+0	3.923924e+0	6.245120e+0	1.601250e-001	1.000000e+0	1.539718e+0
16	1.731590e+0	4.161238e+0	6.622816e+0	1.509932e-001	1.000000e+0	1.731590e+0
17	1.807366e+0	4.251313e+0	6.766174e+0	1.477940e-001	1.000000e+0	1.807366e+0
18	1.839464e+0	4.288898e+0	6.825993e+0	1.464988e-001	1.000000e+0	1.839464e+0
19	1.944133e+0	4.409233e+0	7.017512e+0	1.425006e-001	1.000000e+0	1.944133e+0
20	2.086894e+0	4.568254e+0	7.270601e+0	1.375402e-001	1.000000e+0	2.086894e+0

Figure 3.12. Periods value for 1st and 13th modes

A Free field (Infinite boundary or absorbent boundary) was created as a boundary condition for time-dependent earthquake analysis (Figure 3.13).

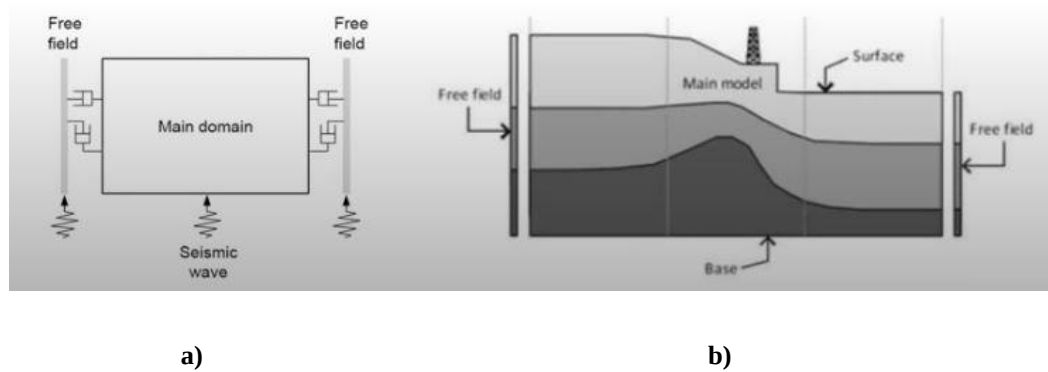


Figure 3.13. Free field a) infinite boundary b) absorbent boundary

In earthquake analysis, the model must be infinitely large so that the earthquake waves are not affected by the reflections at the model boundary. Since this situation is not possible in practice, analysis with free field boundary conditions can be performed in earthquake analysis. An absorbent boundary from the free field boundary condition is used to minimize the effect of model size and eliminate earthquake wave reflections at the substrate boundary.

We used Free field absorbent boundary for time-dependent earthquake analysis. We chose a value greater than 10^4 for the Width factor (penalty parameter) parameter to prevent our earthquake waves from being affected by the reflections at the border.

We performed Stress-Nonlinear Time History analysis using the period values obtained from Eigenvalue analysis. For the Static Slope analysis part, we analyzed it in two stages. In the first stage, we ensured the formation of stresses according to real stress conditions. The first stage is the initial condition. In the second stage, we defined the time-dependent earthquake load and added the dynamic load that we defined. So, at this stage, we shook the system with the earthquake load. In the study area, the effective ground acceleration was determined as $PGA_{475} = 0.546g$ in the Kocaeli-Düzce region. We translated it with a scale factor and used the real earthquake record. The time interval (Δt) is 0.02 seconds and the record lasted is 27.18 seconds of our Kocaeli earthquake (Figure 3.14.). According to the information, the software determined our total time step as 1359.

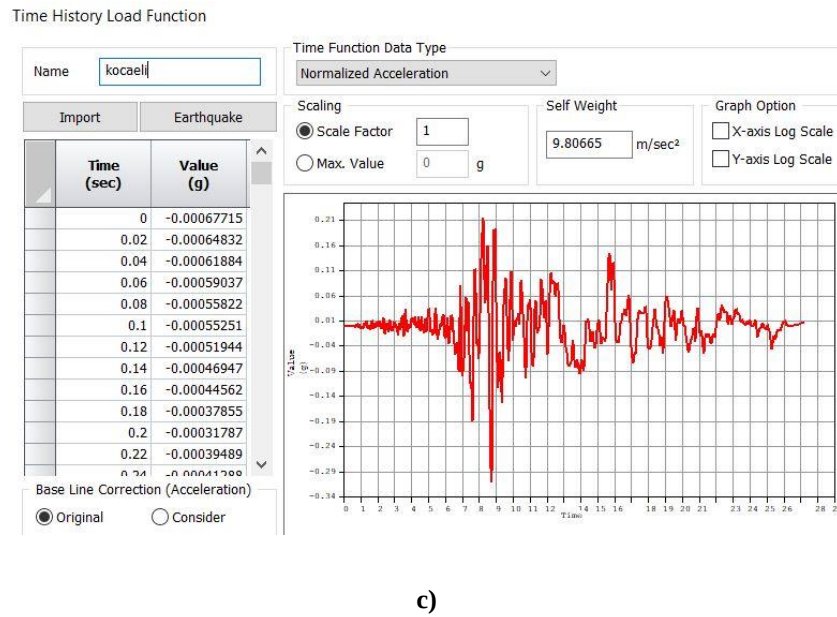
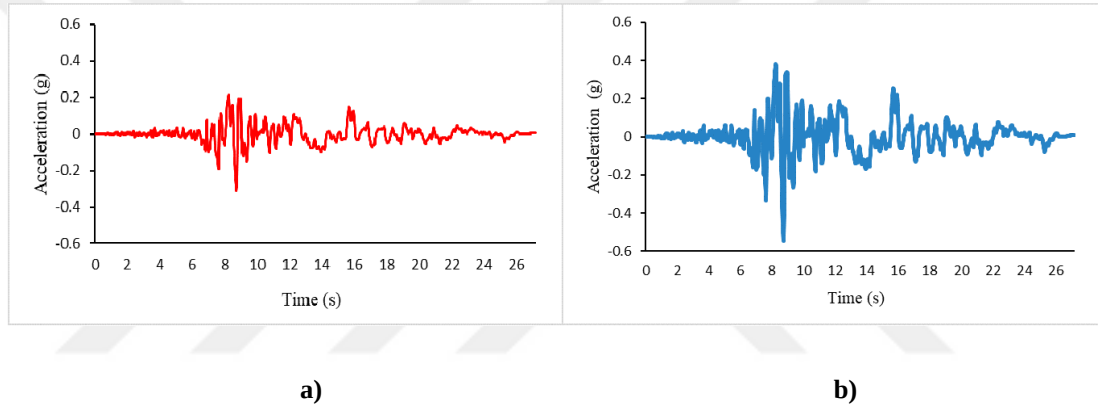


Figure 3.14. a) $PGA=0,309g$ b) $PGA=0,546g$ c) Kocaeli earthquake (1999) – Record obtained in Düzce station

According to MIDAS-GTS NX analysis results, the soil before any ground improvement application is made completely liquefied (Figure 3.15).

