REPUBLIC OF TÜRKİYE

ALTINBAŞ UNIVERSITY

Institute of Graduate Studies

Civil Engineering

NUMERICAL ASSESSMENT OF TUNNEL SHAPE FOR LIQUEFACTION- INDUCED UPLIFT

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Master's Thesis

Supervisor

Asst. Prof. Dr. Mohsen SEYEDİ

Istanbul, 2023

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The thesis titled NUMERICAL ASSESSMENT OF TUNNEL SHAPE FOR LIQUEFACTION-INDUCED UPLIFT prepared by SAMI BAKOUR and submitted on 19/04/2023 has been **accepted unanimously** for the degree of Master of Science in Civil Engineering.

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ABSTRACT

NUMERICAL ASSESSMENT OF TUNNEL SHAPE FOR LIQUEFACTION- INDUCED UPLIFT

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Date: April / 2023

Pages: 61

This study Aims to give an illustration about the effects of the shape and size of the tunnels on the uplift displacement, the study starts with an overview about the liquefaction phenomena and the reason hidden behind it, however, the most convinced description about the liquefaction phenomena is That, The higher hydraulic gradient between the soil beneath the tunnel bottom and the surrounding soil became obvious and the surrounding soil squeezing into the tunnel bottom this is the major reason of tunnel uplifting. The study also has covered many sides of the phenomena such as the effects of the liquefaction and the damages caused during an earthquake hit, also it gives a general solution about ground improvement and some techniques used, such as soil compaction which is hugely used in almost everywhere around the word. Mitigation of the liquefaction also has played a very important role in this research and many specific solutions related to the mitigation of liquefaction have been provided. Mainly in chapter 3, the effect of the size of the tunnel has been discussed and concluded as following, (there is a positive relationship between the increment of the size of the tunnels and the uplift displacement), and this is because of the pressure applied on a bigger area.

Keywords: Liquefaction, Tunnel Displacement, Earthquakes, Tunnel’s Shape, Size.

TABLE OF CONTENTS

Pages

LIST OF TABLES

Pages

LIST OF FIGURES

Pages

ABBREVIATIONS

Sr	:	Saturation (Degree of Saturation)
NATM	:	New Austrian Tunnelling Machine
SCP	:	Soil Compaction Pile
TBM	:	Tunnel Boring Machines
Rd	:	Relative Density
RF	:	Failure Ratio

LIST OF SYMBOLS

Φcv	:	Constant Volume Friction Angle
Φp	:	Peak Friction Angle
C	:	Cohesion
$(N1\ )60$	:	Corrected SPT Value
γ unsat	:	Dry Unit Weight
γ sat	:	Saturated Unit Weight
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
Pref	:	Reference Pressure

1. INTRODUCTION

The characteristics of the soil determine the type of soil surveys used for building projects. The building site report serves as the basis for the foundation design. The first phase in the construction design process is often soil testing to see whether the soil is suitable for the proposed construction work for any building or other structure. The soil responsible for stress emanating from the structure needs to be well tested for better execution. In another word If the soil is not properly tested, the entire building or structure will be collapse or topple over like the Leaning Tower of Pisa. Various tests are carried out on the ground to determine the quality of the soil for the construction of buildings. Some tests can be performed in the laboratory while the other tests can be done in the field.

In 2010, New Zealand was struck by a severe 7.1 magnitude earthquake which shocked most of the Canterbury region, soon after as 6.3 magnitude earthquake, which became known as the Christchurch earthquake devastated the region, leaving 185 dead, and 1000s of buildings suffering the effects of extensive liquefaction damage. Most recently, 2016 saw a 7.8 magnitude earthquake, which not only affected the Canterbury region but also shut down much of the nation's capital. Main Mark have been innovators and seismic remediation and ground improvement since 1989, starting with the Newcastle earthquakes in Australia, and they continue to innovate through the most recent seismic events in New Zealand.

In geotechnical field especially soil mechanics its known that the term "liquefied" was first utilized by the scientist Hazen in reference to the 1918 failure of the Calavera’s Dam in California [1]. liquefaction is one of the most severe damages that causes harm in countries where earthquakes and rains occur. Liquefaction is the process when the soil turns into a liquid, it happens usually in the sandy soil due to its large particles size, and due to the irregular shape of the sand, air gaps would exist in the soil. Liquefaction tends to happen in in low-lying region where the water is underneath, like a water table underneath the soil, this will cause the water molecules to rise up to the earth surface.

The formation of sand boils or mud spouts at the ground surface by seepage of water through ground cracks, when the sand blows up through the face of the earth it's bringing up whatever is down there, this phenomena is a common shape of liquefaction. These sand volcanoes are the result of soil liquefaction as the shaking saturated earth compacts pressure builds up until a vent or fissure forced open, this brings the watery sand mixture blasting to the surface. Harm from liquefaction is rarely due to the boiling of the sand itself, but rather to the loss of strength and hardness in the liquefied soil and related ground deformations which it follows.

The New Zealand government, along with some of the world's leaders in earthquake engineering research undertook what is now known as the ground improvement science trials, on a scale, which the world has never seen before. blast induced liquefaction was used for the first time to evaluate ground improvement for residential construction.

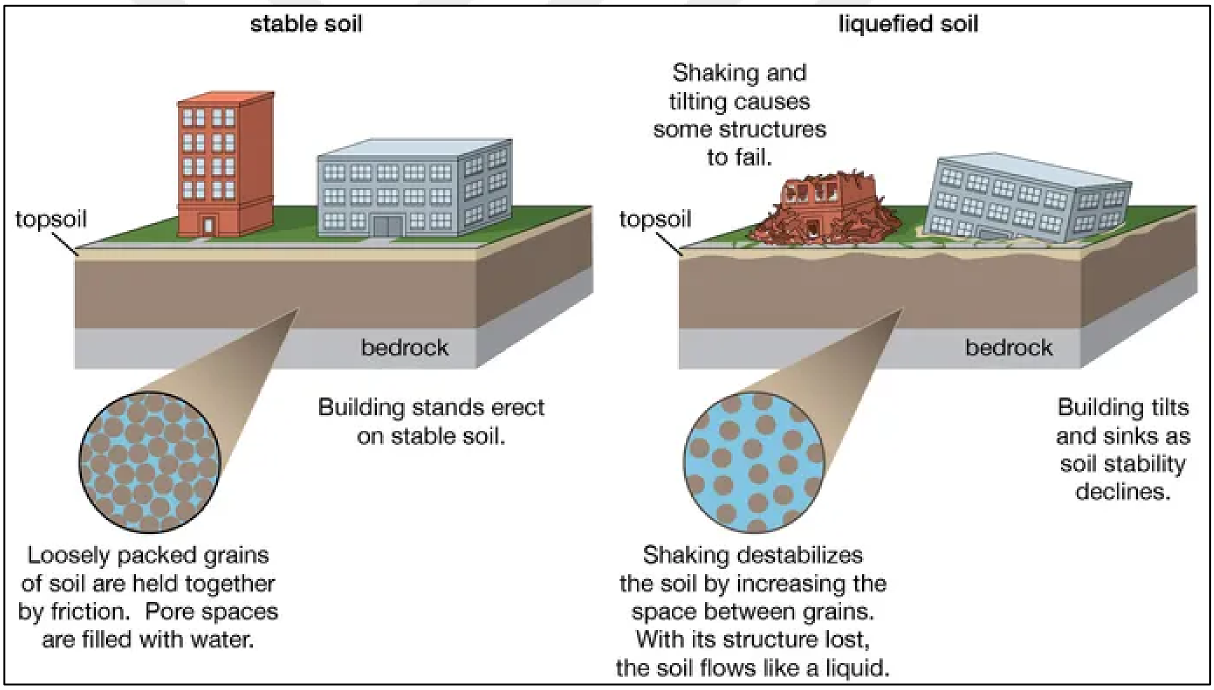

Figure 1.1: The Qualities of Stable Soil Compared with Those of Liquefied Soil.

1.1 EVALUATION OF FOIL AND LIQUEFACTION POTENTIAL

Geotechnical issues such as earthquakes require applicable analysis to the minimum stages of the assessment methodology, same as seismic stress assessment, to do online surveys, to identify any possible, to characterize soil and finally to do an engineering judgment. Uncertainties is definitely essential throughout the assessment method. To balance all phases, more details should be seeked and better choices of the assessment procedure have to be subdued. All together must be overcome as passable for the dimensions of the project, the importance of the facilities planned for the situation, the extent of risk associated with the hazard and potential consequences of failure in terms of loss of life, economic loss, and impacts on communities Due to the difficulties in getting and testing undisturbed samples from doubtless liquifiable sites, in-situ testing is that the approach most well-liked wide for evaluating the phase change potential for the soil deposit. the foremost common procedure employed in engineering apply for the assessment of liquefaction potential of sands and silts is the "simplified procedure.” The "simplified procedure was proposed and developed by Seed and Idris in (1971) following disastrous earthquake in Alaska and Niigata, in 1964 [2].

Shear strength of cohesive soil is highly complex as compared to cohesionless soils, this difference in characteristic between the cohesive and non-cohesive soils plays a big role in the selection and result of engineering techniques for assessing a soil’s response to seismic loading.

The development of analytical procedures for evaluating liquefaction excitement depend on experimental data to give the link between several in situ test indicators and liquefaction resistance. This development is expressed by the following steps.

a. Developing an analytical framework that is solidly based on the physics and mechanics of soil.

b. Gathering case histories that show a variety of observed liquefaction traits, including instances when liquefaction did not take place.

c. Applying the existing framework for analysis to interpret the case histories in order to construct semi-empirical connections that discriminate between the incidence and non-occurrence of liquefaction.

For evaluating liquefaction potential soil, mainly two parameters such as Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR) should be analysed [3].

$$F_s = \frac{CRR}{CSR} \tag{1.1}$$

which CRR = Cyclic resistance ratio CSR = Cyclic stress ratio

1.2 FACTORS INFLUENCING LIQUEFACTION SUSCEPTIBILITY

What are the various factors which influence the liquefaction susceptibility, that is at any region when there is an earthquake or any kind of rapid dynamic load is acting whether that particular soil can be subjected to liquefaction or not. It depends on various factors. The first one is of course the earthquake intensity and duration if the magnitude of earthquake is higher or intensity of earthquake is higher, obviously, the chances of the soil getting liquefied will be more and also if the duration of earthquake is more then obviously, chances are also more for liquefaction. Then second one is soil type what type of soil are there, if it is a loose soil, then it is more susceptible for liquefaction than a dense material or dense soil. Then, soil relative density, in soil type we also can say whether it is a sandy cohesionless soil or it is a c phi soil, that is fines are present it is helpful since the cohesion components provide some strength. So, that's why soil type plays an important factor for liquefaction analysis. particle size distribution is also a factor which effects susceptibility, if the particle size distribution is uniformly graded or poorly that case the chance of liquefaction is more than a well graded soil, so, well graded soil has less susceptibility to liquefaction. Next factor is presence or absence of plastic fine, which Is already mentioned within the soil type, if plastic fines are present, then it will help to some extent reduce the liquefaction effects. So, chances of liquefaction susceptibility will be less for high plastic fines up to a certain level it is not infinite, so, presence or absence of plastic fines by changing the amount of plastic fines the susceptible to liquefaction can also change. The next one is groundwater table location or degree of saturation. When the groundwater table at any particular area is pretty high means very close to the ground in that case more susceptible to liquefaction is expected than another area, where groundwater table is much below the ground. Hydraulic conductivity of the soil is also another important parameter or important factor which influence liquefaction, suppose a soil which is having a very good hydraulic conductivity or which is another terminology is used permeability. So, if the soil is having good permeability or hydraulic conductivity value, like sandy material, so it can release the generated pore water pressure during the shaking process or earthquake shaking process easily. So, having a high value of permeability means less susceptible to liquefaction, whereas low permeability means it is more susceptible because it is not getting sufficient scope to release its water. However, structure loads, overburden pressure and historical liquefaction are also factors influence the susceptibility.

