

DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**AN INVESTIGATION ON THE INFLUENCE OF
JET-GROUT PARAMETERS ON
LIQUEFACTION AND EARTHQUAKE
PERFORMANCE OF CELLULAR JET-GROUT
APPLICATION**

by
Barış KAYMAZ

January,2024

İZMİR

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JET-GROUT PARAMETERS ON
LIQUEFACTION AND EARTHQUAKE
PERFORMANCE OF CELLULAR JET-GROUT
APPLICATION**

**A Thesis Submitted to the
Graduate School of Natural and Applied Sciences of Dokuz Eylül University
In Partial Fulfillment of the Requirements for Master of Science in
Department of Civil Engineering, Applied Geotechnical Program**

by

Barış KAYMAZ

January,2024

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THESIS EXAMINATION RESULT FORM

We have read the thesis entitled “**AN INVESTIGATION ON THE INFLUENCE OF JET-GROUT PARAMETERS ON LIQUEFACTION AND EARTHQUAKE PERFORMANCE OF CELLULAR JET-GROUT APPLICATION**” completed by **BARIŞ KAYMAZ** under supervision of **PROF.DR. GÜRKAN ÖZDEN** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Master of Science.

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ACKNOWLEDGMENT

I am grateful to my supervisor, Prof. Dr. Gürkan Özden, for his invaluable guidance and support throughout my thesis study. His expertise, patience, and encouragement have inspired me on this academic journey. His knowledge and valuable suggestions have played an important role in shaping my research and personal development.

I would like to thank my family for their understanding and endless support. Their encouragement and support during difficult times, helping me to achieve my goals, and always being there for me would have made this study impossible.

Finally, I would like to express my gratitude to my dear friends who supported me throughout this thesis process. Their encouragement, exchange of ideas, and unwavering support enabled me to complete this work. I dedicate this thesis to all those who stood by me with their knowledge and support.

Bariş KAYMAZ

AN INVESTIGATION ON THE INFLUENCE OF JET-GROUT PARAMETERS ON LIQUEFACTION AND EARTHQUAKE PERFORMANCE OF CELLULAR JET-GROUT APPLICATION

ABSTRACT

Soil liquefaction is a seismic event that can cause significant damage. To prevent loss of life and property in earthquakes, it is necessary to improve soil. The technique aims to increase the strength parameters of the soil to which it is applied and to restrict the deformations occurring in the soil. This study comprises two parts. First part is to analyze the effect of the socket length of jet grouting columns and the effect of the unconfined compressive strength of jet grouting columns on the prevention of liquefaction in soil improvement works using jet grouting technique using finite element software. Additionally, it evaluates the success of cellular jet grouting, an innovative method, in preventing liquefaction using finite element analysis. The effective stress based UBC3D model was used to represent soil that exhibited liquefaction behavior in the analyses. The numerical analysis revealed a decrease in shear deformations in the soil and columns with increasing socket length. Analysis of the excess pore water pressure ratio (R_u), a key parameter for liquefaction, showed decrease with increasing socket length. The numerical analysis results indicate that socket length significantly affects liquefaction. It was also revealed that jet grout columns should restrict ground movement instead of moving with the ground. Additionally, the study investigated the impact of two different unconfined compressive strength parameters. The results showed that the uniaxial compressive strength parameter had a significant effect on liquefaction. In addition, innovative method of cellular jet grout column application was found to be successful in preventing liquefaction.

Keywords: Jet-grout, numerical analyses, liquefaction, mitigation

JET-GROUT KOLON PARAMETRELERİNİN SIVILAŞMA ÜZERİNDEKİ ETKİSİ VE HÜCRESEL JET-GROUT UYGULAMASININ DEPREM PERFORMANSI ÜZERİNE BİR İNCELEME

ÖZ

Zemin sıvılaşması ülkemizde ve dünyada önemli hasarlara neden olan deprem olaylarından biridir. Olası depremlerde meydana gelecek can ve mal kayıplarının önüne geçilmesi amacıyla bu tür sorunlu zeminlerde iyileştirme yapılmalıdır. Bu tekniğin temel prensibi uygulandığı zeminin dayanım parametrelerini iyileştirmesi ve zemin içinde oluşan deformasyonların kısıtlanmasıdır. Bu çalışma iki farklı bölümden oluşmaktadır. İlk bölümde jet grout tekniği ile yapılan zemin iyileştirme çalışmalarında sıvılaşmanın önlenmesi için jet grout kolonlarının soket boyunun etkisi ve jet-grout kolonu tek eksenli basınç dayanımının etkisi sonlu elemanlar yazılımı ile irdelenmiştir. İkinci bölümde ise, yenilikçi bir metot olan hücresel jet grout uygulamasının sıvılaşma üzerindeki başarısı sonlu elemanlar yöntemi kullanılarak değerlendirilmiştir. Analizlerde, sıvılaşma davranışı gösteren zemin efektif gerilme tabanlı UBC3D modeli kullanılarak temsil edilmiştir. Yapılan nümerik analizler sonucunda soket boyunun artmasıyla zemindeki ve kolonlardaki kayma deformasyonlarının azaldığı görülmüştür. Sıvılaşma için anahtar parametrelerden biri olan aşırı boşluk suyu basıncı oranı (R_u) incelendiğinde soket boyunun artmasıyla birlikte R_u parametresinin azaldığı görülmüştür. Nümerik analiz sonuçları soket boyunun, sıvılaşma üzerinde önemli bir rol oynadığını göstermektedir, jet grout kolonların zeminle hareket etmemesi, zeminin hareketini kısıtlaması gerektiği ortaya çıkmıştır. Bu analizlere ek olarak tek eksenli serbest basınç dayanımının etkisi iki farklı dayanım parametresi için incelenmiştir. Analizler sonucunda tek eksenli basınç dayanımı parametresinin sıvılaşma üzerinde etkili olduğu anlaşılmıştır. Ayrıca yenilikçi bir metot olan hücresel jet-grout kolon uygulamasının sıvılaşmayı başarıyla engellediği ortaya çıkmıştır.

Anahtar kelimeler: Jet-grout, sayısal analiz, sıvılaşma, zemin ıslahı.

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CHAPTER ONE

INTRODUCTION

Liquefaction is a soil problem that occurs when the soil loses its strength under cyclic loading, such as earthquakes. This phenomenon was first observed during the 1964 Niigata and Alaska earthquakes. Following the earthquakes, researchers have conducted several studies on the soil liquefaction mechanism and its prevention. Jet grouting is a widely used method for preventing liquefaction.

This study investigates the effects of unconfined compressive strength and socket length of jet grouted columns on liquefaction and examines a new application method, the cellular jet grouting method, using data from a wildlife research site, which is a typical liquefaction soil profile. This site exhibits great similarity to the soil profile of İzmir Atatürk Organized Site. The UBCSAND constitutive model was used to obtain the values of the excess pore water pressure parameter (R_u) and shear strain.

The study's first part utilized the Plaxis 2D finite element software to model jet grout columns with different socket lengths and unconfined compressive strengths. To make the modeling more realistic and increase the reliability of the results, a real earthquake record was used as input motion. The second part of the study modeled the effects of cellular jet grouting, an innovative geotechnical engineering method, on liquefaction using Plaxis 3D finite element software.

1.1 Problem Definition and Aim of Study

In the past decades, a few methods have been developed to reduce the destructive effects of soil liquefaction. These methods include cement injection, stone columns, deep soil mixing, and the most used jet grout columns. The fundamental principle behind these methods is to increase soil stiffness and restrict the development of excessive pore water pressure under repeated loading. Research has demonstrated that the potential for liquefaction in cohesionless soils is related to parameters such as magnitude of seismic acceleration and relative stiffness. Therefore, it is critical to understand the factors that influence liquefaction risk and to evaluate how soil improvement techniques perform under dynamic loading.

The study aims to investigate the effectiveness of the jet grouting method as a mitigation strategy for soil liquefaction problems. Specifically, it examines the effects of unconfined compressive strength and socket length of jet grout columns on liquefaction behavior through a series of comparative analyses. The study focuses on the cellular jet grouting method, an innovative approach in geotechnical engineering. The investigations were conducted using Plaxis finite element software under various scenarios.

1.2 Structure of Thesis

This thesis is structured into five main chapters to comprehensively present the research study:

Chapter one presents an overview of the research, including its purpose and scope of research.

Chapter Two presents a literature review that examines the fundamental concepts of liquefaction, the development of methods, liquefaction mitigation techniques, and previous studies. The chapter discusses current and historical approaches to liquefaction potential assessment and provides researchers with an expanded perspective on the topic.

Chapter three presents the site description, ground conditions, and seismic observations used in the study. It then presents the numerical models, the Plaxis program, and the parameters used in the program. Finally, it describes the establishment of the finite element model, the mesh calibration, and the establishment of the 2D and 3D models. This chapter prepares the basis for the analyses presented in the results section.

Chapter 4 presents the results of the research in detail. In this chapter, the effects of socket length and unconfined compressive strength of jet grouted columns on liquefaction are discussed. In addition, the effectiveness of cellular jet grouting is evaluated under two different earthquake scenarios.

Chapter 5 presents the study's conclusion, including a summary of the findings and recommendations for future research.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

In geotechnical engineering, research on liquefaction has been a focus point for engineers and researchers. This phenomenon can be observed during or after a certain time earthquake ceases. However, in the event of an earthquake, liquefaction may or may not occur, depending on the soil conditions. This phenomenon can greatly reduce the load-bearing capacity of the soil and potentially cause structures to collapse.

Many soil improvement methods have been developed to address these critical soil problems. Among these methods, Deep Soil Mixing (DSM) and jet grouting technologies stand out. Both techniques are based on similar principles: Injecting and mixing binding materials (usually cement-based) into the ground to improve the soil to a specified depth. DSM uses center-hole augers or paddle mixers, while jet grouting uses high-pressure jets to inject this mixture into the ground.

The aim of this literature review is to focus on the basics of liquefaction phenomenon, DSM, and jet-grouting technology and to shed light on how these technologies reduce the risk of liquefaction. In addition, detailed information on the advantages of these techniques, their capacity to reduce liquefaction risk and their success in practice will be presented. The review will cover numerical studies in this field and will also address real-life application examples of these techniques.

2.2 Liquefaction

In the geotechnical earthquake engineering community, earthquake liquefaction is a relatively new concept that came to the attention of the scientific community in the 1960s. However, signs of this phenomenon can be found in many historical earthquakes. In 1964, the direct effects of liquefaction were observed during major earthquakes in Alaska and Niigata. These first observations helped us to better understand the process that causes solids to become liquid.

In 1953, Mogami and Kubo coined the term "liquefaction". While there is debate over what liquefaction is, the generally accepted definition is associated with the buildup of excess pore pressure in undrained conditions. This pressure increase is usually the result of compaction of the soil by some kind of external agent and reduces the soil's stability.

During the 1964 Niigata and 1964 Alaska earthquakes, seismic soil liquefaction caused extensive damage, which accelerated research efforts on understanding the seismic soil liquefaction behavior of soils. Recent earthquake events (e.g., 2011 Tohoku, 2010 Darfield, etc.) have caused widespread liquefaction, resulting in significant loss of life and property, and providing additional lessons to be learned.

2.2.1 Definition of Liquefaction

Earthquake liquefaction describes the process by which soils become liquid-like under seismic effects. This phenomenon was first described by Terzaghi and Peck (1948) and liquefaction refers to the sudden loss of strength in saturated sandy soils. Mogami and Kubo used the term in a similar context in 1953.

The main characteristics of liquefaction are the generation of excess pore water pressures and the drainage conditions of non-cohesive soils. If the soil is not fully saturated, there is no risk of liquefaction, in other words, soils with water above the groundwater level are not at risk of liquefaction. Liquefaction occurs during seismic loading (earthquakes), especially in saturated and non-cohesive soils. Earthquakes cause high-frequency vibrations that tend to compress the soil. This squeezing leads to an increase in the effective stress between soil particles. However, due to the presence of pore water, this increase in pressure is not transmitted between the soil particles, resulting in excessive pore water pressure.

According to a recent definition by Idriss and Boulanger (2008), loose, unconsolidated soils tend to compact under cyclic loading. When the soil is saturated and largely unable to drain during shaking, the effective compressive stress in the soil is reduced, resulting in deformation of the soil layers. Increased pore pressure due to this reduction in effective stress is called liquefaction.

Historically, liquefaction has been observed during large earthquakes, such as in Niigata and Alaska in 1964 (Figure 2.1). However, the potential hazards of liquefaction have also been observed in subsequent years, especially during major seismic events such as the 1999 Kocaeli ,2011 Tohoku and 2023 Kahramanmaraş earthquakes. The main factors affecting the liquefaction phenomenon include earthquake characteristics, soil properties and site topography.



Figure 2.1 Tilting of apartment buildings caused by the 1964 Niigata earthquake (I.M. Idriss & R. W. Boulanger, 2008)

2.2.2 Liquefaction Types

Based on Kramer (1996) “Liquefaction phenomena that result from this process can be divided into two main groups: flow liquefaction and cyclic softening. “

When evaluating liquefaction hazards, both flow liquefaction and cyclic mobility are important factors to consider. While flow liquefaction occurs less frequently in the field, its effects are typically more severe. Cyclic mobility, on the other hand, can occur under a wider range of soil and site conditions and can have varying degrees of effect. Comparison of Flow Liquefaction and Cyclic softening shown in Table 2.1 according

to Schneider and Mayne (2000). Liquefaction examples shown in Figure 2.2 and Figure 2.3

Table 2.1 Comparison of Flow Liquefaction and Cyclic Mobility (Schneider & Mayne, 2000)

	Flow Liquefaction	Cyclic Softening	
		Cyclic Liquefaction	Cyclic Mobility
Loading Conditions	Static or Cyclic	Cyclic with stress reversal	Cyclic without stress reversal
Drainage	Undrained	Undrained	Undrained
Soil Response to Shear	Strain Softening	Strain Softening and Strain Hardening	Strain Softening and Strain Hardening
Controlling Stresses	Static Shear Stress	Static and Cyclic Shear Stresses	Static and Cyclic Shear Stresses
Induced Stress State	In-situ shear stresses greater than minimum undrained shear strength	Effective stress state reaches essentially zero	Zero effective stress does not develop
Failure or Deformation Potential	A sufficient volume of soil must strain soften. Failure can result in slide or flow depending upon internal geometry and stress state.	Strain softened shear modulus can lead to large deformations during cyclic loading. Soils will tend to stabilize upon termination of cyclic loading.	Limited deformations, unless very loose soil results in flow liquefaction.
Soil Types	Any metastable saturated soil; very loose granular deposits, very sensitive clays, and loess deposits	Almost all saturated sands, with limited deformations in clayey soils.	Almost all saturated sands, with limited deformations in clayey soils.



Figure 2.2 A small flow slide along the shore of Lake Merced in San Francisco in 1957.(Kramaer,1996)



Figure 2.3 Lateral spreading of very flat ground toward the Motagua River following the 1976 Guatemala earthquake. Note orientation of ground surface cracks parallel

In the NCEER workshop on the assessment of soil liquefaction resistance (1997), flow liquefaction is defined as follows (Youd & Idriss, 2001). “Phenomenon in which the equilibrium is destroyed by static or dynamic loads in a soil deposit with low residual strength. Residual strength is defined as the strength of soils under large strain levels.”

Once instability is triggered, the soil, now considered unstable, undergoes a significant loss in strength and undergoes significant deformations under constant stresses, the so-called steady state (see Figure 2.4). During this process, the confining pressure decreases, but not necessarily to zero.(Mital et al., 2017)

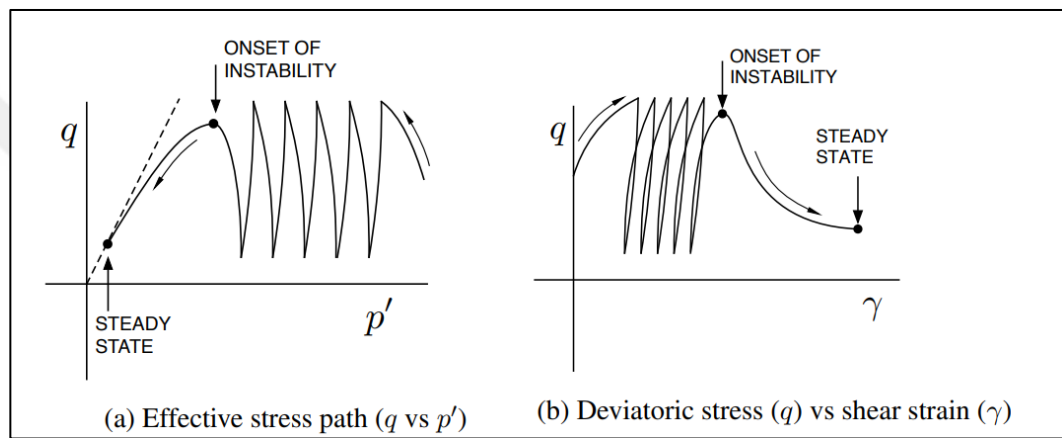


Figure 2.4 Schematic for flow liquefaction during cyclic loading: q is deviatoric stress and p' is the effective pressure. The dotted line is the steady state line. (Mital et al., 2017)

Cyclic mobility can occur when the applied shear stress is less than the residual or steady-state strength of the soil. It is important to note that this phenomenon happens under specific conditions (Vaid & Chern, 1983). This case requires a gradual decrease in shear stiffness as the effective pressure decreases with each loading cycle, resulting in the accumulation of stress. Unlike the apparent starting point in flow liquefaction instability, cyclic mobility has no clear starting point (Kramer, 1996). The accumulation of stress will be accelerated when the effective stress path is aligned with the steady-state line. The steady-state line is a collection of nodes showing the relationship between void ratio, shear stress, and confining pressure corresponding to a soil at steady state (Kramer 1996). Initiation of liquefaction, whether flow liquefaction or cyclic mobilization, is empirically determined at 3% shear stress (I.M.

Idriss & R. W. Boulanger, 2008). Figure 2.5 presents schematic diagrams illustrating the phenomenon of cyclic mobility.

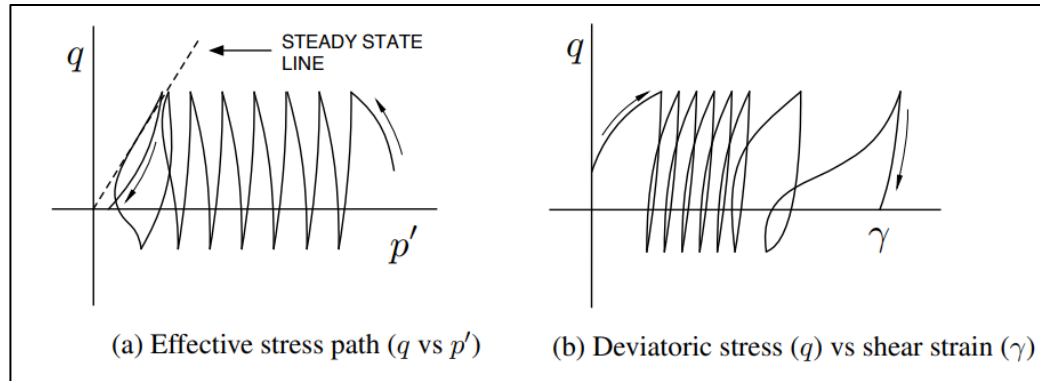


Figure 2.5 Schematic for cyclic mobility during cyclic loading: q is deviatoric stress and p' is the effective pressure. The dotted line is the steady state line. (Mital et al., 2017)

2.2.3 Characteristics of Liquefiable Soils

Liquefaction typically occurs in sandy soils and non-plastic silts, while it is relatively unlikely in plate-shaped silts and clays with plastic properties. Moreover, sensitive clays may suffer a reduction in stiffness and strength under the influence of repeated shear stresses, resulting in liquefaction-like effects. (Wang, 1979) introduced methods for evaluating the probability of liquefaction in soils high in fine content, now known as the Chinese criteria.

Before the earthquakes at Haicheng in 1975 and Tangshan in 1976, the susceptibility to seismic soil liquefaction was attributed only to saturated "clean sandy soils" with a minimum fines content. However, after these earthquakes, some examples of ground failure (Wang, 1979) lead on the fact that cohesive fine-grained soils are also susceptible to liquefaction. This situation continued especially after the 1989 Loma Prieta, 1994 Northridge, 1999 Adapazarı, 2004) and 1999 Chi-Chi earthquakes, reinforcing the idea that both silty and clayey soil layers can exhibit cyclic mobility and cyclic liquefaction soil behavior.

The evaluation of susceptibility in fine-grained soils has evolved from the basic Chinese criteria to more advanced methods established by experts with the advancement of seismic soil liquefaction engineering. Seed and Idriss (1982),

Andrews and Martin (2000), Seed et al. (2003), Bray and Sancio (2006) and Boulanger and Idriss (2006). In the following sections, some of the commonly adopted criteria will be reviewed.

Wang (1979) investigated the liquefaction events after the 1975 Haicheng and 1976 Tangshan earthquakes and determined that certain soil mixtures are susceptible to liquefaction by creating a database. According to Wang (1979), soil mixtures with clay particle size less than 0.005 mm and water content (w/LL) greater than 0.9 are more susceptible to liquefaction.

Seed and Idriss (1982) analyzed the same database and stated that soils with high clay content can liquefy only under certain conditions, conditions given below:

- i) the percentage of particles with particle size less than 0.005 mm is less than 15%
- ii) $LL < 35$
- iii) $w/LL > 0.90$

These criteria were determined by a falling cone device commonly used in Chinese practice and are referred to as the "Chinese Criteria".

Andrews and Martin (1999) suggested improved criteria for identifying soils more sensitive to liquefaction using clay content and fluid limit parameters. In these criteria, clay particle size was set to 0.002 mm and the critical LL value was reduced to 32. Furthermore, according to the criteria in Table 2.2, soils with a clay content (0.002 mm%) of less than 10% and a liquid limit $< 32\%$ are potentially liquefiable, while soils with a clay content of 10% and above require further study.

The Chinese Criteria and subsequent revised versions were applied in practice for more than 2 decades, but doubts have been raised since the mid-1990s with an increasing number of case histories and high-quality test data. Recent studies have emphasized that clay mineralogy as well as particle size is important in determining liquefaction potential. It was also noted that the Chinese Criteria were developed based on data from only two earthquakes, the 1975 Haicheng and 1976 Tangshan earthquakes. Therefore, they need to be evaluated for different earthquakes.

Table 2.2 Liquefaction susceptibility criteria (D. C. Andrews & G. Martin, 1999)

	Liquid Limit < 32%	Liquid Limit $\geq$ 32%
Clay Content (< 0.002 mm) < 10%	Potentially Liquefiable	Further studies required considering plastic non-clay sized grains
Clay Content (< 0.002 mm) $\geq$ 10%	Further studies required considering non-plastic clay sized grains	Non-Liquefiable

According to the study by (J. Bray et al., 2001) they found that the Chinese criterion is insufficient. The Chinese criterion is related to the number of clay-sized particles in the soil. However, recent research has shown that non-plastic fine-grained soils can also liquefy, and the cyclic response of soils is largely controlled by the Plasticity Index (PI).

In 2003, Seed et al. suggested a new set of liquefaction evaluation criteria based on cyclic tests and data collected after the 1999 Adapazarı and Chi-Chi earthquakes. These new criteria classify saturated soils according to specific PI, Liquid Limit (LL) and w_c/LL values. Soils with $PI < 12$, $LL < 37$ and $w_c/LL > 0.80$ are considered susceptible to liquefaction, while soils with $PI < 12$, $LL < 20$, $LL < 37$ and $w_c/LL > 0.85$ require further testing and are considered potentially liquefiable (R. B. Seed et al., 2003). All other soils are not considered susceptible to liquefaction. seed and its team's criteria are shown schematically in figure 2.6.

Seed et al.'s proposed criteria were evaluated based on a specific definition of liquefaction. The model's error rate was found to be 19.6%. This error rate indicates that the criteria cannot accurately predict the liquefaction potential under certain conditions.

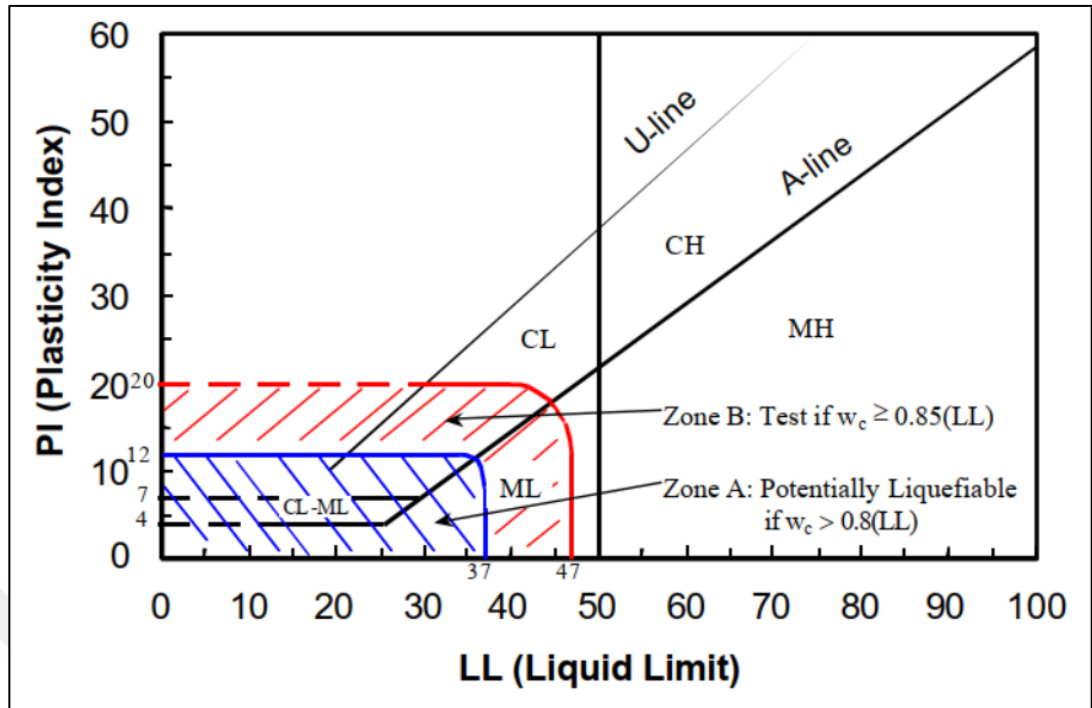


Figure 2.6 Schematic for cyclic mobility during cyclic loading: q is deviatoric stress and p' is the effective pressure. The dotted line is the steady state line. (Mital et al., 2017)

In 2006, (J. D. Bray & Sancio, 2006) conducted a study to reevaluate soil liquefaction susceptibility and develop a more precise analysis method. Their objective was to develop a more accurate understanding of soil liquefaction by performing cyclic tests on undisturbed fine-grained soil samples from the Adapazarı.

The soil samples in the test programs were consolidated isotopically to a compressive stress of 50 kPa and then subjected to CSR levels of 0.3, 0.4, and 0.5. Cyclic loading was sustained until achieving 4% double amplitude axial deformations, which was considered the criterion for liquefaction triggering. According to Bray and Sancio (2006), determining the liquefaction susceptibility of fine-grained soils depends on critical parameters such as the Plasticity Index (PI) and the ratio of water content to Liquid Limit (LL) (w_c/LL). Soils with a Plasticity Index (PI) below 12 and w_c/LL (water content divided by liquid limit) greater than 0.85 are considered susceptible to liquefaction. For soils with a PI below 12 and between 12 to 18, and a w_c/LL greater than 0.80, further testing is recommended. Soils with a PI above 18 were

found to be non-liquefiable at low effective stress levels due to their high clay content, as depicted in Figure 2.7.