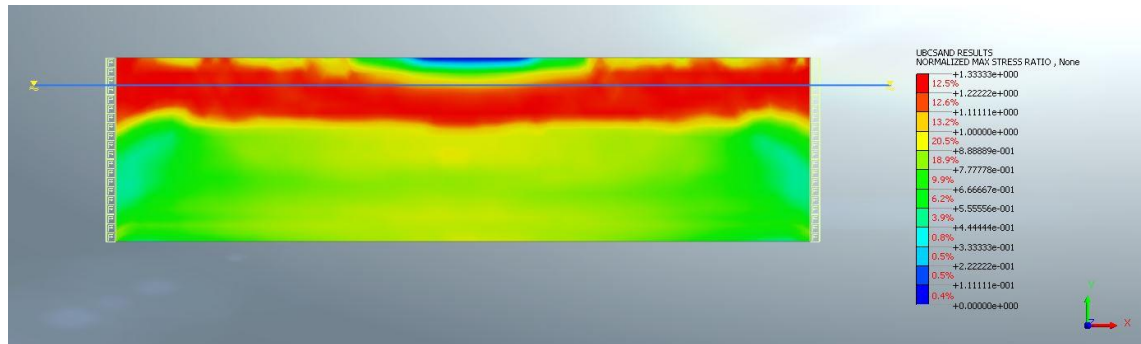


Figure 3.15. UBCSAND results, liquefiable soil

4. SOIL IMPROVEMENT PROJECT with JET GROUT COLUMNS

4.1. Properties of Jet Grout

In the jet-grout column ground improvement method, a water-cement mixture is injected into the ground with high pressure, and the injection liquid is mixed with the ground to ensure the formation of approximately circular cross-sectional improved parts. In the method, injection is made using high-pressure pumps and special rods. Injection rods are first lowered by applying hydraulic water pressure to the lower elevation of the jet-grout column. Then, injection is started under high pressure, the rods are rotated at a certain speed on the one hand, and jet-grout columns are formed by pulling them towards the ground surface at a prescribed speed on the other hand. In this way, cohesion and high-pressure strength are gained to the ground. Jet-grout columns are more rigid than the pre-improvement condition of the soil, but it is accepted that they have a certain degree of flexibility during an earthquake and can monitor soil deformations. Thanks to this feature, jet-grout columns absorb some of the earthquake energy and reduce the repeated shear stress rate that liquefies or softens the ground.

4.2. Numerical Analysis

The shear strength parameters (c and φ) of the improved soil and the deformation modulus (E) can be calculated by taking the weighted average of the jet-grout column and natural soil layer parameters in the plan. In this approach, jet-grout column parameters are found using the following equations (Nishimatsu, 1972):

$$c = \frac{\sigma_c \cdot \sigma_t}{2[\sigma_t(\sigma_c - 3\sigma_t)]^{0.5}} \quad (4.1)$$

$$\tan \varphi = \frac{\sigma_c^2 - 4c^2}{4\sigma_c \cdot c} \quad (4.2)$$

$$\sigma_t = (0.08-0.14) \sigma_c \quad (4.3)$$

σ_c = Unconfined compressive strength

σ_t = Tensile strength

$$\tau_0 = 0.53 + 0.37(\text{UCS}) - 0.0014(\text{UCS})^2 \quad (\text{UCS} \leq 60 \text{ kg/cm}^2) \quad (4.4)$$

τ_0 = 28-day shear strength (kg/cm²)

UCS= 28 days free compressive strength (kg/cm²)

$$E/\text{UCS} = (362 \sim 913)$$

In the weighted average calculations of the improved soil, the effective cohesion of the natural ground is neglected, and the effective friction angle is taken as $\phi = 25^\circ$. The deformation modulus of the natural soil was taken as $E = 14 \text{ MPa}$, considering the average standard penetration resistance and depth of the soil.

According to the designed soil improvement project, the targeted jet-grout column-free compressive strength is $q_u = 15 \text{ kg/cm}^2$. The tensile strength is between 8-14% of the free compressive strength. From here, the tensile strength was calculated as 210 kPa.

Jet grout column;

cohesion $c_{\text{jet}} = 368 \text{ kPa}$,

internal friction angle $\phi_{\text{jet}} = 38^\circ$

elastic modulus $E_{\text{jet}} = 540000 \text{ kPa}$

shear modulus $G_{\text{jet}} = 197000 \text{ kPa}$

Calculation results are in Figure 4.1.

<u>Jet-Grout</u>		<u>Soil</u>	
Unconfined compressive strength		Effective cohesion:	0 kPa
q_u :	1500 kPa	Effective friction angle:	25.0 °
Tensile strength		Elastic modulus:	14000 kPa
α :	0.14		
σ_t :	210 kPa		
Cohesion			
c_{jg} :	368 kPa		
Angle of internal friction			
$\tan\phi$:	0.77		
ϕ_{jg} :	37.7 °		
Elastic modulus			
E_{jg} :	540000 kPa		
		<u>Improved soil</u>	
		Area ratio:	0.09
		c_{ort} :	32.2 kPa
		ϕ_{ort} :	26.1 °
		E_{ort} :	59902.2 kPa
		JG diameter	60 cm
		S	180 cm (S=3.0B)

Figure 4.1. Ground improvement calculations

The diameter of the designed jet-grout columns has been chosen as $B=60$ cm, and the average distance between the column from the axis to the axis $S=3.0B=180$ cm (Figure 4.2.). Constructive principles were also taken into consideration while preparing the column disposition.

Number of horizontal columns;

$$(24.68-2 \times 0.6)/1.8+1=14$$

Number of vertical columns;

$$(65.86-2 \times 0.6)/1.8+1=37$$

Total number of columns;

$$14 \times 37=518$$

4.3. Affecting of Jet Grout

As expected, the improved soil parameters after the jet grout application; cohesion is $c_{av}= 32.2$ kPa, internal friction angle is $\phi_{av}= 26.1^\circ$, modulus of elasticity is $E_{av}= 59902$ kPa and shear modulus is $G_{av}= 21865$ kPa. After the jet grout application, the average shear wave velocity (V_s) of the improvement soil was found as $V_{s(av)} = 542$ m/s by going backwards in the liquefaction analysis correlation. The average value of

SPT-N of the improvement soil was found as $N'_{1,60}$ average 31, based on the change of shear wave velocity (V_s) of the soil.

Reliable foundation bearing capacity value (allowable bearing value) in the improved soil easily provides the foundation base pressure even (1.3 kg/cm²) found according to increased loads (1.4G + 1.6Q).

Settlements and liquefaction on the improved soil using the jet grout method were re-examined, and it was determined that the elastic (immediate) settlement could be 20 mm, and by using the Burland Burbidge method, it was determined that there could be a settlement in the ground at 20 mm.

