



REPUBLIC OF TURKEY
ALTINBAŞ UNIVERSITY
Institute of Graduate Studies
Civil Engineering Department

**NUMERICAL MODELLING OF MITIGATION OF
LIQUEFACTION – INDUCED UPLIFTS IN
TUNNELS**

Ahmed J S SULAIMAN

Master's Thesis

Supervisor

Assist. Prof. Dr. Mohsen SEYEDİ

Istanbul, 2022

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2022

The thesis titled “NUMERICAL MODELLING OF MITIGATION OF LIQUEFACTION – INDUCED UPLIFTS IN TUNNELS” prepared by AHMED J S SULAIMAN and submitted on 15/08/2022 has been **accepted unanimously** for the degree of Master of Science in Civil Engineering.

Assist. Prof. Dr. Mohsen SEYEDİ

Supervisor

Thesis Defense Committee Members:

Assist. Prof. Dr. Mohsen SEYEDİ

Faculty of Engineering and
Architecture,

Altınbaş University

Assoc. Prof. Dr. Sepanta NAİMİ

Faculty of Engineering and
Architecture,

Altınbaş University

Assist. Prof. Dr. Kaveh DEHGHANIAN

Faculty of Engineering,

İstanbul Aydın University

I hereby declare that this thesis meets all format and submission requirements of a Master’s thesis.

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Ahmed J S SULAIMAN

Signature

DEDICATION

I dedicate this work to my parents and my family and my friends whose gave me the encouragement all the time to finish this work and to get the master's degree. and I gratefully dedicate it to all my teachers in every education period, especially Dr. Mohsen SEYEDÍ who kindly guided me on this work and gave me his great knowledge, thanks a lot for all that.



ABSTRACT

NUMERICAL MODELLING OF MITIGATION OF LIQUEFACTION – INDUCED UPLIFTS IN TUNNELS

Sulaiman, Ahmed

M.Sc., Civil Engineering Department, Altınbaş University,

Supervisor: Assist. Prof. Dr. Mohsen SEYEDİ

Date: August/2022

Pages: 128

The earthquakes are the most dangerous natural disasters that facing a lot of countries around the world, and the earthquakes result in many consequences that can lead to human and economic losses, so it is necessary to take measures to mitigate the expected damages. The problem of liquefaction in the soil is one of the common problems when earthquake or any strong motion occur. When liquefaction occurs in the soil, it produces many disasters for the infrastructures and facilities especially tunnels, and it leads to uplift the tunnel from its position. This research addressed this issue and tried to mitigate the uplifting in the tunnels, by making models in PLAXIS 2D software by including mitigation techniques in these models and making comparisons between the results from these models to find if these techniques are successful or not. The mitigation techniques in this thesis to mitigate the uplift of the tunnel centered on replacing the sand around the tunnel by another sand has a higher relative density or replacing the sand around the tunnel by another soil such as clay soil and gravel soil. And this study was done under three different earthquakes such as the earthquake of Kocaeli, Kobe and Northridge.

Keywords: Liquefaction, Uplift, Tunnel, Pore Water Pressure, Mitigation

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ABBREVIATIONS

RD	:	Relative Density
CSR	:	Cyclic Shear Stresses Ratio
CRR	:	Cyclic Resistance Ratio
PGA	:	Peak Ground Acceleration
FEA	:	Finite Element Analysis
SPT	:	Standard Penetration Test
VDC	:	Virtual Data Center
UTC	:	Coordinated Universal Time

LIST OF SYMBOLS

Δu	: The excess pore water pressure
σ'_{3C}	: Minor effective consolidation stress
r_u	: Excess pore water pressure ratio
$K\sigma$	: Overburden correction factor
σ'_{vc}	: Vertical effective stress at consolidation
K_α	: Correction factor for the effects of an initial static shear stress ratio
K_o	: Coefficient of lateral earth pressure at rest
W_s	: The dissipated energy per unit volume of soil split on the initial effective confining pressure
a	: Inconstant parameter relies on type of the soil
α_1	: Constant parameter relies on soil characteristics
β_1	: Constant parameter relies on soil characteristics
Γ	: Shear strain amplitude
σ'_c	: Initial effective confining pressure
S_r	: Degree of saturation
B-value	: Skempton's coefficient
γ_{unsat}	: The unit weights of the sand in unsaturation state
γ_{sat}	: The unit weights of the sand in saturation state