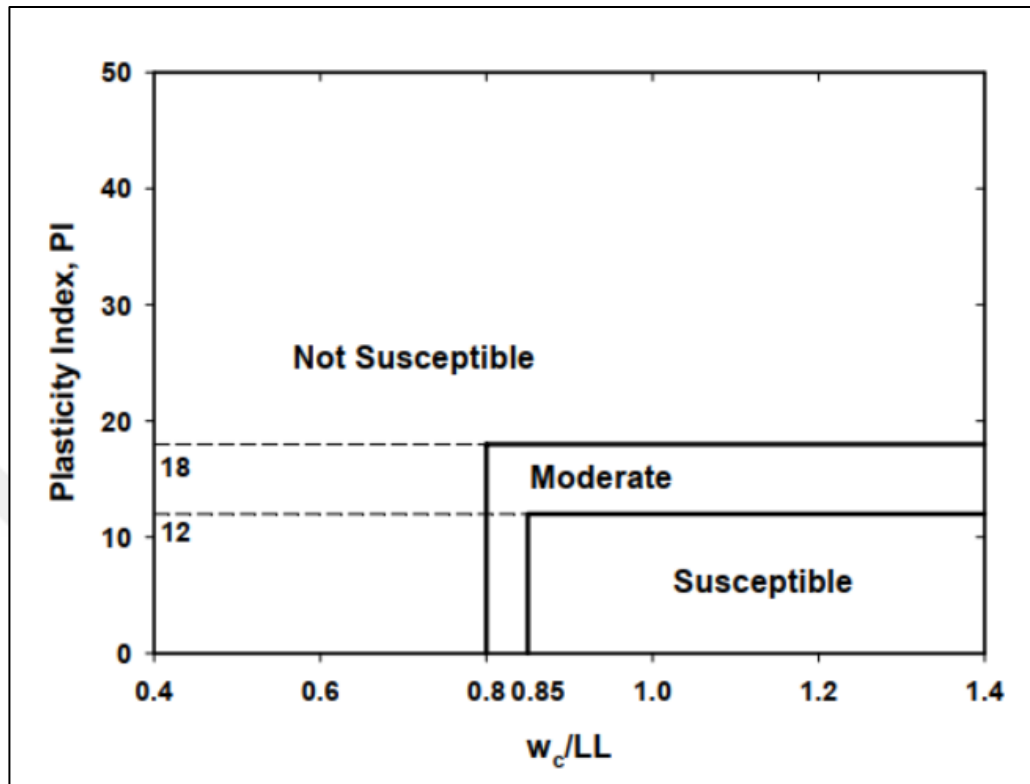


Figure 2.7 Criteria for liquefaction susceptibility of fine-grained sediments. (J. D. Bray & Sancio, 2006)

In 2006, Boulanger and Idriss conducted a comprehensive study on the liquefaction susceptibility of fine-grained soils. The study was based on the categorization of the cyclic response of soils into two categories, "sand-like" and "clay-like". The researchers found that this categorization is important in determining the potential for liquefaction, as sand-like soils have lower cyclic resistance ratio (CRR) values than clay-like soils.

Boulanger and Idriss aimed to objectively assess soil liquefaction potential, independent of in-situ conditions. The study adheres to conventional academic structure and maintains formal register. Adherence to spelling, grammar, and citation standards is ensured. The authors differentiate between sand and clay-like soil responses and consider hysteretic stress-strain cycles of fine-grained soils, the presence of a zone of zero shear resistance, and the generation of porous water

pressure. They stated that the Plasticity Index (PI) is the sole parameter that can account for the variations seen in the responses. A diagram illustrates this methodology Figure 2.8.

Furthermore, they noted a clear shift from sandy soil behavior to clay soil behavior at Atterberg limits, with PI serving as an indicator of this transition. The researchers reported that soil behaves in a "sand-like" manner when the PI value is below 3 and in a "clay-like" manner when it exceeds 7, with a gradual transition in behavior occurring at PI values between 3 and 7. However, it was stressed that the w/LL value cannot provide accurate details on soil behavior. Demonstrated that while w_c/LL and PI criteria play a significant role in ascertaining the liquefaction susceptibility of fine-grained soils, they are not conclusive. The liquefaction susceptibility of shallow, non-plastic silts and clayey silts is a crucial matter, with research indicating that these soils can liquefy under intense seismic loading. Moreover, clayey silts and silty clays with intermediate plasticity may liquefy under intense shaking and loading cycles. Sensitive soils may lose strength severely due to earthquake-induced stress.

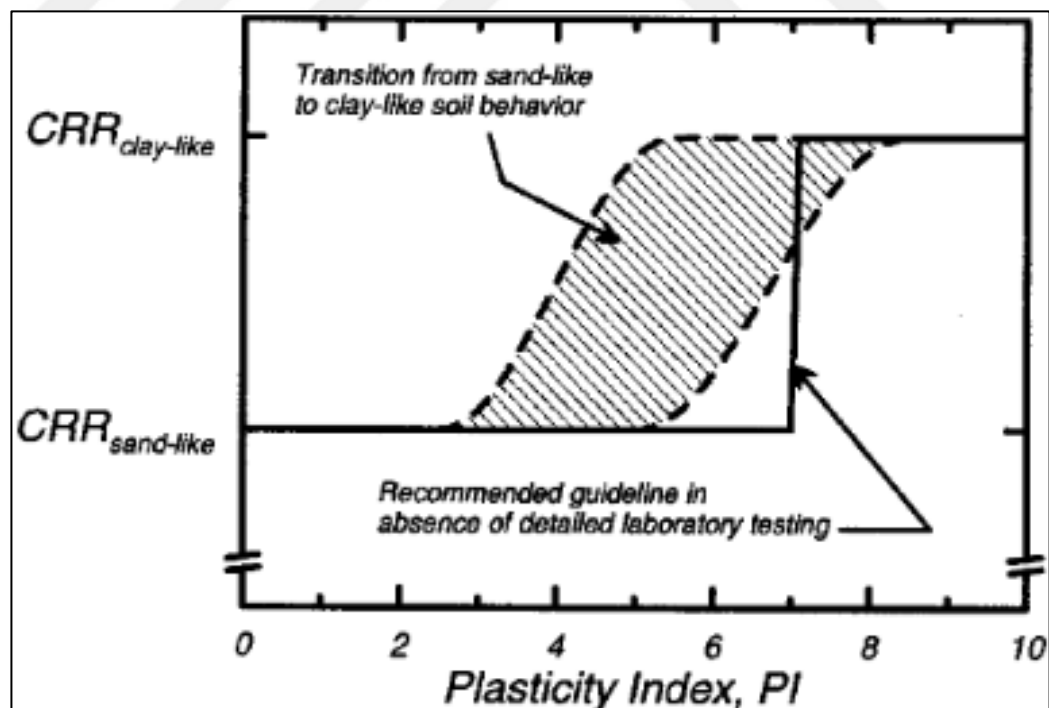


Figure 2.8 Recommended limit values for sand-like and clay-like soil behavior due to plasticity index. (Boulanger & Idriss, 2006)

2.2.4 Determination of Liquefaction Potential

The "Simplified Method," developed by Seed and Idriss in 1971, analyzes soil liquefaction risk utilizing Standard Penetration Test (SPT) findings. The method calculates the cyclic shear stress ratio (CSR) induced by the earthquake and subsequently compares this value with the cyclic shear strength ratio (CRR) to determine the soil's liquefaction resistance. Specific parameters such as peak horizontal ground acceleration (a_{max}), gravitational acceleration (g), total vertical stress (σ_v), effective vertical stress (σ'_v), and stress reduction coefficient (r_d) are used in the calculation of the CSR to determine the factor of safety against liquefaction. The resulting cyclic stress ratio (CSR) is then compared with the cyclic resistance ratio (CRR) to obtain the factor of safety (F_s), calculation of safety factor shown in Eq 2.1. A magnitude scaling correction factor (MSF) is used when calculating F_s .

$$F_s = \frac{CRR}{CSR} * MSF \quad (2.1)$$

The value of the F_s indicates the liquefaction hazard of the soil. If the F_s is less than 1, there is a risk of liquefaction; if the F_s is greater than 1, there is no risk of liquefaction. The cyclic stress ratio (CSR) in the above expression can be calculated with Eq 2.2.

$$CSR = 0.65 \left(\frac{a_{max}}{g} \right) \left(\frac{\sigma_v}{\sigma'_v} \right) r_d \quad (2.2)$$

a_{max} = Peak horizontal ground acceleration at the ground

g = Gravitational acceleration (9.81 m/s²)

σ_v = Total vertical stress

σ'_v = effective vertical stress

r_d = Stress reduction factor

Stress reduction coefficient can be found using Eq 2.3a and Eq 2.3b below proposed by (R. B. Seed et al., 2003)).

$$r_d = 1 - 0,00765z \text{ (For } z \leq 9.15\text{m)} \quad (2.3a)$$

$$r_d = 1,174 - 0,0267z \text{ (For } 9.15 \leq z \leq 23\text{m)} \quad (2.3b)$$

z = Depth from the surface in meters

Existing static shear stresses can have a significant effect on the liquefaction capacity of soils in sloping areas or in soil layers beneath large structures. Seed's 1983 study emphasized the importance of including these initial shear stresses and high surrounding pressures in liquefaction analyses and suggested a correction method for the cyclic shear stress ratio (CSR) to account for these factors(Seed et al., 1983).

$$(CSR_{land})_{a,\sigma} = (CSR_{land})_{a=0,\sigma} = 0, \sigma < 100kPa K_a K_\sigma \quad (2.4)$$

This correction enables more precise liquefaction assessment under high ambient pressures. Later, Idriss and Boulanger (2006) expressed these correction factors mathematically to enhance the correction factor for liquefaction analysis Eq 2.6 and Eq 2.7 . These corrections consider the impact of static stresses and environmental effects, thus increasing the soil liquefaction analysis's realism and reliability.

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_v}{p_a}\right) \leq 1.0 \quad (2.5)$$

$$C_\sigma = \frac{1}{18,9 - 2,55\sqrt{N_{1,60}}} \quad (2.6)$$

The Simplified Method to evaluate liquefaction risk uses the cyclic resistance ratio (CRR) to measure the liquefaction resistance of the soil. Calculating CRR is predicated on the number of impacts (SPT-N) derived from Standard Penetration Test (SPT) findings. However, due to the different experimental setups of the SPT, there may be variations in the number of impacts obtained. Various corrections are made to these impact numbers, considering factors such as the experiment's energy ratio, pit diameter, rod length, and sampling method. The correction factors C_N , C_E , C_B , C_R , and C_S are employed in these calculations to arrive at the corrected SPT pulse number ($N_{1,60}$) (Eq. 2.7). This corrected pulse number is utilized to calculate the CRR, leading to a more reliable determination of the soil's liquefaction resistance. This method is crucial in the risk analysis of soils susceptible to liquefaction and offers the necessary adaptability to accommodate variations in site conditions.

$$N_{1,60} = N C_N C_R C_S C_B C_E \quad (2.7)$$

N = Number of SPT blow counts

C_N = Correction factor of according to vertical geological stress

C_E = Correction factor of the energy rate of the rammer

C_B = Correction factor of the diameter of the drilling pit

C_R = Correction factor of stem bar length

C_S = Correction factor of according to the method of sampling

The correction factor for the vertical geological stress (C_N) is determined by the equation developed by Liao and Whitman (1986) It can be computed using Eq 2.8.

$$C_N = \sqrt{\frac{P_a}{\sigma'_{v}}} \leq 1,7 \quad (2.8)$$

In Eq 2.8, P_a = constant reference stress of 100 kPa and σ'_v is the effective vertical stress. According to Youd et al. (2001a), the CN value should be less than or equal to 1.7.

Seed et al. (1985) conducted a study on liquefaction in sand soils that were impacted by an earthquake with a moment magnitude of 7.5 in 1985. The study analyzed the results of Standard Penetration Tests (SPT) in areas where liquefaction had occurred and where it had not. The minimum cyclic stress ratios required for liquefaction to occur were determined by the researchers and are shown graphically in Figure 2.9. This important study was later revised by NCEER in 1997. This study has provided crucial insights into liquefaction risk assessment and enhanced understanding of soil behavior.

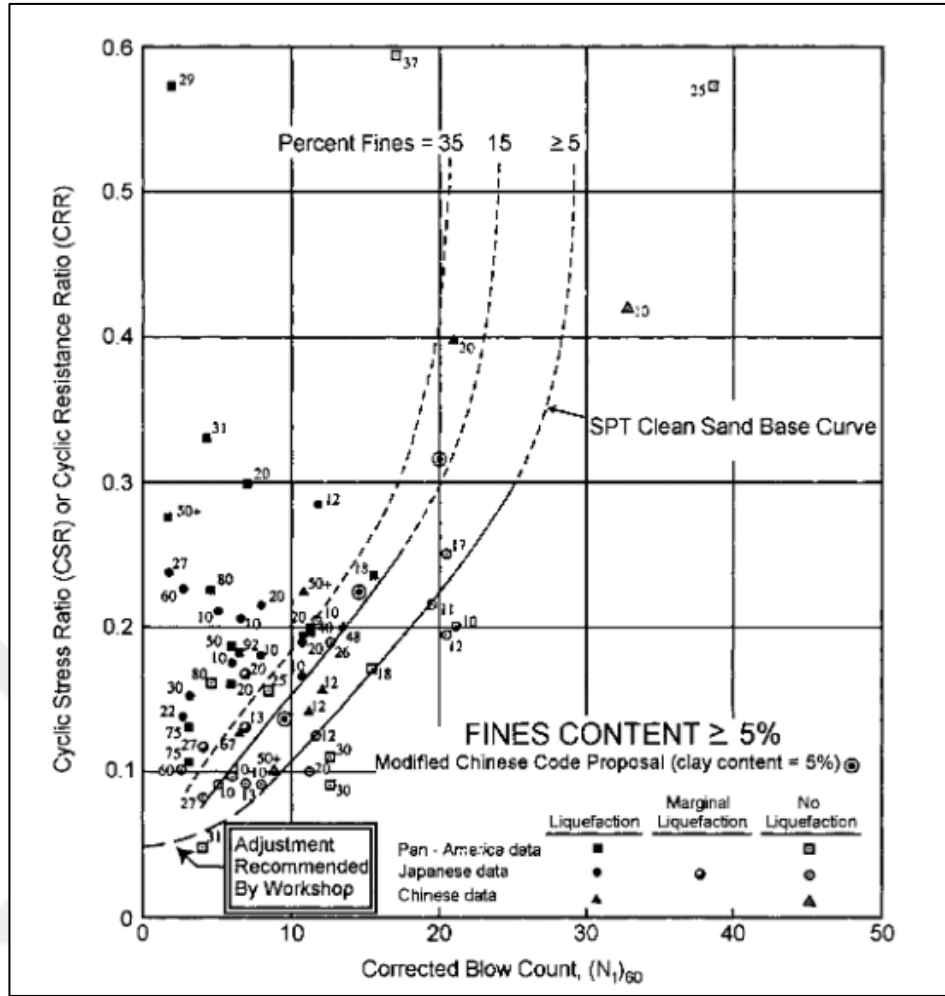


Figure 2.9 SPT Clean-Sand Base Curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Case Histories (Youd & Idriss, 2001)

The mathematical expression of the empirical curve drawn for the cyclic resistance ratio (CRR) for an earthquake magnitude of MW = 7.5 is given in Eq 2.9.

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10 * (N_1)_{60cs} + 45]^2} - \frac{1}{200} \quad (2.9)$$

This statement is valid for cases where the corrected Standard Penetration Test (SPT) impact number is less than 30, because clean sands (those with very few fines) with $(N_1)_{60}$ values of 30 and above are considered quite firm, and such soils are

generally classified as not at risk of liquefaction. Seed et al. (1985) found that the fine grain ratio is directly proportional to liquefaction resistance; that is, as one increases, the other increases, and as one decreases, the other decreases. This relationship has been expressed by Youd and Idriss (2001) in equation 2.10. In this equation, $(N_1)_{60cs}$ represents the number of SPT pulses corrected for fines. This correction is calculated by factors α and β , which vary with the fines content.

$$(N_1)_{60cs} = \alpha + \beta (N_1)_{60} \quad (2.10)$$

Where α and β = coefficients determined from the following relationships:

$$\alpha = 0 \text{ for } \%FC \leq 5\% \quad (2.11)$$

$$\alpha = \exp[1.76 - (1.90/FC^2)], \text{ for } 5\% < FC < 35\% \quad (2.12)$$

$$\alpha = 5.0, \text{ for } FC \geq 35\% \quad (2.13)$$

$$\beta = 1.0, \text{ for } FC \leq 5\% \quad (2.14)$$

$$\beta = 0.99 + (FC^{1.5}/1.000), \text{ for } 5\% < FC < 35\% \quad (2.15)$$

$$\beta = 1.2, \text{ for } FC \geq 35\% \quad (2.16)$$

The cyclic resistance ratio (CRR) values used to evaluate the liquefaction potential of the soil are valid for 7.5 magnitude earthquakes. However, a method proposed by Youd and Idriss (2001) indicates that these values can be adjusted for earthquakes with magnitudes other than 7.5. This adjustment is made using the moment magnitude scaling factor (MSF), and the MSF is calculated according to the moment magnitude (Mw). Equation 2.17 is used to calculate the MSF value.

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \quad (2.17)$$

2.3 The Jet Grouting Method

2.3.1 Introduction

The jet-grout method fundamentally relies on the principle of injecting one or more fluids (such as cement slurry, air, water) into the ground under high pressure.

In this method, fluids are injected under high pressure while being pulled from the application depth to the ground surface through a steel pipe, which has been positioned down to the desired depth. The applied jet disperses perpendicularly to the pipe axis, creating various interactions with the soil mixing and fragmentation, some degree of erosion and surface transport, and penetration into the soil.

In this process, the injected cement slurry (a mixture of water and cement) undergoes hydration over time within the soil. This hydration ultimately forms a hardened mass combined with the soil. Commonly, these hardening structures possess a cylindrical geometry and are referred to as “jet grout columns.” Nowadays, numerous studies and patents for various construction methods and equipment are conducted and proposed to increase the diameters of these columns and create as homogeneous column geometry and material as possible.

The popularity of the jet injection technique in contemporary soil improvement methodologies is preserved for several reasons. Firstly, this technique has the capacity to form large-diameter rigid columns within the soil, without damaging the surrounding ground, following a minor drilling operation. Secondly, by strategically positioning the formed columns, it is possible to produce high-strength and impermeable structural elements (such as curtains, walls, shafts, composite masses, etc.) (Figure 2.10). Lastly, the equipment used for the jet injection method tends to be lighter and smaller in volume compared to those used in other methods, enabling its application even in challenging environmental conditions.

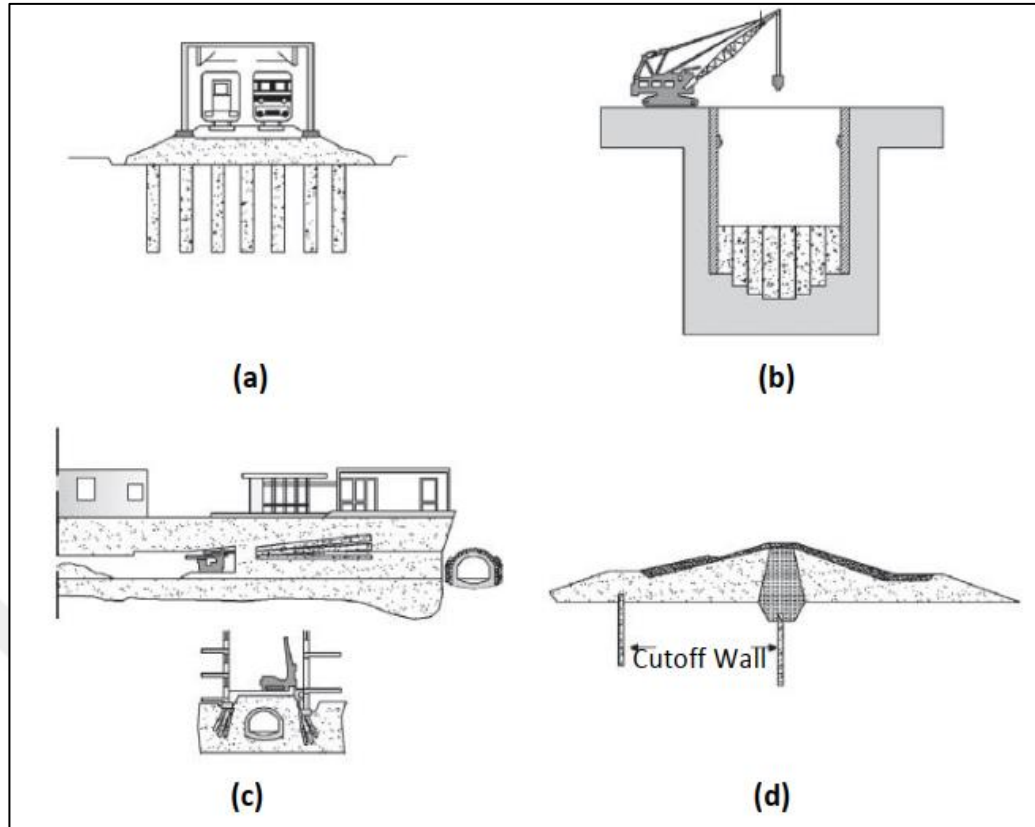


Figure 2.10 Typical jet injection applications: (a) improvement of fill base soils, (b) bottom sealing plugs in excavation-bracing systems, (c) temporary tunnel support systems, and (d) sealing curtains in dams (Croce et al., 2014)

2.3.2 Methods and Technologies

Jet column formation and jet grouting applications exhibit a comprehensive methodology in the field of soil improvement and stabilization. In this technique, rods that are pipes attached to each other, conducting single, double, or triple fluid, are utilized. Fluids within these rods are injected into the ground through a monitor located at the tip of the rod, which houses one or more small-diameter nozzles that will inject the fluid into the ground under high pressure.

The jet grout method has adapting capacity variety of ground conditions. Firstly, holes are drilled with a certain diameter and depth, and these holes are generally between 120 mm and 150 mm, but in some cases, they can be up to 300 mm. Moreover, the liquids used during these processes, such as water, air, cement slurry or foam, carry

As a result, jetting technology and methods supply flexible and suitable solutions in many areas where ground conditions and the general requirements of the project are challenging. The developed technologies and methodologies allow engineering applications to be conducted more efficiently, sustainably, and safely.

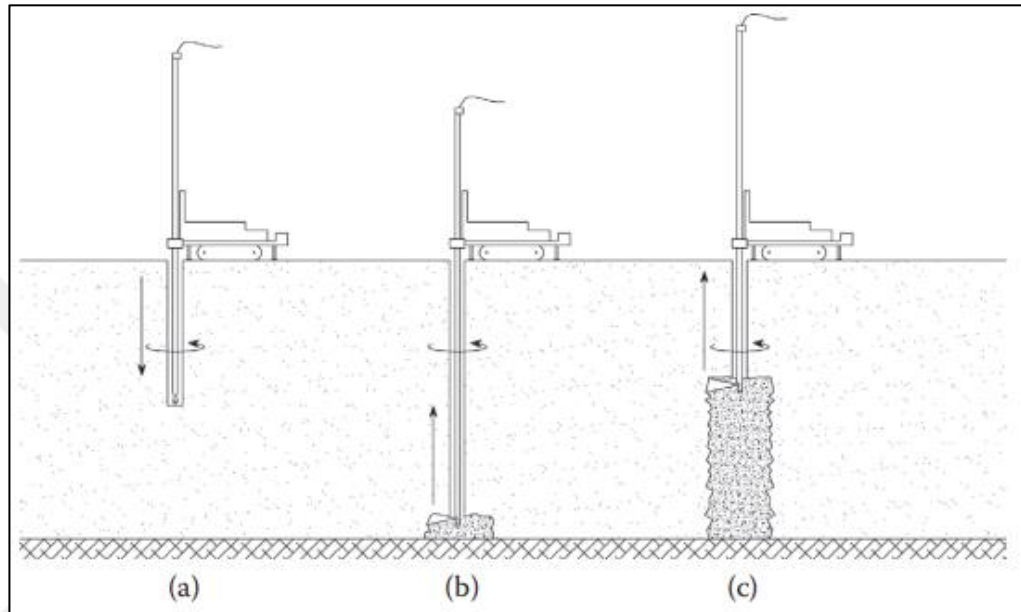


Figure 2.12 Typical jet grouting procedure: (a) drilling; (b and c) jet column formation. (Croce et al., 2014)

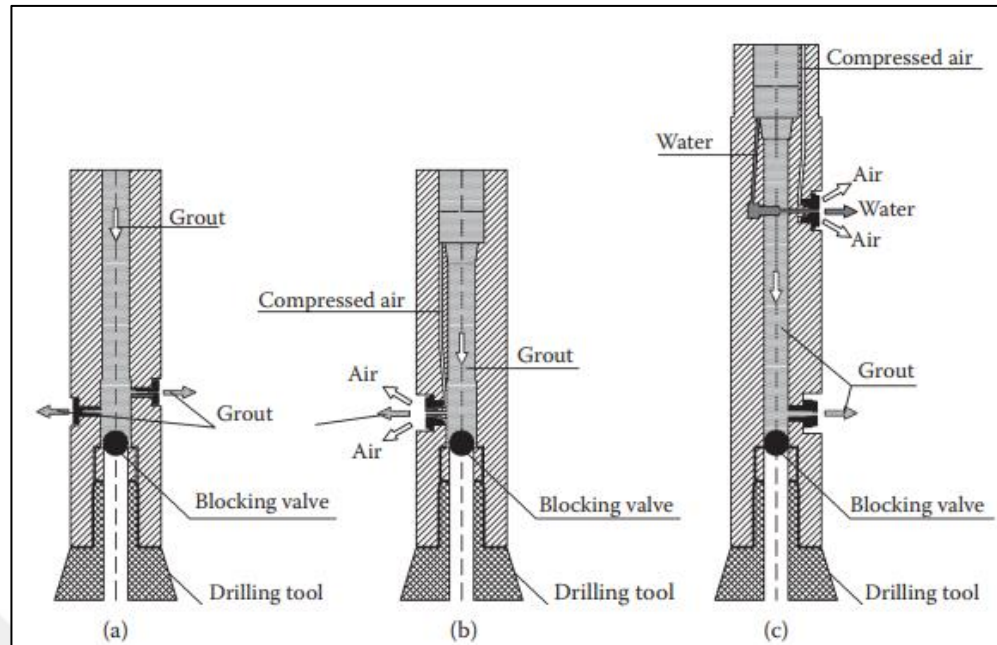


Figure 2.13 Schematic drawings of the monitors: (a) single fluid, (b) double fluid and (c) triple fluid (Croce et al., 2014)

2.3.3 Treatment Parameters

The diameter, strength and homogeneity of jet grout columns depends on basic improvement parameters. These are explained below.

- Geometric parameters: Such as the number of nozzles, nozzle diameter and lifting speed directly affect the characteristics of jet grout procedure.
- Kinematic parameters: Water/cement (w/c) ratio, time step and rotation speed determine the placement and spreading of material during process.
- Grout Parameters: The flow rate and pressure of the grout fluid are critical factors that affect the properties of material and how it is spreading.

For instance, the basic parameters provided by (Croce et al., 2014) can be summarized in Table 2.3.

Table 2.3 List of fundamental treatment parameters (Croce et al., 2014)

Parameter	Definition	Symbol	Unit	
			S.I. unit	Practical unit
Geometrical	Number of nozzles	M	–	–
	Nozzle diameter	d	m	mm
Kinematic	Lifting step	$\Delta s \Delta t$	m s	cm s
	Time interval per step	ω	rad/s	round/min
	Rotational velocity	W/C	–	–
	W-C ratio by weight			
Injected fluids	Fluid pressure ^a	p_g, p_w, p_a	MPa	bar
	Fluid flow rate ^a	Q_g, Q_w, Q_a	m ³ /s	L/min

^a the pedex indicates, respectively, grout (g), water (w) and air (a).

Each jet grout method (single, double, and triple fluid) has a unique set of parameters associated with the injected fluids. Some parameters are interdependent and cannot be selected independently.

Table 2.4 provides typical ranges of treatment parameters. These ranges are indicative, as values outside of them may be chosen in specific cases. Additionally, technological developments in pumping equipment, monitors, nozzles, and procedures are very rapid, and the ranges shown in the Table 2.4 are likely to change significantly in the near future (Paolo Croce et al., 2014).

Table 2.4 Typical values of treatment parameters (Croce et al., 2014)

Treatment parameter	Symbol	Unit	System		
			Single fluid	Double fluid	Triple fluid
Lifting step	Δs	mm	40–50	40–80	40–100
Average lifting speed	v_r	mm/s	4–10	1–8	0.5–5
Rotational velocity	ω	rpm	5–40	3–30	1–40
Nozzle diameter	d	mm	2–8.0	2–8	2–8
Number of nozzles	M	–	1–2	1–2	1–2
Grout pressure	p_g	MPa	30–55	20–40	2–10
Air pressure	p_a	MPa	–	0.5–2.0	0.5–2.0
Water pressure	p_w	MPa	–	–	20–55
Grout flow rate	Q_g	L/s	2–10	2–10	2.0–5
Air flow rate	Q_a	L/s	–	200–300	200–300
Water flow rate	Q_w	L/s	–	–	0.5–2.5
W-C ratio by weight	W/C	–	0.60–1.25	0.60–1.25	0.40–1.0

2.4 Liquefaction Mitigation by Means of High Modulus Soil-Crete Columns

2.4.1 Jet Grout

This section describes research into the advancement of jet grouting treatment techniques. The primary objective of treatment methods to prevent liquefaction is to densify the soil and enhance its shear stress capacity. By this method, the generation of excess pore water pressure in the treated soil during ground movements such as earthquakes is reduced.

The phenomenon of liquefaction in loose sandy soils can be effectively treated through jet grouting. Jet-grouted columns have become a popular method to reduce liquefaction risks in geotechnical engineering applications. Studies by Rayamajhi et al. (2014) Rayamajhi et al. and Martin et al. (2004) have emphasized that jet-grouted columns demonstrate greater resistance to earthquake-induced shear stress in soil. This significantly reduces the potential liquefaction and impact of soil with high modulus columns. Furthermore, the following sections will provide details on some studies.

Cooke (2000) investigated the application of jet grouting on an embankment slope under existing highway bridges. Results showed that the jet grouted soil structure had the ability to resist liquefaction and liquefaction-induced softening during seismic movement. Deformations were restricted depending on the stiffness of the jet grouted soil. The untreated soil is significantly less stiff than the jet-grouted soil. This study highlights a crucial finding that the shear strength capacity of jet-grouted soil relies on the strength of the grout mortar used.