Liquefaction prevention calculations for jet grout column soil improvement project for soil,

Average groundwater level, GWT = 3.0 m

Current corrected average SPT resistance in profile, SPT- N_{60} = 13

Soil average shear modulus, $G \cong 10$ MPa

Jet grout column features,

Manufacturing method: Jet-I

Diameter, $B = 60$ cm; Length, $L = 11$ m; Distance between axes, $S = 3B = 1.8$ m

Targeted free compressive strength, $q_{u-jg} = 10$ kg/cm²

Modulus of elasticity, $E_{jg} = 540$ MPa

Poisson ratio, $\nu = 0.25$

Shear modulus, $G_{jg} = 197$ MPa

These columns have a reducing effect on the dynamic shear stresses affecting the soil layers between the jet-grout columns. This effect can be expressed by the shear stress ratio, K_G . This ratio is written as the ratio of reduced shear stress, τ_s , to unreduced stress, τ acting on the soil between the columns and is given by the following equation:

$$K_G = \frac{\tau_s}{\tau} = \frac{1}{G_r} \cdot \frac{1}{[A_r + \frac{1}{G_r}(1 - A_r)]} \quad (4.5)$$

$$G_r = G_{jg} / G_{soil} \quad (4.6)$$

$$A_r = A_{jg} / A \quad (4.7)$$

G_r = Shear modulus ratio

G_{jg} = Jet-grout column shear modulus

G_{soil} = Soil shear modulus

A_r = Displacement ratio

A_{jg} = Jet-grout area

A = Foundation area

$$G_r = 197 / 5.1 = 38.6$$

$$A_{jg} = (\pi \times 0.6^2 / 4) = 0.283 \text{ m}^2$$

$$A_r = 0.283 / 1.8^2 = 0.087$$

$$K_G = 0.462$$

The average rate of shear stress causing liquefaction in the soil is $CSR_{av} = 0.33$

The average ratio of slip resistance against liquefaction in the soil is $CRR_{av} = 0.20$

A factor of safety against liquefaction before jet-grout column manufacturing:

$$F.S = CRR_{av} / CSR_{av} \quad (4.8)$$

$$F.S = 0.61 < 1.0 \quad \text{There is a liquefaction risk.}$$

Shear stress-causing liquefaction in the soil after Jet-Grout column manufacturing,

$$CSR'_{av} = K_G \times CSR_{av} \quad (4.9)$$

$$CSR'_{av} = 0.462 \times 0.33$$

$$CSR'_{av} = 0.152$$

A factor of safety against liquefaction after Jet-Grout column manufacturing,

$$F.S = CRR_{av} / CSR'_{av} \quad (4.10)$$

$$F.S = 1.32 > 1.0 \quad \text{No risk of liquefaction.}$$

4.3. Assessment of Liquefaction Risk by Using MIDAS GTS-NX

4.3.1. Constitutive model

As mentioned, the jet grout method was used to improve liquefied soil. In the MIDAS-GTS NX program, the ground improvement with jet grout was analyzed. After the improvement was done, it was investigated whether soil became liquid or not.

jet grout columns were activated and static analysis was performed (Figure 4.2.). The strength ratio and area ratio applied in the field were 38.6 and 8.7%, respectively.

Numerical analyzes (nonlinear stress-time analysis) were performed for different strength and area ratios to show the effectiveness of jet grout columns in reducing the liquefaction potential in the field. For this purpose, by keeping the shear modulus of the soil constant, by changing the column strengths and the distances between the columns; strength ratios and area ratios, and numerical parametric analyzes were performed.

Jet grout analyzes were repeated statically and dynamically for each different spacing (2B, 3B, 4B, 5B and 6B) between adjacent columns. Jet grout columns were activated and static analysis was performed. The jet grout column spacings presented correspond to the area change ratio of 2.2%, 3.1%, 4.9%, 8.7% and 19.6%. Strength change ratios are 20, 38.6, 60, 80 and 100.

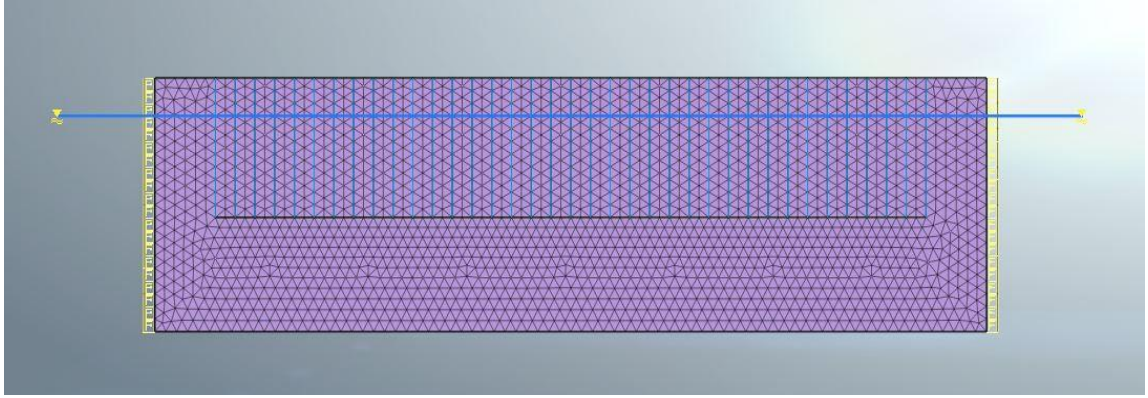


Figure 4.2. Finite Element Model with Jet Grout (MIDAS-GTS NX)

In addition, the varying strength of jet grout columns was taken into account as relatively less rigid, rigid and very rigid with respect to the ground.

4.4. Parametric Study

To show the effectiveness of jet grout columns in reducing the liquefaction potential in the field, numerical analyzes (nonlinear stress-time analysis) were made for different strength and area ratios. For this purpose, by keeping the shear modulus of the ground constant, by changing the column strengths and the distances between the columns; Numerical parametric analyzes were performed with $G_r = 20, 38.6, 60, 80, 100$ strength ratios and $A_r = 2.2\%, 3.1\%, 4.9\%, 8.7\%, 19.6\%$ area ratios.

The strength ratio and area ratio applied in the field were 38.6 and 8.7%, respectively (Figure 4.3).