E_{50}^{ref}	: The secant stiffness in standard drained triaxial test
$E_{\text{oed}}^{\text{ref}}$	: The tangent stiffness for primary oedometer loading
$E_{\text{ur}}^{\text{ref}}$	: The unloading and reloading stiffness from drained triaxial test
G_0^{ref}	: The reference shear modulus at very small strains such $\varepsilon < 10^{-6}$
P_{ref}	: Reference pressure
ε	: Strain
m	: The rate of stress dependency or the power for stress-level dependency of stiffness
$\gamma_{0.7}$	: The modulus decreasing curve to the cyclic shear strain level
G_0	: The initial or very small-strain shear modulus
G_s	: The secant shear modulus
ϕ'	: The friction angle
ψ	: The dilatancy angle
R_f	: The failure ratio
$(N1)_{60}$	: The corrected SPT value
K_G^{*e}	: The elastic shear modulus factor
K_B^{*e}	: The elastic bulk modulus factor
K_G^{*p}	: The plastic shear modulus factor
m_e	: The rate of stress-dependency of elastic bulk modulus
n_e	: The rate of stress-dependency of elastic bulk modulus

n_p	: The rate of stress-dependency of plastic shear modulus
C	: The cohesion
φ_p	: Peak friction angle
φ_{CV}	: Constant volume friction angle
f_{dens}	: Densification factor
f_{Epost}	: Post-liquefaction stiffness degradation
E'	: Young's modulus at reference level
$S_{u,ref}$	: Undrained shear strength at reference level
ν'	: Poisson's ratio
E'_{inc}	: Young's modulus inc.
y_{ref}	: Reference level
$S_{u,inc}$	: Undrained shear strength inc.
C'_{inc}	: Cohesion inc.
k_x	: Horizontal permeability
k_y	: Vertical permeability
E	: Young's modulus
G	: Shear modulus
E_{oed}	: Oedometer modulus
V_s	: Shear wave velocity
V_p	: Compression wave velocity

EA : Normal stiffness

EI : Flexural rigidity

w : Weight

M_w : Moment magnitude

M_s : Surface-wave magnitude

Dip : The angle of the fault in decimal degrees

$r_{u,\sigma'v,max}$: Max excess pore pressure ratio

1. INTRODUCTION AND PURPOSE

Liquefaction means the state when the undrained sand loses shearing strength and behaves as a liquid during strong earthquake or any strong shaking, this loss in the shearing strength didn't come back until the dissipation of the excess pore-water pressure. There are a lot of damages to the building or any structure resulting from the liquefaction during the earthquakes, so it's the main attention to the public and researchers. the sand boil on the ground surface is a common appearance of liquefaction, it occurs by leakage of water throughout ground cracks. Figure 1.1 illustrates the sand boil when the liquefaction occurs.



Figure 1.1: The sand boil shape when the liquefaction of underlying sediments occurred by the 1978 Miyagi-ken-Oki, Japan earthquake [1].

There are a lot of potential outcomes of liquefaction, these outcomes depend on the nature of any structure on the site, the characteristics of the seismic load and site status. These results involve instability or deformation of soil, increase in the lateral stresses versus retaining structures, loss of the bearing support and lateral support for foundations and embedded structures, buoyancy of buried structures such as reservoir tanks or tunnels by accumulation of the pore water pressure at the base of the buried structures, settlement of the buildings by reconsolidation of soil after occurrence of liquefaction. Figure 1.2 and Figure 1.3 illustrate the damages when the liquefaction occurs in the sand.



Figure 1.2: The uplifting of buried tank in liquefied soil by the 1964 Niigata earthquake [1].



Figure 1.3: Sloping of the buildings by the 1964 Niigata earthquake [1].

Under cyclic loading resulting from earthquakes, loose sand attends to shrink or contract, so the normal stresses in the sand can transfer to the pore water when the soil is undrained condition, because in this case the soil is saturated and unable to drain during earthquakes. so, a decreasing in the effective confining stress inside the soil will happen, and there will be some deformations to the soil deposit resulting from the loss of stiffness and strength of the soil [1].