2.4.2 Deep Soil Mixing (DSM)

This section explores the use of deep mixing (DSM) in mitigating earthquake-induced liquefaction. DSM proves to be an essential technique in preventing liquefaction, particularly in fine-grained soils with a high potential for liquefaction. By constructing soil-cement columns, walls, and grids, the method enhances soil structure, increasing its stability under seismic loading. Consequently, it significantly reduces the risk of soil liquefaction during an earthquake.

In previous earthquakes, notable soil improvements made with the DSM method have been documented. Specifically, the effectiveness of the DSM grid in protecting the pile foundations of a 14-story waterfront pier building during the 1995 Kobe Earthquake serves as a concrete example of this method's success. Similarly, the ability of DSM-treated structures and infrastructure to resist liquefaction during the 2011 Tohoku Earthquake is further testament to the substantive capabilities of this technology. These cases offer valuable insights into the successful application of DSM treatments to soil prone to liquefaction. Specifically, DSM treatments play a crucial role in protecting the integrity of structures and infrastructures located in high-risk liquefaction areas. The effective implementation of the DSM method is enabled by advanced analysis techniques in structural and geotechnical engineering and comprehensive evaluation of ground conditions.

2.4.2.1 Liquefaction Mitigation by Columns

Preventing the initiation of liquidation is a goal that can be reached by using DSM columns. The effectiveness of DSM columns depends on the characteristics of the columns, the characteristics of the liquefaction-prone layers, the initial static loading conditions, and the ground motion characteristics (e.g., frequency content, density). The unit cell analysis method can be used to evaluate the effectiveness of DM columns when the foundation area is relatively large, and the ground surface is approximately flat.

Various DSM configurations have been used to estimate the reduction of seismic shear stresses applied to the local soil by assuming shear strain compatibility between the local soil and the DSM columns. Baez (1995) proposed this assumption for vibro-displaced stone columns with shear moduli only a few times that of the local soil, which was later extended to much stiffer soil-cement treatments (H. Durgunoğlu, 2005) However, as the difference in shear modulus between the local soil and the DSM columns increases, the shear strains can become increasingly unequal.

Several studies have demonstrated that the tilting and rotation of separate columns is not as effective as anticipated based on the compatibility of shear strain and does not reduce the seismic shear stresses in the local soil. Goughnour et al., (1998)

examined the mechanical response of an elastic column in elastic soil using analytical expressions and concluded that columns encountered in practical settings will not substantially decrease shear stresses in soil for column length-to-diameter ratios.

Rayamajhi et al. (2014) conducted a study on the distribution of shear stresses between DSM columns and the native soil, and developed a relationship in which the columns reduced the mean cyclic stress ratio (CSR) in the native soil (Figure 2.14). Additionally, centrifugal model tests were performed, which showed no significant effect of DSM columns on reducing excess pore pressure in the native soil Rayamajhi et al. (2015) (Figure 2.15). These tests demonstrated that discrete columns are not effective in preventing liquidation triggering or soil settlement, but have the potential to support the structures above them.

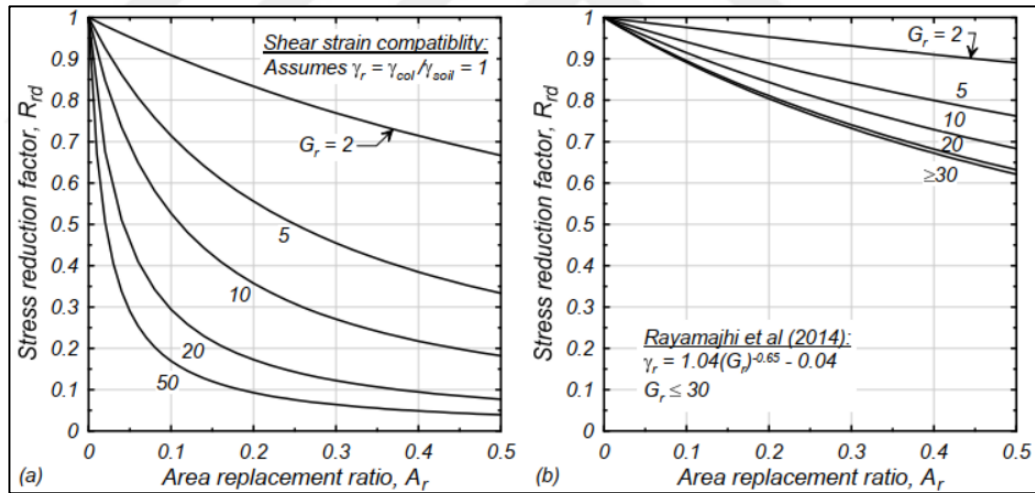


Figure 2.14 Shear stress reduction factor (R_{rd}) for periodic arrangement of columns based on: (a) assumption of shear strain compatibility, and (b) relationship by Rayamajhi et al. (2014)

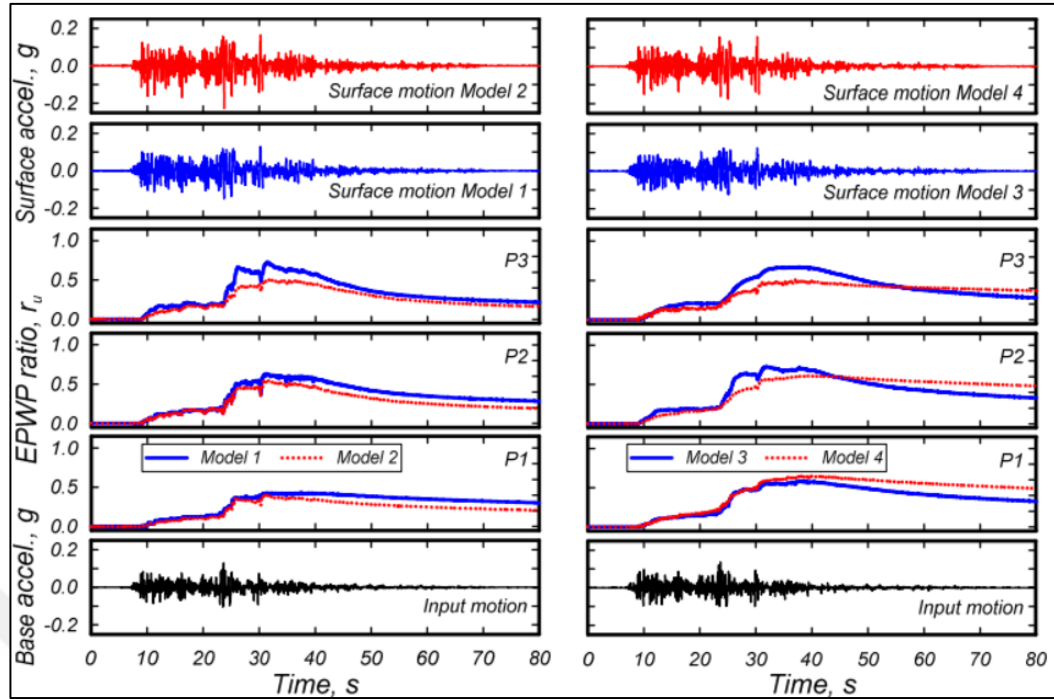


Figure 2.15 Acceleration and excess pore pressure time histories for the same shaking motion for centrifuge models of level, saturated sand with 8 m (prototype) of loose sand overlying either dense sand (models 1 and 2) or cemented sand (models 3 and 4)

The research conducted by Demir and Özener (2020), involved 2D and 3D nonlinear dynamic analyses of the centrifuge model tests by Rayamajhi et al. (2015) and showed that the DSM columns did not significantly reduce the dynamic shear stresses or the generation of excess pore pressure in the soil.

These findings suggest that DSM columns may not be effective in preventing liquidaion triggering but can support the structures above them if their lateral movement is limited.

2.4.2.2 Liquefaction Mitigation by Grid Shape

DSM grids and columns are important in soil engineering for hardening the soil and preventing liquefaction under seismic conditions. The structures' effectiveness depends on several factors, such as the physical properties of the grids and columns, the characteristics of the liquefiable layers, the initial static load conditions, and the ground motion characteristics. Researchers have proposed several analysis and design approaches in this field. This section presents a summary of these studies.

The study by Nguyen et al. (2013) established relationships to decrease the mean shear stress of soil within grids using linear-elastic 3D dynamic analyses (Figure 2.16). These analyses considered the thickness, height, spacing, and liquefiable layer thicknesses of the grid walls. Findings indicate that the height-to-width ratio (H/W) of the grid modifies according to the location (horizontal and vertical) within the treatment interval and the frequency content of the input motion (Figure 2.17) (Figure 2.18). Analyses of harmonic input motions demonstrated how the frequency of the input modifies this phenomenon.

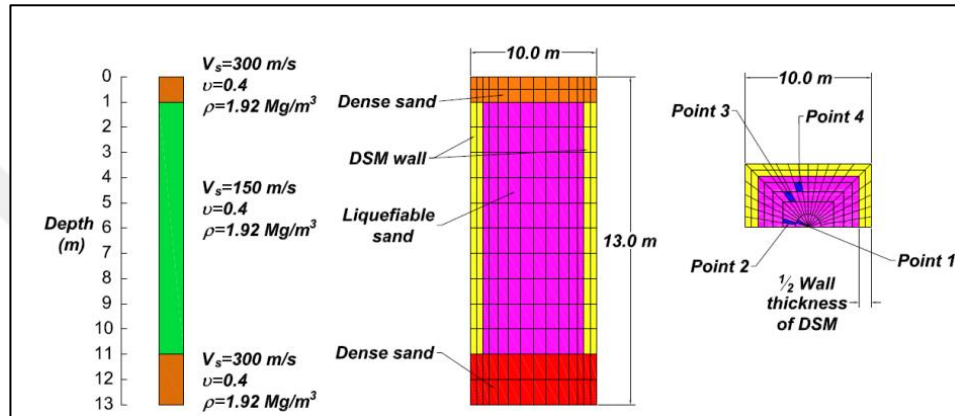


Figure 2.16 Finite element model for unit cell from periodic grid arrangement with wall thickness of 1.0 m, wall spacing of 10 m, and liquefiable layer thickness of 10 m (Nguyen et al., 2013)

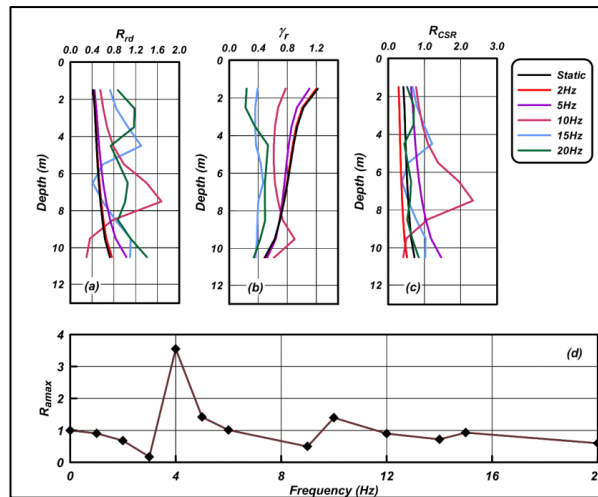


Figure 2.17 Effect of harmonic input motion frequency on (a) R_{rd} , (b) γ_r , (c) R_{CSR} , and (d) R_{amax} for wall spacing of 10 m ($A_r = 19\%$) and shear modulus ratio $G_r = 13.5$ (Nguyen et al., 2013)

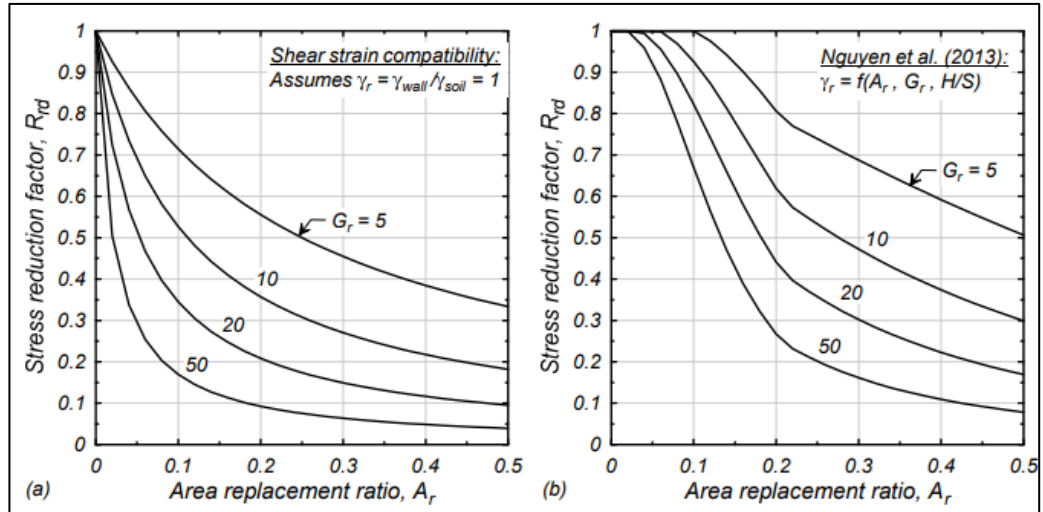


Figure 2.18 Shear stress reduction factor (R_{rd}) for periodic grid reinforcement based on: (a) shear strain compatibility with $CG = 1$, and (b) relationships by (Nguyen et al., 2013)

Gueguin et al. (2013) developed shear stress reduction factors for different values of A_r . These findings suggest that the lateral stiffness of the grating is dominated by walls parallel to the loading direction, while perpendicular walls contribute little to the lateral stiffness. Additionally, their research indicates that grating effectiveness is influenced by key factors including wall spacing (S), wall height (H), and depth (d) within the grating design. For instance, (Ishii et al., 2017) established a guideline indicating that the internal spacing ($L = S - t$, t = wall thickness) should not exceed $0.8H$. Additionally (Kitazume & Terashi, 2013) proposed that the S/H ratio should be kept below 0.8 to prevent liquefaction triggering.

Namikawa et al. (2007) conducted 3D dynamic analyses at various grid widths, and these analyses showed that the grids can restrict the formation of excessive pore water pressure in the soil within them. The studies demonstrate the effectiveness of DM grids in mitigating liquefaction under a wide range of conditions. Also demonstrate that DSM grids can help reduce foundation settlement even when liquefaction is induced in the soils within them.

The results of these studies indicate that DSM grids and columns can be an effective solution for the prevention of liquefaction. Detailed analysis should be performed for each project, considering grid and column properties, soil conditions, and seismic

effects. In addition, the effectiveness of these systems should be verified not only by theoretical studies and modeling, but also by means of field applications and the observed performance during seismic events. While the design and installation of soil-cement grids and columns has been acknowledged to be a significant development in the field of soil engineering and earthquake risk reduction, more research needs to be done on the application and effectiveness of these systems.

2.5 Case Studies

This section presents jet grouting projects from around the world. The aim is to highlight the main factors that support the modeling and success of the jet grouting method. The topic will be approached in two different directions. Recent and past case studies and projects are discussed and several types of jet grouting applications such as inclusion, liquefaction prevention, linear and comprehensive treatments are discussed. Although detailed descriptions of these procedures are not presented in this chapter, selected case histories from the past twenty-five years illustrate the various applications of jet grouting. By reviewing the published case histories, it is possible to get an estimate of the intended use and level of activity of jet grouting in different countries. Among the cases reviewed, jet grouting was found to be widely used in Europe, Japan, and Singapore.

2.5.1 Liquefaction Control

2.5.1.1 Carrefoursa Shopping Center, Turkey

Durgunoglu et al. (2001) investigated the efficiency of jet grout columns against liquefaction after the 1999 Kocaeli earthquake. Their research focused on the protection of the Carrefoursa Shopping Center, which was reinforced with jet grout columns, against the possibility of liquefaction-induced collapse. Although the mall and adjacent two-story parking lot were reinforced with these columns, Area C, which has a shallow foundation design, was not treated, and liquefied (Figure 2.19). The foundation design of the complex varied from section to section; Section A had spread footings linked by tie beams, while Section B had a raft foundation.

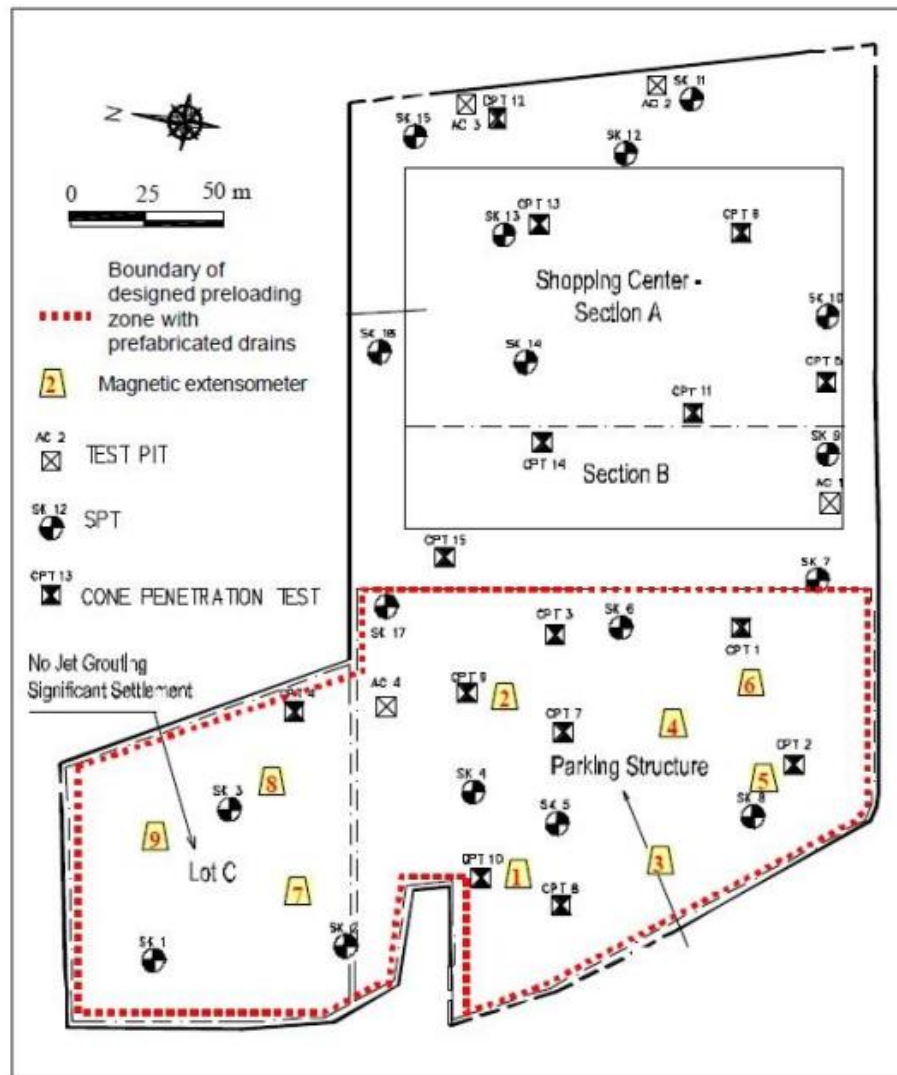


Figure 2.19 General information about structure.

The actual Pick Ground Surface Acceleration (PGA) of the site was designed to be 0.5g, although it was recorded as 0.24 g during the earthquake. After the earthquake, liquefaction induced settlements in lot C were verified by magnetic extensometers while there was no structural damage (Figure 2.20). Post-earthquake, liquefaction induced settlements were measured and compared with the predictions from empirical models, a volumetric deformation of about 3.3% and settlements between 4.0 cm and 4.5 cm were observed, in close agreement with the values predicted by (Tokimatsu & Seed, 1987) of 4.6 cm and Ishihara (1993) of 4.8 cm.

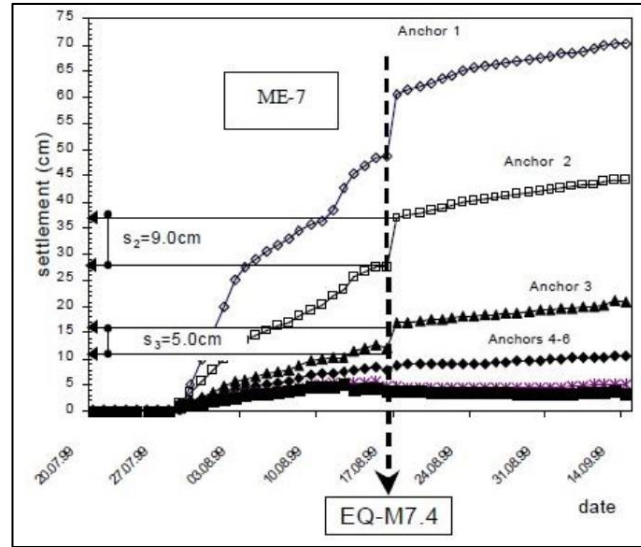


Figure 2.20 Settlement at lot C after earthquake, measured by extensometers.

This study emphasizes the importance of pre- and post-construction measures in the prevention of geotechnical hazards, especially those related to liquefaction. It is worth noting that the jet grouting technique not only consolidates liquefiable soils but also provides additional structural support for structures. The study conducted by Durgunoglu et al. (2001) proved that jet grouting is an effective method to reduce the liquefaction potential and increase the bearing capacity of the soil, while at the same time showing significant success in minimizing the displacements and permanent soil deformations that can occur because of soil liquefaction. The results of this study provide valuable information to civil engineers and geotechnical specialists and provide guidance on methodologies that can be applied when faced with similar geotechnical problems.

The study by Ozener et al. (2015) investigates the seismic performance of soil sections, both with and without improvement, during a seismic event at the Carrefour shopping center that was under construction using a numerical models (Figure 2.21). The earthquake behavior of the jet grout columns and the actual cause of their effectiveness turned out to be different from what was first thought. Furthermore, the reinforced soil did not function as a composite soil mass, as commonly designed. Dynamic shear stresses and stresses in the ground were not significantly reduced by

the columns. Instead, it is believed that the columns are effective in providing vertical support, preventing earthquake-induced collapse.

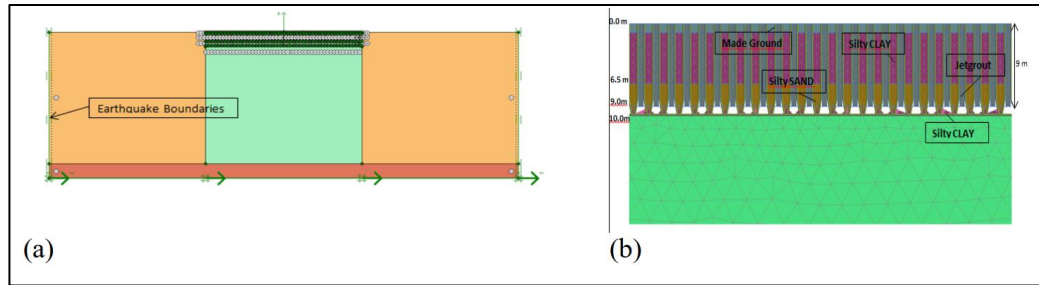


Figure 2.21 Simplified finite Element Model of the Improved Site with Jet Grout Columns

The analyses showed that the reinforced soil did not behave like a composite soil mass because of the stress mismatch between the soil and the stiff columns. The columns were only slightly stressed as they were 50-100 times stiffer than the surrounding soil. Moreover, they experienced negligible shear deformations compared to the peak strain in the soil mass. Additionally, the analysis revealed that the columns and the soil had different modes of deformation during the earthquake. Mostly, the columns did not exhibit shear deformation and behaved as bending beams during shaking instead of shear beams. They did not absorb a significant portion of the seismic shear load. These findings indicate that columns do not significantly contribute to the reduction of dynamic shear stresses, strains, and excess pore pressures in the soil. Numerical modeling suggested that the high vertical stiffness of the soft soil profile is the main contribution of the soil in preventing earthquake-induced collapse, not in reducing seismic shear stresses.

2.5.1.2 İpekkağıt Tissue Factory, Turkey

Durgunoglu et al. (2004) examined the efficacy of jet grout columns in preventing liquefaction at the İpekkağıt Tissue Factory during the 1999 Kocaeli earthquake. The site plan shown in Figure 2.23 The facility, situated approximately 4 km from the epicenter, consists of machinery buildings, water tanks, storage structures, and multipurpose edifices. The construction of the water tanks was completed before the earthquake, but that of the Paper Machine Building (PM3) was still ongoing. It is a

high-risk liquefaction site that comprises of layered soil and has a groundwater level 2 meters beneath the ground surface (Figure 2.22).

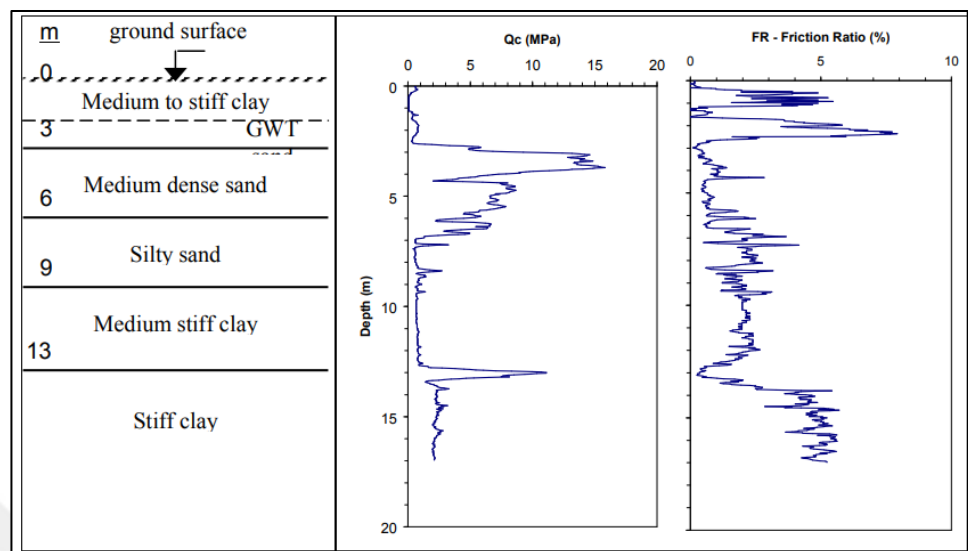


Figure 2.22 Soil profile of research area (Durgunoglu et al., 2004)

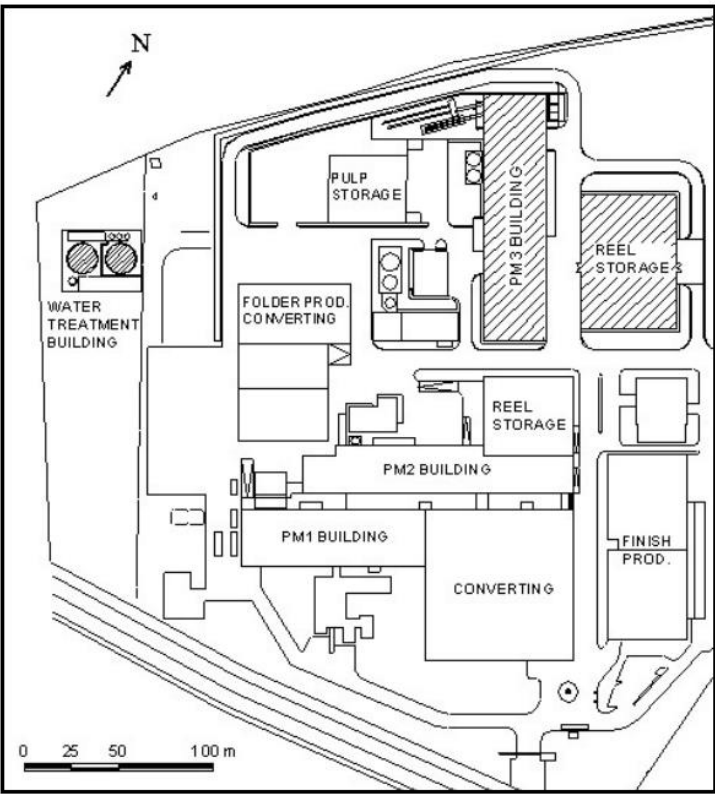


Figure 2.23 Site plan (Durgunoglu et al., 2004)

Jet grouting was used for the PM3 building, water tanks and reel storage buildings. The 60 cm diameter columns are installed at varying intervals and intended only for static loads. Footnote style and formatting features, as well as consistent citation, are consistently applied. The length of each column is 12 meters, with the outer two rows measuring 8 meters in length. The shear modulus ratio of jet grouting compared to soil in this project is approximately 30 times higher, and the 28-day compressive strength of soilcrete columns amounts to 3.0 MPa.

Although jet grout columns are designed to support heavy structures, calculations show that the improvement provided by jet grout columns is sufficient to mitigate liquefaction. The area change ratio within the jet grouted column arrangement varies between 0.07 and 0.10, with the corresponding cyclic shear stress reduction factors (CSR) calculated at 0.33 and 0.27, respectively. Seismic events are expected to cause significant shear stresses in the soil, leading to a reduction of up to 30% of their theoretical value.