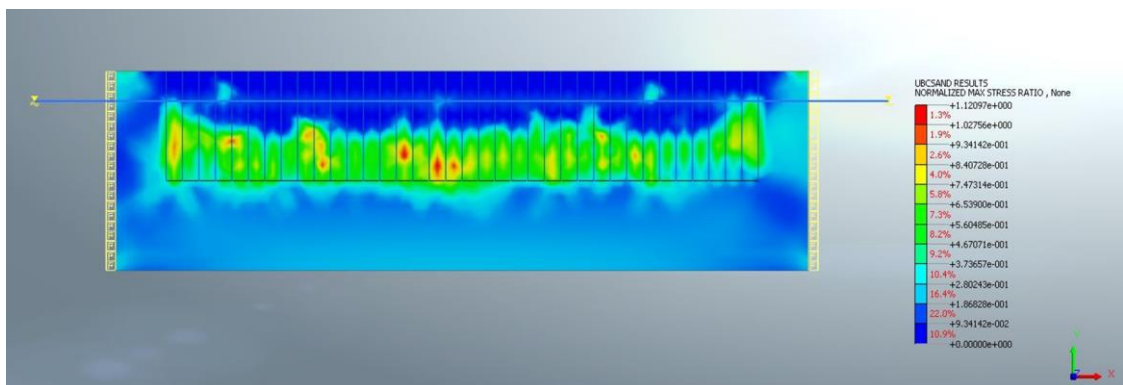


Figure 4.3. UBCSAND results, improved soil with jet grouts

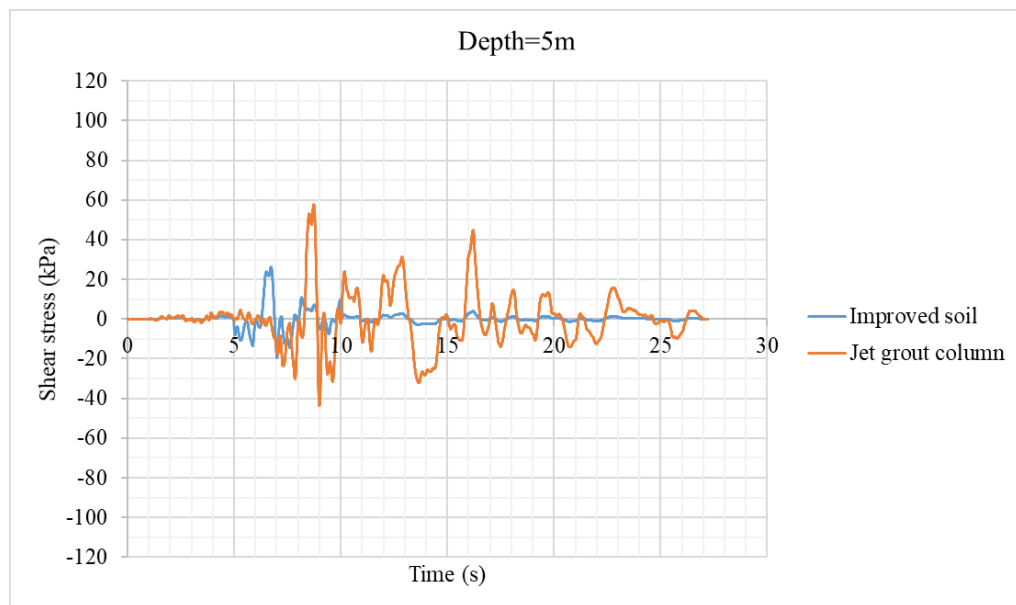
The analysis results were evaluated based on the effect of jet-grout columns the liquefiable soil layer on the shear stress, shear deformation and excess pore water pressure distribution and shown as a function of the depth in the liquefiable soil layer.

4.4.1. Shear stress and shear strain distribution

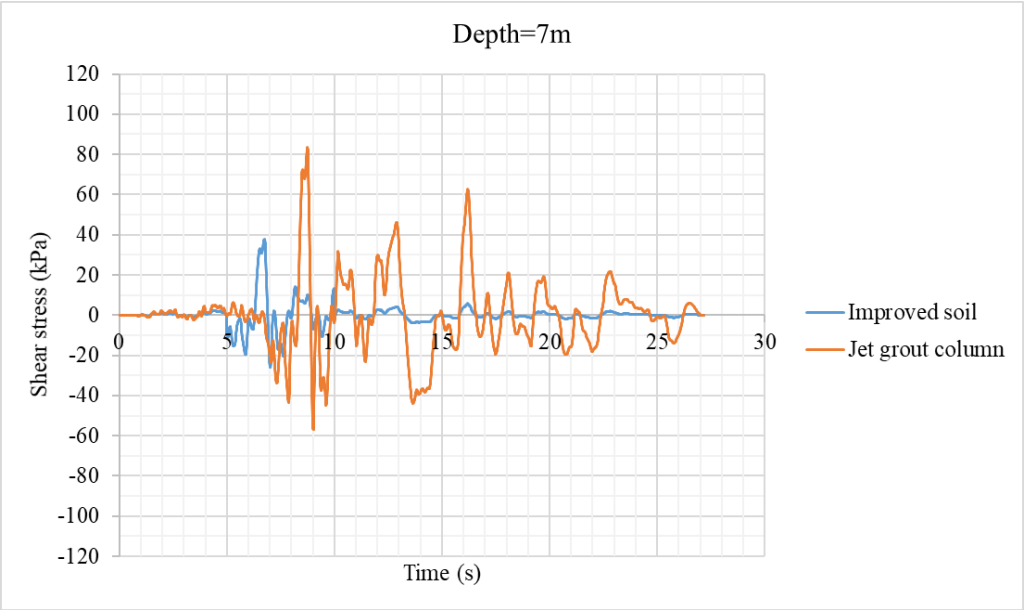
The results of the numerical analyses were evaluated considering the varying strength ratios and area ratios of the seismically induced shear stresses and shear strains. These results were generated considering the liquefiable soil layer at a depth of 3.0 m to 20.0 m. In Figure 4.4., and Figure 4.6., the variation of shear stresses and shear strains over time are shown at various 5m, 7 m, 9 m and 11 m depths for $G_r = 38.6$ and $A_r = 8.7\%$.

In Figure 4.5, the graph is of shear stress maximum values at different depths for improved soil and jet grout column variation.

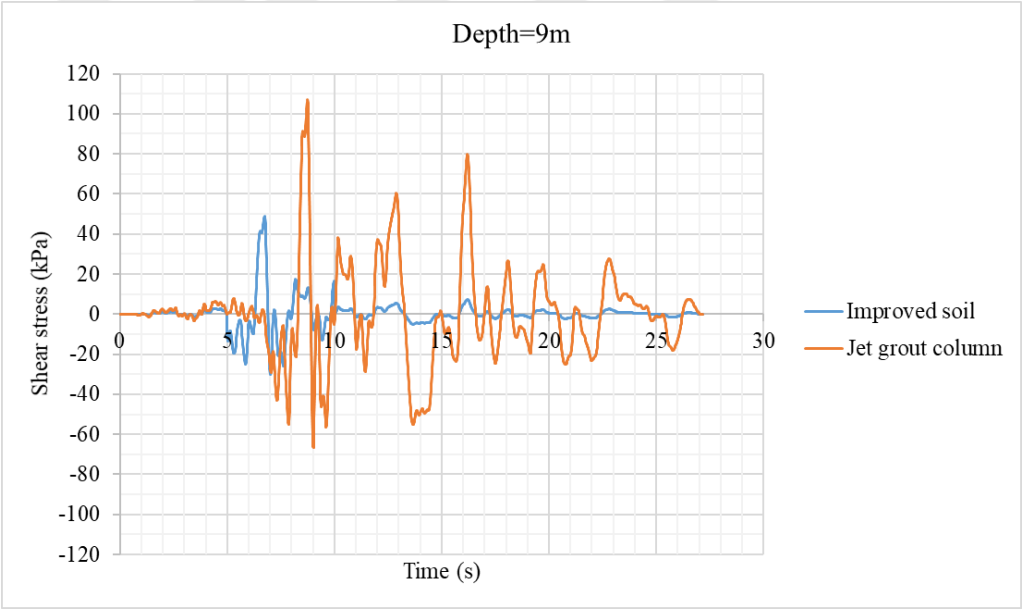
In Figure 4.7, the graph is of shear strain (%) values at different depths for improved soil and jet grout column variation.