The time series of earthquake-induced cyclic stresses includes many cycles with various intensities, and the effects of the nonuniform time series rely on the cycles number and the stress volumes of each cycle. Several studies have shown that a nonuniform time series can be estimated by a uniform time series with an equipollent number of uniform cycles that depends on the amplitude of the uniform cyclic stress [1].

Factors that influence the liquefaction potential in a soil include: the relative density of the soil deposit, grain size distribution, confining pressures on the soil, thickness of the soil deposit, duration and severity of ground shaking, applied loading pattern, amount of the air that entrapped in the soil deposit, the properties of the deposit that related to drainage, strain history of the soil deposit, soil structure and cementation.

Under cyclic loading, the stress-strain response of sand is highly dependent on the maximum effective stress in the sand, relative density (RD), stress history, and other factors. The theory of the energy-per-unit-volume could be the perfect way to determine the potential of liquefaction in the soil deposit, to measure the energy per unit volume required to reach liquefaction; several tests were performed at various confining pressures, strain amplitudes, and relative densities by using hollow cylinder apparatus, and then relationships have been inferred between the energy per unit volume of the soil and the different parameters that changed during the tests [2].

Different numerical and analytical methods have been improved to assess the liquefaction induced by earthquakes [9]. These methods include three fundamental estimation approaches to assess the soil liquefaction: energy-based procedures, strain-based procedures, and stress-based procedures. The energy-based approach uses the dissipation of the energy in the soil when the cyclic loading occurs, to know the variation in the pore-water pressure occurred in the soil.

Under undrained conditions, the soil grain structure bounces to the range that necessary to remain the volume constant, and this interaction between the volume reduction and soil structure bounce determines the magnitude and the extent of the excess pore water pressure in the soil, and the sand reaches to the state of liquefaction when the pore water pressure equals the normal pressure or effective confining pressure, because the sand in this case loses shear strength, and this causes an increase in the pore water pressure rapidly and it undergoes high strains. When

an earthquake occurs, the sand particles are rearranged due to the cyclic load, this rearrangement in the sand particles causes some deformation of the soil, and that dissipates a specific amount of energy and the area of the hysteresis loop represents the energy per unit volume which dissipated in one cycle of loading, and this energy is related to the energy required to change the volume of the soil, so the build-up of the pore water pressure and the occurrence of liquefaction are associated to the quantity of energy dissipated in the soil [2]. Figure 1.4 illustrate the hysteresis loop.

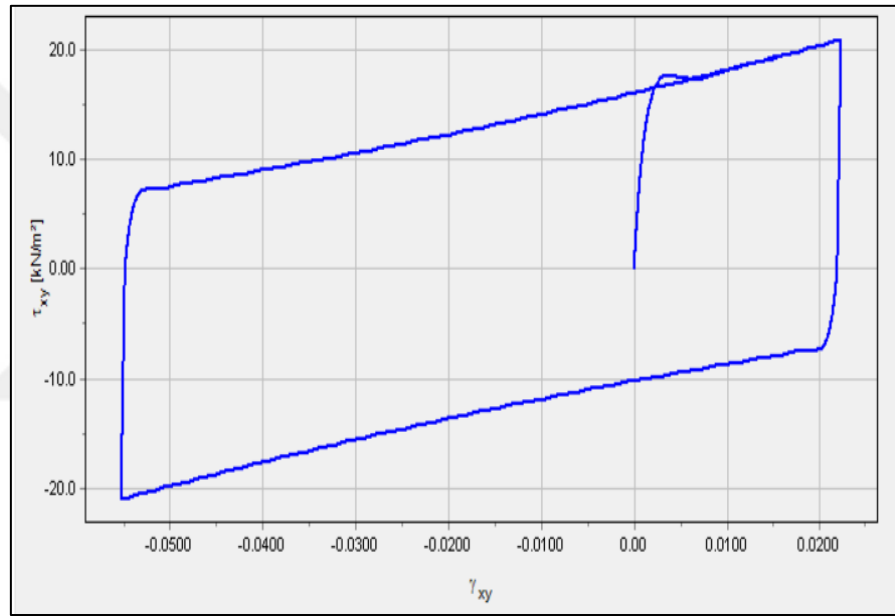


Figure 1.4: The Hysteresis Loop from the current project.