1.3 TESTS OF SOIL PROPERTIES INDICATION

Each year, the Louisiana Department of Transportation and development spends millions of dollars to design, construct and maintain its highway system. A significant portion of these funds is needed for the foundation systems, all structures including bridges, pavements walls and banquets and buildings are ultimately supported by the existing terrain for safety and economy. The design of these structures is based on the properties of the building materials and upon the properties of the foundation materials in the natural ground. Therefore, the designer of these structures must know the surface and subsurface soil conditions in order to create safe and efficient designs. geological conditions are such in Louisiana, that a thorough investigation of subsurface materials is vital to the design engineer, because dense bearing layers are often encountered well below the ground surface. bridge structures in Louisiana are supported on deep foundations. Piles carry the superstructure loads deep into the ground, either in end bearing or by side friction. The safety and stability of pile supported structures depends on the load carrying capacity of these piles. Current D O T D practice of pile design is based on a static analysis using soil properties obtained from field and laboratory testing. Taking deep borings is an expensive, time-consuming process, obtaining samples for the design of deep foundations involves drilling into the earth to retrieve the soil found at various depths for the purpose of determining its strength properties. To recover undisturbed samples and perform standard penetration soundings wet rotary drilling equipment is needed. Standard equipment includes a drilling rig, a water trucks a utility truck, and a carry all. Wet drilling is always messy and sometimes hazardous work. drilling mud is pumped into the hole to provide stability for raising and lowering sampling tools to the required depths. Shelby tubes are used to retrieve undisturbed soil samples. These samples are extracted and carefully packaged for shipment to the laboratory. When dense materials are encountered, standard penetration tests or SPT are conducted. Penetration soundings involve driving a steel rod into the soil to determine the resistance and blows per foot. Pouring logs are manually kept in the field to record depth of sample visual classification and SPT or pocket penetrometer strength depending on the depth of the exploration. Crews are often capable of completing two borings per day laboratory testing of the soil samples includes soil classification, soil density, Atterberg limits, unconfined compression tests and tri axial tests. Department of transportation and development (DoTD) district crews perform the majority of subgrade soil surveys used in the design of pavement structures. Each district operates a pouring rig capable of recovering disturbed soil samples. samples are collected as the auger was advanced into the ground. A drilling log is maintained in the field to record location and depth of sample. Samples are placed in moisture proof containers for testing in the laboratory. laboratory testing includes soil classification Atterberg limits, sieve analysis and moisture content. DEP engineers use the subgrade soil survey produced from these borings for the design of pavements besides subgrade properties, the design and evaluation of pavement beside subgrade properties the design and evaluation of pavement structures needs a substantial amount of supporting data such as traffic loading characteristics, environmental conditions, and materials properties of the base and subbase. Currently, empirical correlations developed between laboratory material properties and field are used to obtain subgrade performance characteristics. These correlations do not satisfy the design and analysis requirements because they do not address all possible failure mechanisms in the field. Most of the subgrade properties used in the correlations do not represent the strength conditions of the subgrade environmental variability or the repeated traffic loading on the pavement. Because of the considerable time and expense associated with conventional exploration operations. There is a need for more rapid and accurate site characterization techniques.

1.3.1 The Standard Penetration Test (SPT)

During sampling the soils, the engineers usually encounter some difficulties with the cohesionless soil, therefore the engineers have to test the soil in different way. The Reymond concrete and pile company has developed a test which can be done in the field, the test is either valid for cohesion and cohesionless soil, the SPT test can indicates many important characteristics of the cohesionless soil such as relative density and the shear resistance angle. SPT also can be done on the cohesive soil to determine the unconfined compressive strength. This test must be done under some conditions, each change in the stratum of the soil should be considered and the interval shouldn’t be more than 1500mm (1.5m).

1.3.2 Cone Penetration Test

Cone penetration testing is a quick and precise method of subsurface research. A CPT rig drives a steel cone into the earth using hydraulics. A steady forward motion of the cone is created. The instrumented cone measures the dynamic pore water pressure, chip resistance, and sleeve friction continuously. An electronic probe is literally pushed into the earth at a pace of two centimetres per second during a cone penetration test, or CPT. At its tip, the tool has a load cell that measures the resistance the soil presents to incursion. Additionally, it has a friction sleeve to gauge the degree of friction between the soil immediately around the probe and that soil. You may utilize the CPT tip resistance and sleeve friction statistics.

1.4 DRAINED AND UNDRAINED SOIL SHEAR STRENGTH

Whether the soil performs best under undrained or drained applied load circumstances must be considered when choosing a soil strength for the design. An improper resistor might cause failure since each offers distinct resistor values and selecting options. Depending on the weight of the load and the permeability of the soil, any terrain may encounter any circumstances. Since the permeability of coarse-grained soils is great and water may easily flow through the continuous and substantial void spaces, these soils are typically thought of as drained materials. However, the void spaces in fine-grained soils like silts and clays are substantially smaller. There is frequently no direct path for water to flow freely and the flow is frequently not continuous.

1.5 LABORATORY TESTING OF FIELD SAMPLES

Significant ambiguity could be part of the procedure of sampling, assessment, analysis, and style for cost-efficient and secure soil structures over sampling disturbance in "undisturbed" soil samples. If the way within which turbulence affects the properties of the soil measured within the laboratory is known, this entered uncertainty cab be partly removed or avoided a minimum of. the target of the inquiry explicit here is to advance the understanding of however sampling disturbance affects laboratory measured soil parameters by reviewing relevant recent studies that are conducted to assess the result of sampling disturbance on laboratory measured soil properties.

1.6 EFFECTS OF SAMPLING DISTURBANCE

Sampling is linked to the extraction of the soil or rock from the bored hole. The sampling of soil for the objective of in situ characterization is a situation in which the presence of an observer alters the nature of the experimentation. Soil cannot be removed from its environment on site, regardless how carefully it is implemented, without releasing stresses on site, disturbing the material, and changing its mechanical properties. soil properties irreversibly changed to a greater or lesser extent by sampling.

The amount of disturbance occurring is determined by:

a. Soil type, whether cohesive or granular.

b. The characteristics materials such as sensitivity, water content (saturation), density, OCR or Over Consolidation Ratio, etc.

c. Types of Samples it can be either tube sample or block sample.

d. Specimen size.

e. Geometric design of the sampler and sampling tube used.

f. The depth of the sampling.

Each type of soils has its own specific problems regarding to sampling disturbance whether the soil is granular soil or cohesive soil. Clay soil, which is a cohesive soil, is subject to a disturbance in its structure as a result of sampling, and the rate of its exposure to disturbance increases as the sensitivity of the material or sample increases. Some researchers hypothesize that the strength in cohesive clay soils depends in its function on time in the procedure related to the gradual change in stress and the change in the distribution of water content. While Non-cohesive granular soils can be subjected to a change in density in the course of sampling, which affects the results of laboratory strengths. All scholars who have studied the problem concur that (all other things being equal), the larger a sample, the less disruption it will experience during sampling. Additionally, block samples that are cut by hand are less disturbed than tube samples, but block samples may only be collected by digging to the necessary level of interest. Particularly if the required level is below the water table, this can be costly and challenging. Finally, studies show that sample depth affects the disruption that results from sampling. The likelihood of a sample disturbance increases with sampling depth. Simply because all sampling-related processes become more complex and time-consuming as depth rises, logic would support this argument.

Sampling can be divided into two groups:

a. Undisturbed soil sample is taken out for laboratory tests properties, without disturbing the soil texture, structure, natural water content, density.

b. Disturbed sample A sample is a portion of soil whose structure has been sufficiently altered that only the characteristics of the soil grains will be tested as representing the soil's structural qualities under in-situ circumstances.1.7 Triggering of Liquefaction:

The way that cohesionless sediments exist in the soil can have major effect on the liquefaction degree. One of the most important parameters that should be considered during the study of liquefaction is the age of sediments which determine if they are the most susceptible to liquefaction where if they are young (recently buried) or old (buried long time ago). The age of sediments leads to more resistant as much as they become older. Another critical factor that should be considered, is the earthquake effect on the sediments which makes them soft and sensitive.

Cohesionless soils is the behavior of clays and plastic silts. Depths can be a concern due to its relation with the age of the sediment which can be young when its less than 15m but greater depths are more serious because they can lead to embankments constructed of loser material. It is very important to review the historical records of the site because soils that liquefy in few earthquakes usually liquefy subsequent earthquakes on the other hand the areas of very recent natural buried and unstable sites have high possibility to liquefy.

The most widely used approach to analysis framework for developing liquefaction triggering correlations is the stress-based approach that consider both the earthquake-induced cyclic stresses and the cyclic resistance of the soil then observe the expected zone of liquefaction.

1.7 CONSEQUENCES OF LIQUEFACTION

First consequences can be named as the alteration of the ground motion. And so, its known that when the soil liquefies, essentially what happens is that the shear modulus goes down drastically. And if the shear modulus goes down drastically, then the natural period of the soil goes up drastically as well. So the soil loses stiffness it loses strength, and it increases in its natural period because of that, that soil layer is going to filter out a whole bunch of the low period, high frequency ground motions they won't get through that liquefied layer. The layer acts like a base isolation layer. instead. What's going to come through that liquified layer are the very high period low frequency motions, however, if the built structure has a very high period, the occurrence of liquefaction can amplify high period ground motions. while if the structure has very low period structure the onset of liquefaction can actually filter out ground motions that might be detrimental to the structure. So, in terms of just pure ground shaking the liquefied soil may help by filtering out motions that could cause the structure to resonate.

The second effect is post liquefaction settlement. And so, anytime when there is liquefied soil, the soil particles get closer together water gets ejected and the volume of the soil changes so the soil has the appearance of compacting or settling and it can settle through tremendous amounts like figure (1.2), in this picture over a metre of settlement has occurred between the structure and the ground outside it. And what happened at least with this structure is the structure was founded on piles and the ground all around it settled and so it gives the appearance that the structures pop out of the ground. So, anytime when there is lifelines like the pipe coming in fig (1.2), it can imagined that this type of effect can be really devastating and damaging.

Figure 1.2: Soil Settling by Loma Prieta Earthquake San Francisco1989.

A third effect is the loss of bearing capacity. So, if the soil is liquefied that are shallow and very close to the ground surface, and the soils relying on the shear strength to keep the structures upright, a significant problem may occur. liquefies soil drastically loses its shear strength and as a result a stuff like fig (1.3) happening where buildings are tipping over. Or it can work the other way too. Like if there are Barry tanks, tanks that can pop out of the ground and start floating like they're on water. All because of a loss of shear strength and so this is an important effect to consider, especially if when dealing with shallow footings, embankments, anything that are relying on shallow soils to provide shear strength, if the foundations are deep such piles or drill shafts, this loss of bearing capacity isn't that problem.

Figure 1.3: Tilting Of Apartment Buildings Caused By The 1964 Niigata Earthquake (Photo: National Information Service for Earthquake Engineering, EERC, University of California, Berkeley).

Flow failures is the scariest consequence. if the Liquify soil can't support its own weight and it's on a slope, usually the slope is going to be equal to or greater than a six-degree gradient. If that's the case, then in the soil liquefies, a very large soil deformations can be noticed, and as known, this type of deformation is widely considered to be the most dangerous of all the effects of liquefaction because anything that's located down slope of the failing ground, can get buried or if in the case of like this, slower San Fernando dam, as in fig(1.4) there's a city right down below the dam, where the water could have easily flooded it.

Figure 1.4: Lower San Fernando Dam After The San Fernando Earthquake (1971). (Courtesy Of The National Information Service For Earthquake Engineering, EERC, University Of California, Berkeley).

1.8 GROUND IMPROVEMENT

Clay, silt, and sand make up soil. Therefore, the combination is heterogeneous. The term "ground enhancement" describes a method that enhances the treated soil mass's engineering qualities. Shear strength, stiffness, and permeability are often adjusted attributes. The qualities of the ground at the time of creation, the beginning of the project, and any potential changes to the ground over the design life of the building should all be taken into account when designing a structure. Grounds need to be improved When we come across poor soil conditions at the site, such as loose and soft clay, high organic deposits, or dumped heterogeneous materials, and there aren't enough high-quality, granular building materials available for building dams, embankments, and roadways. Today, the majority of the land is used for industrial or urban development. Since quality building sites are hard to come by, soil improvement techniques are typically the most practical and cost-effective solution.

The stiffness or strength of the structure can be increased using a variety of approaches to withstand the stress and deformation that will be delivered to the liquefied soil. Another thing that may be done and is frequently done is to simply enhance the soil's quality while attempting to reduce or eliminate its capacity to begin or alter its susceptibility to liquefaction. The detrimental consequences of liquefaction can be completely prevented if either one or both of them are removed. Therefore, we usually refer to this soil enhancement as ground improvement. So, in addition to ground renovation, what more may be done to the soil? Change the soil's physical or chemical makeup Additionally, if the soil is unstable because of landslides or slope problems, if there are organic soils that may settle too much, and the list goes on and on. If there are unfavourable soil conditions and there is no way to construct the building to resist the consequences that are anticipated. We frequently rely on improving the ground. So, for instance, in this picture (1.5) below, a machine is inserting reinforcing materials into the ground along a shear plane that engineers are concerned about because it might have a slope stability issue. So, ground approvement today is typically designed and installed by geotechnical specialty contractors. the question, why don't engineers just do ground improvement for every site! It's because of it's high cost. ground improvement in general, based on many experts experience in consulting, does generally increases the total cost of the project by three to 10%. So, if you weren't anticipating that, you can imagine that this could be a very significant increase in the budget.