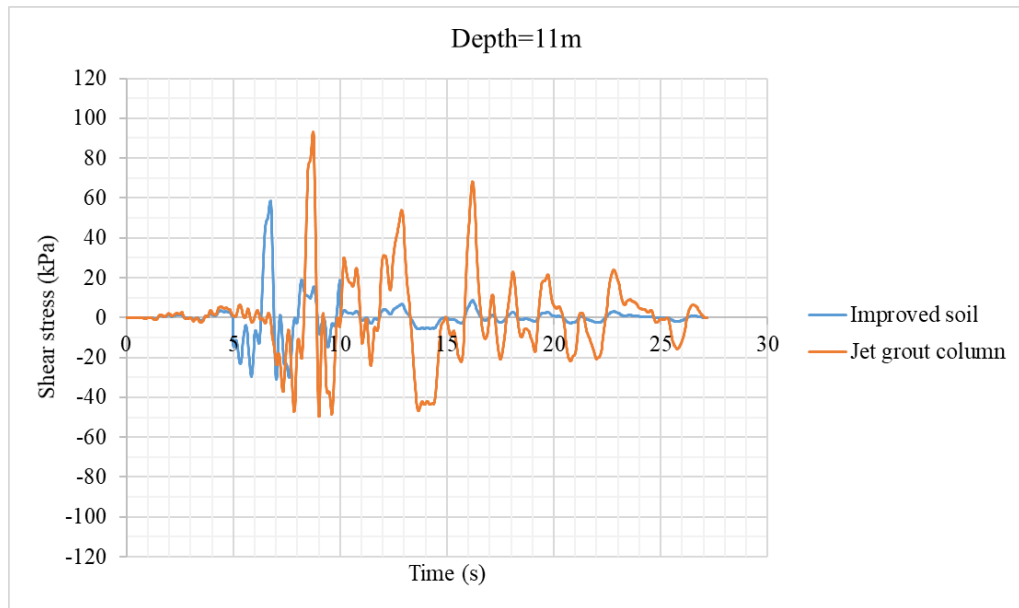
a)



b)



c)



d)

Figure 4.4. Time-dependent shear stress variation at different depths a) 5m b) 7m c) 9m d)11m

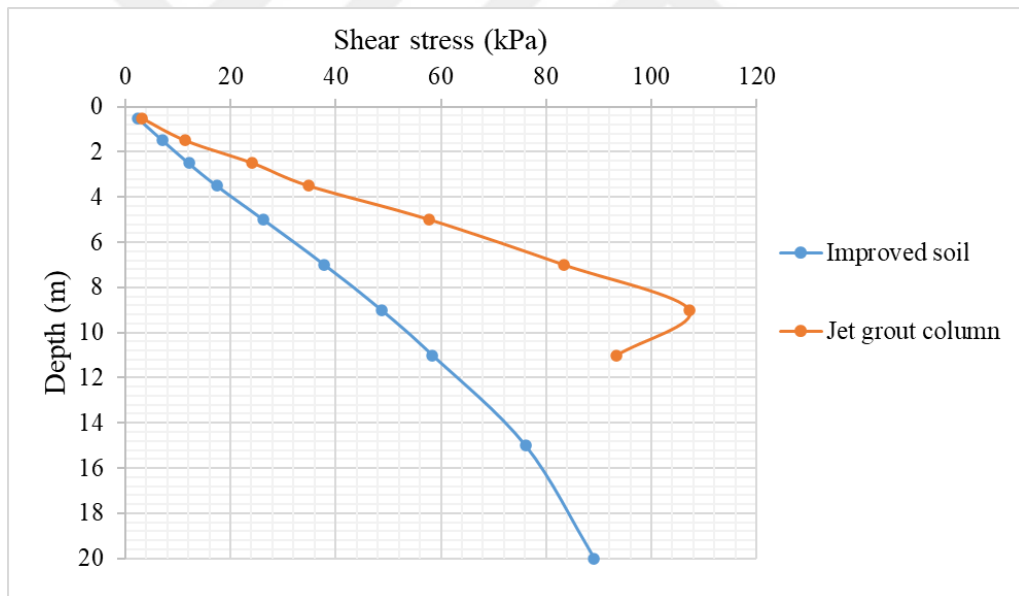
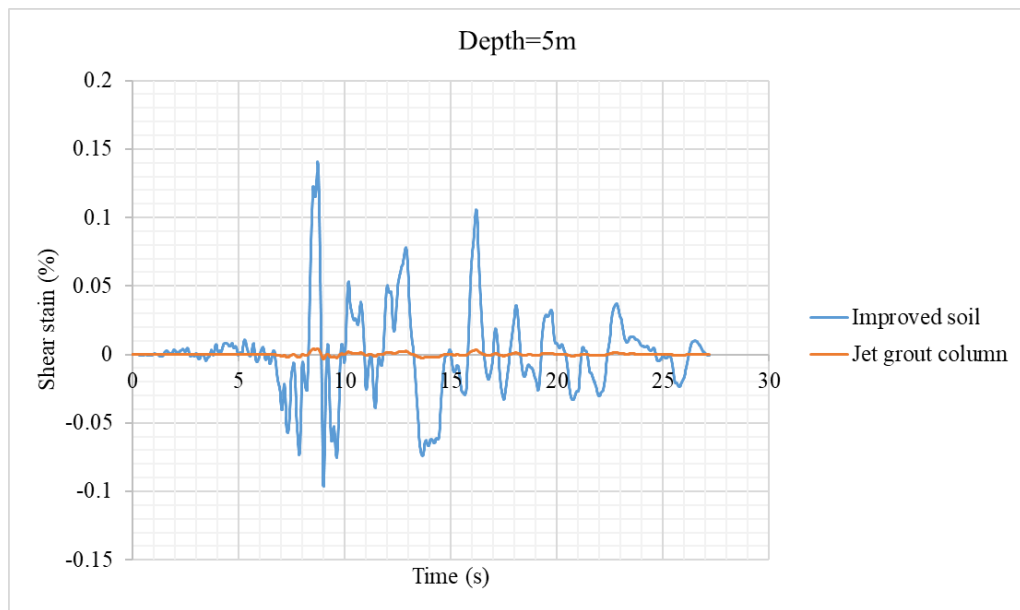
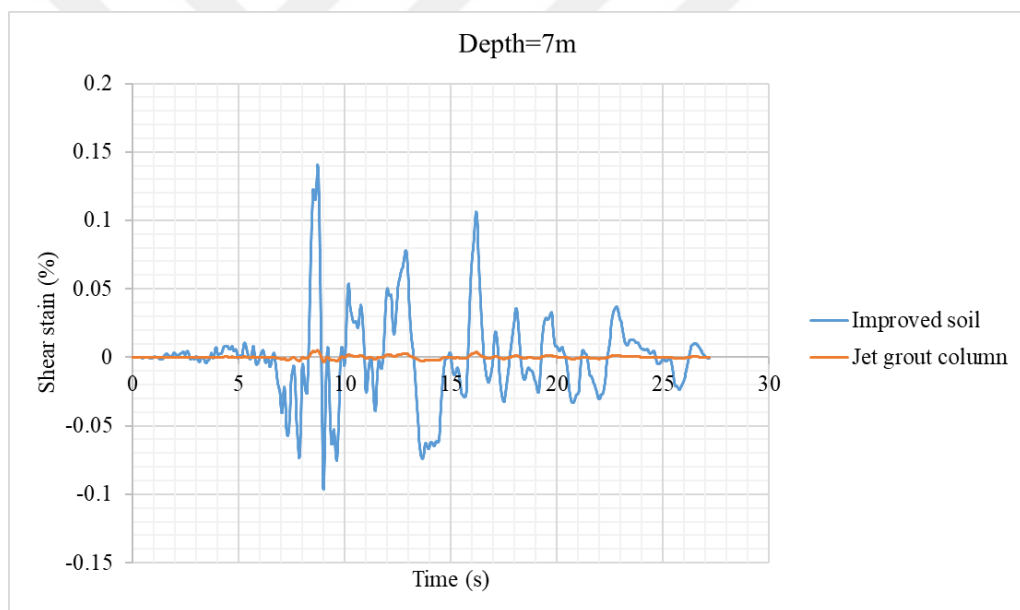