“The excess pore water pressure (Δu) generated during undrained cyclic triaxial loading is normalized by the minor effective consolidation stress (σ'_{3C}); this ratio is called the excess pore water pressure ratio (r_u)” (I. M. Idriss, R. W. Boulanger, 2008, p.22), this can be simplified by the formula (1.1) for calculate the excess pore water pressure ratio (r_u):

$$r_u = \Delta u / \sigma'_{3C} \quad (1.1)$$

the maximum value for the excess pore water pressure ratio (r_u) is 1.0 (or 100%), this happens when $\Delta u = \sigma'_{3C}$. When this condition happens ($r_u = 1.0$) this called initial liquefaction [1].

During an earthquake, the shear stresses produced at any depth in a soil deposit of a ground with a level surface are primarily resulting from the vertical propagation of the horizontal shear waves, while the earthquake-induced cyclic stresses under flat ground surfaces refers mainly to the impacts of the horizontal shaking. When a vertical shaking occurs in the soil profile causes additional transient variations in total horizontal stress, total vertical stress, and pore water pressure, but the horizontal and vertical effective stresses remain without affect. the liquefaction in the soil occurs at depths which the induced stresses from shaking exceed the cyclic resistances, so the design methods require determining the earthquake-induced cyclic shear stresses ratio (CSR) and cyclic resistance ratio in the site (CRR). The CRR of soil in the site depends on the period of vibration (i.e., this is similar to the loading cycles number), the effective overburden stress on the soil (K_σ and σ'_{vc}), and the existence of slop in the ground (i.e., K_α) [1].

There are many factors that can largely affect on the liquefaction-induced deformations in a soil:

- a) Soil characteristics such as fines content in the soil, relative density, degree of saturation, gradation, age of the soil, cementation, previous stress and strain history, depositional environment, and K_o .
- b) Characteristics of the ground motion that resulting from the earthquake such as period of shaking, level of shaking, and frequency content of shaking.
- c) The stratigraphy and topography of the site, that can affect the site response, void redistribution, production and spreading of excess pore water pressure in the soil, and deformation types.
- d) Other complicating phenomena, like the laterally spreading mass may be partially constrained by the neighbouring steady soils (three-dimensional effects); the soil cracks that allow water pressures to escape and breakdown of water films that may have created under layers of lower permeability; and the impact of the foundations and embedded structures.

1.1 MITIGATION OF SOIL LIQUEFACTION

The successful liquefaction mitigation design requires an overall understanding for the possible impact of liquefaction on the structure, the hazards of liquefaction, performance targets, and the availability of materials and methods for construction. The eventual chosen of the mitigation plan relies on the risk management approach and can range from the heuristic assessment to the more formal risk assessment.

To assess the suitability of a ground improvement method for a certain site, it is necessary to:

- a) Realizing the fundamental mechanisms of the soil improvement so the engineering design steps, and the quality monitoring or verification standards are sensible.
- b) Realizing the constructability cases so that the chosen method or procedure is constructible and affordable.

1.1.1 Ground Improvement Methods

1.1.1.1 Vibro-replacement method

A vibrating probe frequently penetrates the soil and enhances the liquefiable soil deposits through increase the soil density, increase the soil lateral stress, improve the soil using stiffer columns of fill material (reinforcing the soil), and provide increase in the drainage of the excess pore pressures from the soils, as in Figure 1.5 [1].

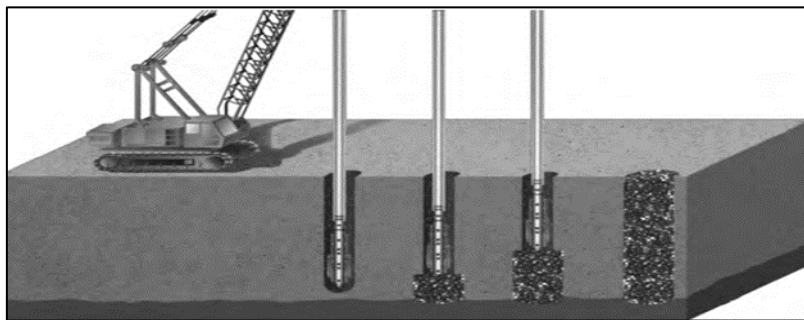


Figure 1.5: Vibro-replacement method [1].