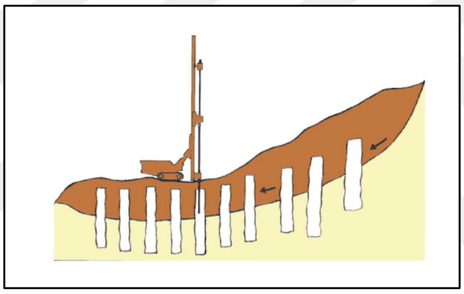

Figure 1.5: Installing Reinforcement Machine

1.9 LIQUEFACTION MITIGATION

The liquefaction phenomenon can have catastrophic effects on both above and below ground structures. Liquefaction can result in bearing capacity failures, lateral spreading and intolerable settlements for bridges, tunnels, roadways and embankments.it can also cause intolerable differential and total settlements and in some cases a total loss of bearing capacity for buildings and above ground structures. The weight and strength of the overburden soils are used to support constructions built in the earth below the water table. These airport tunnels and utilities float to the ground surface in seismic zones because to the liquefaction of near-surface sands. Due to this huge hurt of liquefaction, the researchers have to figure out a solution to this phenomena by mitigate the occurrence of it, throughout a series of laboratory tests and numerical models the researchers tried to eliminate the damage of liquefaction by eliminate the water content in zones of liquefiable soil deposits. The techniques provide an increasement in the compressing of the pore fluid with generating an amount of gas in the soil of completely saturated sand pore by flipping the soil from a fully saturated state into a partially saturated state.

Not all of the soils are saturated, some of soils has an undissolved gas bubbles which have a great positively effects on the soils due to their important role in increasing the pore fluid compressibility and it can also increase the resistance of the liquefaction, and that's why the geotechnical engineers prefer to insert some chemical materials to the soil, because it generates some gas bubbles, while other engineers have tried to change the state of soil from a fully saturated into a partially saturated state by other ways such as injecting air into saturated soils to resist the liquefaction.

1.9.1 Dewatering

Dewatering method is a type of ground modification process which is also useful for liquefaction mitigation, through the process of draining an excavated regions that are flooded with rainwater or ground water before starting a construction. It is done while digging for foundations and basements or when there is a buildup of ground water at the location where water or sewer lines need to be installed. The water is removed using dewatering pumps with large air-handling capacities. Deep wellpoint method, eductor wells, open sump pumping, and wellpoint method are the major four dewatering techniques.

1.9.2 Earthquake Drains Mitigate Liquefaction

During an earthquake shaking, loose silts and sands tend to densify into a tighter configuration. When these soils are located below the water table and are partially to fully saturated, the reorientation of sand grains results in an increase in pore water pressure in the soil skeleton. When this pressure exceeds 60% of the soil strength, intolerable seismic settlements may occur when pore water pressure reaches 100% of the soils initial effective stress, silts and sands liquefy. earthquake drains or geosynthetic drains installed vertically through a soil profile to provide a shortened drainage path for the rapid dissipation of earthquake generated excess pore water pressures during and immediately after a seismic event. The drains consist of perforated corrugated HDPE pipes wrapped in a specially designed non-woven filter fabric. The pipes are sized to provide positive drainage with little to no back pressure on the in-situ soils. The filter fabric has been specially designed to effectively block fine soil particles from entering the drains while allowing water to freely pass through the filter. Each earthquake drain design is site specific and based on the index properties of the subsurface soils and the magnitude and peak ground acceleration of the design earthquake. Responsible spacing of earthquake drains typically ranges from four to seven feet centre to centre in an equilateral triangle pattern. For new projects earthquake drains are installed with a self-contained vibratory rig. The earthquake drains are bottom fed into an installation pipe or mandrel and then proven to treatment depth. The sacrificial endplate is used to anchor the drain in the appropriate soil layer, vibratory installation techniques densify in situ soils, further increasing their resistance to liquefaction. during an earthquake as pore pressure builds, water flows radially to the nearest drain and then upward towards the ground surface. The tight spacing of earthquake drains allows each pipe to effectively serve as a pressure relief valve during ground shaking. Depending on the depth of the water table. The drain itself may contain enough storage to hold the expelled water if not, an artificial or naturally occurring reservoir must be designed for the project. At the conclusion of the design earthquake, the expelled water seeps back to the static groundwater table level and the earthquake drains are ready for the next seismic event. For retrofitting applications where vibrations may be detrimental to the existing or nearby structures, earthquake drains are installed with drilling equipment on these projects, drains are top fed through hollow stem augers to the design depth.

1.9.3 Jet Grouting

A deep foundation system, a floating foundation, or a soil stabilization alternative such ground improvement techniques like grouting must all be considered if the anticipated settlement for a planned project is too great. Grouting often refers to the injection of suspensions, solutions, and emulsions into soil pores in an effort to weaken or improve soil permeability. However, it must consistently travel the necessary distance through the earth and arrive in a usable condition in order for it to be successful. Although permeation grouting is the most frequent kind of route used in construction, grout may also be used to either replace or displace soils, as in compaction and hydro fracture grouting or jet grouting. A rather hard, impenetrable column is the ultimate product when the grout has dried and hardened. Some of the coarse particles in granular soil are left in and added to the grout. The choice of an appropriate grout material, together with the right drilling tools, techniques, and grout hole patterns, are essential to the strategy of a successful grouting program. The vertical thickness and order of the grouting phases for each hole, as well as the arrangement of the holes and the order in which they are grounded, are all included in the grout pattern. The arrangement of the holes may be based on a certain geometric design, such as staggered, square, or rectangular, or it may be altered depending on the state of the ground.

The New Zealand government, along with some of the world's leaders in earthquake engineering research undertook what is now known as the ground improvement science trials, on a scale, which the world has never seen before. blast induced liquefaction was used for the first time to evaluate ground improvement for residential construction. The Christchurch residential red zones spend 630 hectares of land once home to 1000s. The land has now been deemed unlikely to be rebuilt on due to the extremely high risk of liquefaction in future events. Determined to develop a technology for liquefaction mitigation main Mark reentered the red zone to complete further research and testing of a technology for liquefaction mitigation that can be applied beneath existing properties which are at risk of liquefaction induced damage from future seismic events.

Resume injection is a method which has long been used nationally and internationally to lift and re level structures. Expanding resin mixes are injected at shallow depth beneath the structure which results in ground heave and building lift. My Mark hypothesized that this method would also result in densification of the ground testing was carried out using production panels to mimic the structure. The ground was loaded with a uniform surcharge made up of concrete blocks and a strip footing was also superimposed to simulate foundation loads. Injection tubes were arranged in a triangular pattern at 1.2-meter centers and inserted to a treatment depth of eight meters below ground level. Custom Built resin extraction units were connected by hose to mobile plant containers which has the pumps power generators and materials. Extensive pre and post ground investigations consisted of geophysical testing, which included Cone Penetration Testing cross hole shear wave velocity testing, and seismic dial atomic testing. Each of these tests were carried out within the treatment zone and a further two meters below the ground and pregnant zone to the mainland could determine the effect of the injections and the associated results had increasing depths. The final stage of testing was hydro excavation, which allowed main mark to see the final structure of the injected resin which were then 3d scanned in order to find approximate volume replacement ratio of the resin. A total of five test panels were conducted and all of the results showed significant improvement in ground density. And stiffness greatly reduced surface damage potential and an improvement in static bearing capacity. Results from the A series of testing proved this technology could be delivered beneath an existing structure and had opened up a lot more opportunities for main mark to innovate further, as well as providing engineers with a new tool for liquefaction mitigation. Now that their research has been internationally peer reviewed mine Mark are now able to present a series of papers nationally and internationally sharing their findings with the engineering community. Starting with the 2017 New Zealand society of earthquake engineering conference in New Zealand. main mark success in this area has already seen international interest from Japan and the United States for potential applications and these liquefaction susceptible zones main Mark are also continuing to explore delivery innovations including bespoke equipment and the unsought design package which aims to predict the increase in ground strength which can be achieved in various soil types. Taking their science to the real world, my mark undertook and successfully completed its first large scale liquefaction mitigation project beneath a large retail complex in Christchurch, resume was injected to eight meters below ground level beneath load bearing walls and four meters below ground level beneath floor slabs as shown in fig (1.6) being a large structure over 3000 injection points in total were required for the project like fig (1.7) and may marks work was carried out outside of business hours to reduce interruption and prevent loss of income to the tenants and acid owner. Equipment machinery and data logging systems were bespoke creations for this project, some of which required up to eight months of research and development before starting the project. The resume injection and tube extraction systems were integrated with an automatic data logging system. The system was synced such that the data could be immediately analyzed after each nightshift. This required innovation and both software and mechanical engineering and the resulting custom iPad application saved precious injection time to both the machine operators and project managers. Further innovation was also required for angled injections to be conducted beneath hygiene areas fig (1.8). This meant that no reinstating placement work to floor slabs was required which would have been an extensive cost and interruption to the tenants.

Figure 1.6: Injected Resume Beneath The Structure, 09/2017 Newezland.

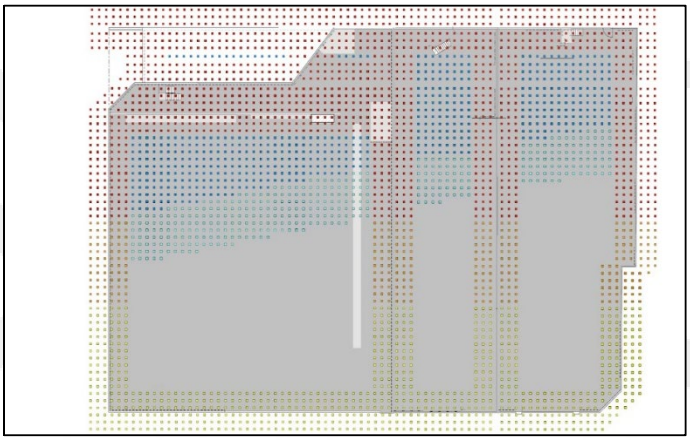

Figure 1.7: 3000 Injection Points 09/2017 New Zealand.

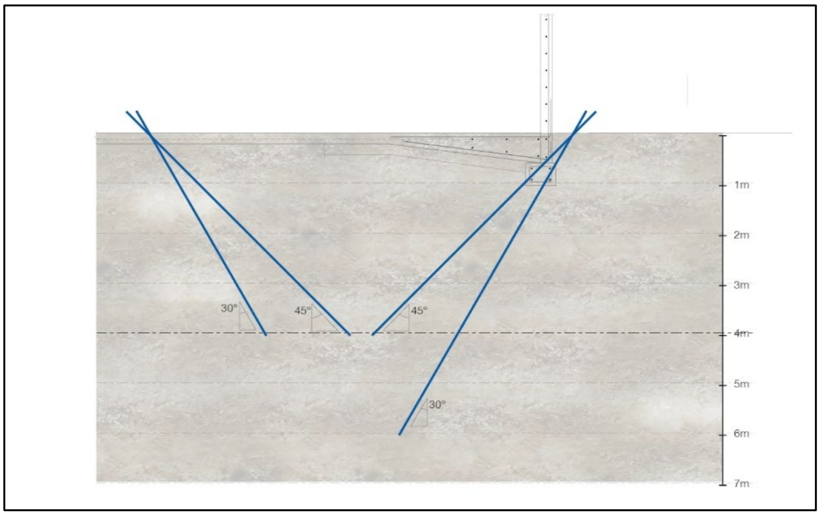

Figure 1.8: Angled Injections 09/2017 New Zealand.

1.10 TUNNEL EXCAVATION AND THE BORING MACHINE:

1.10.1 Natm Tunnelling Method

Engineering challenges the insurmountable, with the advance of the NATM technique, tunnels are no longer considered difficult, unsafe and expensive construction works. In addition to major roadways this technique is used in homes and companies without size or obstacle limits. Diversification in the use of tunnels in engineering projects is due primarily to the development of the new Austrian tunnelling method. Fig (1.9)

Figure 1.9: Tunnel Boring Machine, It Was Developed Between 1957 And 1965 In Austria.

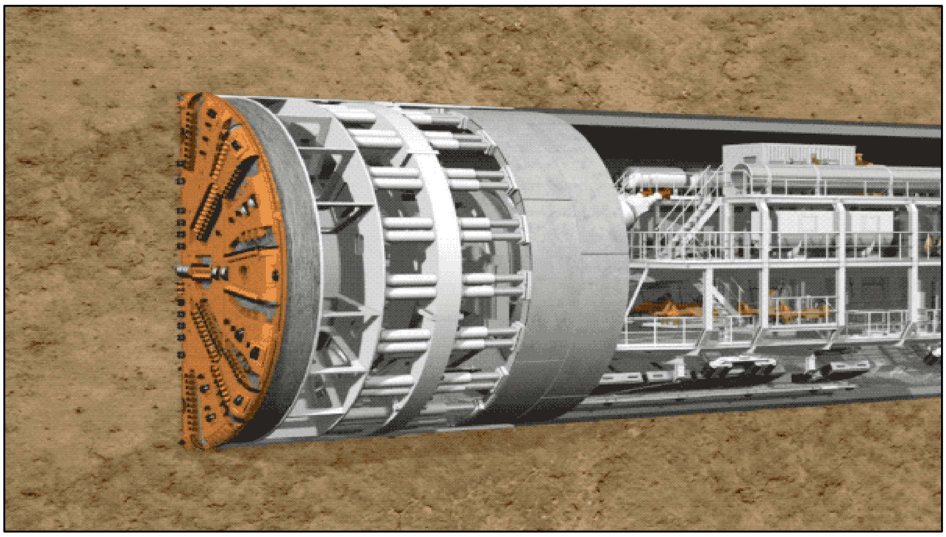

Figure 1.10: NATM, Developed Between 1957 And 1965 In Austria.