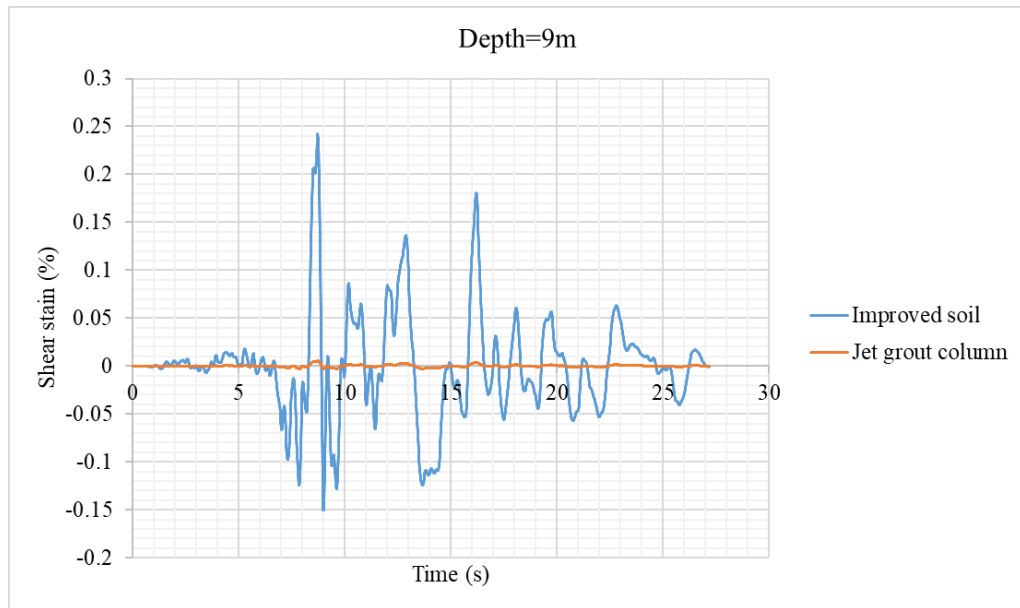
Figure 4.5. Maximum shear stress variation at different depths improved soil and jet grout column



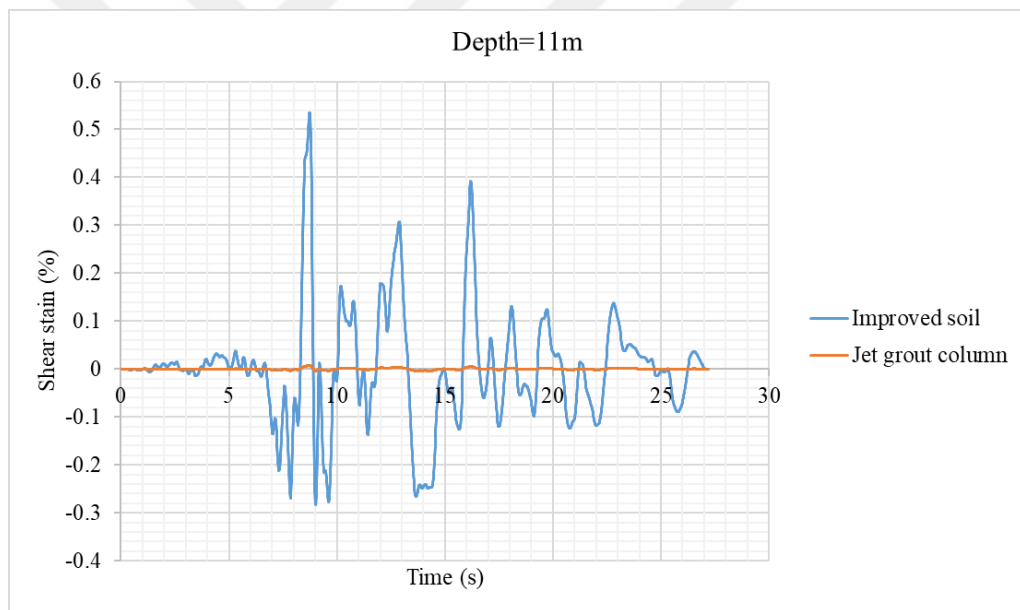
a)



b)



c)



d)

Figure 4.6. Time-dependent shear strain variation at different depths a) 5m b) 7m c) 9m d)11m

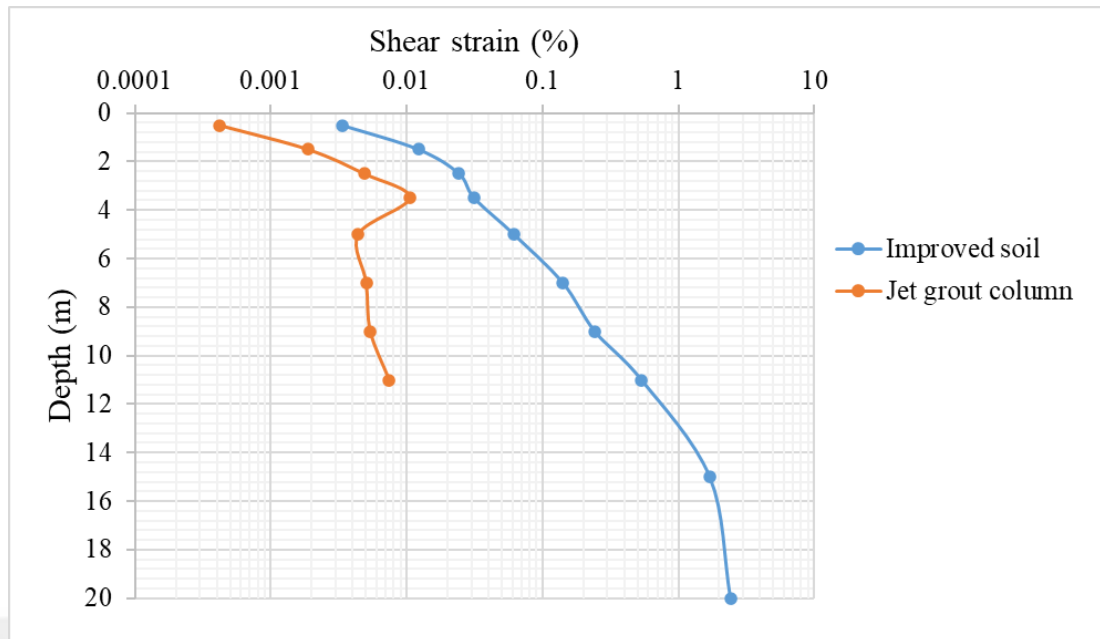


Figure 4.7. Shear strain (%) variation at different depths, improved soil and jet grout column

Jet-grout columns appear to take on most of the shear stresses compared to the surrounding soil, as they are more rigid than the liquefiable soil layer. On the other hand, as can be seen from the plot of shear deformation versus time, the rigid columns did not deformation much compared to the surrounding ground.