After the earthquake, no damage was observed at the factory site, except for the PM2 building, where liquefaction-induced settlement was observed. This study demonstrates that utilizing jet grouted columns can be an efficient solution for liquefaction prevention applications and a useful reference for ground improvement in structural projects. In this context, the use of jet grouted columns is acknowledged as a significant method for enhancing the ground conditions during the construction of underground structures as well as strengthening existing structures in regions with a greater risk of earthquakes.

2.5.2 Other Applications

2.5.2.1 Tunnel Construction in French Alps

Guilloux (2000) describes the geotechnical problems encountered during the construction of the 1200-metre-long, double-track Les Hurtières tunnel for the A43 highway in the French Alps. Despite the predominant metamorphic rock excavation, geological evaluations showed the existence of valleys comprising alluvial material. Typically, blasting and shotcrete techniques are utilized for tunneling through the rocky formation. During construction of the western tube in Les Hurtières, two violent

water and mudflows collapsed into the tunnel, halting work, and leaving 2000 m³ of debris. Proper remediation measures were taken, and work resumed once the site was secure. Further investigations uncovered a canyon with a complex shape at a depth of 60-80 m. The canyon extends deeper in the eastern tube and spans approximately 50 m of the tube, including a saturated sand material section that is 15 m (Figure 2.24).

To address the soil instability issue, the decision was made to implement jet grouting as a reinforcement method. A two-tiered umbrella consisting of 15-meter long, 0.6-meter diameter, horizontal jet grouting columns was constructed. Additionally, seven jet grouting micropiles, with the same dimensions as the umbrella columns, were installed at the base of the umbrella and along the tunnel's foundation to distribute loads to the surrounding soil (Figure 2.25) . Moreover, grouting was performed to fill any existing voids and compact soft areas. Some columns used steel tube reinforcement, with each set of jet columns and piles repeated every 7 meters of advancement.

A single-fluid system widely utilized in horizontal jet grouting was employed, resulting in soilcrete compressive strengths ranging from 10 to 77 MPa (with an average of 30 MPa). Excavation deformations were minimal, not exceeding a few millimeters, and overall performance surpassed expectations.

Roughly 6200 meters of jet grout columns were installed, allowing the completion of tunnel excavation within a 50-meter zone along the valley in just 7 months. The study showed that jet grouting can be a reliable technique for operating in challenging ground conditions, developing robust and cohesive zones, and adapting flexibly to the various demands encountered during tunneling applications.

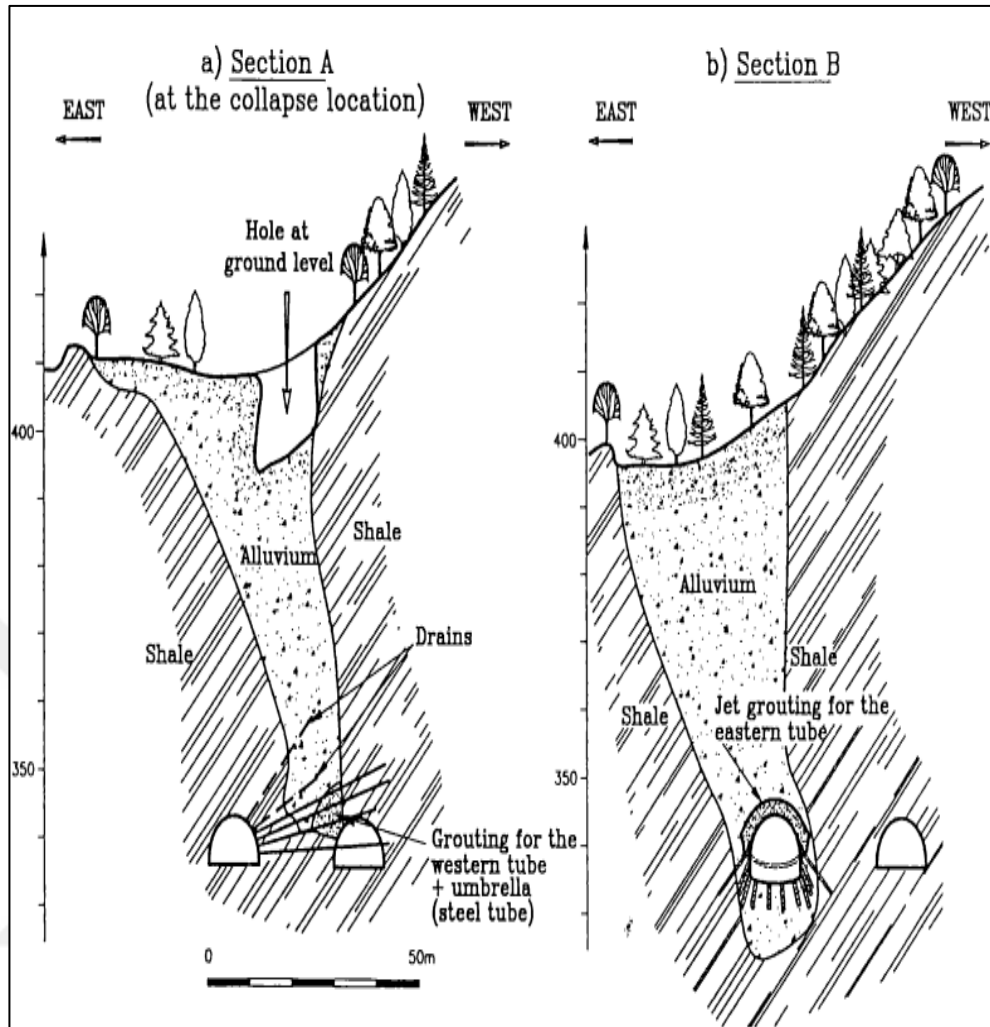


Figure 2.24 Cross Sections (Guilloux, 2000)

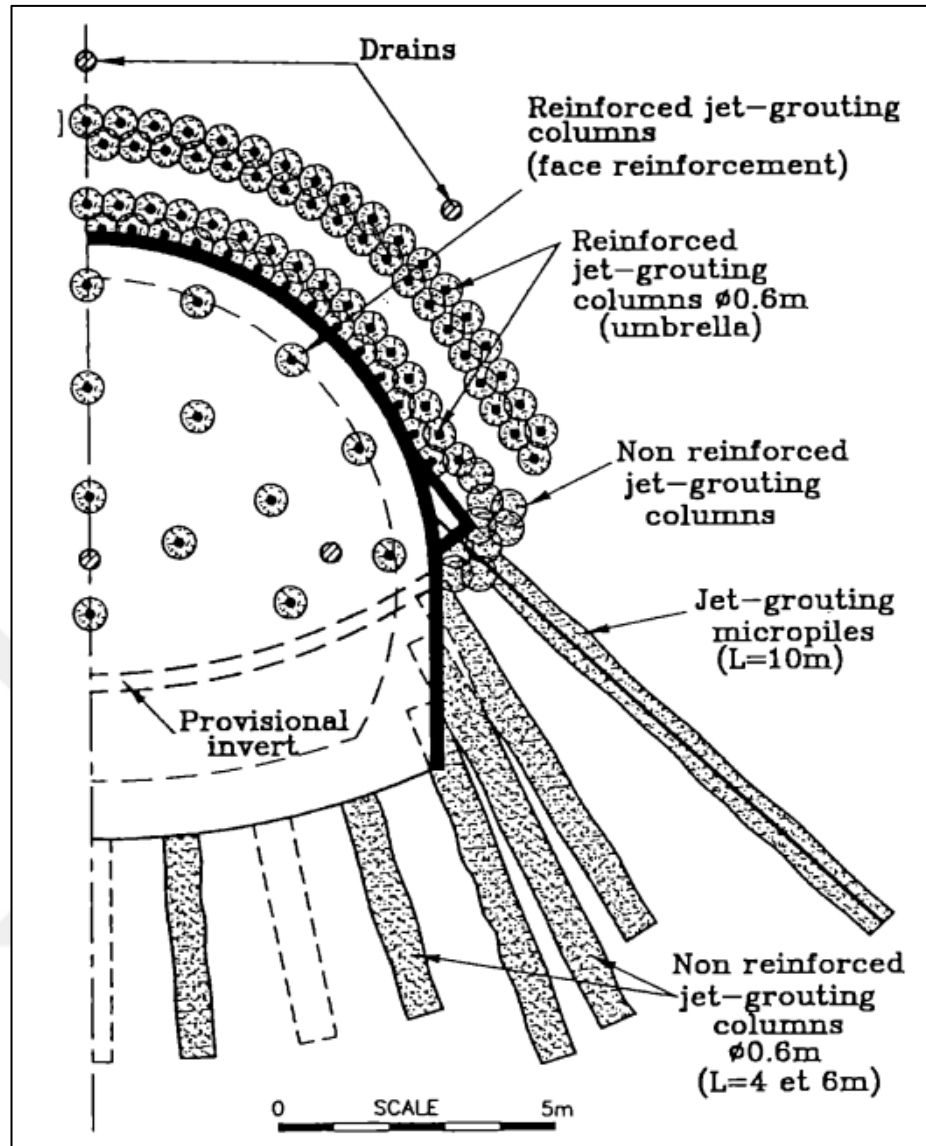


Figure 2.25 Jet-Grout columns shape

2.5.2.2 Basement excavation for groundwater control, Jersey

Newman et al. (1994) examined the effectiveness of jet grouting as a measure to prevent seepage around a 4-meter-deep basement to be constructed as part of a project for redeveloping the library campus of Midland Bank in Jersey. The plan involved demolishing the existing building and constructing a new one with a basement. The renovation came across challenges caused by highly compressible soil and the proximity of historic buildings adjacent to the site. As a result, remedial measures had to be taken without causing any permanent damage to the ground.

The soil profile consisted of a 7 m alluvial layer consisting of a thin layer of silt and peat with about 10% clay content, overlying a top layer of dense sandy clay, as shown in Figure 2.26. As depicted in the figure, the alluvial layer exhibited hydrostatic water pressure, while the head layer showed semi-artesian groundwater. Challenges during the excavation process involved dealing with buoyancy pressures from semi-artesian flows, maintaining surrounding groundwater levels to prevent drawdown and subsequent subsidence, and minimizing ground movements. Additionally, there was a potential risk of liquefaction of the excavation floor due to seepage pressures building up at ground level.

A proposal was made to construct a semi-impermeable enclosure using jet grout columns. This enclosure would effectively limit the infiltration of groundwater into the excavation zone, allowing for proper drainage during construction. To construct a 1.5-meter-deep foundation shelf, a three-fluid system was implemented to grout numerous 1.2-meter-diameter columns. During the jet grouting process, a three-stage sequencing plan was utilized to ensure adequate overlap and proper connection between the columns.

The jet columns proved successful with a range of 1 to 4 L/min vertical seepage measured during excavation, which was effectively controlled by a drainage blanket. Permeability reduced from approximately 10^{-3} m/s to 6×10^{-8} m/s.

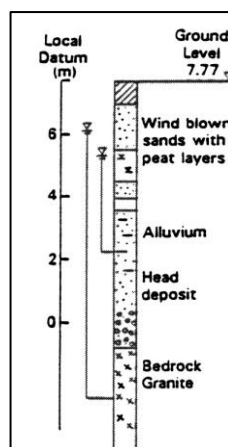


Figure 2.26 Soil profile (Newman et al., 1994)

CHAPTER THREE

FIELD CONDITIONS AND NUMERICAL MODEL

3.1 Introduction To Study Area

This chapter introduces the Wildlife Site in the Imperial Valley, a significant geotechnical earthquake engineering research location. The behavior of the ground in this area post-1981 Westmorland earthquake has garnered attention from scientists, making it a crucial site for research. The 1981 Westmorland earthquake, with a magnitude of $M_s = 5.9$, is suspected to have caused liquefaction events in the region. In 1982, the US Geological Survey (USGS) installed multiple tools at the Wildlife Site to research the behavior of liquefaction (Holzer et al., 1989).

In 1987, there were two major earthquakes in the Imperial Valley that happened 12 hours apart. The first earthquake took place at Elmore Ranch, which is 23 km west from the Wildlife Site, and had a moment magnitude of $M_s = 6.2$. The maximum ground acceleration recorded at the ground surface was 0.13 g. The Superstition Hills earthquake, which had a moment magnitude of $M_s = 6.6$ and a maximum ground acceleration of 0.20 g, had a more severe impact and occurred 31 km southwest of the Wildlife Site. It was found that the earthquake caused liquefaction in the study area. Figure 3.1 displays the epicenter of these earthquakes in the Wildlife Site area.

This chapter presents the Wildlife Site located in the Imperial Valley, a significant research site for geotechnical earthquake engineering. This region is a typical example of a liquefaction zone. Zone, exhibiting similar characteristics to the Izmir Atatürk Organized Industrial Zone.

The chapter will present a description of the installation work completed and provide specifics regarding the research area's location.

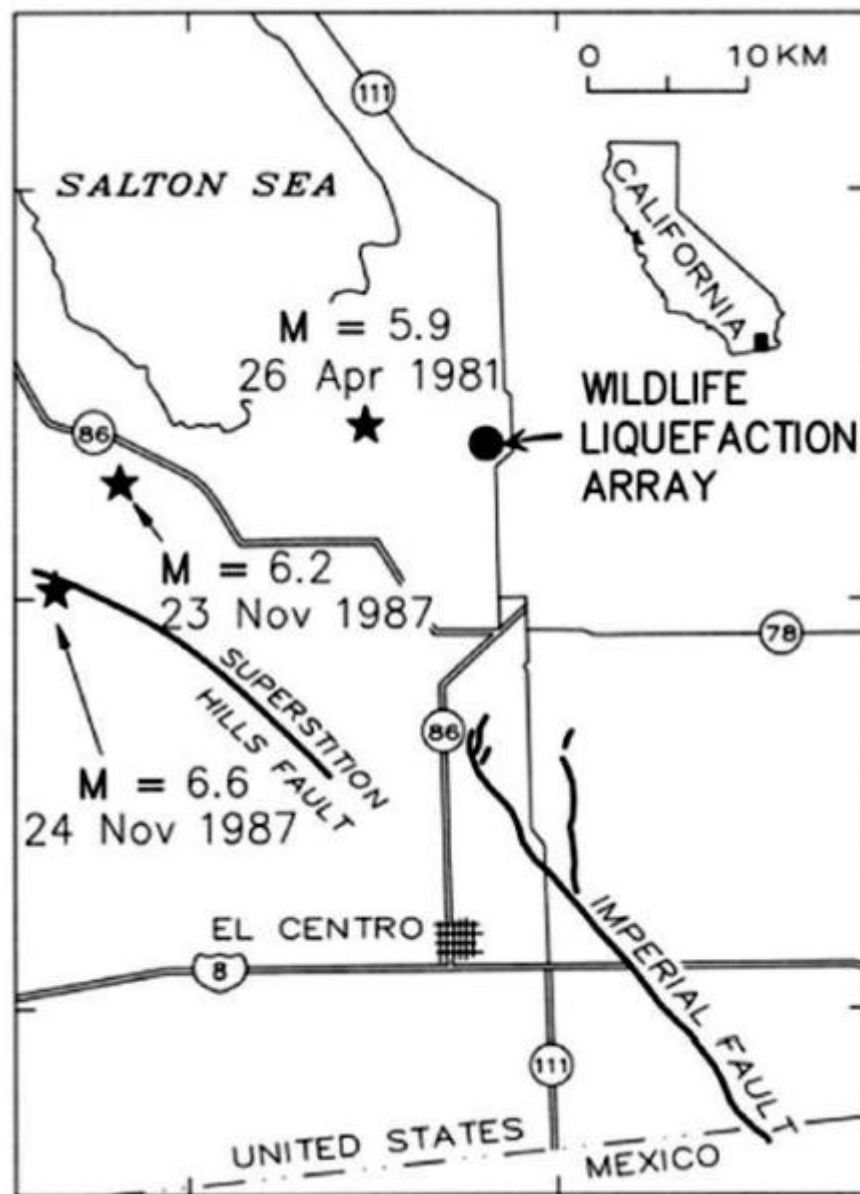


Figure 3.1 Earthquake epicenters in the Wildlife Site area (Holzer et al., 1989)

3.1.1 Site Location and Properties

The Wildlife Site is situated in the alluvial valley of the Alamo River, 3.2 km south of Calipatria, California, in an area with a desert climate and intensive agricultural irrigation.

According to Holzer et al. (1989) studies, the former bed of the Alamo River is filled with saturated floodplain alluvium near the surface of the study area, which resulted from major river flooding between 1905 and 1907. The Alamo River flows in

a channel that is 3.7 m deep, located 23 m east of the Wildlife Site. This site controls the groundwater level at a depth of about 1.2 m. The river is primarily used for irrigation and drainage purposes. Just beneath the surface lies a layer of flat-lying silt, measuring 2.5 meters in thickness. Beneath this is a 3.3-meter layer of silty sand, followed by a 5-meter layer of silt that contains clay. This clay-containing layer constitutes the top layer of a dense sediment unit that extends extensively. The fine-grained content of the silty sand layer, which is said to be the most prone to liquefaction ($<75\ \mu\text{m}$), varies from 16% to 60%, with an average of 33%.

In a study conducted by Gu et al. (1994), the soils that were identified as prone to liquefaction during the Superstition Hills earthquake were a 2.5 m thick layer of silt and a 3.3 m thick layer of silty sand. The first layer is composed of typical flood plain sediment with low resistance to penetration and a liquidity index greater than 1, demonstrating its liquefaction susceptibility. Approximately 20% of this sediment is susceptible to liquefaction, as demonstrated by Seed et al. (1983). These sediments possess a liquidity index greater than 0.9, a liquid limit of less than 35%, and a sediment fraction ($<0.005\ \text{mm}$) of less than 15%. The second layer, composed of 3.3 meters of silty sand, liquefied, and created sand boils due to its porous sedimentary structure, high water table, and similar characteristics to sand boils. The top layer, which resides predominantly above the water table, is believed to have remained un-liquefied. In Bennett et al. (1984)'s report, tests were conducted to determine the soil profile at the Wildlife Site. Figure 3.2 shows the percentages of sand, silt, and clay, as well as the combined soil classification, cpt tip resistance, and water table location.

In the study conducted by Davis and Berrill (2001), it was determined that the average shear wave velocity for the Wildlife Site was 125 m/s. A different study by Holzer et al. (1989) reported the same shear wave velocity value. Table 3.1 presented by Gu et al. (1994) contains the soil parameters determined for the Wildlife Site. Table 3.2 shows the depths at which the piezometers are located.

Accelerometers were installed at the surface and 7.5 meters depth, linked to the same triggering mechanism as other instrumentation. The subsurface accelerometer is positioned near the upper surface of the firm clay. Inclinator cases and piles encircle the instrument array at the site to monitor lasting horizontal subsurface deformations.

The installed instrumentation at the Wildlife Site has provided valuable data on seismic motions and liquefaction events. Specifically, piezometers and accelerometers provide crucial information for analyzing soil behavior during earthquakes. Such data are vital in assessing the potential for liquefaction and the effectiveness of jet-grouting applications.

Table 3.1 Soil Parameters

Layer	I	II	III	IV	V
Depth (m)	0.0-1.2	1.2-2.5	2.5-3.5	3.5-6.8	6.8-10
Total Density (kg/m ³)	1600	1940	1970	1970	2000
Bulk Modulus (N/m ²)	2.61*10 ⁷	2.44*10 ⁷	4.50*10 ⁷	4.50*10 ⁷	5.83*10 ⁷
Shear Modulus (N/m ²)	1.57*10 ⁷	1.47*10 ⁷	2.08*10 ⁷	2.08*10 ⁷	2.70*10 ⁷
Cohesion (Pa)	2*10 ³	2*10 ³	-	-	-
Friction angle	21.3	20.0	22.0	22.0	35.0
Dilation	21.3	20.0	19.0	18.0	5.0
SPT average	6	6.25	7.65	10.65	10
Permeability (m ³ .sec/kg)	5*10 ⁻¹¹	5*10 ⁻¹¹	5*10 ⁻¹¹	2.1*10 ⁻¹⁰	1*10 ⁻¹³
Porosity	0.4047	0.4431	0.4253	0.4253	0.4075

and piles encircle the instrument array at the site to monitor lasting horizontal subsurface deformations.

The installed instrumentation at the Wildlife Site has provided valuable data on seismic motions and liquefaction events. Specifically, piezometers and accelerometers provide crucial information for analyzing soil behavior during earthquakes. Such data are vital in assessing the potential for liquefaction and the effectiveness of jet-grouting applications.

Table 3.2 Piezometer identification and depths.

Identification	Depth (m)	Layer
P1	5.0	Silty Sand
P2	3.0	Silty Sand
P3	6.6	Silty Sand
P4	3.5	Silty Sand
P5	2.9	Silty Sand
P6	12.0	Silt

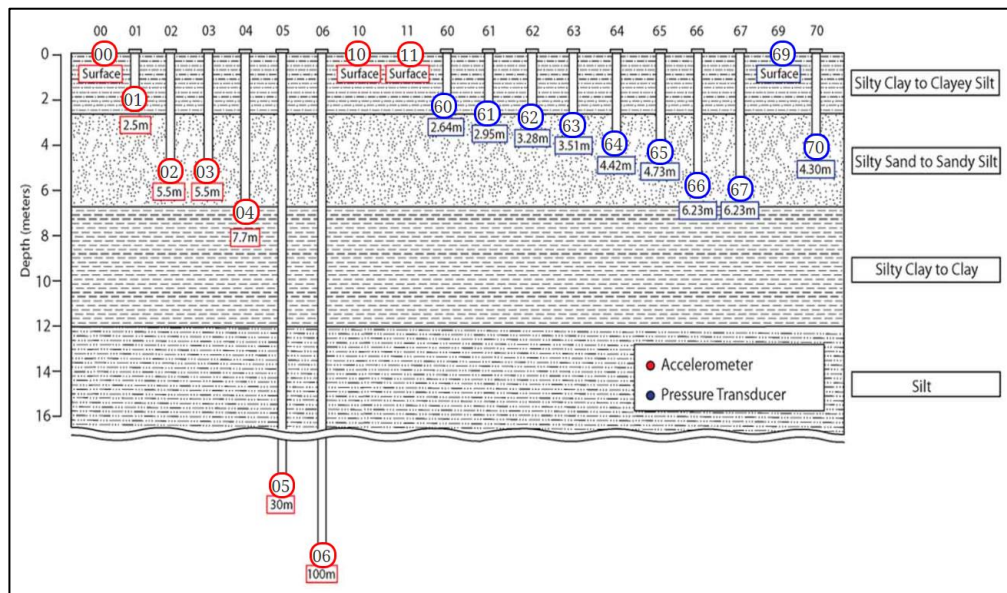


Figure 3.3 Schematic representation of the instrumentation

3.1.3 Earthquake Impact at the Wildlife Site

During the Superstition Hills earthquake in 1987, signs of liquefaction were observed in the silty sand layer at the Wildlife Site. The pore water pressures and prominent cracks in the ground surface, as documented by Holzer et al. (1989), indicated the occurrence of this liquefaction event. Notably, large ground cracks linked to localized subsidence on the west bank of the Alamo River served as evidence of lateral spreading. Sand flowing out of the crevices provided conclusive evidence of liquefaction. Figure 3.4 displays a photograph, depicting sand boils at the Wildlife Site.



Figure 3.4 Sand boils section at the Wildlife Site

3.2 Overview Of Numerical Model

This section examines the fundamental principles of numerical modeling, which has widespread application in geotechnical engineering and liquefaction analysis and plays a crucial role in complex projects. Numerical modeling is a powerful tool for understanding complex soil behavior and can effectively simulate seismic soil behavior. The intent of this chapter is to introduce Plaxis, the numerical modeling tool used in our study, and its methods, and to explain why this approach was chosen.

Modern geotechnical engineering relies strongly on numerical analysis to understand and simulate complex soil behavior, especially in areas such as liquefaction and soil-structure interaction. This study utilizes advanced numerical modeling software, such as Plaxis, to analyze soil behavior and simulate its characteristics.

This section explains the Plaxis software, the selection of soil constitutive models, and how these models represent soil behavior. The setup process of the numerical model using this software is also described, including parameter selection, analysis steps, and calibration. Additionally, this numerical model's ability to answer research questions and contribute to a better simulation of complex soil behavior will be discussed.

3.2.1 Numerical Analysis Tool: Plaxis

Plaxis 2D is a software that utilizes finite element methodology for the analysis and design of geotechnical engineering projects. Delft University of Technology in the Netherlands initially developed Plaxis in 1987 to provide an easy-to-use 2D finite element code for analyzing river embankments situated on soft soils in the country. The software can model a wide range of ground conditions. The Dutch Ministry of Public Works and Water Management initiated the software's develop. Plaxis 2D can model various soil behaviors thanks to a large library of advanced soil models. This Plaxis software includes linear elastic, Mohr-Coulomb, hardening soil and small deformation hardening soil models, allowing users to use their own defined soil models in calculations.

The software features a graphical interface for computer-aided design (CAD), which enables the creation of structures and the assignment of loads, displacements, and boundary conditions to the numerical model.

Two types of analysis can be performed in Plaxis 2D: plane deformation and axisymmetric geometry. The plane deformation model applies to projects with a consistent cross-section and loading pattern along a specific length. In contrast, the axisymmetric model is utilized for circular structures, where the deformation and stress state remain identical in all radial directions. Figure 3.5 shows two different model cases.

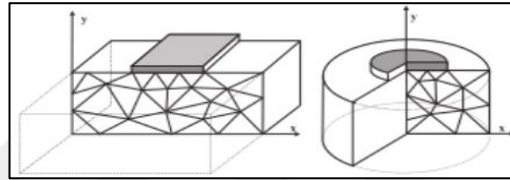


Figure 3.5 Plain strain model (left) and Axisymmetric model (right).

The software is divided into two primary modes: geometry and calculation, with geometry mode further divided into two sub-modes: soil and structures. The soil mode is utilized to establish soil stratigraphy, initial conditions, and water level, while the structures mode is utilized to create interfaces, loads, and plates. The calculation mode includes the creation of a mesh, flow conditions, and phased construction. Creating a finite element mesh of the project's geometric configuration is achieved using meshing, which is a crucial step that ensures consistent results. Flow conditions mode is implemented to adjust water levels, perform flow calculations, or simply analyze them for various stages. Phased construction enables to simulate construction steps, evaluate different load conditions that impact the model, and toggle the model's geometry and structure.

The features and capabilities provided by Plaxis software are essential in soil engineering. Its two-dimensional and three-dimensional modeling capabilities offer effective understanding and analysis of soil behavior and geotechnical engineering issues. Plaxis combines theoretical knowledge with practical applications, contributing

to a better comprehension and enhancement of ground phenomena and geotechnical designs.

3.2.2 Soil Models and Input Parameters

Various soil models have been developed in geotechnical engineering to represent various properties of soil, including anisotropy and behavior under cyclic loading. This section provides a brief description of the soil models used in this research, followed by a table of the parameters utilized in the numerical models. The objective of this section is to provide an overview of the soil models and parameters used in this research.

3.2.2.1 Linear Elastic Model

Based on Hooke's law and the isotropic elasticity it provides, the Linear Elastic model comprises Young's modulus (E) and Poisson's ratio (ν). While it may be useful in modeling rigid volumes such as concrete walls and foundations, this model is not suitable for soil modeling due to the nonlinear and irreversible nature of soil. The study employs this soil constitutive model to represent the foundation in 2D analyses, with the model parameters displayed in Table 3.3. Foundation is modeled as impermeable, i.e., non-porous.

Table 3.3 Foundation parameters

Symbol	Value	Unit
Y_{unsat}	24	kN/m^3
E_{ref}	3.00E+07	kN/m^2
G_{ref}	1.36E+07	kN/m^2
E_{oed}	3.07E+07	kN/m^2
V_s	2.36E+03	m/s
V_p	3.54E+03	m/s
R_{inter}	6.70E-01	-
K_0	Automatic	-

3.2.2.2 Mohr-Coulomb model (MC)

The parameters required for the Mohr-Coulomb, a linear elastic and perfect plastic model, are shown in Table 3.4. This model does not include stress dependence, stress path dependence, or anisotropic stiffness, although it does account for the increase in stiffness with depth. Below the failure line, the stiffness behavior is assumed to be linearly elastic according to Hook's law. The MC soil constitutive model fails to accurately predict pre-failure deformation behavior, particularly in cases where the stress level changes, or multiple different stress paths are followed. This study employs the MC soil constitutive model to simulate jet grout columns in 2D and 3D models, using the parameters listed in Table 3.5.