For every tunnel to be constructed, operating sites are identified and set up by installing necessary infrastructure machinery and equipment. Before actual tunnelling process starts, false tunnel section is excavated and RCC lining is done, and then actual tunnel excavation begins. actual tunnelling starts with portal construction with size greater than actual tunnel size. Pipe for polling with high pressure cement grouting is done for roof stabilisation. And then, actual face drilling starts with twin boom jumble drilling machines followed by charging and blasting with explosive drilling blasting is carried out from centre in small units for a distance of 10 to 15 metres creating pilot holes eventually the full shape and size of the tunnel is achieved. Repetitive tunnelling method begins, geologists and surveyors analysed class of rock after every round of blasting operation and find Q value and RMR value, depending on the class of rock rest of tunnelling steps to be undertaken is formulated. Geologists and surveyors collect data with help of profiler a machine and plot them on paper. charging patterns change with change in rock class. Based on this data drilling and blasting pattern is prepared, drilling and blasting is carried out as per design pattern. These along with charging pattern is the heart of successful tumbling, it is followed by Defuming for about 30 minutes ‹water is sprayed all over blasted surface to settled dust and to reveal cracks and fractures. Next step is mucking once muck is removed using excavator geologists and surveyors again evaluate region provide support system design and collect data for the next blast. Primary shotcreting is done with robot arm shotcrete machine. Rockball drilling and grouting is done as per rockball design given by geologists. Depending on rock class blades are fixed and bolted and predefined amount of torque is applied. rock bolts are then tested for load bearing capacity after 15 days as per requirement and based on geological evaluation done by geologists earlier preliminary support structures like wire mesh or lattice girder may be erected after which final layer of shotcreting is done then the process is then repeated for next round of blasting as per the requirements. Monitoring of critical aspects with instruments such as extensor metre, pressure cell and load cell are carried out after every 10 metres of tumbling, if there are any variations of problems, adequate steps are taken to correct them like changing supports and so, on. In case of tunnel construction from both sides, when 10 metres is remaining in between a central hole is drilled from one side to the other, to check and maintain alignment then one side is closed and digging is completed from the other side. For rock class three and above, the tunnelling is done first with headings and then in the same process, benching is completed. once tunnel has been excavated from one end to the other, entire tunnel is clean. Geologists evaluate and survey once more. gantries are introduced into tunnel, if possible, from upward direction to downward direction, geotextile membrane and waterproof membrane are fixed to a certain no seepage water comes after lining. kicker gantry boasts concrete on two sides, creating side slabs perforated PVC pipes are embedded into them. Then kicker gantry moves forward. Main gantry is then fixed, which bores concrete by pumping and complete concrete in process. Once entire tunnel is completed, Kinder gantry and main gantry is dismantled, after that floor is washed and cleaned. RCC Concrete is poured in on the ground. Kicker arch and invert concreting are completed.

1.10.2 Types of Tunnels Depends on Shape

The shape of the cross - section of a tunnel depends upon the pressure of the ground which the tunnel lining can resist and the purpose for which the tunnel is constructed.

a. Circular.

b. egg shaped.

c. horseshoe

d. Segmental.

e. Elliptical.

f. polycentric.

1.10.2.1 Circular shape

Circular shape tunnel is the Best for resisting external as well as internal pressure, hence they are suitable for carrying water under pressure Fig (1.11).

a. They can be easily constructed in soft soil and soft rocks.

b. If these shapes are to be used for carrying traffic, then the invert portion of the tunnel needs to be filled to level surface excavation.

Figure 1.11: Circular Tunnel.

1.10.2.2 Elliptical section

They are used in grounds softer than rock in order to have better resistance to external pressure, the major axis of these tunnels is kept vertical. These tunnels serve as water and sewage conduits. They cannot be used as traffic tunnels because of their narrow base. Fig(1.12)

Figure 1.12: Elliptical Shape Tunnel.

1.10.2.3 Rectangular tunnel

They are the best suited for hard strata, they prove costly as they completely made of R.C.C. & difficult to construct, Depth is less and hence suitable for pedestrian traffic, but can also be used for vehicular traffic with restriction on vehicle height.

2. LIQUEFACTION AROUND TUNNELS (LITERATURE REVIEW)

When an earthquake imposes the soil to liquify, the effective stress and the shear strength of the soil reduce significantly and huge deformation occurs, this problem put the liquefaction issue on the top of the geotechnical engineer concerns. The tunnels constructed in a liquefiable layer may be exposed to a settlement, deformation, however the Underground structures are common susceptible objects to floating and liquefaction during an earthshaking event. During an earthquake a lot of damages occurs therefore it’s important to investigate and study these damages, by investigate its uplift behavior through few modelling tests. A series of test were conducted on a rectangular tunnel embedded in liquefiable soil, these tests provide the 3 factors influence the magnitude of tunnels uplift displacement, those factors are tunnels, buried depth, the number of loading cycles, and the input amplitude of base acceleration. The reason that makes the surrounding soil of the tunnel squeeze into the tunnel and push it up, is the hydraulic gradient difference between the tunnel’s surrounding soil and the soil beneath the tunnel and this is the main reason of tunnel uplifting due to an earthquake.

as seen in Fig. 2.1. The theory of force mechanism presented by China (2012) can help us comprehend the consequences of soil liquefaction on tunnels and other subterranean constructions. The soil's shear strength is mobilized on planes above the tunnel in the static condition, resulting in a vertical component of force that includes the tunnel's own weight (FT), force from the planes (FSP), and the weight of the soil above the tunnel (FWS), all of which are completely in equilibrium with the buoyant force that is exerted upward (FB). If liquefaction occurs, the accompanying loss of shear strength will reduce FSP, which will result in an extra upward force from EPP acting on the tunnel's base (FEPP).

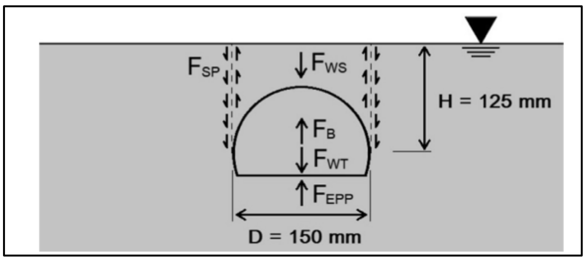

Figure 2.1: Force Mechanism Theory [4].

2.1 LIQUEFACTION CAUSED BY EARTHQUAKES

Numerous studies have been published on the destructions caused by uplift pressure in the tunnels below (Aydingun, 2003; Chou, 2001; Design Specification of Taiwan's Building Code, 1997; Khoshnoudian, 2002); (Koseki J. M., 1997). 1984 Chubu Nihonkai, 1990 Luzon, 1964 Nigata oki-Kushiro and oki-Hokkaido in 1993 Nansei earthquakes are a few instances of the research described (Japanese Society of Soil Mechanics and Foundation Engineering, 1986; Japan Society of Civil Engineers, 1993; Khoshnoudian, 2002; Koseki J. M., 1997); undertaken with scientists in relation to previous significant causes of earthquakes. Liu and Song (Liu, 2005) discovered the effects of soil liquefaction on the phenomena of subsurface constructions in this thought experiment. (Fig. 2.2) illustrates how liquefaction caused the structure to be lifted higher, and the experiments they conducted demonstrate how the uplift pressure behaved. [5] conducted the surface transformation in the location of tunnels showed (Fig. 2.3.) According to these numbers, the ground surface heaviness and the structure's uplift are 36 and 34 cm, respectively. These factors alter the constraining conditions of the subsurface structures and are likely to have detrimental consequences on surface structures (Liu, 2005). The previous findings are consistent with Khoshnoudian's research on the impact of liquefaction on tunnels dug on liquefied soils (Fig. 2.4). According to these research, the internal loads in subterranean buildings are different when the surrounding soils are liquefied, even when the characteristics of the loads do not significantly influence them (Khoshnoudian, 2002). As a result, it is vital to take into account the uplift of embedded structures as a major factor in the deformation of subterranean structures during the liquefaction of the soil.

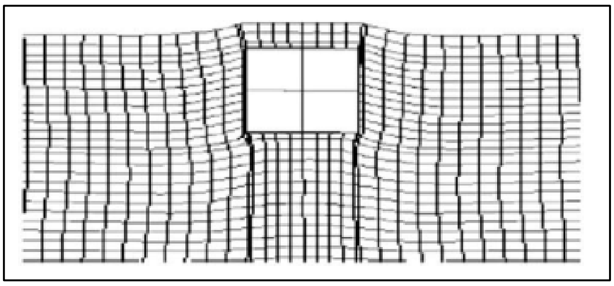

Figure 2.2: Shows How The Dynamic Loading Can Cause Uplift In Underground Structure [6].

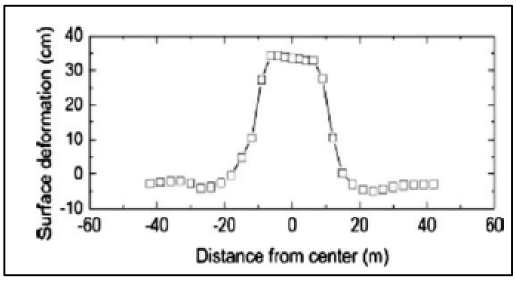

Figure 2.3: Shows How The Dynamic Loading Can Cause Uplift In Surface Ground [6].

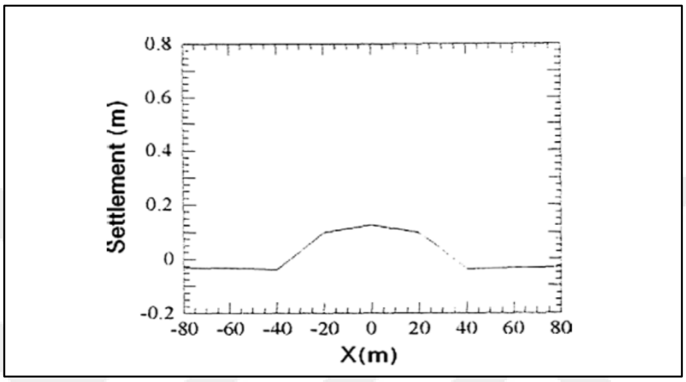

Figure 2.4: Khoshnoudian's Research On The Impact Of Liquefaction On Tunnels Dug On Liquefied Soils [6].

Depending on Bird et al. [7], in terms of accuracy this method should find out the possibility of liquefaction and the predict the ground damage between others. Usually, two types of methods are used to determine the possibility of liquefaction to happen. The first one depends on the equivalent-linear method and the simplified process suggested by Seed and Idriss [8] enhanced to take into account the uncertainties at several process levels [9] [10] [11] [12]. The second method determine the liquefaction possibility by models built on the elasto-plasticity theory to show the soil behaviour into the soil profile [13] [14] [15] [16] [17] [18]. The efficiency of this type of models better than equivalent-linear method as they show a rational mechanical procedure. In practice, to reduce the negative effects of earthquake that cause liquefaction in engineering structures, the counter-measure approaches such as gravel drains, soil densification or confinement walls among others are used. Such approaches are used in many numerically studies [19] [20] [21]or via tests [22] [23] [24] [25], these studies conclude to that any efficient solution should consider many parameters (such as soil properties and input signal characteristic). [26]

2.2 SOLUTIONS TO STOP LIQUEFACTION

There have been investigations on the behavior of tire chips as a geotechnical material (Edil and Bosscher, 1994; Yajima et al., 2006; Kawata et al., 2008; Tafreshi et al., 2012). Liquidification can be avoided by using a tire chip and sand combination (TS) (Uchimura et al., 2008; Yajima et al., 2009). When shaking a soil bag filled with tire bits and gravel, Yoshida et al. (2008) saw a decrease in the amount of surplus pore water pressure. In the current investigation, No. 7 silica sand and tire chips with a D50 of 10.54 mm were combined. Depending on the pipes being utilized, the size of the tire chips can be chosen in practice. It is remarkable that TS has virtually the same permeability as pure silica. The goal of the current investigation was to determine how colloidal silica grout affected loose sands' ability to resist liquefaction. Sands that have been treated with cement or chemical grouts have seen fewer liquefaction investigations than sands that have not. To assess the dynamic qualities or fatigue life for the construction of machine foundations, certain dynamic testing investigations have been conducted on sands treated with chemical grouts (Maher MH R. K., 1994; Rosenfarb JL, 1981). (Vipulanandan C, 2000). Sands that have been naturally and artificially cemented, as well as silicate-grouted sands, have all been the subject of several liquefaction experiments (Clough GW, 1989; Saxena SK, 1988). (Maher MH R. K., 1994). Clough et al. (Clough GW, 1989) looked at how poor cementation affected sand's ability to withstand liquefaction.