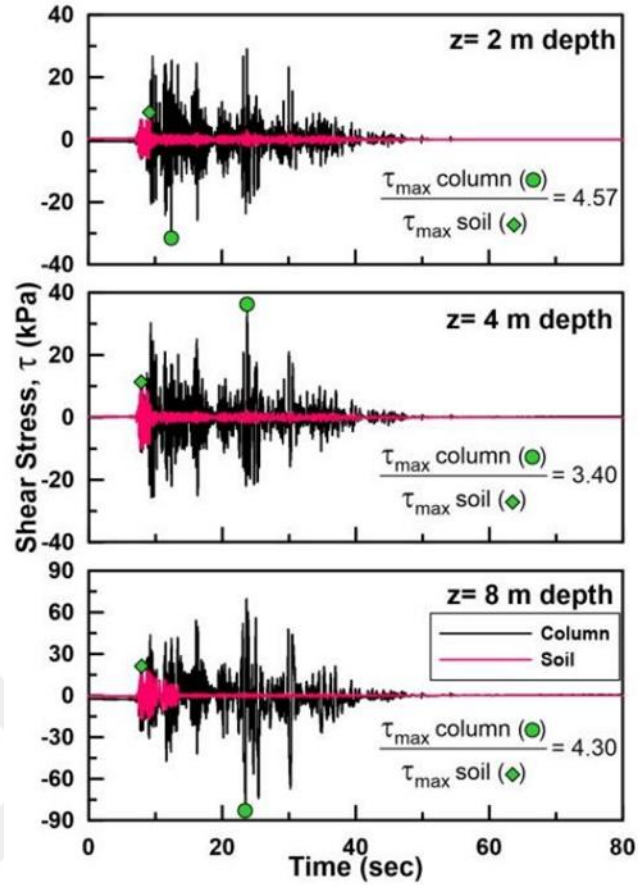


Figure 4.8. Time-dependent shear stress variation at different depths (Demir and Özener, 2017)

This case, according to Demir and Özener (2017) is consistent with the results stated in Figure 4.8.

This result shows that the jet-grout columns do not provide the expected shear deformation compatibility. Considering Figures 4.4. and 4.5, the analysis results obtained for cases with different strength ratios and area ratios are evaluated and the change in the maximum values of these shear stresses. The shear stress ratio (R_{ss}) in the graphs is the ratio of the soil shear stress to the shear stress of the jet grout column.

Based on the analysis results, as seen in Figure 4.9, The shear stress ratio (R_{ss}), has been plotted along with the depth for different area ratios for $G_r = 38.6$. Figure 4.10. shows shear stress ratio for different strength ratios and $A_r = 8.7\%$.

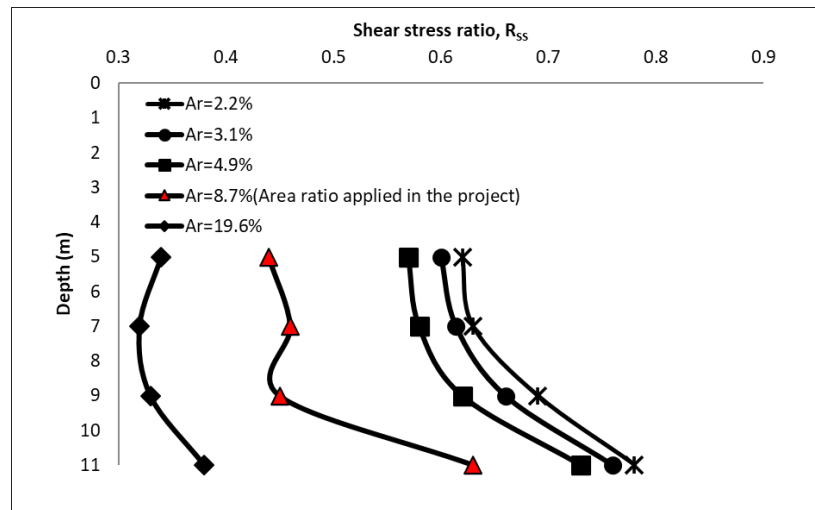


Figure 4.9. Variation of shear stress ratio with depth for different jet-grout area ratios (Gr=38.6)

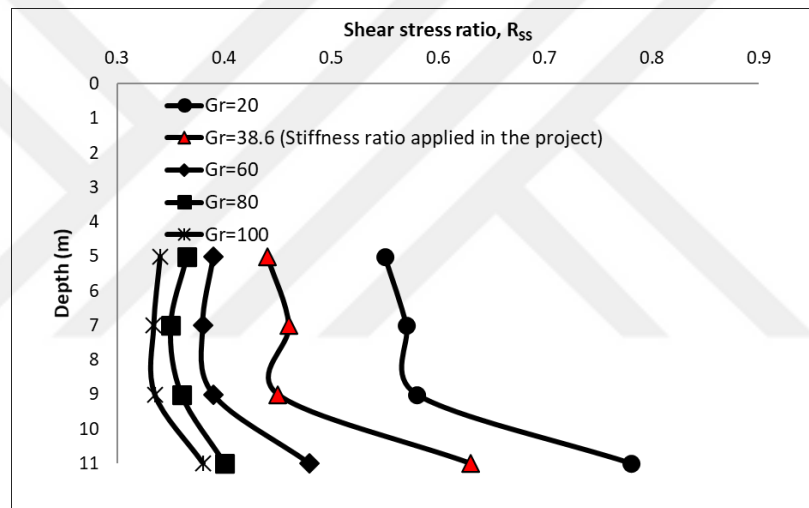


Figure 4.10. Variation of shear stress ratio with depth for different jet-grout strength ratios (Ar=%8.7)

Thus, as the jet grout area ratio increases, the shear stress taken by the jet increases as seen in Figure 4.9. When we examine in Figure 4.10, we can say that increasing the area ratio is more logical than increasing the stiffness of the jet.

It has been calculated that jet grout columns are subjected to higher shear stresses than liquefiable soils' shear stresses, since jet grout columns are more rigid than the soil. However, based on the analyses results, it is considered that they do not have sufficient shear deformation to significantly reduce the shear stresses in the liquefiable soil layer. The same conclusion was reached by Olgun and Martin (2008),

who examined the role of jet grout columns with total stress analysis (DIANAFLOW), which does consider pore water pressure generation.