Table 3.4 Parameters of the Mohr-Coulomb model

Symbol	Description	Unit
E_{ref}	Young's modulus	[kN/m ²]
ν	Poisson's ratio	[-]
G	Shear modulus	[kN/m ²]
E_{oed}	Oedometer modulus	[kN/m ²]
c	Cohesion	[kN/m ²]
ϕ	Friction angle	[°]
ψ	Dilatancy angle	[°]
σ_t	Tension cut-off and tensile strength	[kN/m ²]
V_P	Compression wave velocity	[m/s]
V_S	Shear wave velocity	[m/s]

Table 3.5 Jet-Grout column parameters

Symbol	Value		Unit
	$q_u=15 \text{ MPa}$	$q_u= 2 \text{ MPa}$	
Y_{unsat}	18	18	kN/m^3
ν_{nu}	0.3	0.3	-
E_{ref}	$3.10\text{E}+06$	$5.00\text{E}+05$	kN/m^2
G_{ref}	$1.19\text{E}+06$	$1.92\text{E}+05$	kN/m^2
E_{oed}	$4.18\text{E}+06$	$6.73\text{E}+05$	kN/m^2
V_s	807.1	323.7	m/s
V_p	1510	605.7	m/s
C_{ref}	3685	324	kN/m^2
ϕ	38	54	°
ψ	0	0	°
Tensile Strength	1530	160	kN/m^2
R_{inter}	1	1	-

3.2.2.3 Hardening soil model

The hardening soil model is a general type of true quadratic model for various geotechnical applications for both soft and hard soils. The Hardening Soil model is a common model used in geotechnical applications for both soft and hard soils. The model precisely predicts soil displacements and failures in various geotechnical uses and generally applicable types of soils. However, the model does not include anisotropy and time-dependent behavior. Table 3.6 presents the input parameters of the soil constitutive model. The soil constitutive model with clay parameters was used to simulate the 2D and 3D analyses in this study due to its ability to simulate dynamic soil behavior. The clay parameters utilized are displayed in Table 3.7.

Table 3.6 Parameters of the Hardening Soil Model (Plaxis Manual, 2022)

Symbol	Description	Unit
ψ	Angle of dilatancy	[°]
σ_t	Tension cut-off and tensile strength	[kN/m ²]
E_{50}^{ref}	Secant stiffness in standard drained triaxial test	[kN/m ²]
E_{oed}^{ref}	Tangent stiffness for primary oedometer loading	[kN/m ²]
E_{ur}^{ref}	Unloading / reloading stiffness (default $E_{ur}^{ref} = 3E_{oed}^{ref}$)	[kN/m ²]
m	Power for stress-level dependency of stiffness	[-]
ν_{ur}	Poisson's ratio for unloading reloading (default $\nu_{ur} = 0.2$)	[-]
p_{ref}	Reference stress for stiffnesses (default $p_{ref} = 100 \text{ kN/m}^2$)	[kN/m ²]
K_{nc}^0	K_0 -value for normal consolidation (default $K_{nc} = 1 - \sin\phi$)	[-]
R_f	Failure ratio q_f / q_a (default $R_f = 0.9$)	[-]
$\sigma_{tension}$	Tensile strength (default $\sigma_{tension} = 0$ stress units)	[kN/m ²]
c_{inc}	As in Mohr-Coulomb model (default $c_{inc} = 0$)	[kN/m ³]

Table 3.7 Clay layers parameters

Symbol	Value			Unit
	Clay Layer 1	Clay Layer 2	Clay Layer 3	
Y_{unsat}	18	18	18	kN/m ³
Y_{sat}	20	20	20	kN/m ³
G_0^{ref}	36000	65000	122000	-
$Y_{0.7}$	2.00E-04	4.00E-04	4.00E-04	-
v_{ur}	0.2	0.2	0.2	kN/m ²
E_{50}^{ref}	1800	10000	30000	kN/m ²
$E_{\text{oed}}^{\text{ref}}$	1800	9000	30000	kN/m ²
$E_{\text{ur}}^{\text{ref}}$	8000	35000	90000	kN/m ²
p^{ref}	100	100	100	kN/m ²
m	0.6	0.75	0.7	-
c	10	20	40	kN/m ²
ϕ	24	25	28	°
ψ	0	0	0	°
R_{inter}	0.67	0.67	0.67	-
K_0^{nc}	0.5933	0.5774	0.5305	-

3.2.2.4 UBC3D-PLM Model for Liquefiable Soil

The UBC3D-PLM utilized in this study was created by Puebla et al. (1997) (UBCSAND) and later developed further by Beaty and Byrne (1998). The purpose of the UBCSAND model is to forecast sandy soil liquefaction behavior, and the purpose of the UBCSAND model is to simulate the liquefaction behavior of sandy soils. This rule uses the mobilized internal friction angle with a plastic slip function.

UBC3D-PLM is a 3D version of the 2D UBCSAND model with a Mohr-Coulomb yield condition and a modified principal stress function based on the Drucker-Prager criterion. The yield function is a generalized 3D Mohr-Coulomb in the principal stress space.

Here, the three primary stresses can be ordered as $\sigma_{\min} \leq \sigma_{\text{int}} \leq \sigma_{\max}$. Each principal stress has a unique direction in the Cartesian stress space. In the principal stress space, plotting the six planes together generates a comprehensive 3D Mohr-Coulomb criterion surface. The surface displays only values within the range of $\sigma_{\min} \leq \sigma_{\text{int}} \leq \sigma_{\max}$ (Figure 3.6).

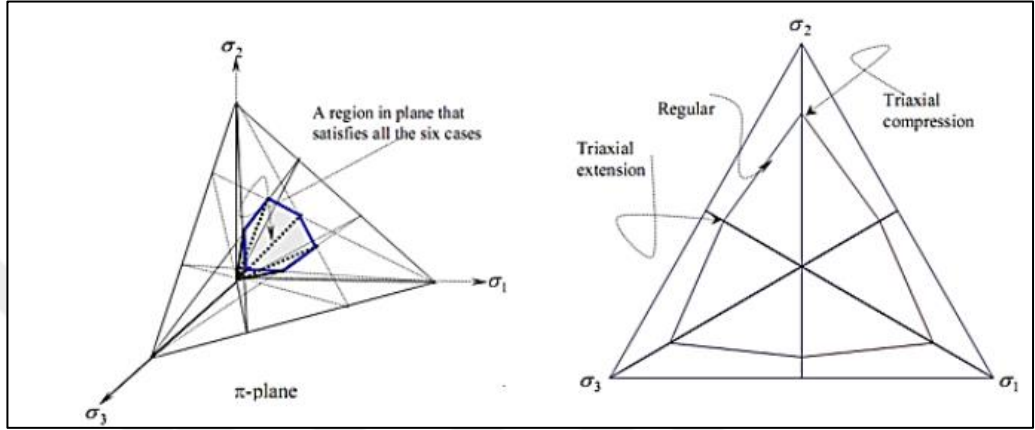


Figure 3.6 Mohr-Coulomb yield surface in 3D principal stress space (Petalas et al., 2013)

The model is solved in a series of steps. The first step is to calculate the principal stresses of the stress matrix. by solving the eigenvalue problem. The eigenvalues are used to determine the principal stresses and the eigenvectors are used to determine the directions of these principal stresses. According to Petalas et al. (2013), in the UBC3D-PLM model, the directions of the principal stresses are unchanging, indicating that the response of the material does not depend on the direction. The stress plane is then defined in the model. The model's critical stress surface is where the difference between the two main stresses is the largest and is ordered as $(-\sigma_1 \geq -\sigma_2 \geq -\sigma_3)$. This is represented by the Equation 3.1.

$$f_m = \frac{\sigma'_{\max} - \sigma'_{\min}}{2} - \left(\frac{\sigma'_{\max} - \sigma'_{\min}}{2} + c' \cot \varphi_p' \right) \sin \varphi_{mob} \quad (3.1)$$

The model generates three different stress paths as it develops the yield surface and analyzes them correspondingly. - triaxial compression, triaxial extension, and regular stress path. The model assumes elastic behavior in the region defined by the

yield surfaces. However, as soon as the stress path touches the yield surface, plastic deformation begins. Elastic deformations are a function of shear or normal effective stress changes. When the elastic stress limit is exceeded, the deformations become permanent. The model's plastic deformations are determined using hyperbolic relations obtained from the current stress ratio, and the direction of these deformations is independent of the yield surface. Figure 3.7 graphically illustrates this situation.

The relationship between stresses and deformations is determined by the shear modulus and the bulk modulus. Both modules are isotropic and nonlinear in nature, they depend on the current average stress state.

$$G = K_G^e P_A \left(\frac{\sigma'}{p_A} \right)^{n_e} \quad (3.2)$$

$$B = \alpha K_e^G \quad (3.3)$$

$$B = K_B^e P_A \left(\frac{p'}{p_A} \right)^{m_e} \quad (3.4)$$

G = Shear modulus

B = Bulk modulus

K_e^G = Shear modulus dependent on relative density

K_B^e = Bulk modulus dependent on relative density

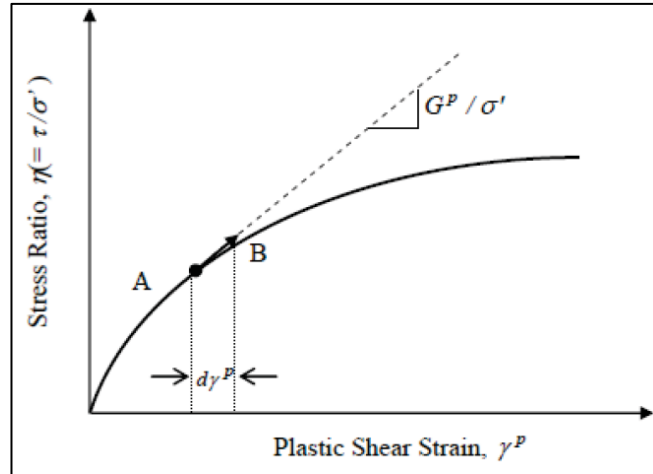


Figure 3.7 Plastic strain increment and plastic modulus

The magnitude of plastic volumetric deformations is related to shear deformations by the angle of dilation. Volumetric deformations can be contractive or dilative and their nature is determined by the proximity of the soil to the critical state line. Volumetric deformations can be contractive or dilative and their nature is determined by the proximity of the soil to the critical state line. The constant volume friction angle indicates this proximity. Soils with lower strain rates in terms of this angle tend to contract, while those with higher strain rates tend to dilate, as illustrated in the Figure3.8. Soils on the critical state line do not develop volumetric stresses.

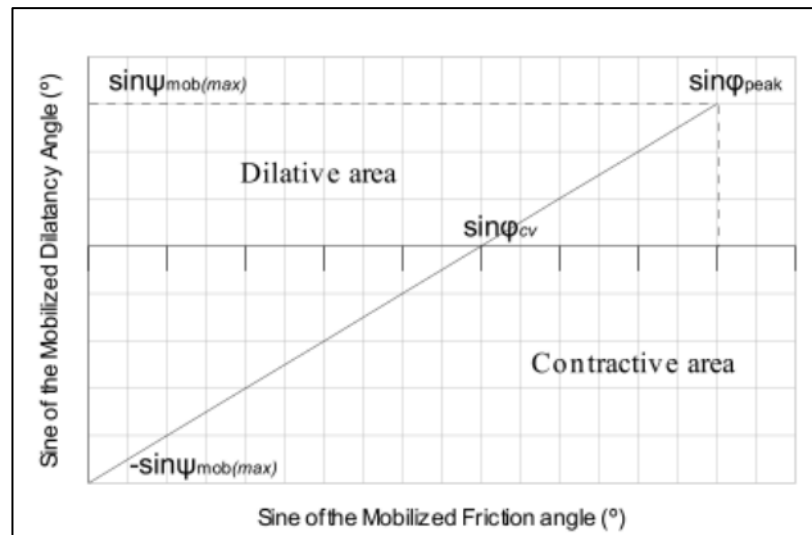


Figure 3.8 Modified Rowe's flow rule used in UBC3D-PLM (Alexandros Petalas et al., 2013)

The yield surface begins as a small radial line from the origin but expands as the soil Positive and negative strain rates are monitored separately, so that the hardening law is followed separately for positive and negative strain rates. Elastic behavior is monitored, and the soil is "reloaded" until it returns to the last maximum or minimum stress ratio. Plastic deformations occur during reloading with a significantly more rigid shear modulus than during initial or virgin loading.

Volumetric restraint is an important factor in the cyclic liquefaction of sands. When the stress path reaches the yield surface defined by the peak friction angle, the evolution of volumetric deformations becomes constant due to the formulation of the flow rule. Because of this limitation, the reduction in soil stiffness described in experimental studies, such as the post-liquefaction behavior of loose cohesive soils or the cyclic mobility of dense cohesive sands, cannot be predicted. The UBC3D-PLM formulation addresses this problem by using an equation that gradually decreases the plastic shear modulus due to the plastic deviator deformation generated during soil element expansion.

Most of the UBC3D-PLM model parameters can be obtained using cyclic direct simple shear (CDSS) or drained triaxial tests. These tests are commonly used in the laboratory and in the field to better understand the properties of the soil, both mechanical and dynamic. The UBC3D-PLM model uses a customized formulation based on these tests, and the input parameters are determined based on the results of the tests. The model input parameters are detailed in Table 3.8.

Beaty and Byrne (2011) suggested some correlations to obtain soil model parameters using the clean sand equivalent parameter $((N_1)_{60})$. These correlations were found based on measurements of clean sand equivalent SPT blows, and the parameters are obtained using the following equations (Equation 3.5-3.9).

$$K_G^e = 21.7 \times 20 \times (N_1)_{60}^{0.333} \quad (3.5)$$

$$K_B^e = 0.7 \times K_G^e \quad (3.6)$$

$$K_G^P = K_G^e \times (N_1)_{60}^2 \times 0.003 + 100 \quad (3.7)$$

$$\phi_{cv} = 20^\circ + \sqrt{15.4 \times (N_1)_{60}} \quad (3.8)$$

$$\phi_{pi} = \phi_{cv} + \frac{(N_1)_{60}}{10} + \max(0; \frac{(N_1)_{60}-15}{5}) \quad (3.9)$$

Table 3.8 UBC3D-PLM input parameters

Symbol	Description	Unit
k^{*e_B}	Elastic bulk modulus factor	[-]
k^{*e_G}	Elastic shear modulus factor	[-]
k^{*p_G}	Plastic shear modulus factor	[-]
me	Rate of stress-dependency of elastic bulk modulus	[-]
ne	Rate of stress-dependency of elastic bulk modulus	[-]
np	Rate of stress-dependency of plastic shear modulus	[-]
p_{ref}	Reference pressure	[kN/m ²]
ϕ_{cv}	Constant volume friction angle	[°]
ϕ_p	Peak friction angle	[°]
c	Cohesion	[kN/m ²]
σ_t	Tension cut-off and tensile strength	[kN/m ²]
R_f	Failure ratio	[-]
$(N_1)_{60}$	Corrected SPT value	[-]
f_{dens}	Densification factor	[-]
f_{Epost}	Post-liquefaction factor	[-]

The Wildlife National Geotechnical Instrumentation Site was selected as the primary site for this study to determine material model parameters relevant to liquefaction analysis. The University of British Columbia sand model, the calibrated UBC3D-PLM available for the Wildlife site, and the soil profile parameters were obtained from this study. The soil parameters assigned to the finite element model are

listed in Table 3.9. Petalas et al. (2013) explained the physical meaning of these parameters and the details of the model, which can be found in the previous chapters.

Table 3.9 UBC3D-PLM model parameters

Symbol	Value	Unit
ϕ_{cv}	22	°
ϕ_p	23	°
c	0	kN/m ²
K_G^e	855	-
K_G^p	250	-
K_B^e	599	-
n_e	0.5	-
n_p	0.5	-
m_p	0.5	-
R_f	0.81	-
P_A	100	-
σ_t	0	kN/m ²
fac_{hard}	0.2	-
$N_{1.60}$	8	-
fac_{post}	0.02	-

3.3 Establishment Of the Finite Element Model

In this study, both two-dimensional and three-dimensional finite element models have been developed to investigate the effects of jet grouted columns on liquefaction in detail. These models were prepared to evaluate different soil, seismic, and structural scenarios.

The effects of socket length and unconfined compressive strength of the jet grout column on the liquefaction behavior of the jet grout column were investigated in detail

in the 2D models. Many models with different socket lengths and unconfined compressive strengths were established.

Three-dimensional models were used to investigate the effects of cellular jet grouting on liquefaction behavior. Earthquakes of different periods were used in the models and their effects were also studied.

Mesh calibration was to be performed to improve the consistency of the finite element analysis. The DeepSoil program was used to perform a series of comparative analyses and calculate the optimal mesh size.

3.3.1 Mesh Calibration

In this study, mesh calibration was performed to increase the reliability of the soil behavior of the analyses. In Plaxis software, dynamic input motion was applied to the soil profile (Free field conditions) shown in Figure 3.9. The model is 80 meters wide and 36 meters high. As a result of the analysis, the soil period vs. $P_{sa}(g)$ relationship was obtained on the graph. The same soil conditions were then modeled using the DeepSoil software and a 1D soil response analysis was performed. The soil parameters of the DeepSoil model are shown in Figure 3.10. The Soil Period vs. $P_{sa}(g)$ plot was obtained from this model and used for comparison.

When the Soil Period vs. $P_{sa}(g)$ plot shown in Figure 3.11 is analyzed, the medium mesh and 3% damping rate gave the most compatible result. The rest of the study was performed according to this mesh size and damping ratio. The Dinar earthquake record was used as the input motion in both models for the comparisons.

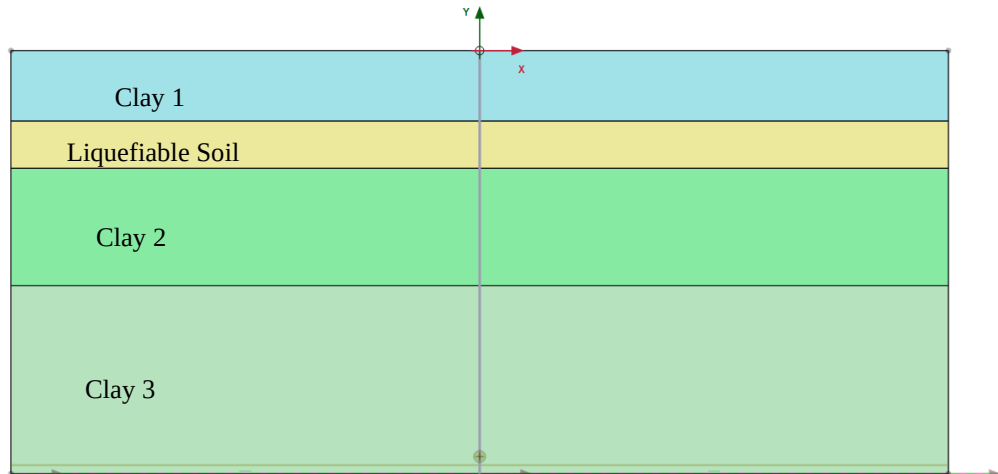


Figure 3.9 Plaxis model for mesh calibration

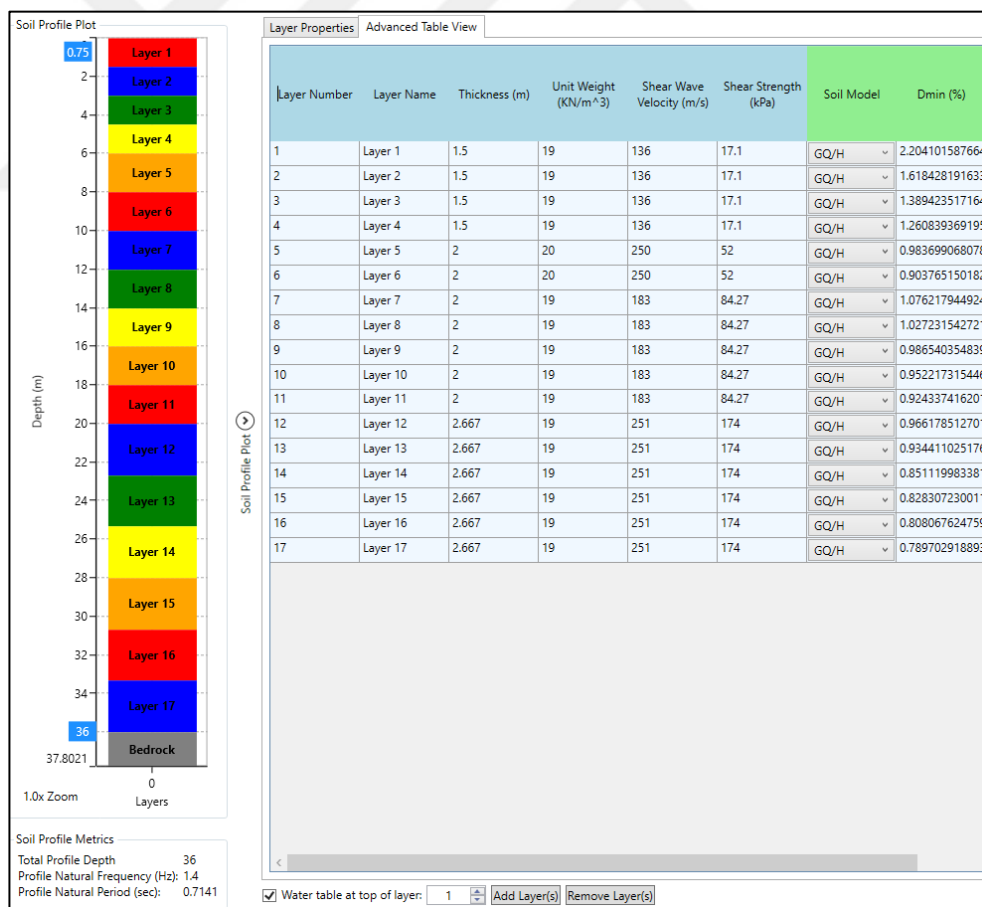


Figure 3.10 DeepSoil Profile

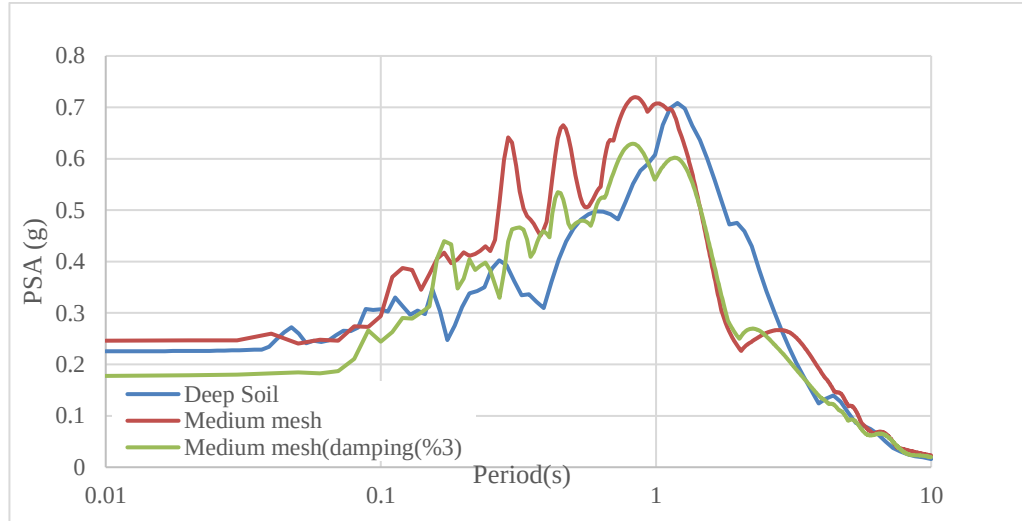


Figure 3.11 Period vs. Psa(g) Graph obtained from Plaxis and DeepSoil

3.3.2 Establishment of 2D Models

Numerical modeling utilized Plaxis 2D software running under plain-strain conditions and a finite element method (FEM). The FEM effectively manages the complexity of the boundary conditions. The examination model spans 80m in width and 36m in depth and is made up of four sequential soil layers. Soft clay exists from the surface to a depth of 6m, liquefiable sandy soil flows between 6m and 10m, whereas medium stiff clay defines the 10m to 20m range. The model contains a layer of stiff clay, measuring from 20m to 36m deep. The soil profile depth in the model is 36 meters and the natural frequency of this soil is 1.4 Hz. Understanding the seismic behavior of the ground and its effects on the model analysis is important. The boundaries in the x-direction of the model achieved "free field" boundary conditions (Refer to Figure 3.12). The jet grout column's diameter was set as $B=60$ cm with a center-to-center spacing of $S=3B$. While investigating the influence of a basement and impermeable foundation that covers the liquefiable soil layer is beyond the scope of this proceeding, the results presented in this study pertain to a model where socket length and the strength of jet-grout columns were the sole variables. The jet-grouting section of the model is bounded on both sides by impermeable diaphragm walls, and the columns themselves are located between elevations -6 and -10.

When evaluating the results of the analysis, the data obtained from the area in the center of the remediated area was used (Figure 3.12). as a reference to evaluate the liquefaction effects in the 2D study. This area was selected because it facilitates the analysis of the response of the jet grout columns as well as the surrounding soil.

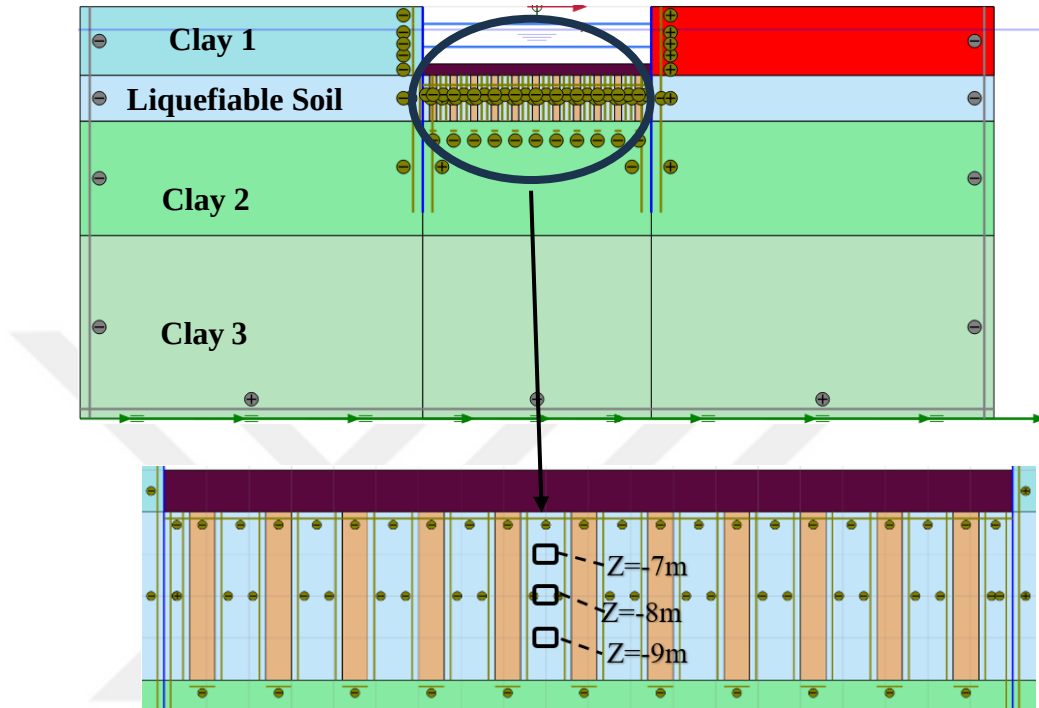


Figure 3.12 Plaxis 2D model and cross-sectional view

3.3.2.1 Review of 2D models

The study of the effect of the socket length and the unconfined compressive strength of the jet grout column on the liquefaction behavior of the jet grout column was performed in detail in 2D models. The analysis involved eight different models, with the "FF" model illustrating the initial state without any remediation (Figure 3.13), and the "A0" model showing jet grout columns with a compressive strength of 2 MPa and without sockets (Figure 3.14). Model "A1" depicted a compressive strength of 2 MPa and a socket length of 1m (1.67B) (Figure 3.15). The "A3" model has a compressive strength of 2 MPa and a socket length of 3 meters (Figure 3.16), while the "A6" model has a length of 6 meters (Figure 3.17). The "B0" model includes columns with a compressive strength of 15 MPa and no socket (Figure 3.18). This model was used to compare the effects of compressive strength with the "A0" model. The Model "B3"

depicted a compressive strength of 15 MPa and a socket length of 1m (5B). Model "B6" depicted a compressive strength of 15 MPa and a socket length of 6m (10B). These models are an important tool to understand the effect of different parameters on liquefaction and the results obtained will be analyzed in detail later in the study. Models designated as B code were also utilized for investigating the performance of jet grout columns when socketed. Details of the models are summarized in Table 3.10.

Table 3.10 Summarized of models.