Finally, the soil is practically unliquefiable after a specific amount of cementation is attained. When compared to uncemented sands, Saxena et al. (Saxena SK, 1988) discovered that even a modest quantity of cement greatly boosts the cyclic strength. Comparable to what Saxena et al. (Saxena SK, 1988) observed, thick uncemented sands and loose cemented sands exhibit similar behaviors. They added that axial stresses were "frequently more uneven at the zero strain axis" during cyclic testing of cemented sands than for untreated sands. Additionally, they discovered that the relative density and curing duration both boosted the cyclic strength of treated sands.

An aqueous suspension of minute silica particles made from saturated silicic acid solutions is called colloidal silica (RK, 1979). When diluted, colloidal silica . Noll et al. [27] and Gallagher [28] reported gel times of up to 49 days and 200 days, respectively. The rate of particle-to-particle contact, which is influenced by a number of factors, determines the time before gelation, or gel time (DuPont, 1997). These elements include the silica solids content, silica particle size or surface area, pH, and salt concentration. As the silica solids concentration decreases and the size of the silica particles grows, the gel time increases. In general, minimum gel times happen between pH 5 and 6. If the pH is above or below this range, much longer gel durations can be obtained. As the salt content in the colloidal silica solution falls, the gel time lengthens. When no salt is added to a colloidal silica solution, the longest gel durations are seen. Colloidal silica is a desirable grouting material for passive site restoration due to its density, gel time, and viscosity properties. Stabilizers must have low viscosities to move quickly through the formations and lengthy, regulated gel times to solidify when they reach their destination if the procedure is to be successful. Excellent durability qualities may be found in colloidal silica. [29], [30], It is non-toxic and chemically and physiologically inert. Finally, compared to other liquefaction remediation techniques for built sites, it may be utilized in low quantities, which may make it more affordable.

Although the effectiveness of ground modification techniques for liquefaction reduction has only received a small amount of investigation. [31] [32], The majority of everyday designs for liquefaction mitigation have been built on conceptual knowledge or skewed analysis. As a result, there is a critical need for trustworthy numerical methods to evaluate the reduction of liquefaction-induced soil deformations to levels suitable for building. A recent design research is presented here to simulate the efficacy of the liquefaction mitigation approach of cement grouting for ground improvement of a sluice gate using rigorous finite element calculations. The seismic behavior of liquefiable soils may be appropriately realized by dynamic soil-water coupled analysis together with cyclic elastoplastic constitutive relations of soils based on cutting-edge computational geomechanics (Zienkiewicz OC, 1999) In this study, an effective stress based, fully coupled model is used to predict the seismic behavior of buildings in liquefiable soils. The code was created by the co-authors and other associates beginning in the 1990s (Shibata T, 1991), and it was first made available to the general public in (N. P. Marasini, 2015). (LIQCA Development Group 2002). The numerical simulation essentially makes use of the following theoretical hypotheses: (1) a two-phase fully coupled analysis using a u-p formulation (displacement for the solid phase and pore pressure for the fluid phase); (2) a cyclic elastoplastic constitutive model for sand based on a nonlinear kinematic hardening rule of the Armstrong-Frederick type. Through comparison with a number of laboratory experiments, including triaxial compression, simple shear, and hollow-cylinder torsional testing, the used elastoplastic constitutive model was well calibrated (N.P. Marasini, 2015). In order to conduct liquefaction analysis on real engineering projects, the finite element method was also used to case studies and experimental test findings, and then confirmed (Oka et al. 1994; N. P. Marasini, 2015; Huang et al. 2004, 2006, 2007). These prior and current work to evaluate liquefaction mitigation of soil deformations for structures may be seen as a further evolution of this study.

With enough compaction, as has been frequently done since the 1970s, the risk of liquefaction in a sandy backfill can be decreased. For sandy backfills, it has been advised to obtain a degree of compaction (Dc) of greater than 90%. Sewage pipelines distorted and floated during previous earthquakes including the 2007 Noto Peninsula Earthquake and the 2007 Niigata Chuetsu-oki Earthquake because liquefaction took place on sandy backfills with Dco90%. The performance of well-compacted sandy backfills with Dc>90% during the earthquakes, on the other hand, was good. It should be noted that compaction work might damage subterranean pipelines, making it difficult to complete in the constrained area of a backfill trench. One of the main causes of significant damage to buildings, oil tanks, bridges, tunnels, underground pipelines, and maritime structures, according to investigations into damage sites following earthquakes (N.P. Marasini, 2015), is soil liquefaction (Abdoun et al. 2005; Lee 2005). Recent large earthquake occurrences have shown that underground constructions situated in liquefiable soil layers are prone to floating up during earthquakes (Tobita et al. 2010, 2012; Chian and Madabhushi 2010, 2012). It is challenging to conduct in-depth studies of liquefaction phenomena since earthquakes are unpredictable and uncommon. An alternative to geotechnical earthquake engineering that has been utilized to provide light on failure processes is small-scale physical modeling. Geotechnical modeling needs to replicate the stiffness and strength related to soil behavior. The amount of stress and the history of stress have a big impact on soil behavior. Centrifuge modeling provides the low-cost, small-scale replication of complicated scenarios. In liquefiable soil deposits, underground constructions are prone to floating during earthquakes. Large earthquake occurrences have shown these damage instances. The uplift behavior of a rectangular tunnel buried in liquefiable soils was investigated using a series of centrifuge tunnel model shaking table experiments. According to the test findings, the amount of tunnel uplift displacement is influenced by the buried depth of the tunnel, the input amplitude of base acceleration, and the quantity of loading cycles. The surrounding soil pressing onto the tunnel bottom is the primary source of tunnel rising because of the larger hydraulic gradient between the soil beneath the tunnel bottom and the surrounding soil, which became apparent. Reusing waste glass has been more popular recently; for instance, 210,000 t of glass bottles were abandoned in Japan in 2013 without being recycled or utilised (Glass Bottle 3R Promotion Association, 2014). Noteworthy is the extraordinary chemical stability of broken glass used as backfill. Crushed glass backfill's strong hydraulic conductivity can provide effective liquefaction resistance.

This characteristic is comparable to the widely used approach of employing coarse aggregates. When used as backfill under traffic loads, broken glass has been observed to be somewhat stiff if it is sufficiently compacted (Masahide Otsubo, 2016) Mikami et al. (2010) suggested using a geo-grid to effectively condense this material (Masahide Otsubo, 2016). In triaxial compression testing, crushed concrete shown strong results in 2005 and according to Tatsuoka et al. (2013). Crushed concrete was utilized by Orense et al. (2003) to make stone columns, and they found that it worked well to prevent subterranean pipes from floating. As a result, it is anticipated that the high permeability of the crushed concrete backfill may lessen liquefaction. Additional cementation between the pieces of broken concrete could happen if some.

2.3 AIR BUBBLES INJECTION LIQUEFACTION MITIGATION

In a study on sandy soil, it was discovered that adding air bubbles causes the degree of soil saturation (sr) to decrease, which increases resistance to liquefaction. based on research on the sand compaction pile method (SCP method), which improves the engineering qualities of in-situ sandy soils (M.hatanka). After executing the sand compaction as depicted in the figure, it was discovered that the velocity wave had been greatly reduced due to air injection into the soils. Figures 2.5 and 2.6 show, respectively, the soil profiles of the sample site and the P-wave velocity findings of P-S wave-logging experiments conducted before and after sand compaction.

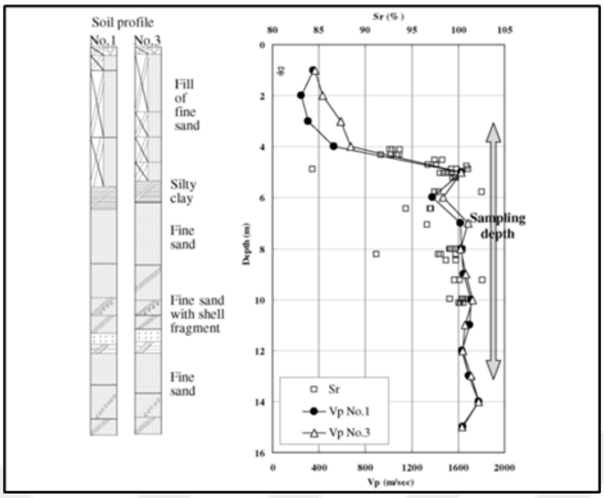

Figure 2.5: Soil Profile Vp and Sr Of Sampling Site (Before Compaction) [33].

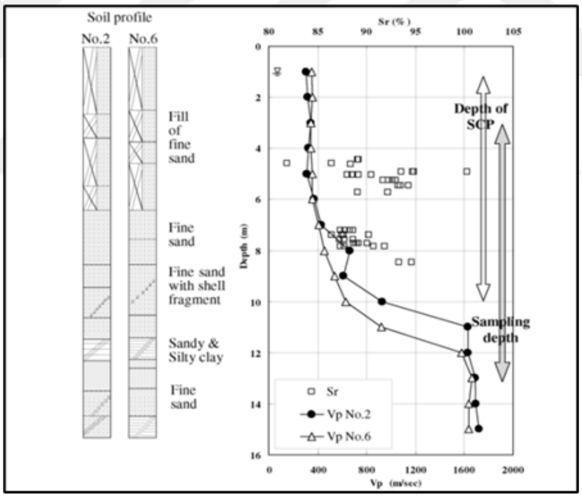

Figure 2.6: Soil Profile Vp And Sr Of Sampling Site (After Compaction), (M. Hatanaka, 2008).

In order to determine the liquefaction strength and the degree of saturation accurately, the insitu freezing method were applied to obtain a continuous undisturbed sample with 150mm diameter to a depth of 3 to 12m from the ground surface were recovered. The sample at point A was obtained before applying the sand compaction Fig (2.5&2.7), while that at the point B the sample was taken after the sand compaction Fig (2.6&2.7). P-S wave logging test using the method of suspension type were performed in the field to measure the Vp before no

(1&3) and after (no2&6) sand compaction. It was clear that vp was sharply reduced to about 300 to 700 m/sat depth of 5 to 10 m after sand compaction within which the soil was improved by scp execution Fig 2&3. The injection of air bubbles into the ground during performing SCP caused the decrease of the degree of saturation sr Resulting in the dramatic decrease in theprimary waves strength. However, the application of the sand compaction on the insitu sandy soil has reduced the degree of saturation by 84-95% at depth of 5 to 9 meter.

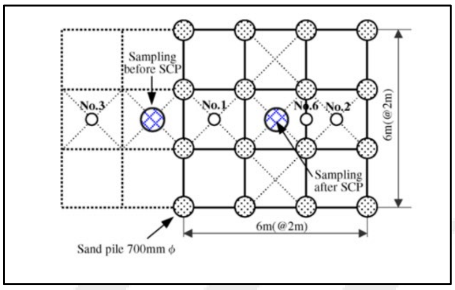

Figure 2.6: Locations Of Freezing Sampling, Field Test And SCP, *Tunnels Modelling By Plaxis 2D* [33].

3. TUNNELS MODELLING BY PLAXIS 2D

The most liquefiable soil type is sandy soil, which is simulated by the UBC3D-PLM model. There are several models for various soil types. This model is a complex one for simulating liquefaction behavior in dynamic applications. The UBC3D-PLM model employs two yield surfaces to make sure that the transition is smooth as it comes near to the mobilized friction angle. The Mohr-Coulomb yield condition and a hardening law similar to the Hardening Soil model are both employed in the UBC3D-PLM model. A densification law for the secondary state is included in the UBC3D-PLM model through a second yield surface with a kinematic hardening rule that is dependent on the quantity of loading cycles. The evolution of the excess pore pressure is more accurately described because to this relationship.

a. Parameters used for the UBC3D-PLM Model rely on assumed relative density of 40%, is mentioned below

Rd=40%.