1.1.1.2 Deep dynamic compaction method

A crane frequently drops a big mass weighing up to 33 tons from height up to 30 m onto the ground surface, creates a hole whose depth relies on the surface soil stiffness, and spreads dynamic stress waves across the subsurface soils. These dynamic stress waves cause liquefying or at least generating high excess pore water pressures in the soils under the impact point, and the dissipation of these excess pore water pressures increases the density and the lateral stress of the soil, as in figure 1.6 [1].

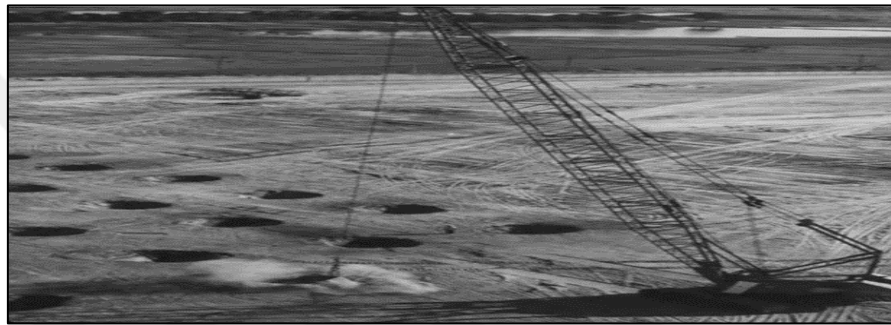


Figure 1.6: Deep Dynamic Compaction method [1].

1.1.1.3 Compaction grouting method

Compaction grouting includes injecting thick grout such as mortar that displaces the neighbourhood soil, instead of penetrates the soil. Compaction grouting can enhance the liquefiable soils through increase the soil density, increase the soil lateral stress, and increase the soil reinforcement, as in figure 1.7 [1].

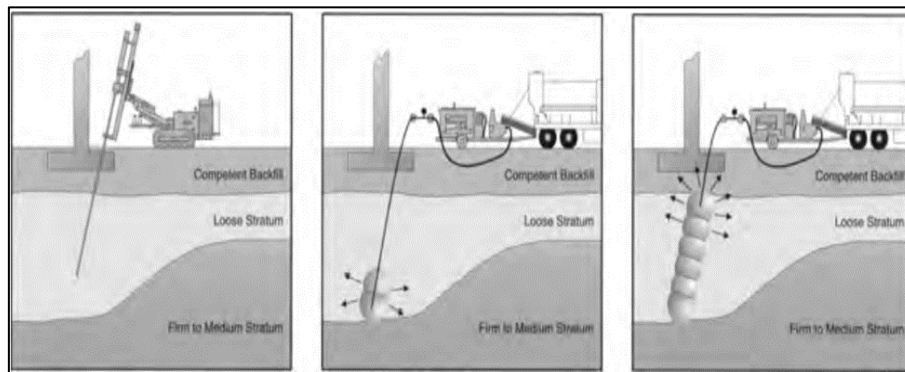


Figure 1.7: Compaction Grouting method [1].

1.1.1.4 Deep soil mixing method

Deep soil mixing includes the soil mixing, and sometimes partial replacement of the soils with another material such as grout or cementitious materials. Deep soil mixing contains different mixing augers or groups of augers. the augers penetrate the soil and then the partial replacement occur across the auger stems by injecting the grout or cementitious materials, and then the augers keep revolving and combine the injected material with the main soils. this method can be used to build underground shear walls by interfering the soil cement columns. the underground shear walls minimize the earthquake-induced shear strains, because these shear walls are stiffer in shear than the surrounded soil, and these shear walls catch a larger portion of the horizontal inertial forces when the shaking occurs. Figure 1.8 shows the process of the deep soil mixing method [1].