Analyses Name	Description
A0	$q_u = 2 \text{ MPa}$, $L_{\text{socket}} = 0$
A1	$q_u = 2 \text{ MPa}$, $L_{\text{socket}} = 1.67B$
A3	$q_u = 2 \text{ MPa}$, $L_{\text{socket}} = 5B$
A6	$q_u = 2 \text{ MPa}$, $L_{\text{socket}} = 10B$
B0	$q_u = 15 \text{ MPa}$, $L_{\text{socket}} = 0$
B3	$q_u = 15 \text{ MPa}$, $L_{\text{socket}} = 5B$
B6	$q_u = 15 \text{ MPa}$, $L_{\text{socket}} = 10B$
FF	Free Field

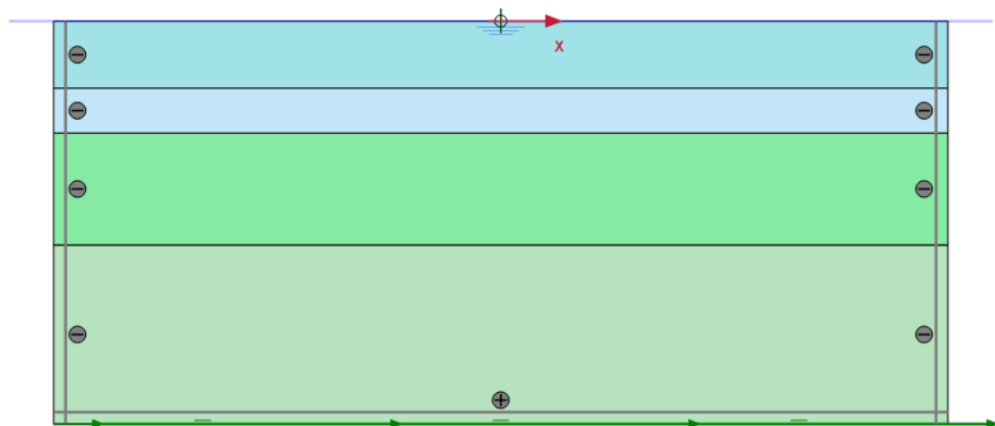


Figure 3.13 Free Field (FF)

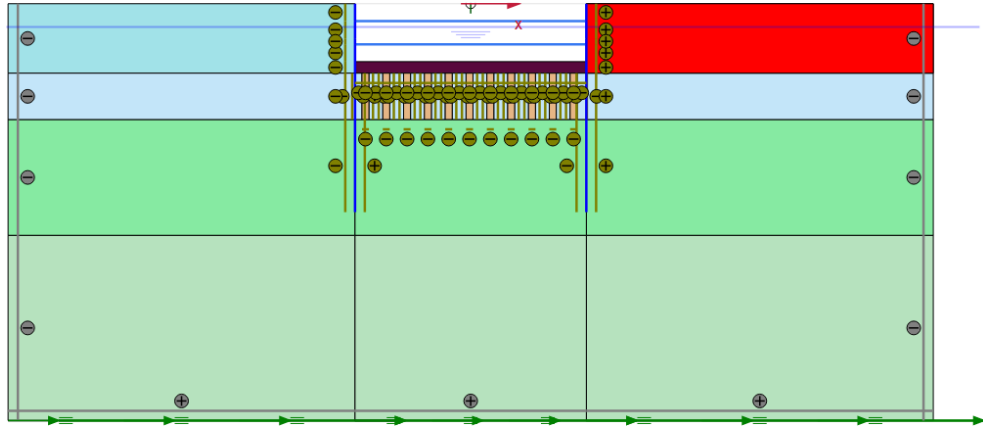


Figure 3.14 A0 model with $q_u = 2 \text{ MPa}$, $L_{\text{socket}} = 0$

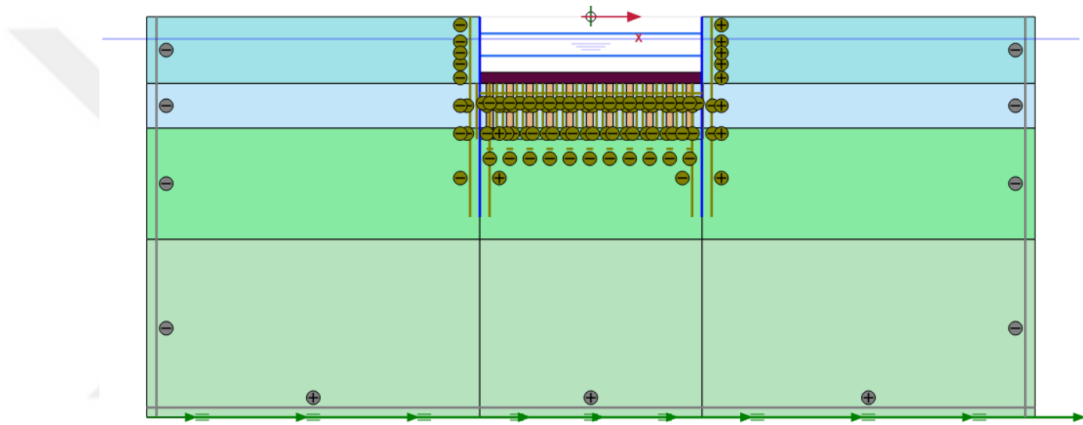


Figure 3.15 A1 model with $q_u = 2 \text{ MPa}$, $L_{\text{socket}} = 1.67B$

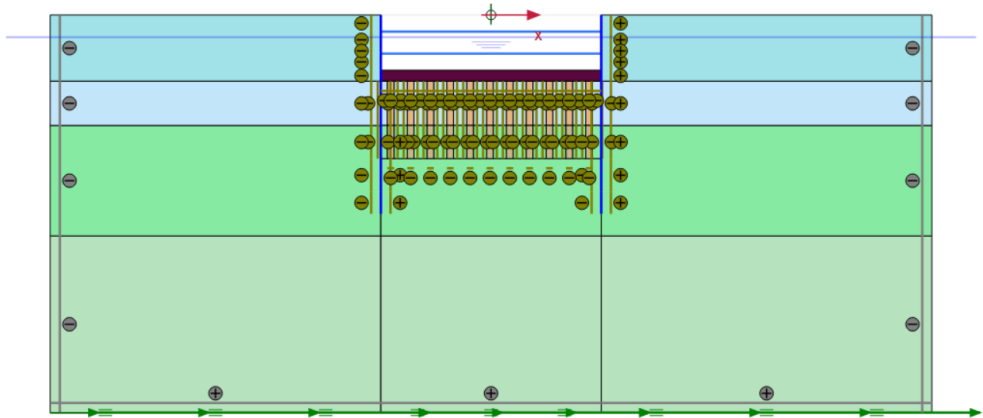


Figure 3.16 A3 model with $q_u = 2 \text{ MPa}$, $L_{\text{socket}} = 5B$

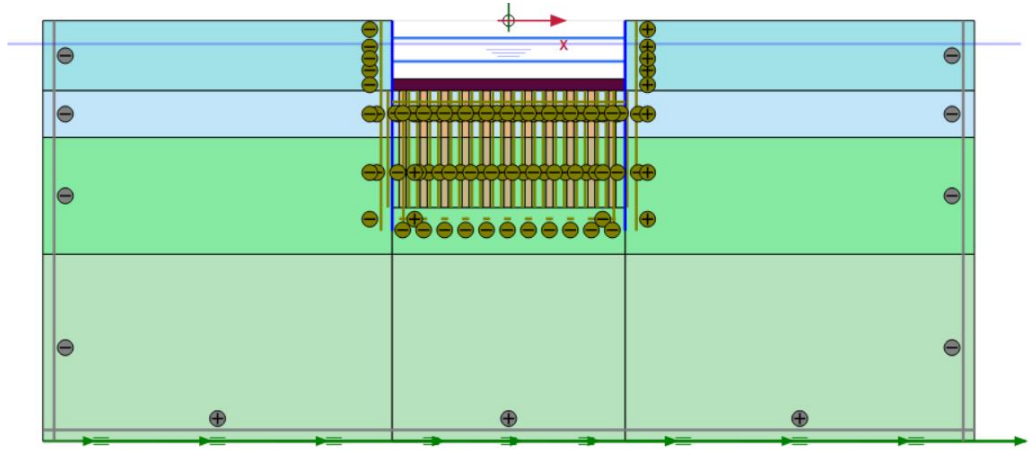


Figure 3.17 A6 model with $q_u = 2 \text{ MPa}$, $L_{\text{socket}} = 10B$

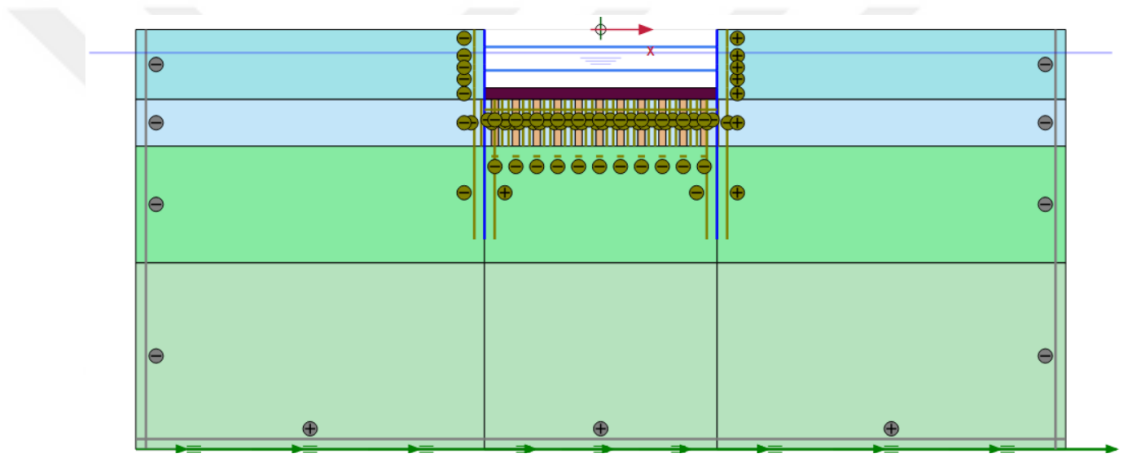


Figure 3.18 B0 model with $q_u = 15 \text{ MPa}$, $L_{\text{socket}} = 0$

3.3.2.2 Dynamic Input Motion

This section describes the earthquake record used in the 2D analysis and its application to the Plaxis program. The main objective of the study is to obtain a liquefied soil layer under dynamic loading and to investigate the effect of jet grout columns on liquefaction. For this purpose, an earthquake record was selected that can cause liquefaction. As a result of the research, the Hector Mine earthquake that occurred in the USA on October 16, 1999, with a magnitude of 7.1 was used. This earthquake was triggered by the 1992 Landers earthquake and deformed nearby fault lines both vertically and horizontally. The focus of the earthquake was recorded at a depth of 20 kilometers and the epicenter was located at 34.603° N latitude and

116.265° W longitude. The earthquake properties used in analysis are shown in Figure 3.19. The maximum ground acceleration was about 0.59 g.

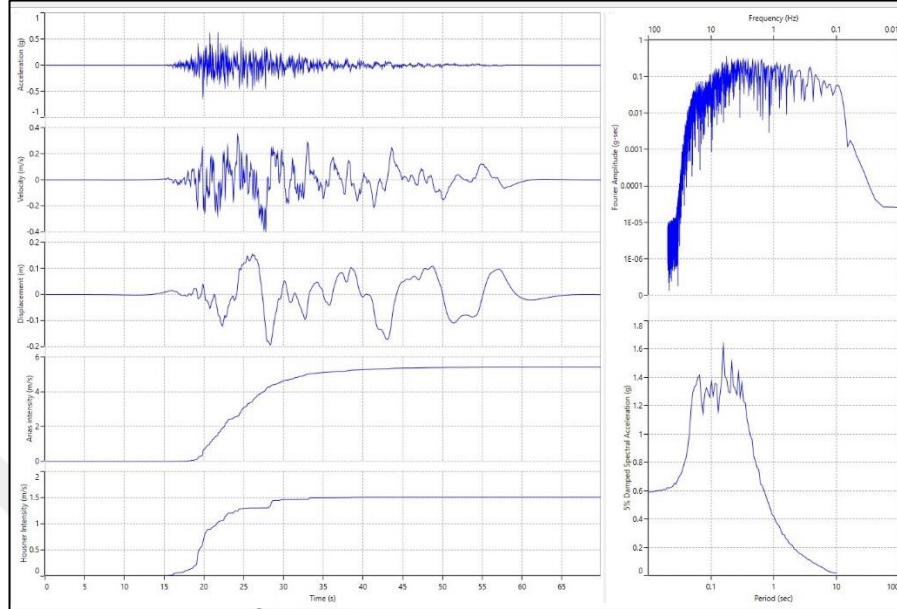


Figure 3.19 Hector-Mine earthquake properties.

The earthquake record was subjected to some processing to obtain more consistent results in the Plaxis software. First, the earthquake record was passed through a 20 Hz low pass filter. This was used to remove high frequency components, often referred to as 'noise', which can potentially distort the analysis results. The effective duration of the earthquake recording was then determined, and a baseline correction was applied to this duration. This correction was applied to make the analysis more accurate and reliable. Previous studies have shown that velocity inputs more consistently model soil behavior. Therefore, the earthquake input motion is given as velocity in this study. This method provides a more accurate simulation of the seismic response of the soil and increases the accuracy of the analysis. The final dynamic motion input to the Plaxis program is shown in Figure 3.20 at the end of these processes.

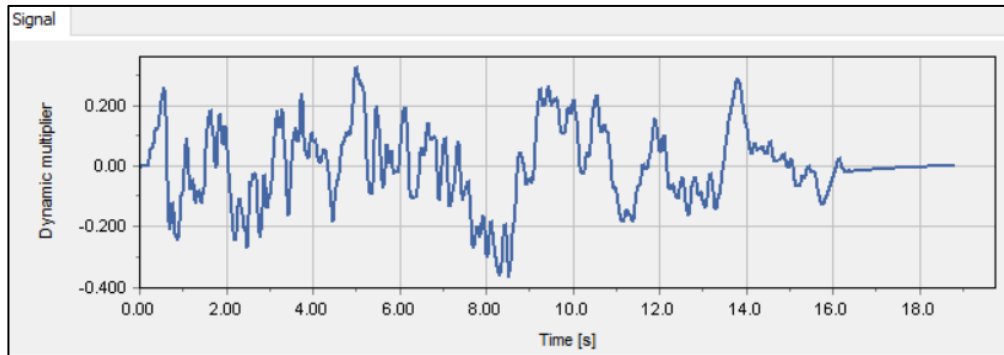


Figure 3.20 Velocity-time history applied in models.

3.3.2.3 Analysis Phases

During the design of the models, 8 different phases have been used.

- Phase 1: The soil profile was defined and the K0 procedure was applied.
- Phase 2: Diaphragm walls and interface elements are defined. The plastic analysis type is used.
- Phase 3,4,5: The area between the diaphragm walls was gradually excavated and rigid struts were placed between them. Plastic type of calculation is used.
- Phase 6: jet grouted columns were defined, and necessary interface elements were defined. Plastic type of calculation was used.
- Phase 7: The non-porous foundation above the jet grouted columns was defined.
- Phase 8: In this phase, the line displacement at the base of the foundation is defined and earthquake record multipliers are applied. This is the stage where the dynamic analysis is performed.

The plastic calculation type calculates the permanent deformation of the soil and structural elements. The dynamic type analyzes the behavior of the soil under dynamic loads.

3.3.3 Establishment of 3D Models

The 3D analyses conducted in this study were performed using Plaxis 3D finite element software. The study's main purpose is to investigate the effectiveness of a new application method, cellular jet grouting, on liquefaction. Jet grouting cells were installed at the liquefiable soil's center (Figure 3.22). The model features liquefiable sand soil from 0 to -10, and medium stiff and stiff clay in the lower layers. The modeling parameters have been detailed in the previous sections. It is observed that the compressive strength of the jet grout cells in these models is 2 MPa. The dimensions in both x and y directions are modeled as 27 m, and the boundary conditions are modeled as free field. The model is shown in Figure 3.21.

The study investigated the effects of the seismic characteristics of the two earthquakes on liquefaction. The dynamic input motion segment contains details about the earthquakes. The results section provides a detailed analysis of the scenarios.

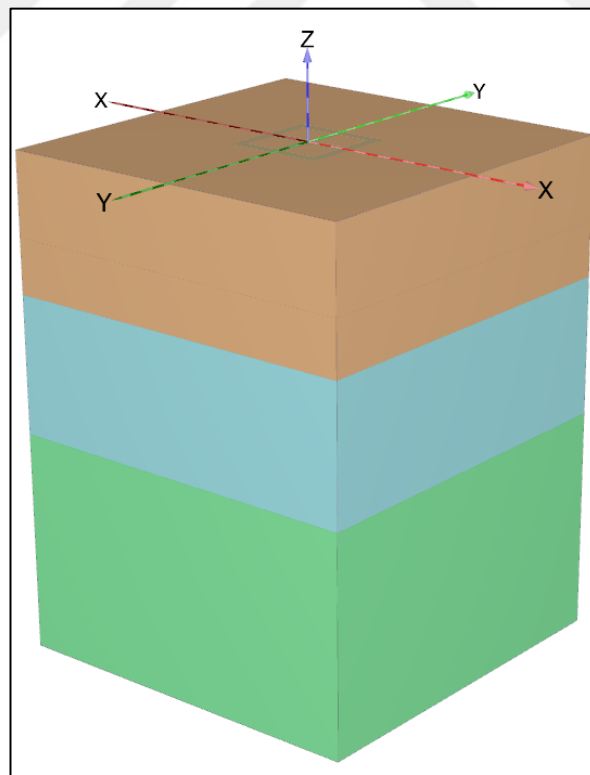


Figure 3.21 Plaxis 3D model, cellular jet grout

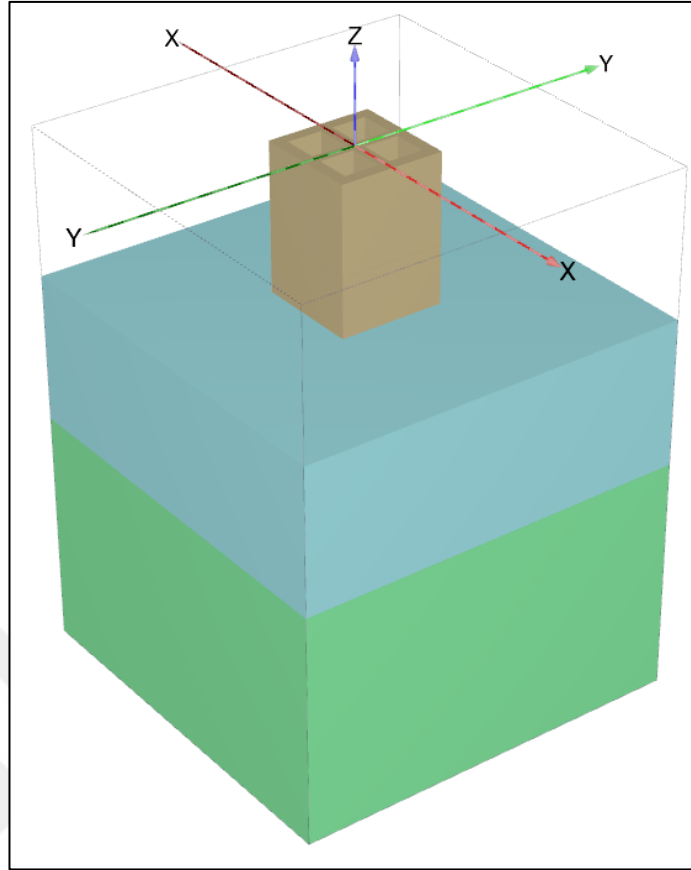


Figure 3.22 Cellular jet grout application

3.3.3.1 *Dynamic Input Motions*

In this section of the study, the Hector Mine and Loma Prieta earthquake records are analyzed to investigate the effects of liquefaction. The Hector Mine earthquake, occurred on October 16, 1999, in California, USA, with a magnitude of 7.1. With a magnitude of 7.1, it has been the subject of numerous studies due to its high value of acceleration. On the other hand, the Loma Prieta earthquake, which also happened in California, USA on October 17, 1989, is notable for its long duration.

The maximum ground acceleration recorded during the Hector Mine earthquake was 0.59g in the x direction (Figure 3.23) and 0.686g in the y direction (Figure 3.24). This earthquake has a dominant period of about 0.2 seconds. The Loma Prieta earthquake was scaled based on the maximum surface acceleration of the Hector Mine earthquake and had a dominant period of around 0.8 seconds (Figure 3.25-3.26).

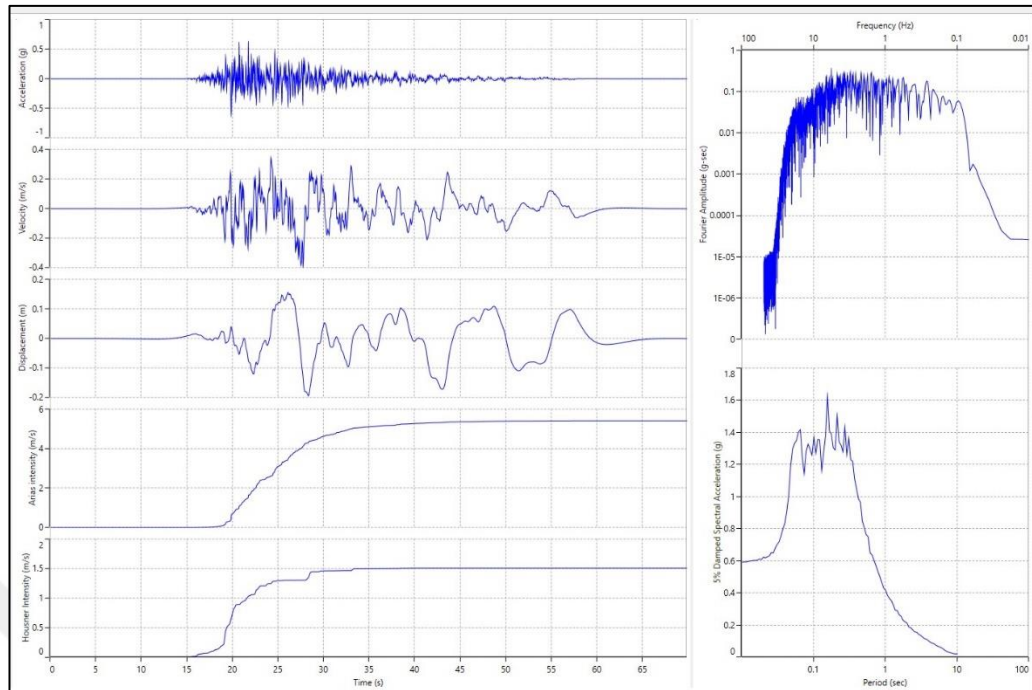


Figure 3.23 Hector Mine earthquake record, x direction

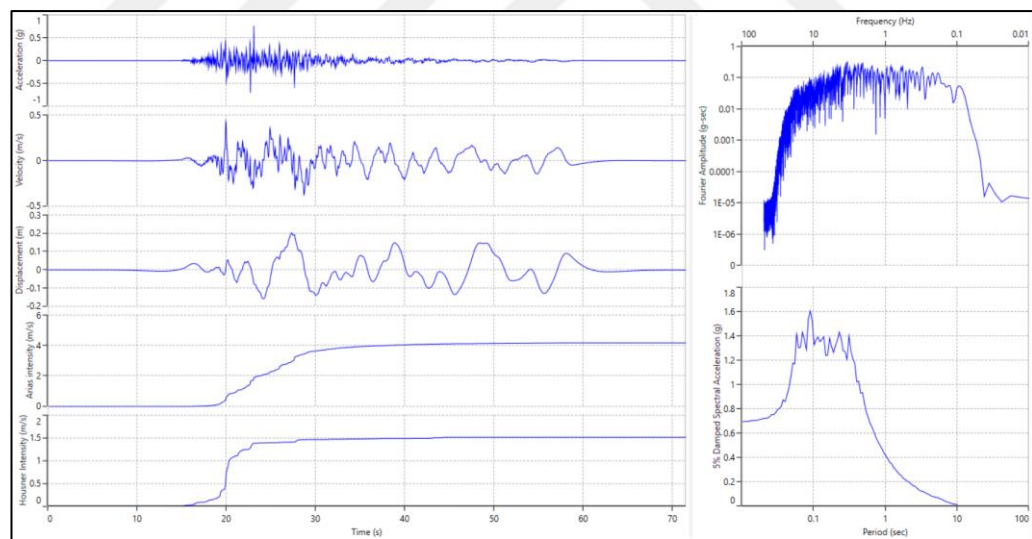


Figure 3.24 Hector Mine earthquake record, y direction

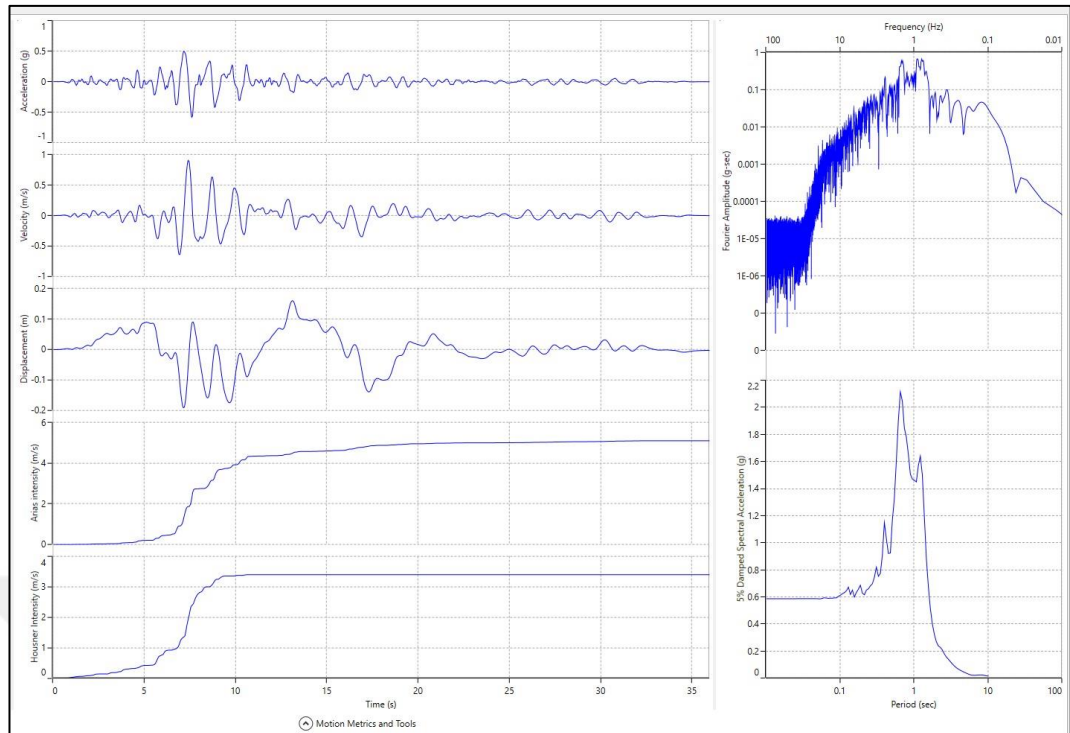


Figure 3.25 Loma Prieta earthquake record scaled, x direction.

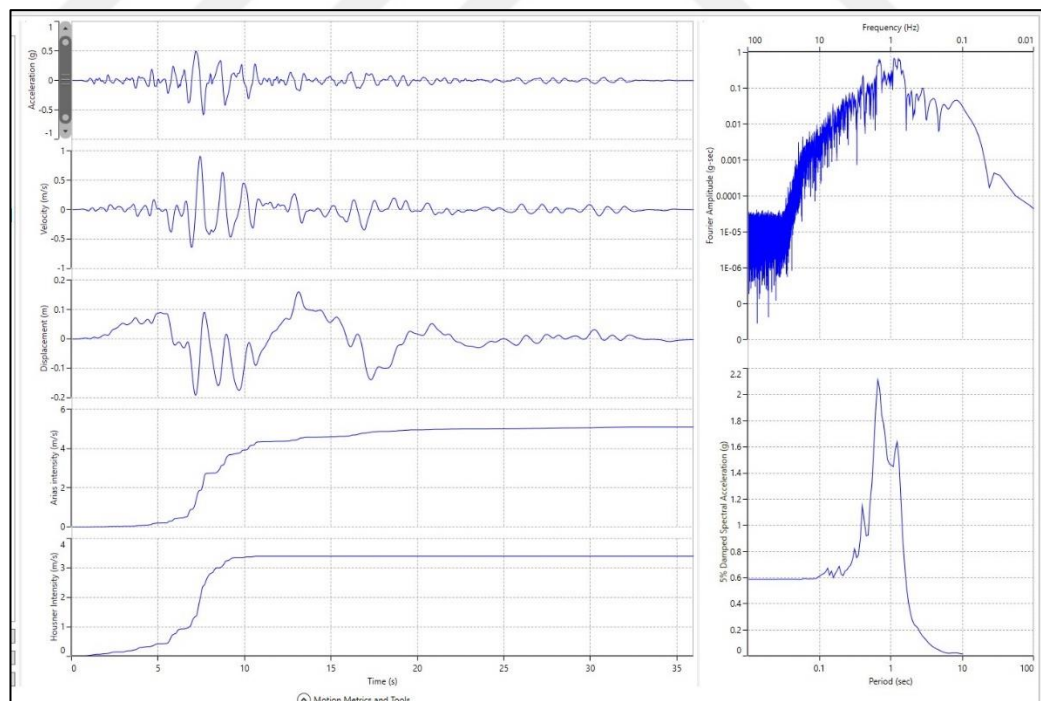


Figure 3.26 Loma Prieta earthquake record scaled, y direction.