4.4.2. The effect of Jet-Grout columns on the formation of excess pore water pressure

In this study, the effect of jet grout columns on mitigating the negative effects of liquefaction in soils was investigated by evaluating the effects on excess pore water pressure production with the help of the effective stress-based UBCSAND model. The development of excess pore water pressure is expressed as the excess pore water pressure ratio (r_u) and the change of r_u over time is shown in Figures 4.8. and 4.9. for the liquefiable soil layer (at 7 m) for different strength and area ratios.

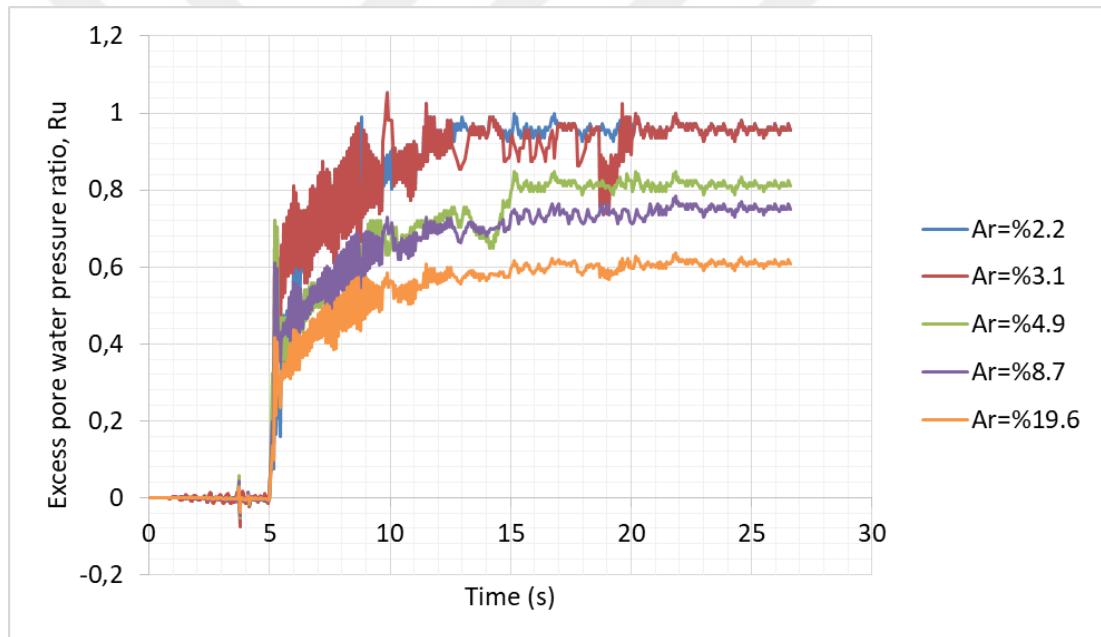


Figure 4.11. Variation of excess pore water pressure with time for different area ratios (Gr=38.6) (7 m)

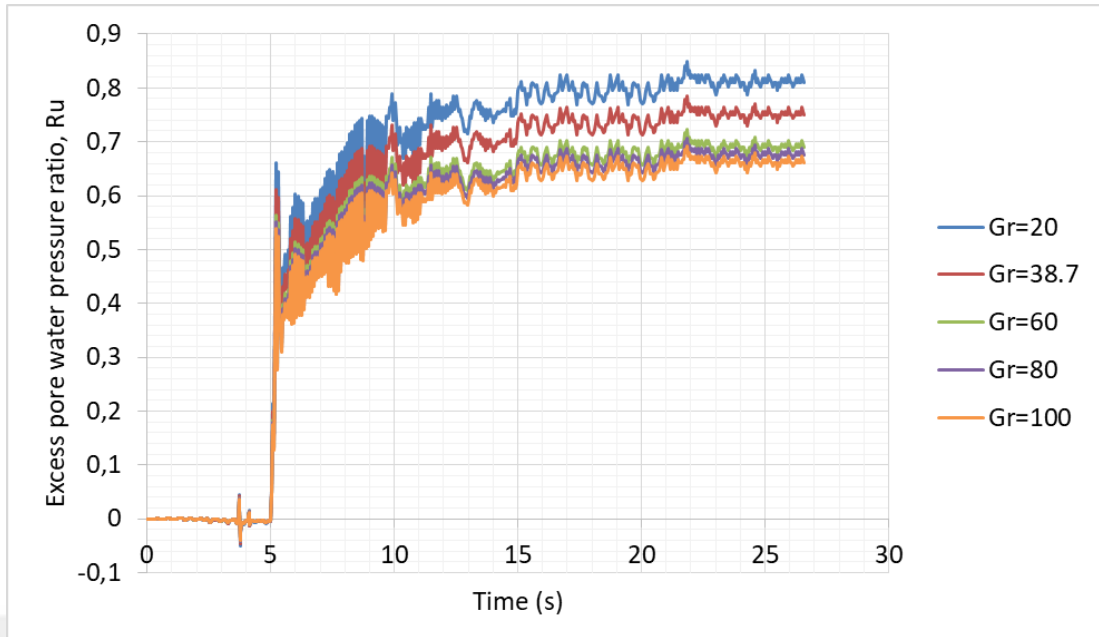


Figure 4.12. Variation of excess pore water pressure with time for different strength ratios (Ar=8.7%)

In Figure 4.11., when we examine the graph, we can say that increasing the jet grout area ratio has a prevention liquefaction effect. Because, when the area ratio rises above 10%, we see that the excess pore water pressure drops below 0.7.

Increasing the strength ratio of jet grout is effective in preventing liquefaction, but when we compare it with increasing the area ratio of jet grout, it seems more logical to increase the area ratio. Because as can be seen in Figure 4.12., increasing the strength ratio value above 50 does not have much effect anymore.

5. RESULTS AND DISCUSSION

Within the scope of this dissertation, the performance of jet grout columns in a field with has the bearing capacity, settlement and liquefaction problems were analyzed through analytical methods and finite element analyses. The quantitative contributions of the factors affecting the efficiency of the soil improvement method (jet grout) were determined by the numerical models. The accuracy of the improved soil parameters determined after the jet grout application has been investigated by back analysis with various correlations.

The results of the numerical analysis carried out to investigate the effectiveness of jet grout columns in preventing liquefaction through a field study were evaluated in terms of shear stresses, shear deformations and excess pore water pressure ratios by considering different columns strength ratios and area ratios. The calculated shear stresses showed that the jet-grout columns undertake most of the shear stresses compared to the surrounding soil, as it is more rigid than the liquefiable soil layer.

The role of jet-grout columns in reducing the negative effects of liquefaction is expressed by the shear stress ratio (R_{ss}), assuming compatibility with shear strain. The shear stress ratio (R_{ss}) in the graphs is the ratio of the soil shear stress to the shear stress of the jet grout column.

As a result of the analysis, it can be said that the jet grouts absorbed some of the shear stresses, but it did not have much effect on the shear deformation. Moreover, increasing the area ratio is more effective than increasing the stiffness of the jet.

As a result of the numerical analyses, it is thought that numerical analysis studies help to understand the mechanisms occurring in the fields reinforced with jet-grout columns. The real data obtained from the instrumented fields and the experimental data obtained from the model tests can provide a better understanding of the behaviour of jet grout columns during liquefaction.

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