Table 3.1: UBC3D-PLM Model Parameters [34]

Parameter	**Definition**	**formula**	**value**
k*eB	Elastic bulk modulus factor	kB *e = 0.7 × kG *e	584.138
k *e G	Elastic shear modulus factor	kG *e = 21.7 × 20 × (N1) 60 0.3333	834.48
k *p G	Plastic shear modulus factor	kB *p = kG *e × (N1) 60 2 × 0.003 + 100	226.55
me	Rate of stress-dependency of elastic bulk modulus	Through previous study	0.5
ne	Rate of stress-dependency of elastic bulk modulus	Through previous study	0.5
np	Rate of stress-dependency of plastic shear modulus	Through previous study	0.4
Pref	Reference pressure	Through previous study	100
φcv	Constant volume friction angle	Assumption	33

Table 3.1: UBC3D-PLM Model Parameters [34] "Table Continued"

φp	Peak friction angle	φp = φcv + (N1) 60 10 + max (0; (N1) 60 ï 15 5)	33.7111
C	Cohesion	No cohesion for sandy soil	0
Rf	Failure ratio	R f =1.1((N1) 60) ï0.15 < 0.99	0.8196
(N1)60	Corrected SPT value	N1) 60 = RD 2 152	7.111
γ unsat	Dry unit weight	15+4*Rd /100	16.6
γ sat	Saturated unit weight	19+1.6Rd /100	19.64

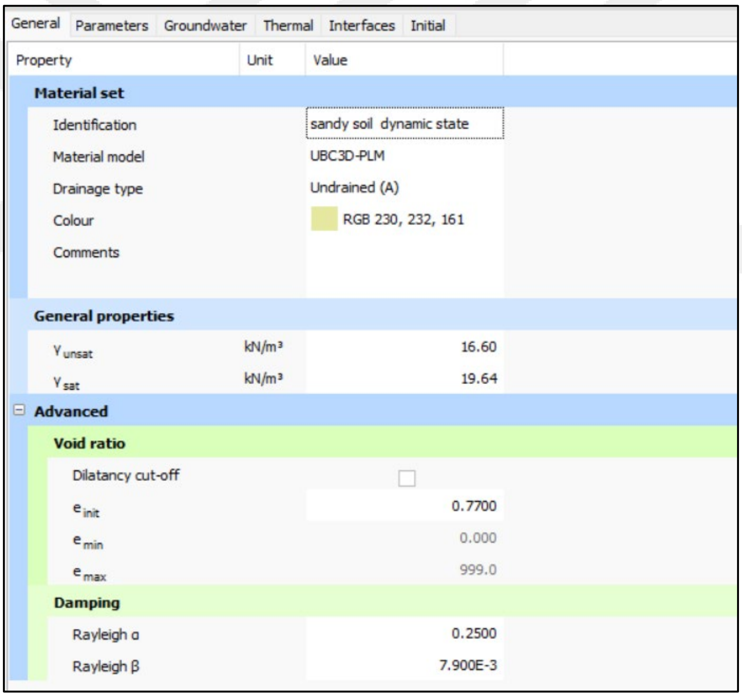

Figure 3.1: UBC3D-PLM General Parameters.

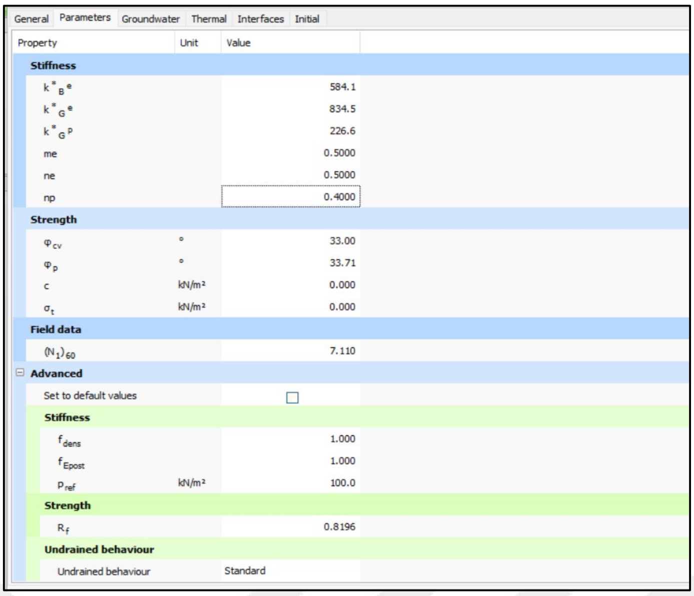

Figure 3.2: UBC3D-PLM Parameters.

b. Parameters used for the Hardening Soil model with small-strain stiffness (HSsmall) Model rely on assumed relative density of **40%**, is mentioned below

Table 3.2: The Hardening Soil Model With Small-Strain Stiffness (Hssmall) Model [35]

Parameter	**Definition**	**Formula**	**value**
Stiffness parameter	E_{50}^{ref}	=60000Rd /100 KN/M2	24000
	E_{oed}^{ref}	=60000Rd /100 KN/M2	24000
	E_{ur}^{ref}	=180000Rd /100 KN/M2	72000
Strength-related properties	ϕ '	=28+12.5Rd/100	33
	ψ	=-2+12.5Rd/100	3
	Rf	=1-Rd/100	0.95
Small strain	G_0^{ref}	=60000+68000Rd/100 KN/M2	872000
	Γ0.7	(2-Rd/100)*10^-4	0.00016

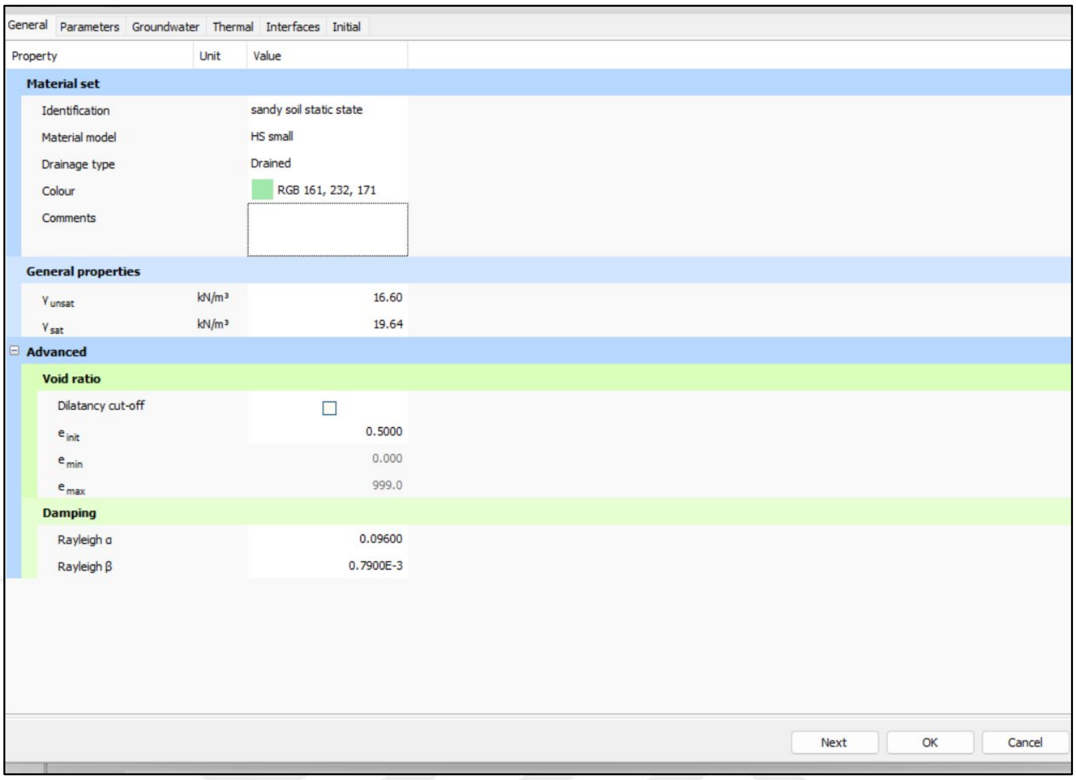

Figure 3.3: The Hardening Soil Model With Small-Strain Stiffness (Hssmall) G.Parameter.

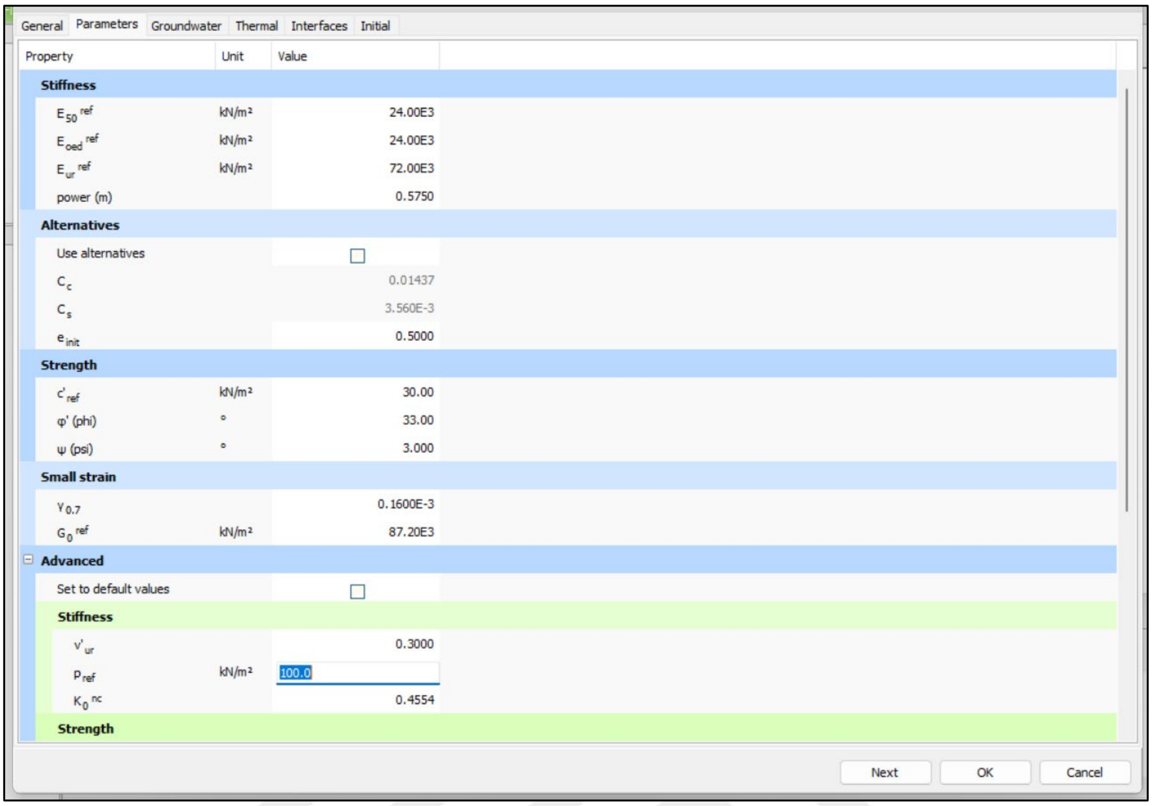

Figure 3.4: The Hardening Soil Model With Small-Strain Stiffness (Hssmall) Additional Parameter.

3.1 CIRCULAR TUNNEL ANALYSIS

3.1.1 Test 1

The study starts with figure (3.5) below, as following, a sandy soil sample with size of 60*24M was used to run this test, a tunnel with radius of ***2.5*** Meters were put with depth of ***10m*** from the surface to the middle of the tunnel (radius), the green layers shown in the figure(24) represent the sandy soil in the static codition, while the pink barriers layers at both edges right&left represent the free fieled, and the grey layer at the bottom is the bedrock layer. After running the test we can see the maximum defomation in the tunnel by 0.6579 as in fig(25) and the chart of the dynamic time with the excess pore water pressure. Fig (3.6)

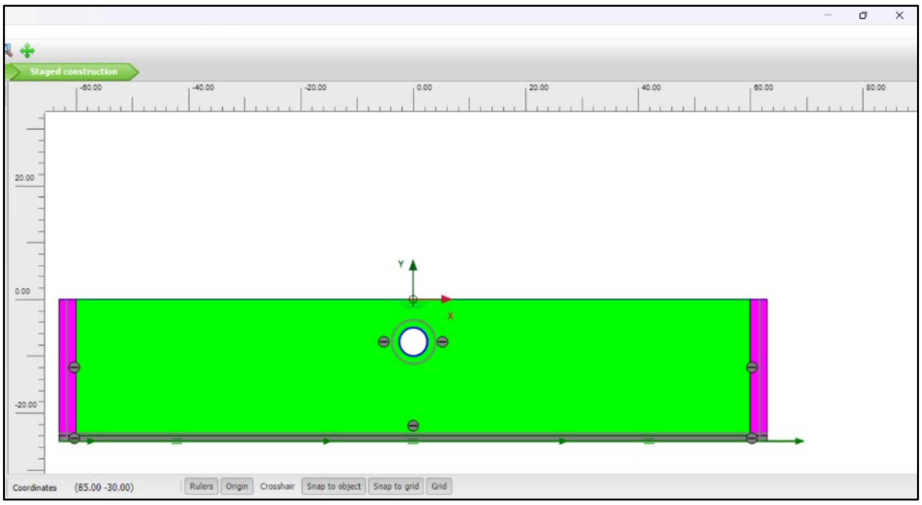

Figure 3.5: Plaxis2D Application Interface.

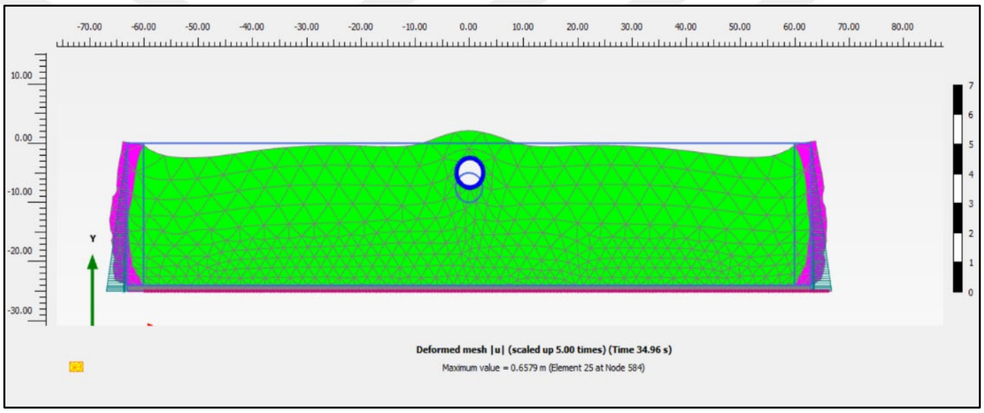

Figure 3.6: Uplift Of The Circular Tunnel 1st Test.