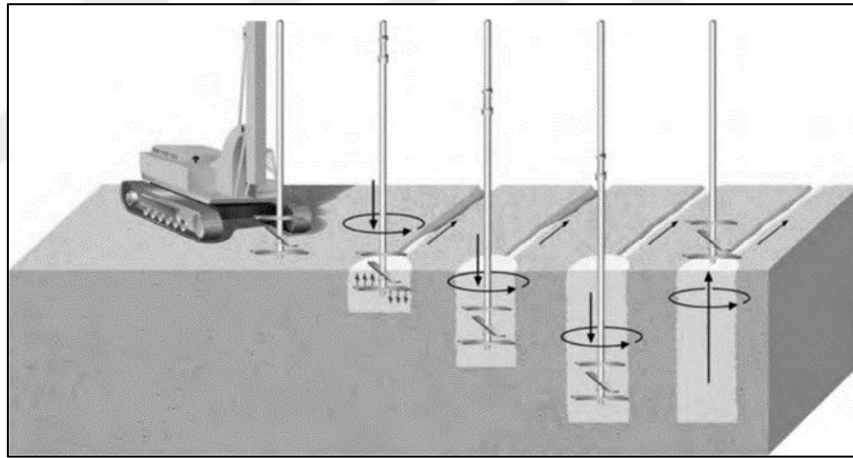


Figure 1.8: Process of the deep soil mixing method [1].

1.1.1.5 Jet grouting method

Jet grouting includes the soil mixing and the soil partial replacing with another material such as cementitious grouts, but the mixing here is achieved through high-pressure jets of water, air, and grout. Jet grouting can produce. this method can be used to build the underground shear walls by interfering soil-cement columns. Figure 1.9 shows the process of the jet grouting method [1].

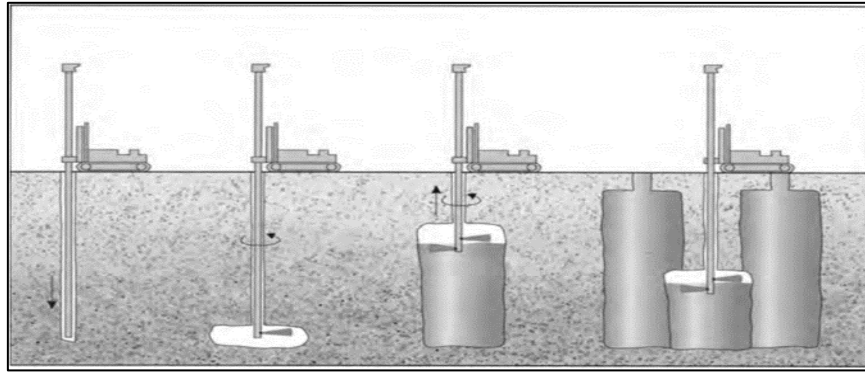


Figure 1.9: Process of the jet grouting method [1].

1.1.1.6 Permeation grouting method

Permeation grouting includes injecting a grout material, this material permeates the pore void spaces of the neighbourhood soil before the grout starts to harden. This method enhances the soil through cementing the soil particles with each other and through filling the pore void spaces (thus decreases the possibility for shrinking under cyclic shearing). Figure 1.10 shows the permeation grouting method [1].



Figure 1.10: The permeation grouting method [1].

1.1.1.7 Explosive compaction method

Explosive compaction or blasting includes detonating charges placed at different depths at the boreholes in the site. The blasts spread dynamic shear stresses across the soil, these stresses lead to liquefy the soils. The discharge of the excess pore water pressures leads to densify the soil, with related soil surface settlement, and sometimes this leads to create the sand boils as in Figure 1.11 [1].



Figure 1.11: The sand boils that caused by the discharge of the excess pore water pressures through the explosive compaction or blasting method [1].

1.1.1.8 Removal and replacement method

The direct method for enhancing the liquefiable loose soil is excavating this soil and then exchange it with a consolidated soil [1].

1.1.1.9 Drainage method

Drainage rises the dissipation averages of the excess pore water pressures that are created through shaking, or it can adjust the dissipation manners, and the drains rise the average of pore water pressure dissipation by quantities that are adequate to keep the excess pore pressures at the allowable low levels during the shaking, and the drainage techniques include unique vertical drain segments built in a closely spaced net, the drain segments usually contain coarse aggregates, and the drainage method is restricted to high-permeability soils. Figure 1.12 shows the grid of gravel drains to mitigate the liquefaction of the soil [1].