Low-pass and baseline correction were conducted before defining the input motions to the model. A frequency of 20 Hz was used as a reference for the low-pass filter, which effectively removed high-frequency components from the recording and provided more accurate results. The purpose of baseline correction is to fix the velocity and displacement components that remain in the recording after the motion. After making these corrections, we defined the input motions as the velocity at the base of the model for the effective duration of the earthquake, aim is to shorten the analysis time. Input motion for Hector Mine earthquake shown in Figure 3.27 and Loma Prieta earthquake shown in Figure 3.28 as velocity-time histories.

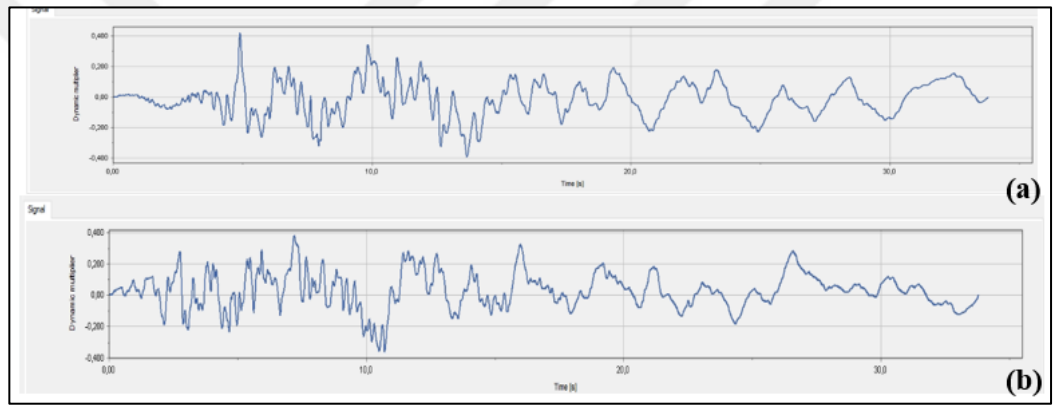


Figure 3.27 Hector Mine earthquake input motions (m/s), (a) x direction, (b) y direction

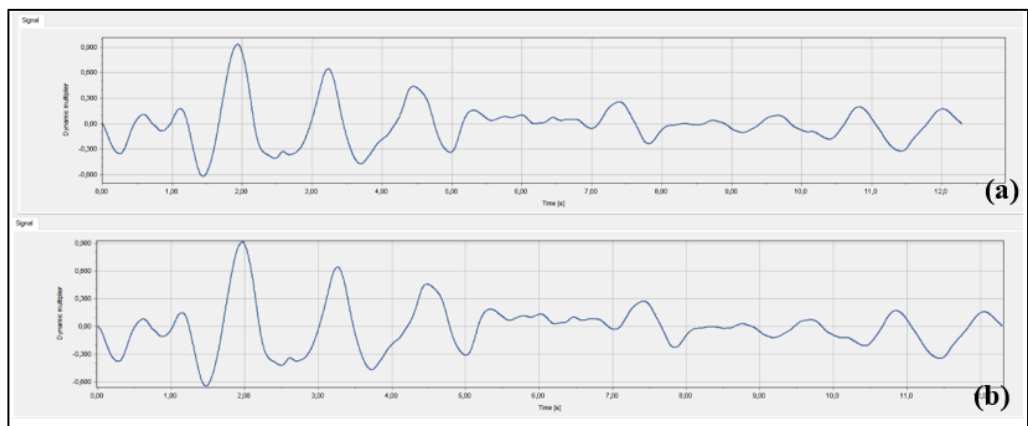


Figure 3.28 Loma Prieta earthquake input motions both directions (m/s), (a) x direction, (b) y direction

3.3.3.2 *Analysis Phases*

During the design of the models, 3 different phases have been used.

- Phase 1: The soil profile was defined and the K0 procedure was applied.
- Phase 2: jet grouted cells were defined. Plastic type of calculation was used.
- Phase 3: In this phase, the line displacement at the base of the foundation is defined and earthquake record multipliers are applied. This is the stage where the dynamic analysis is performed.

The plastic calculation type calculates the permanent deformation of the soil and structural elements. The dynamic type analyzes the behavior of the soil under dynamic loads.

CHAPTER FOUR

RESULTS

4.1 Introduction

This section provides a detailed discussion of the results obtained from 2D and 3D analyses. The 2D analysis results explain the effect of jet grouting column length and unconfined compressive strength on liquefaction with visual and numerical results. The 3D analysis results are evaluated using a new application method, cellular jet grouting, with different earthquake inputs. The results are explained with visual and numerical findings. The analysis results obtained are derived from the model properties, input motions, and soil parameters outlined in chapter 3. These analyses were evaluated based on the parameter of excess pore water pressure (R_u), shear stresses and shear deformations in jet-grout columns and surrounding soil. The studies present an innovative approach to the development of liquefaction mitigation methods.

4.2 Effect Of Socket Length and Uniaxial Compression Strength on Liquefaction

4.2.1 Influence of Unconfined Compressive Strength

The 2D analyses demonstrated that increasing the unconfined compressive strength values delayed, but did not completely prevent, the liquefaction behavior of the soil. Figure 4.1 presents data obtained from the center of the liquefiable soil and summarizes its behavior during the earthquake. The soil loses strength and liquefaction begins when the R_u parameter reaches approximately 0.8.

The A0 analysis refers to the case when the unconfined compressive strength of the grout columns is 2 MPa. Similarly, the B0 analysis refers to the case where the value of the unconfined compressive strength is 15 MPa. The dynamic input motion in these analyses was based on the velocity-time record of the Hector Mine earthquake.

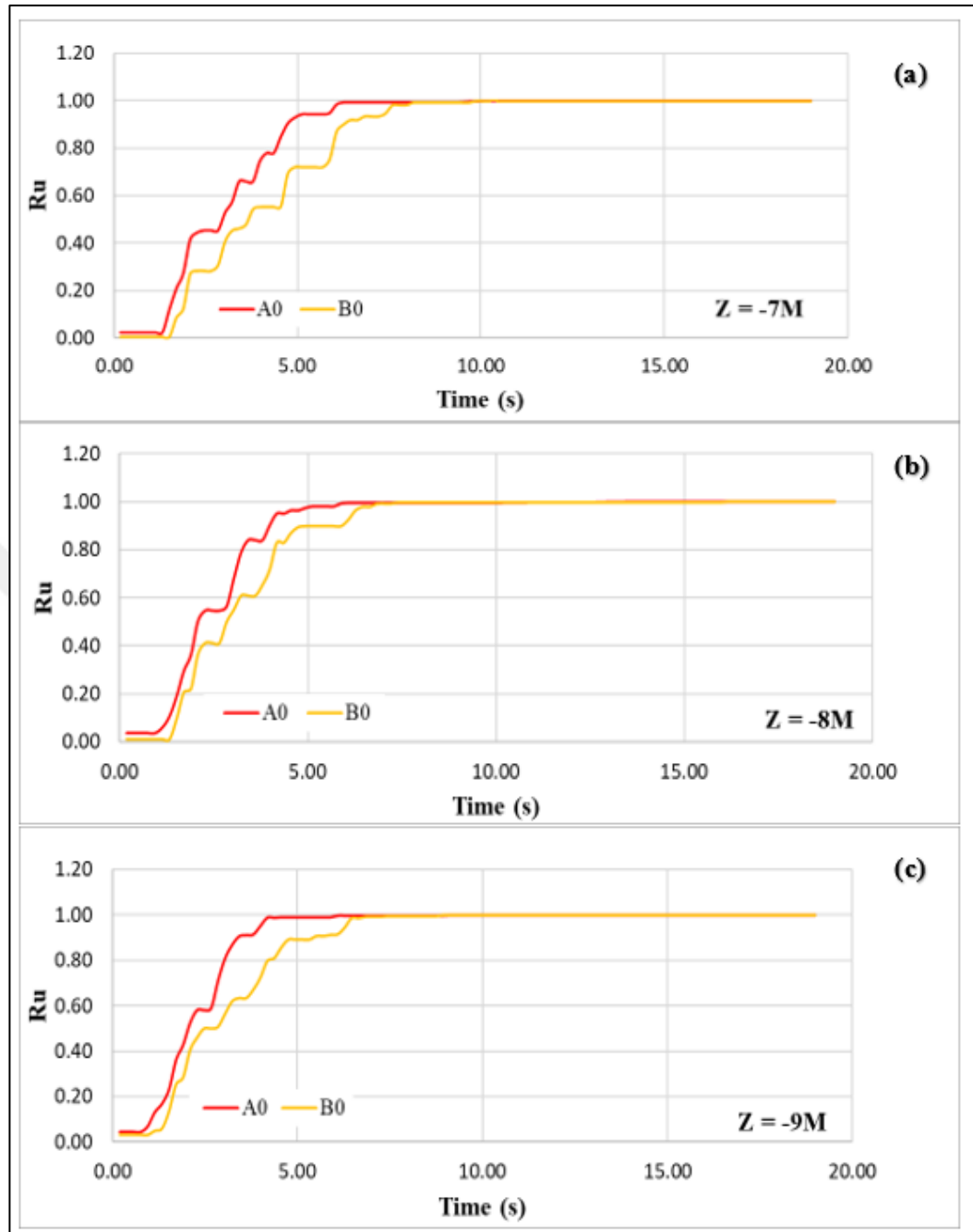


Figure 4.1 . Variation of R_u parameter in A0 and B0 Analyses a) $z = -7m$, b) $z = -8m$, a) $z = -9m$

As the unconfined compressive strength increases, the shear strain is reduced in the soil. This is because the soil has a more rigid composition due to the increased unconfined compressive strength. Figure 4.2 presents the data obtained at different depths of the soil during the earthquake.

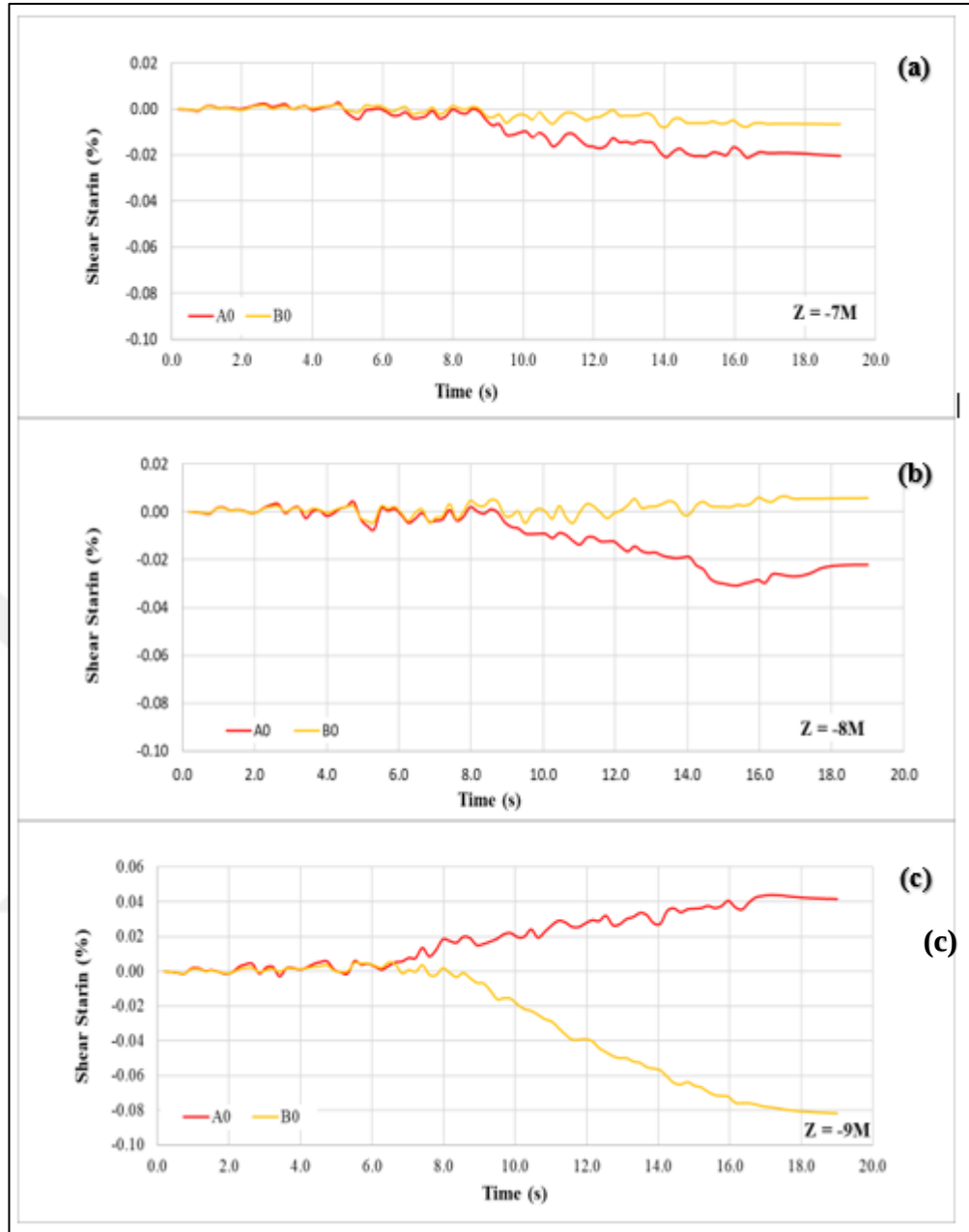


Figure 4.2 Variation of Shear strain in A0 and B0 Analyses a) $z = -7\text{m}$, b) $z = -8\text{m}$, a) $z = -9\text{m}$

4.2.2 Influence of the Socket Length

This section analyzes in detail the effects of the socket length of jet grout columns on liquefaction. The definition of the analysis is presented in detail in Table 3.1. The evaluations are based on the excessive pore water ratio parameter R_u and strain values obtained from the soil. The results of the two-dimensional analysis show that the R_u parameter decreases as the socket length to the clay increases. This is particularly

noticeable in the A6 analysis, where liquefaction is completely prevented. Figure 4.3 presents R_u data obtained from the center of the liquefiable soil and summarizes its behavior during the earthquake. The primary reason for the prevention of liquefaction is that jet grout columns behave like embedded beams when socketed, working in the first mode during earthquake motion. As a result, deformation is significantly reduced. This will be explained in detail in the following sections of this chapter.

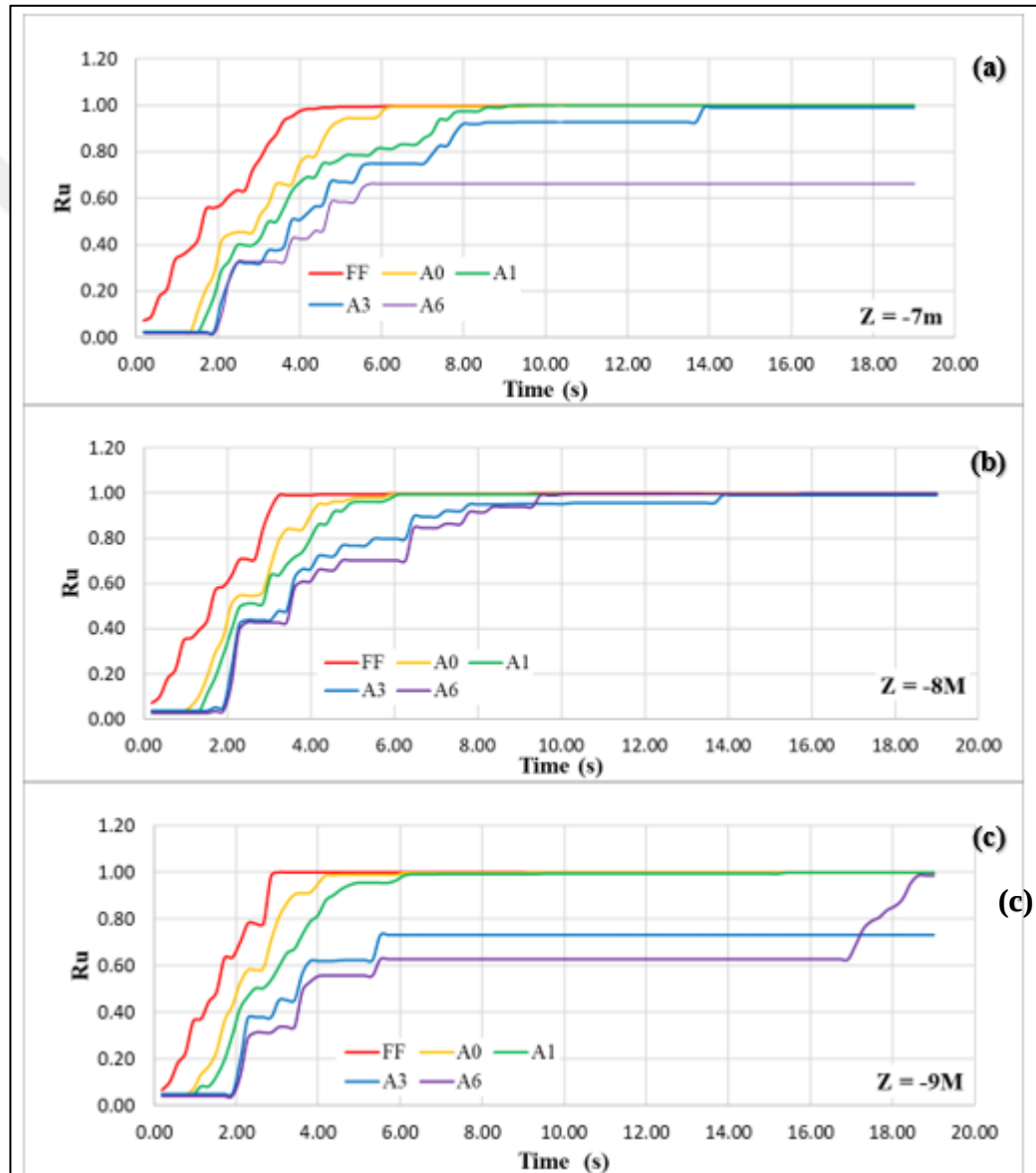


Figure 4.3 Variation of R_u in FF, A0, A1, A3, and A6 Analyses a) $z = -7m$, b) $z = -8m$, a) $z = -9m$

Figure 4.4 shows that increasing the socket length of the jet grout columns resulted in an increase in strain values. This is because jet grouted columns behave like embedded beams. Additionally, a decrease in strain values is observed as the jet grout columns approach the socket zone (around $z = -9\text{m}$), as seen in Figure 4.4. This statement confirms that the jet grout columns are performing in Mode 1. The column behavior in Mode 1 is shown in Figure 4.5.

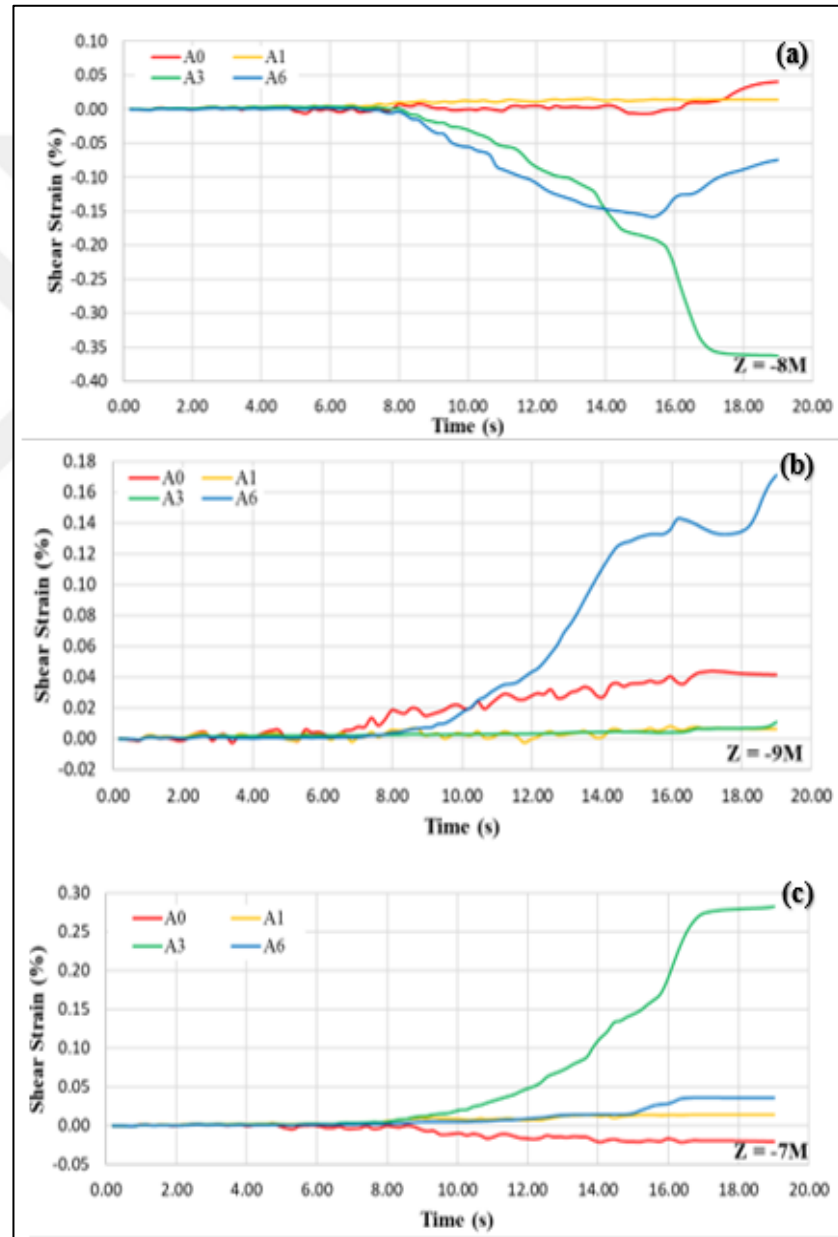


Figure 4.4 Variation of Shear Deformation in A0, A1, A3, and A6 Analyses

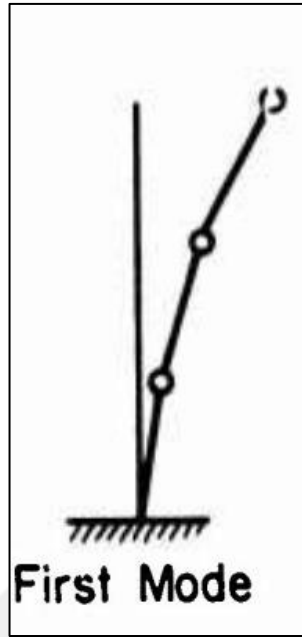


Figure 4.5 Schematic representation of first mode

Two different models were established to investigate the embedded beam behavior of jet grouted columns. The first model, named B3, assumes jet grouted columns have an unconfined compressive strength of 15 MPa and a socket length of 5 B. The second model, named B6, assumes jet grouted columns have an unconfined compressive strength of 15 MPa and a socket length of 10 B. Upon evaluating the model results, it was observed that both models had the same strain values when approaching the socket zone ($z=-9\text{m}$), but when moving away from the socket zone, both strain values increased, and the strain compatibility of the models was lost. These findings demonstrate that jet grout columns operate in the first mode when socketed. The related graphs are presented in Figure 4.6.

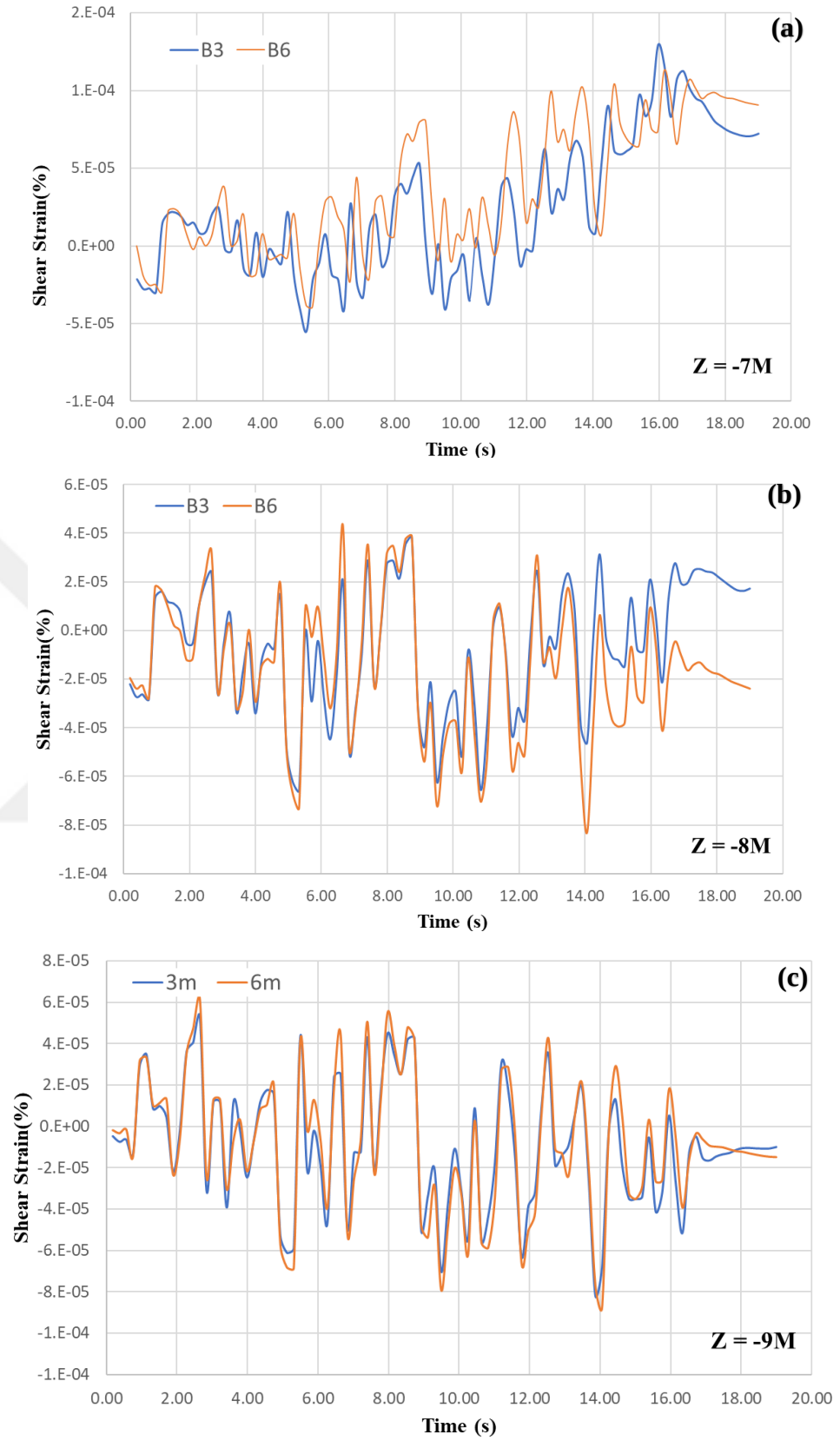


Figure 4.6 Variation of Shear strain in B3 and B6 Analyses a) z = -7m, b) z = -8m, a) z = -9m

4.3 Cellular Jet-Grout

This chapter evaluates the liquefaction performance of cellular jet-grouting based on the model and input motions presented in chapter 3. The effects of two different earthquake contexts are considered: the Hector Mine earthquake and the Loma Prieta earthquake. The results of the studies are detailed, with an evaluation of the shear strains in the ground and the R_u parameter, which is the excessive pore water parameter. The relations of these results to the general objective of the thesis are emphasized.

For the evaluation, data from the center of the jet grout cell was utilized. This area is shown in Figure 4.7. As the model is symmetrical, the values in the center of a single jet grout cell are equivalent to those in the other cells. Consequently, only one point was evaluated.

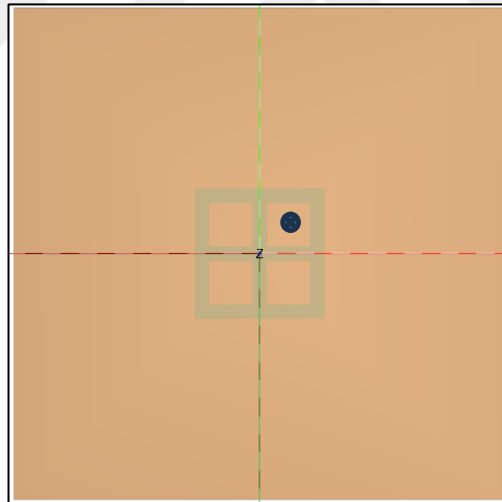


Figure 4.7 The data location is taken from the center of the jet grout cell.

4.3.1 Hector Mine Earthquake

This section examines the analysis results of the Hector Mine earthquake. Figure 4.8 shows that the R_u parameter values at -2 m in the free field model reach a value of 1 around 7 seconds, indicating that liquefaction is occurring. Liquefaction is delayed as the depth increases, but liquefaction is finally realized. Upon analyzing the cellular

jet grouting model, it is evident that the R_u parameter's maximum value remains between 0.7-0.8, indicating the absence of liquefaction. The graph in Figure 4.8(a) illustrates that the R_u parameter exhibits a more rapid increase in the improved soil. However, ultimately, the improved soil prevented liquefaction.