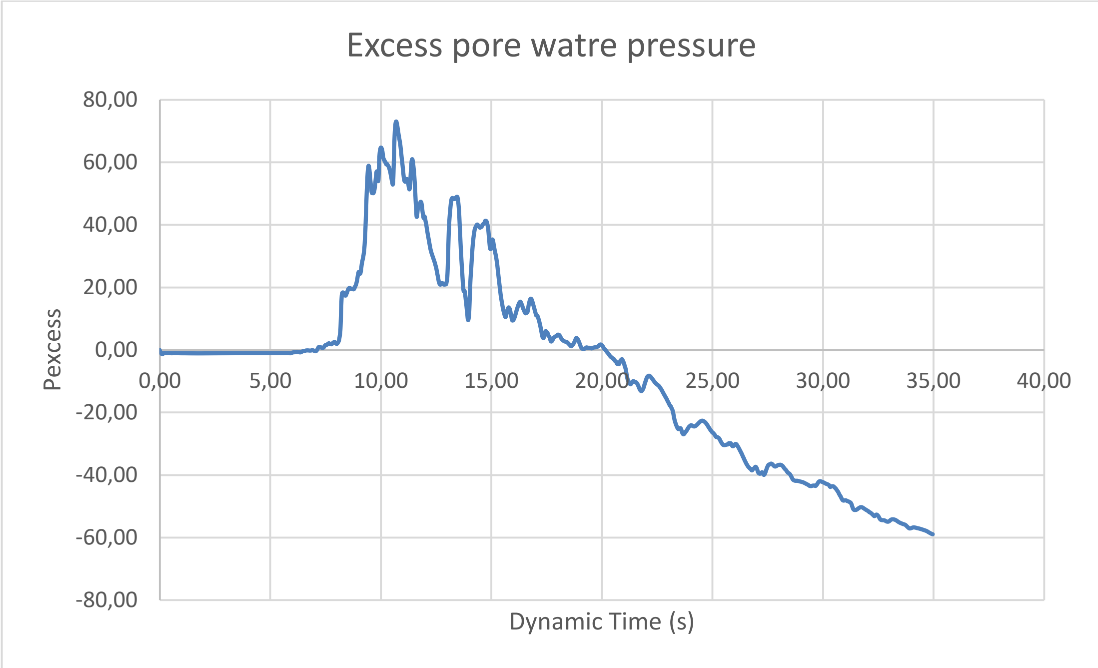

Figure 3.7: Chart Of The Excess Pore Water Pressure Values, With The Dynamic Time.

3.1.2 Test 2

In the second test the radius was increased from 2.5 to 3.5 with an approximate same depth of the previous test.

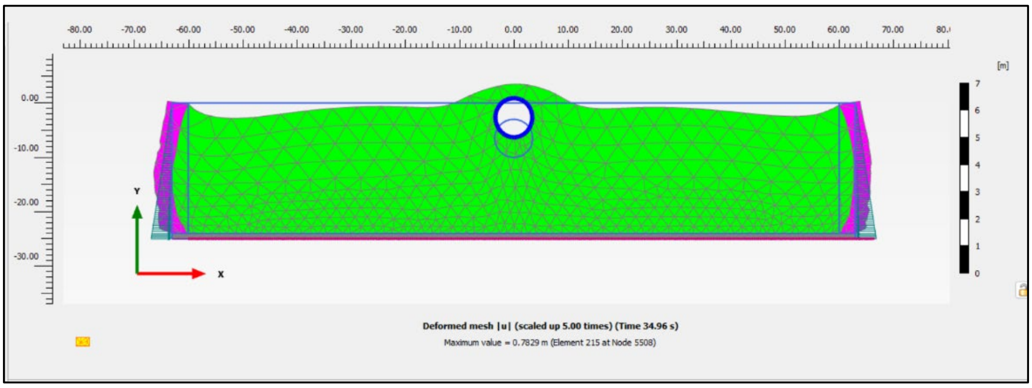

Figure 3.8: The Uplift Has Been Changed With Changing The Diameter.

3.1.3 Test 3

tunnel radius was increased to 5 m so the diameter is 10, and the maximum dispalcement has reached above 1m as shown in the fig(3.9) below.

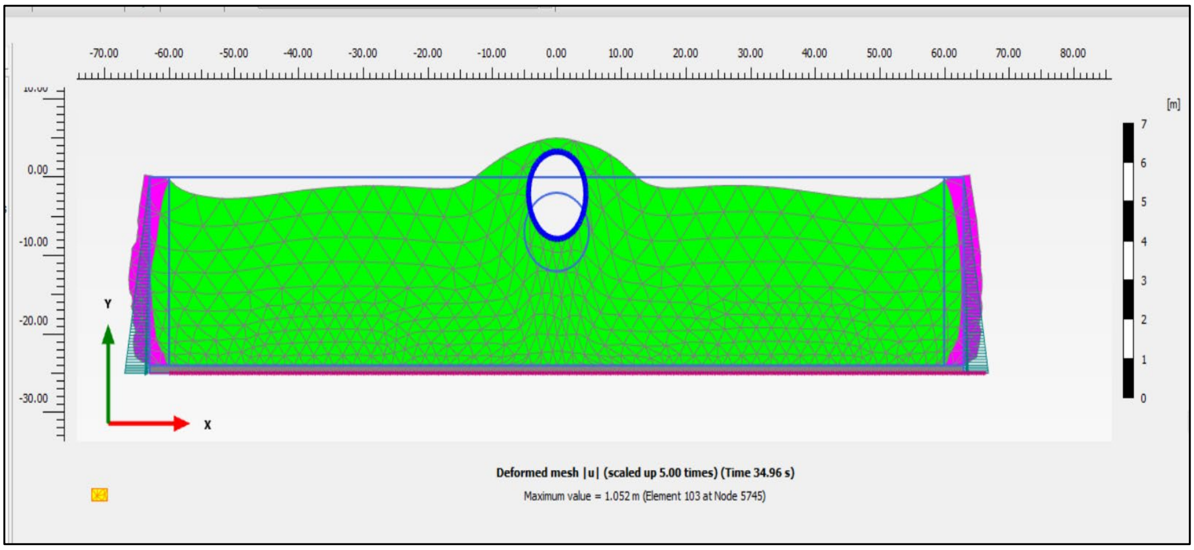

Figure 3.9: Increasing The Radius Increases The Uplift Displacement.

3.2 SQUARE TUNNEL UPLIFT ANALYSIS

3.2.1 Square Shape Analysis Side 5m and 7m Depth To The Middle Of The Tunnel

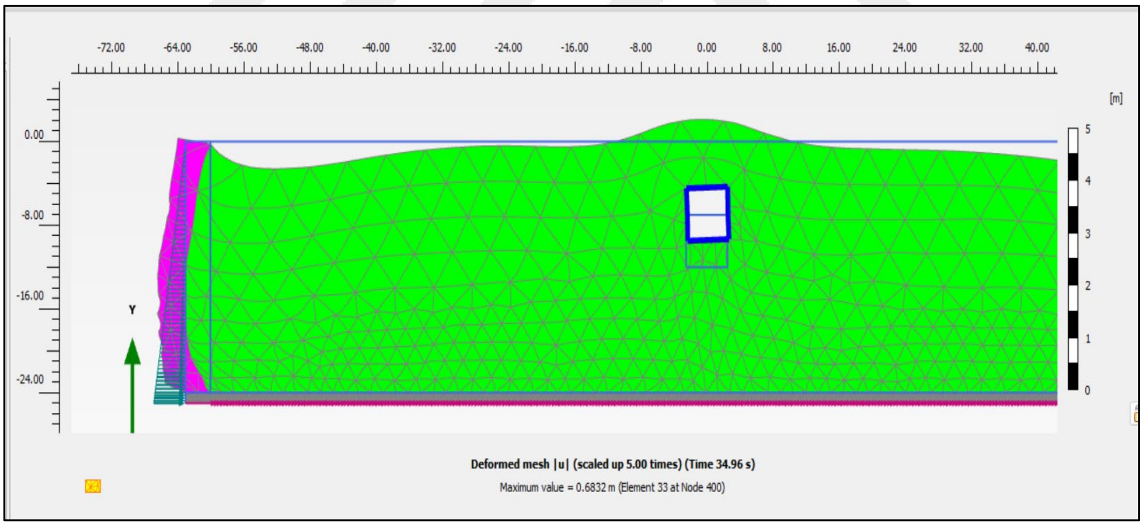

Figure 3.10: 5M Length Tunnel Displacement.

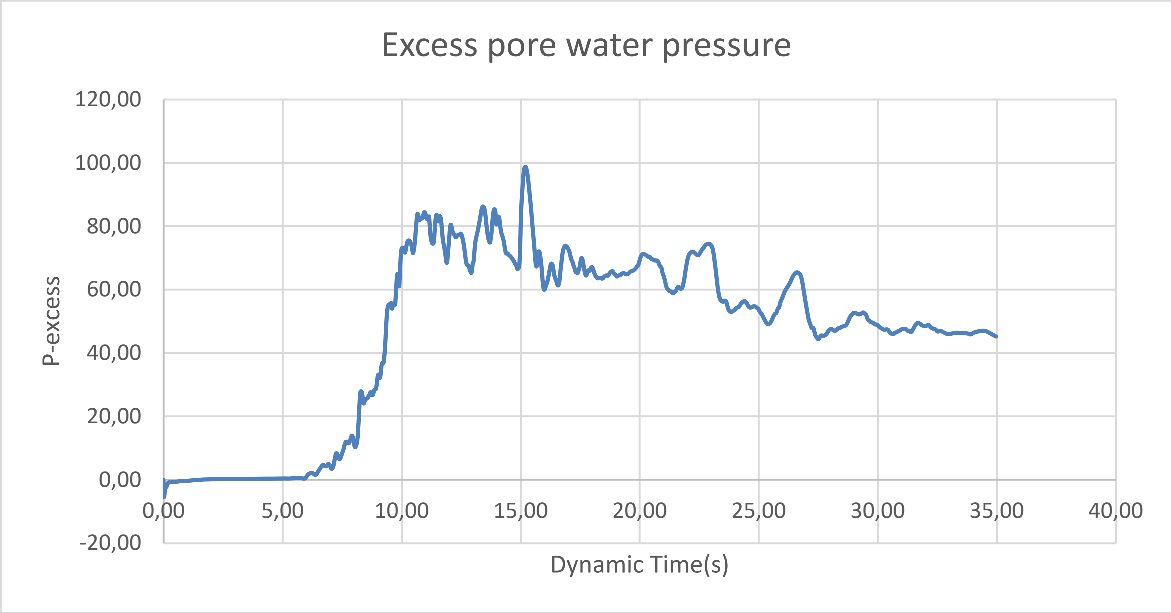

Figure 3.11: Chart Of The Excess Pore Water Pressure Values, With The Dynamic Time

3.2.2 (7*7M) Square Tunnel Test

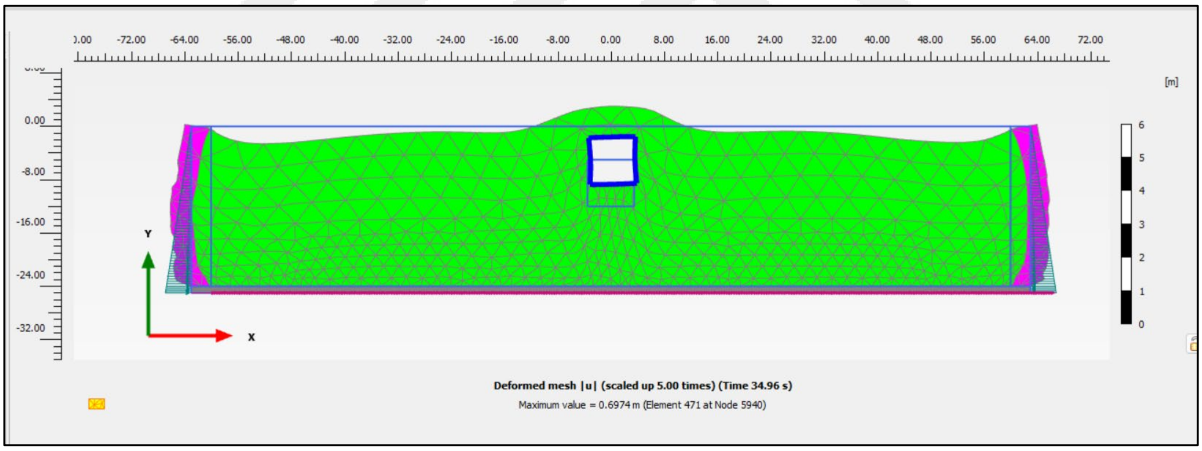

Figure 3.12: Displacement Of The 7m Length Square Tunnel.