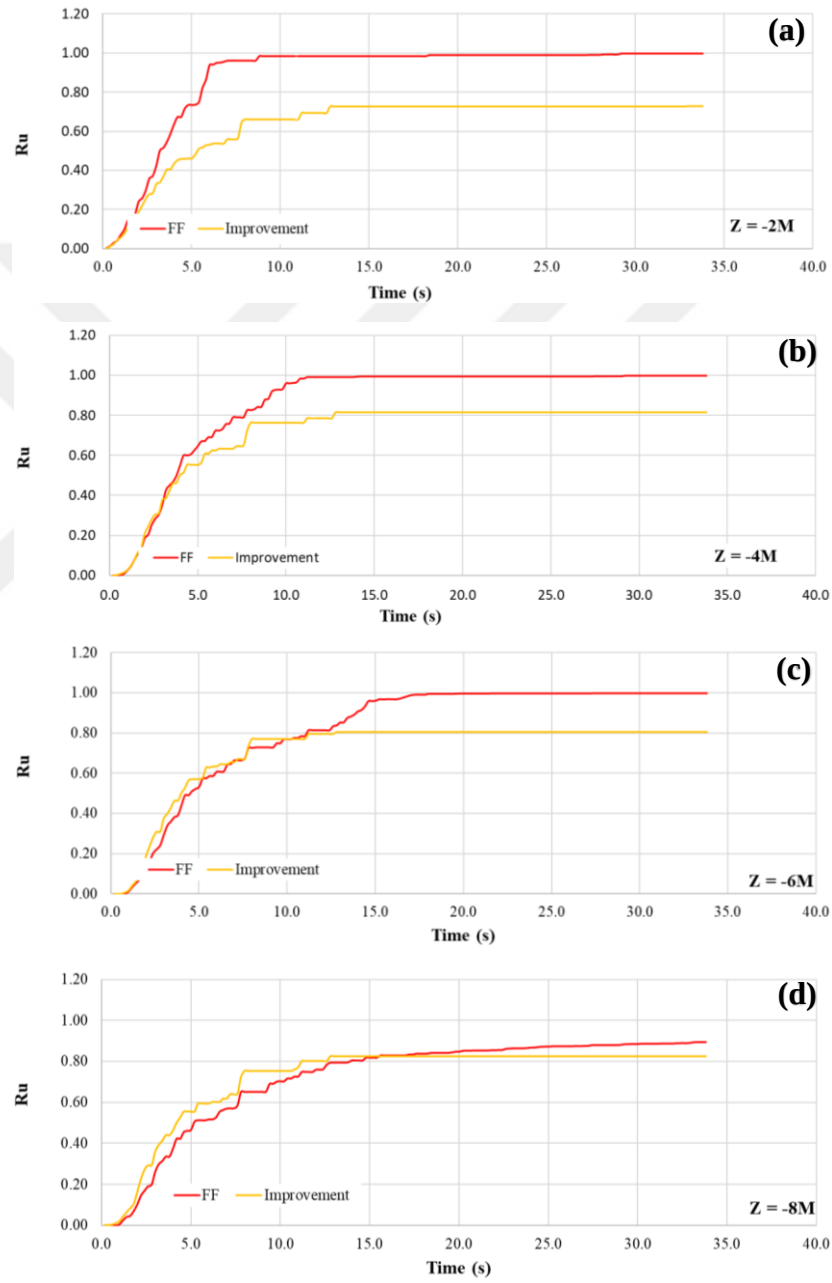


Figure 4.8 Variation of R_u in FF and Improvement a) $z = -2m$ b) $z = -4m$, c) $z = -6m$ d) $z = -8m$

Figure 4.9 shows the graphs of earthquake duration vs shear strain values. When the graphs are examined, the shear strain values in the improved soil are significantly lower. This demonstrates that liquefaction has been prevented. These findings demonstrate the success of the cellular jet-grouting application.

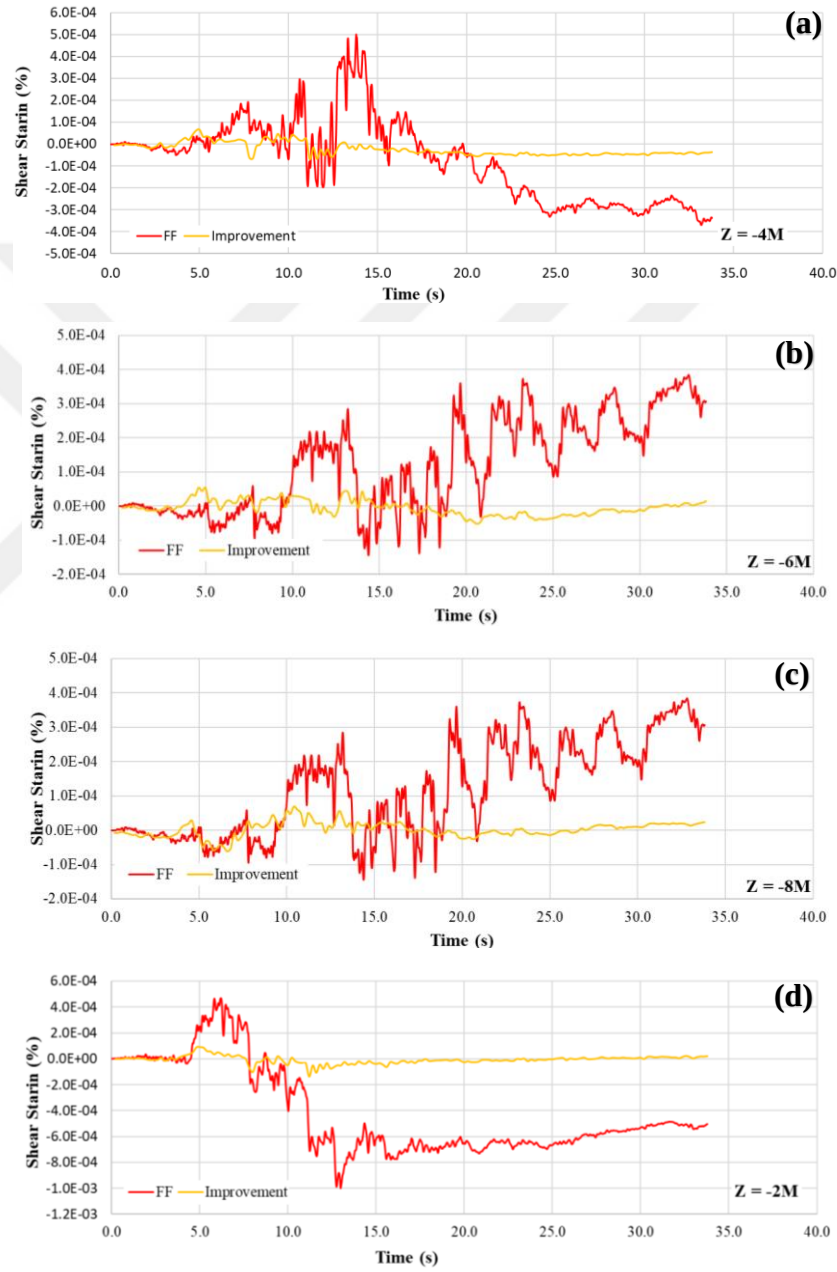


Figure 4.9 Variation of shear strain in in FF and Improvement a) z = -2m b) z = -4m, c) z = -6m
d) z = -8m

4.3.2 Loma Prieta Earthquake

This section evaluates the results of the analysis conducted using the input parameters of the Loma Prieta earthquake. The purpose of this investigation is to examine the liquefaction prevention mechanism when the dominant periods of the soil and the earthquake are compatible. Chapter 3 provides a detailed description of the parameters used in the analysis, model specifications, and input motion.

When analyzing the results obtained, liquefaction behavior is observed in the free field (FF) soil, while liquefaction is prevented in the modified soil. Figure 4.10 displays the R_u parameter obtained at different depths, which remained in the range of 0.7-0.8 in the improved soil. A detailed visualization of the liquefaction behavior for each depth is presented in the appendix of the study.

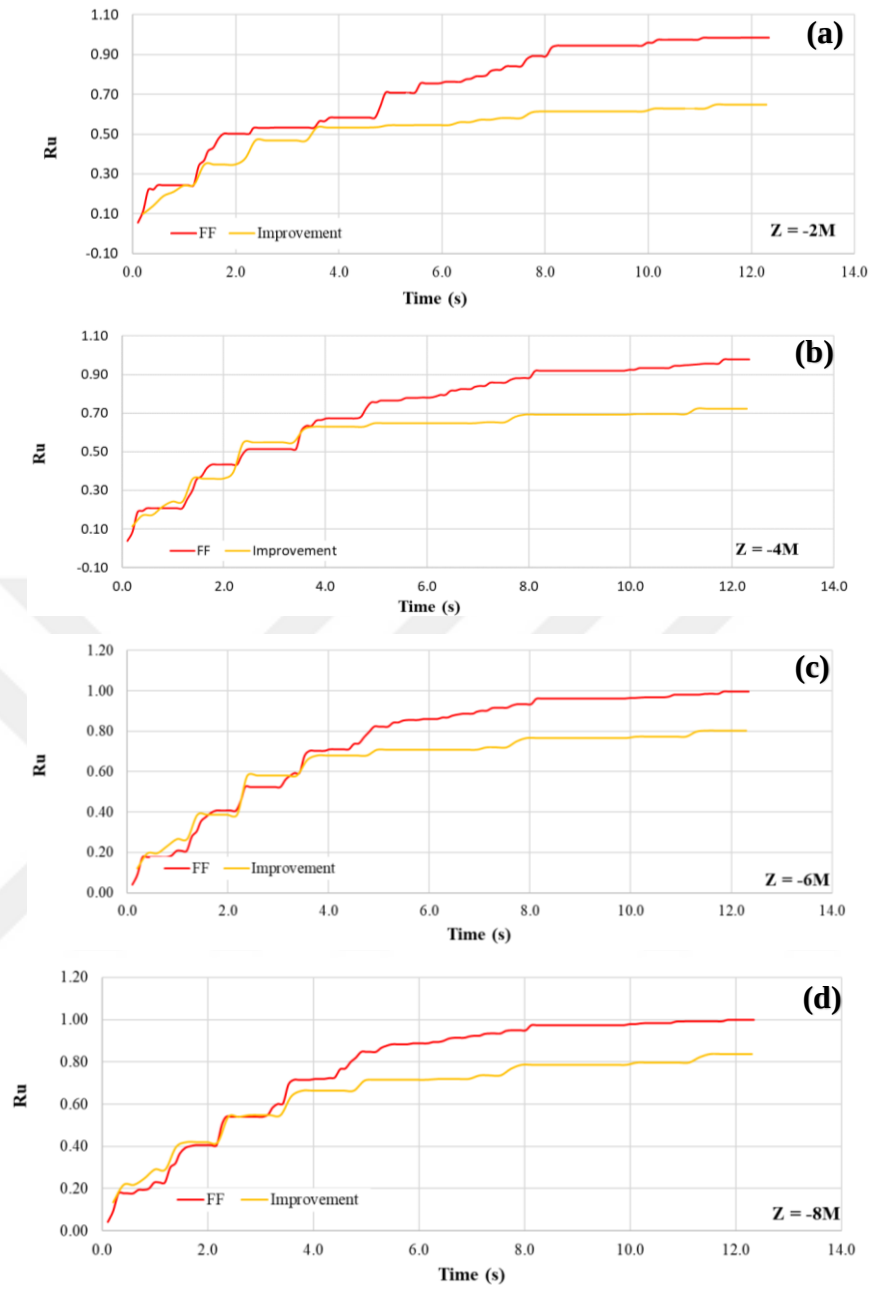


Figure 4.10 Variation of R_u in FF and Improvement a) $z = -2m$ b) $z = -4m$, c) $z = -6m$ d) $z = -8m$

Figure 4.11 illustrates the relationship between earthquake duration and shear strain values during the Loma Prieta earthquake. The analysis of the graphs shows significantly lower shear strain values in the improved soil, indicating the prevention of liquefaction. This finding reveals the success of the cellular jet-grouting application.

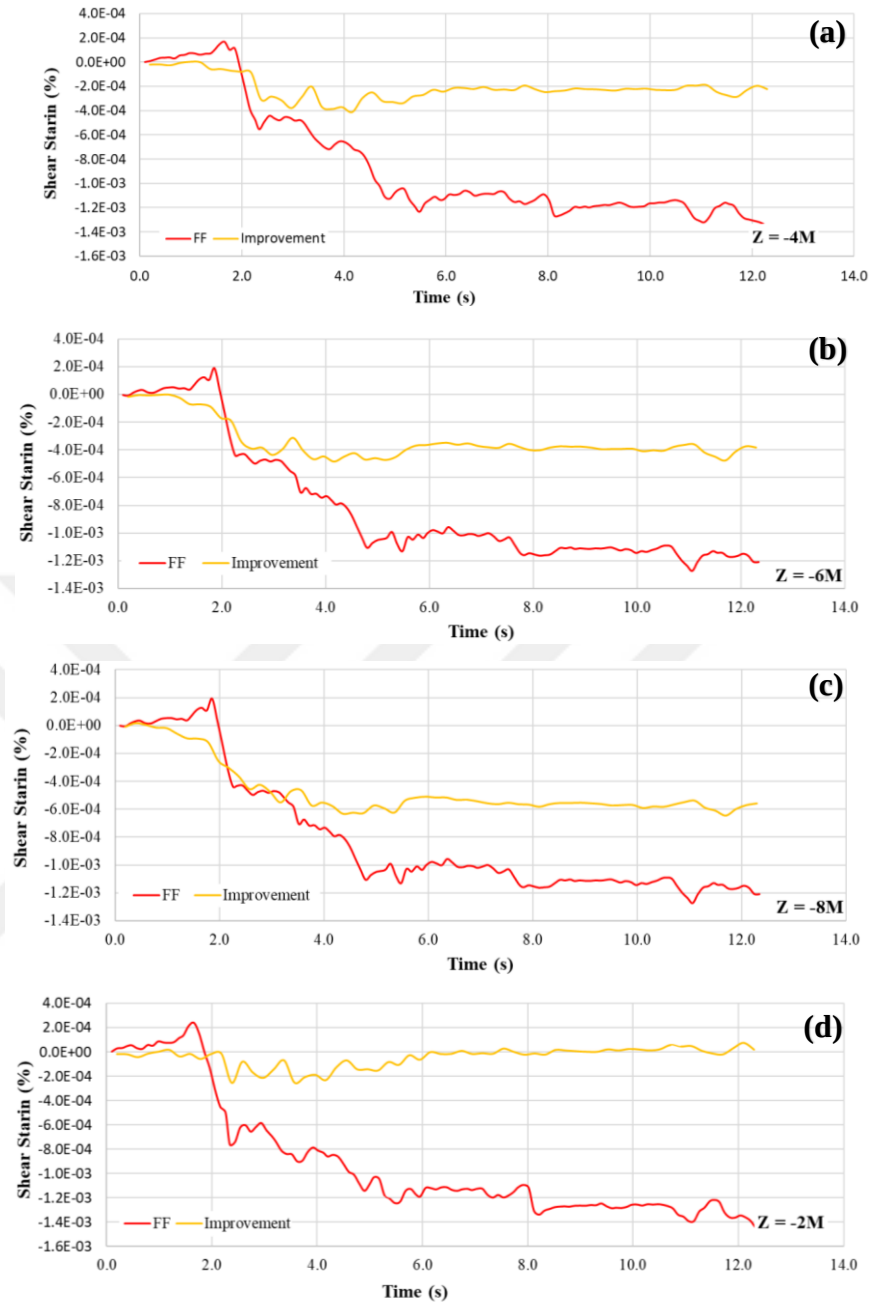


Figure 4.11 Variation of shear strain in in FF and Improvement a) $z = -2m$ b) $z = -4m$, c) $z = -6m$
d) $z = -8m$

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

This study aims to investigate the effects of socket length and unconfined compressive strength of jet grout columns on soil liquefaction and to evaluate the success of cellular jet grouting in mitigating liquefaction. The study was conducted in the wildlife liquefaction array area, where the liquefaction behavior was analyzed using Plaxis 2D and Plaxis 3D finite element software. The first part of the study focuses on the effect of jet grouting columns, socket length and unconfined compressive strength, while the second part focuses on the effectiveness of a new application method, cellular jet grouting. The analyses showed that jet grouting columns have a significant potential to prevent soil liquefaction and increase soil resistance. The key findings of the study are presented below:

- The study found that increasing the socket length of jet grout columns resulted in a decrease in the R_u value, indicating a reduction in excessive pore water pressure. This delay in liquefaction behavior of the soil was also observed to decrease the deformation of the soil as the socket length increased.
- As the unconfined compressive strength of the jet grout columns increased, it was observed that the R_u value, which is the excess pore water parameter, decreased and the deformations in the liquefiable soil decreased.
- As the socket length of the jet grouted columns increased, the deformations decreased as the columns approached the socket area. This can be explained by the embedded beam behavior and the first mode behavior during the earthquake.
- Cellular jet grouting successfully prevented liquefaction as an innovative method. The method was evaluated with two different earthquake scenarios. Additionally, liquefaction behavior was prevented when the dominant period of the earthquake and the soil matched.

This thesis evaluates the physical properties of jet grouted columns and the effects of the new application method of cellular jet grouting on liquefaction. Suggestions for improving research on this subject are presented below:

- By repeating and improving the analyses using different earthquake scenarios and different soil parameters, the effectiveness of jet grouting applications can be evaluated under a wider range of conditions.
- Performing scaled centrifuge experiments and comparing their results with numerical analysis allows us to better understand the agreement between experimental and theoretical approaches.
- Analyses can be performed using various cell widths to investigate the effectiveness of this method under different application conditions.
- The experimental improvement rates can be presented graphically, making it easy to interpret and compare the data.
- The study of deformations and crack mechanisms in jet grout columns can provide insight into the behavior of the columns under shear stress in the subsurface since the stiffness of the jet grout is the main property against shear stresses. If cracks form in the jet grout columns, stiffness will be reduced, and pore water pressure generation increases.

REFERENCES

- Alexandros Petalas, Petalas, A. L., & Galavi, V. "Plaxis liquefaction model UBC3D-PLM." Plaxis report (2013).
- Baez Satizabal, J. I. (1995). A design model for the reduction of soil liquefaction by vibro- stone columns.
- Beaty, M., & Byrne, P. (1998). An effective stress model for predicting liquefaction behaviour of sand. *Geotechnical Earthquake Engineering and Soil Dynamics III*. <https://www.semanticscholar.org/paper/An-Effective-Stress-Model-for-Predicting-Behaviour-Beaty-Byrne/fc1c9942ad200a19e1e713ced76481c1706e79c4>
- Beaty, M. H., & Byrne, P. M. (2011). UBCSAND constitutive model version 904aR. Itasca UDM Web Site, 69.
- Bennett, M. J., McLaughlin, P. V., Sarmiento, J. S., & Youd, T. L. (1984). Geotechnical investigation of liquefaction sites, Imperial valley, California. In *Open-File Report (84-252)*. U.S. Geological Survey,. <https://doi.org/10.3133/ofr84252>
- Bray, J. D., & Sancio, R. B. (2006). Assessment of the liquefaction susceptibility of fine-grained soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 132(9), 1165–1177. [https://doi.org/10.1061/\(asce\)1090-0241\(2006\)132:9\(1165\)](https://doi.org/10.1061/(asce)1090-0241(2006)132:9(1165))
- Bray, J., Sancio, R., Durgunoglu, H., Onalp, A., Seed, R., Stewart, J., Youd, T., Baturay, M., Christensen, C., Karadayılar, T., & Emrem, C. (2001, September). C.6 Ground failure in Adapazari.
- Cooke, H. G. (2000). Ground improvement for liquefaction mitigation at existing highway bridges.
- Croce, P., Flora, A., & Modoni, G. (2014). Jet Grouting: technology, design and control.

- D. C. Andrews & G. Martin. (1999). Criteria for liquefaction of silty soils. in proc., 12th World Conf. on Earthquake Engineering (pp. 1-8). Upper Hutt, New Zealand: NZ Soc. for EQ Engrg.
- Davis, R. O., & Berrill, J. B. (2001). Liquefaction at the imperial valley wildlife site. Bulletin of the New Zealand Society for Earthquake Engineering, 34(2), Article 2. <https://doi.org/10.5459/bnzsee.34.2.91-106>
- Demir, S., & Özener, P. (2020). Parametric investigation of effectiveness of high modulus columns in liquefaction mitigation. Soil Dynamics and Earthquake Engineering, 139, 106337. <https://doi.org/10.1016/j.soildyn.2020.106337>
- Durgunoglu, H. T., Chinchelli, M., Ikiz, S., Emrem, C., Hurley, T., & Catalbas, F. (2004). Soil improvement with jet-grout columns: a case study from the 1999 Kocaeli earthquake.
- Goughnour, R. R., Pestana, J. M., & International Conference on Ground Improvement Techniques (Singapore) 1998: (1998). Mechanical behavior of stone columns under seismic loading. WorldCat.
- Gu, W. H., Morgenstern, N. R., & Robertson, P. K. (1994). Postearthquake deformation analysis of wildlife site. Journal of Geotechnical Engineering, 120(2), 274–289. [https://doi.org/10.1061/\(asce\)0733-9410\(1994\)120:2\(274\)](https://doi.org/10.1061/(asce)0733-9410(1994)120:2(274))
- Gueguin, M., de Buhan, P., & Hassen, G. (2013). A homogenization approach for evaluating the longitudinal shear stiffness of reinforced soils: Column versus cross trench configuration. International Journal for Numerical and Analytical Methods in Geomechanics, 37(18), 3150–3172. <https://doi.org/10.1002/nag.2183>
- Guilloux, A. (2000). Facing a major geological hazard for a highway tunnel in the french alps. 87–92. <http://pascal-francis.inist.fr/vibad/index.php?action=getRecordDetail&idt=6178106>
- H. Durgunoğlu. (2005). Utilization of high modulus columns in foundation engineering under seismic loadings. In The US 8th National Conf. On Earthquake Engineering.

- Holzer, T. L., Youd, T. L., & Hanks, T. C. (1989). Dynamics of liquefaction during the 1987 superstition hills, California earthquake. *Science*, 244(4900), 56–59.
- Idriss, I. M., & Boulanger, R. W. (2006). Semi-empirical procedures for evaluating liquefaction potential during earthquakes. *Soil Dynamics and Earthquake Engineering*, 26(2), 115–130. <https://doi.org/10.1016/j.soildyn.2004.11.023>
- Idriss, I. M., & Boulanger, R. W. (2008). Soil liquefaction during earthquakes: Earthquake engineering research institute. Monograph MNO-12. Soil liquefaction during earthquakes. Earthquake Engineering Research Institute.
- Ishii, I., Towhata, I., Hiradate, R., Tsukuni, S., Uchida, A., Sawada, S., & Takahiro, Y. (2017). Design of grid-wall soil improvement to mitigate soil liquefaction damage in residential areas in Urayasu. *Journal of JSCE*, 5(1), 27–44. https://doi.org/10.2208/journalofjsce.5.1_27
- K Ishihara. (1993). Liquefaction and flow failure during earthquakes. *Geotechnique*, 43(3), 351–415. <https://doi.org/10.1680/geot.1993.43.3.351>
- Kitazume, M., & Terashi, M. (2013). Design of improved ground by the deep mixing method. 289–394. <https://doi.org/10.1201/b13873-9>
- Kramer, S. L. (1996). *Geotechnical earthquake engineering*. Prentice Hall.
- Liao, S. S. C., & Whitman, R. V. (1986). Overburden correction factors for spt in sand. *Journal of Geotechnical Engineering*, 112(3), 373–377. [https://doi.org/10.1061/\(asce\)0733-9410\(1986\)112:3\(373\)](https://doi.org/10.1061/(asce)0733-9410(1986)112:3(373))
- Martin, J. R., Olgun, C. G., Mitchell, J. K., & Durgunoglu, H. T. (2004). High-modulus columns for liquefaction mitigation. *Journal Of Geotechnical and Geoenvironmental Engineering*, 130(6), 561-571.
- Mital, U., Mohammadnejad, T., & Andrade, J. E. (2017). Flow liquefaction instability as a mechanism for lower end of liquefaction charts. *Journal of Geotechnical and Geoenvironmental Engineering*, 143(9), 04017065. [https://doi.org/10.1061/\(asce\)gt.1943-5606.0001752](https://doi.org/10.1061/(asce)gt.1943-5606.0001752)

- Namikawa, T., Koseki, J., & Suzuki, Y. (2007). Finite element analysis of lattice-shaped ground improvement by cement-mixing for liquefaction mitigation. *Soils and Foundations*, 47(3), 559-576. <https://doi.org/10.3208/sandf.47.559>
- Newman, R. L., Essler, R. D., & Covil, C. S. (1994). Jet grouting enables basement construction in difficult ground conditions. 385–402. <https://doi.org/10.1680/gitg.19287.0027>
- Nguyen, T. V., Rayamajhi, D., Boulanger, R. W., S. A. Ashford, S. A. Ashford, Ashford, S. A., Lu, J., Elgamal, A., & Shao, L. (2013). Design of dsm grids for liquefaction remediation. *Journal of Geotechnical and Geoenvironmental Engineering*, 139(11), 1923–1933. [https://doi.org/10.1061/\(asce\)gt.1943-5606.0000921](https://doi.org/10.1061/(asce)gt.1943-5606.0000921)
- P. Ozener, M. Dulger, & M. Berilgen. (2015). Numerical study of effectiveness of jet-grout columns in liquefaction mitigation. In 6th International Conference on Earthquake Geotechnical Engineering, Christchurch, New Zealand.
- Paolo Croce, Alessandro Flora, & Giuseppe Modoni. (2014). 1-Jet grouting technology design and control—2014.pdf. CRC Press.
- Puebla, H., Byrne, P. M., & Phillips, R. (1997). Analysis of Canlex liquefaction embankments: prototype and centrifuge models. *Canadian Geotechnical Journal*, 34(5), 641–657. <https://doi.org/10.1139/t97-034>
- Rayamajhi, D., Nguyen, T. V., Thang V. Nguyen, Ashford, S. A., Boulanger, R. W., Lu, J., Elgamal, A., & Shao, L. (2014). Numerical study of shear stress distribution for discrete columns in liquefiable soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 140(3), 04013034. [https://doi.org/10.1061/\(asce\)gt.1943-5606.0000970](https://doi.org/10.1061/(asce)gt.1943-5606.0000970)
- Rayamajhi, D., Tamura, S., Khosravi, M., Boulanger, R. W., Wilson, D. W., Ashford, S. A., & Olgun, C. G. (2015). Dynamic centrifuge tests to evaluate reinforcing mechanisms of soil-cement columns in liquefiable sand. *Journal of Geotechnical and Geoenvironmental Engineering*, 141(6), 04015015. [https://doi.org/10.1061/\(asce\)gt.1943-5606.0001298](https://doi.org/10.1061/(asce)gt.1943-5606.0001298)

- Schneider, J. A., & Mayne, P. W. (2000). Soil liquefaction response in mid-America evaluated by seismic piezocone tests.
- Seed, H. B., & Idriss, I. M. (1971). Simplified procedure for evaluating soil liquefaction potential. *Journal Of the Soil Mechanics and Foundations Division*, 97(9), 1249-1273.
- Seed, H. B., & Idriss, I. M. (1982). Ground motions and soil liquefaction during earthquakes.
- Seed, H. B., Idriss, I. M., & Arango, I. (1983). Evaluation of liquefaction potential using field performance data. *Journal of Geotechnical Engineering*, 109(3), 458–482. [https://doi.org/10.1061/\(asce\)0733-9410\(1983\)109:3\(458\)](https://doi.org/10.1061/(asce)0733-9410(1983)109:3(458))
- Seed, R. B., Cetin, K. O., Moss, R. E. S., Kammerer, A. M., Wu, J., Pestana, J. M., Riemer, M. F., Sancio, R. B., Bray, J. D., Kayen, R. E., & Faris, A. T. (2003). Recent advances in soil liquefaction engineering: a unified and consistent framework.
- Tokimatsu, K., & Seed, H. B. (1987). Evaluation of settlements in sands due to earthquake shaking. *Journal Of Geotechnical Engineering*, 113(8), 861–878. [https://doi.org/10.1061/\(asce\)0733-9410\(1987\)113:8\(861\)](https://doi.org/10.1061/(asce)0733-9410(1987)113:8(861))
- Vaid, Y. P., & Chern, J. C. (1983). Effect of static shear on resistance to liquefaction. *soils and foundations*, 23(1), 47–60. <https://doi.org/10.3208/sandf1972.23.47>
- Wang, W., Conservancy, B. W., & Department, H. P. S. R. I. E. E. (1979). Some findings in soil liquefaction. earthquake engineering department, Water Conservancy and Hydroelectric Power Scientific Research Institute. <https://books.google.com.tr/books?id=nz7XGwAACAAJ>
- Youd, T. L., & Idriss, I. M. (2001). Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 127(10), 817–833. [https://doi.org/10.1061/\(asce\)1090-0241\(2001\)127:10\(817\)](https://doi.org/10.1061/(asce)1090-0241(2001)127:10(817))