3.2.3 (10*10 M) Square Tunnel Analysis

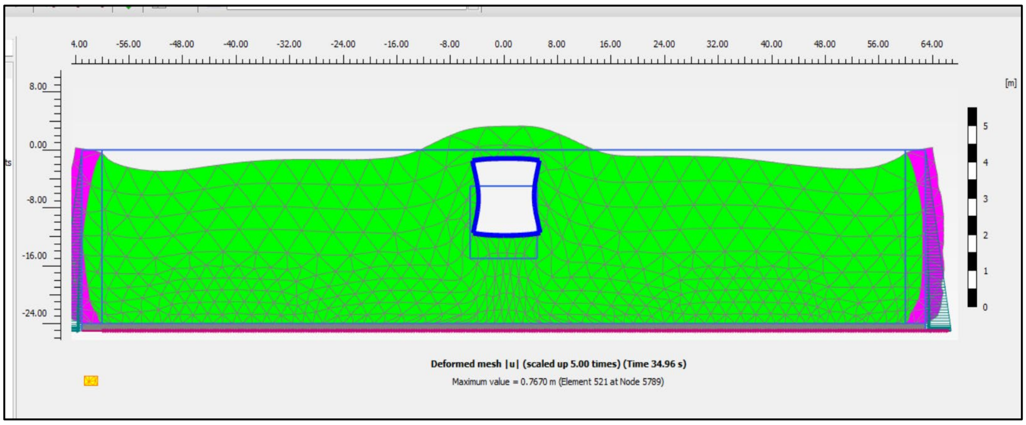

Figure 3.13: 10M Side Length Square Tunnel's Uplift

4. DISCUSSION AND COMPARISON

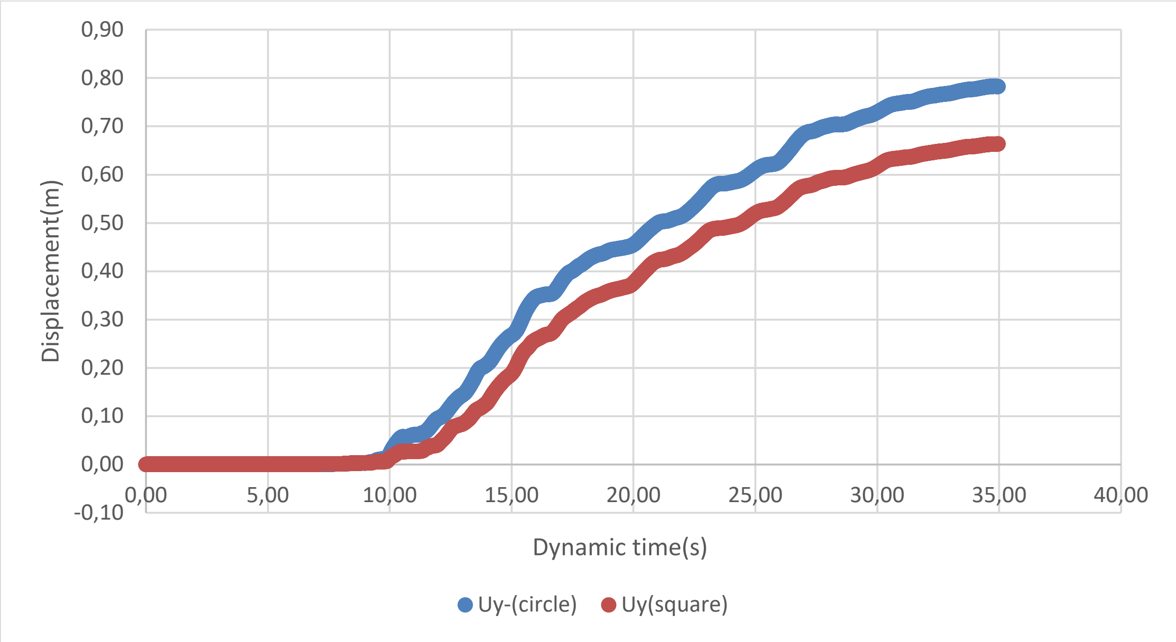

Figure 4.1: Comparison Between The Displacement Of Circular And Square Tunnel L=D= 7 m

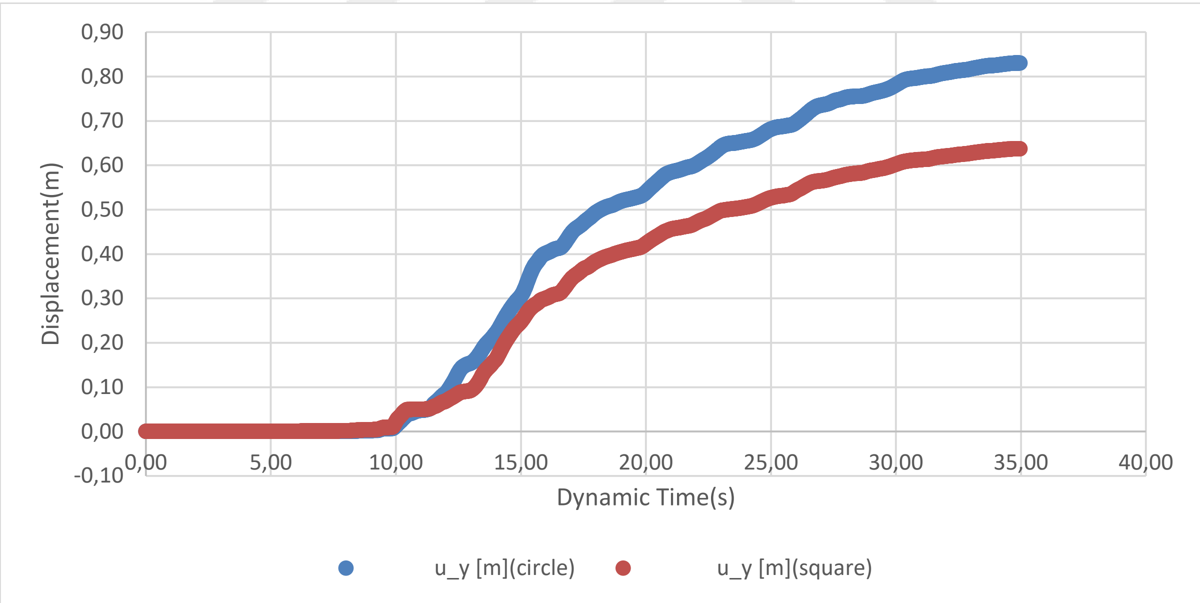

Figure 4.2: Comparison Between The Displacement Of Circular And Square tunnel L=D= 10 m.

Finally, we can see the differences in values between the two tunnel shapes and the increase in displacement value that occurred as the size of the tunnels was increased as in figure 34. Also, we can see through figure 33, that if we have a specific dimension for the two shapes, the square tunnel will have less displacement, whereas as the dimension or size of the tunnels is increased, we will see that the circle tunnels will have a larger u_y with values exceed 1 meter. Fig 35&36 describe the variation of the behaviour of excess pore water pressure with the dynamic time.

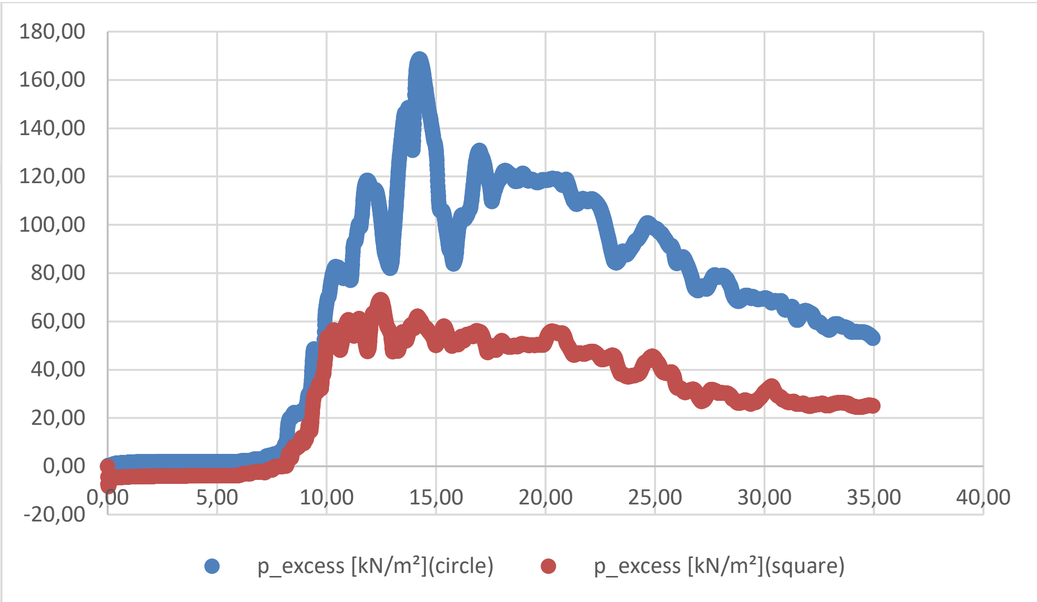

Figure 4.3: Excess Pore Water Pressure Comparison Among Circular And Square Tunnel (D=L=7M).

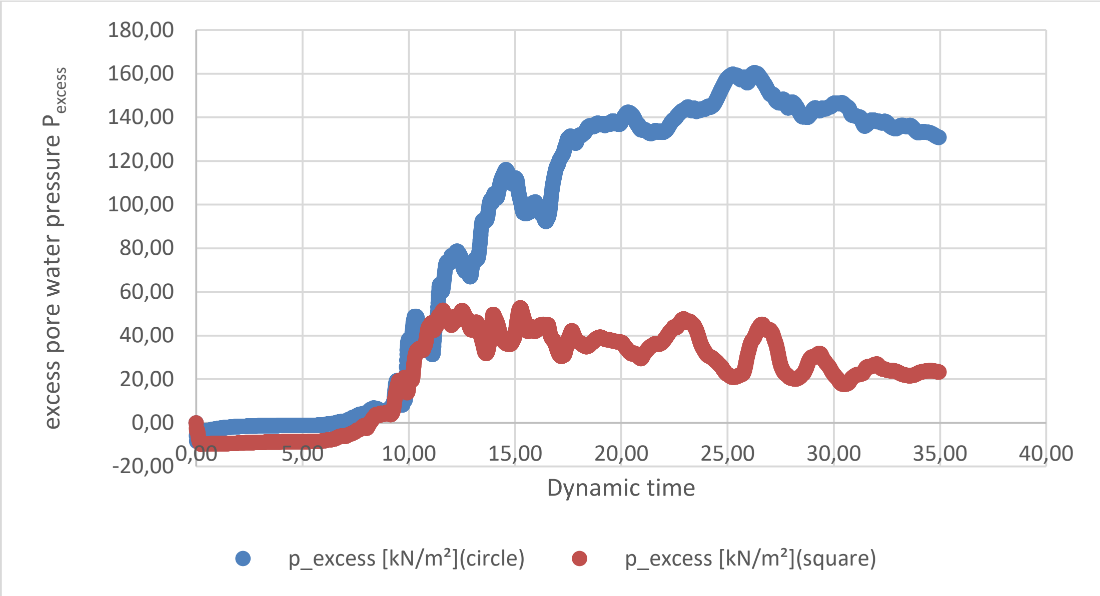

Figure 4.4: Excess Pore Water Pressure Variation With Time When D=L=10M.

5. CONCLUSION

The aim of the study is to evaluate the liquefaction phenomena and to limit the damage caused by it. Through many conducted research and studies, the hurt of the liquefaction due to an earthquake was successfully eliminated by many techniques such as Jet grouting, air bubbles injection. In this study, the effect of the shape and the size, was discussed and conclude that, the uplift of a tunnels increases with increasing the size of a tunnel, such as increasing the diameter of the tunnel in case of circular tunnel and increasing the dimensions of the sides lengths in case of the square or rectangular shape. Although in one study it was mentioned that increasing the diameter of an object, decrease the settlement occurrence, but the study was for those structure above the ground surface or same level of GS. The reason behind the increasement of the uplift displacement is that when increasing the area or dimension of a tunnel, the pressure on the tunnel increases too, there is a direct correlation between the area and the pressure, regardless on the effect of the weight of the tunnel, its assume that same weight here but different size like for example less used quantity in the bigger tunnel. Whatever, as shown in table (1) the uplift in the circular tunnel is more comparing with the square one. So, for the liquefaction uplift displacement solution related to the shape, according to the current study, its recommended to use a square tunnel with less diameter to have better safety.

T**able 5.1 :** Summary And Analysis Of Displacement For Circular And Square Tunnel.

Test Category 1	**No of test**	**Diameter**	**uplift (m)**
Tunnel (Circular shape) with approximate same depth from the surface to middle of the tunnel.	1	5	0.6579
	2	7	0.7642
	3	12	1.052
Test Category 2	No of test	Length	uplift (m)
Tunnel (square shape) with approximate same depth from the surface to middle of the tunnel.	4	5*5	0.6832
	5	7*7	0.6974
	6	10*10	0.7670

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