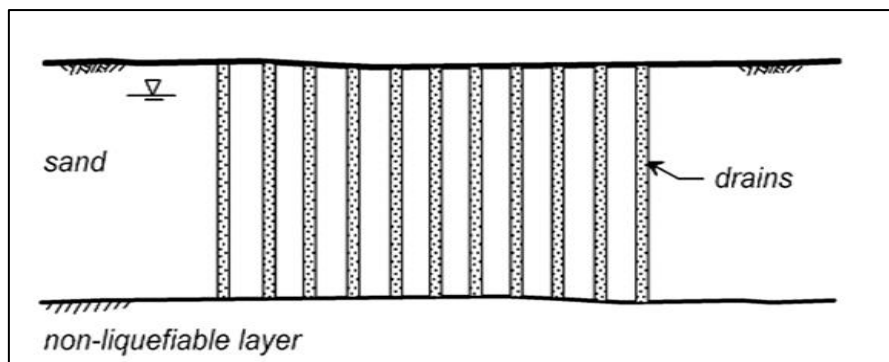


Figure 1.12: Grid of gravel drains to mitigate the liquefaction of the soil [1].

1.2 MITIGATION OF LIQUEFACTION INDUCED UPLIFTS OF THE UNDERGROUND STRUCTURE, ESPECIALLY TUNNELS

The underground structures like subways, and highway tunnels that shallowly buried in liquefiable loose sand are exposed to liquefaction-induced uplifts of these structure and that can be a disaster and goes with the ordinary life to a standstill. So, there are a need to solve this problem by different methods to mitigate the liquefaction-induced uplifts of the underground structures, especially tunnels. So, the good perception of the uplift process and the mitigation of these uplifting are substantial criteria must be considered when building a tunnel in a liquefiable soil.

The uplifting is one of the dangerous appearances of tunnel's instability through the time of shaking. The principal factors of this uplifting are the liquefaction of the soil with the large depression of the soil shear modulus under the tunnel.

The decreasing in the effective stress of the soil that leads to soil liquefaction, and the underground structures will be uplifted during the soil liquefaction by the soil's lateral deformation at the base of the structure, and a hydraulic gradient occurs in the region under the structure due the exist of the structure, that leads the pore fluid to go to the base of the underground structure then a further uplifting will be occurs to the structure, and the structure takes the final position after the soil reconsolidation through the dissipation of the pore fluid [6].

The existence of the underground structure minimizes the overburden stress in the soil under the structure compared to the neighbouring soil, and this causes to a big hydraulic gradient among these two regions which causes to the transmission of the pore fluid to the base of the underground structure [7].

Figure 1.13 and Figure 1.14 illustrate the transmission of the pore fluid and the uplifting mechanism at the underground structure.

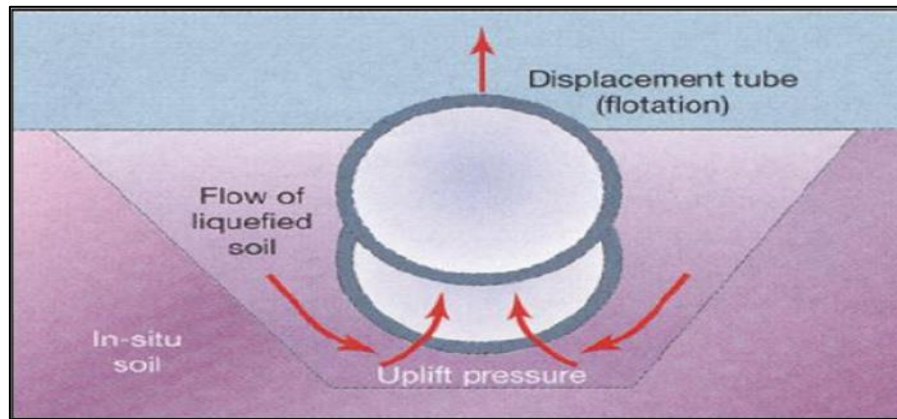


Figure 1.13: The uplifting mechanism at the underground structure [12].

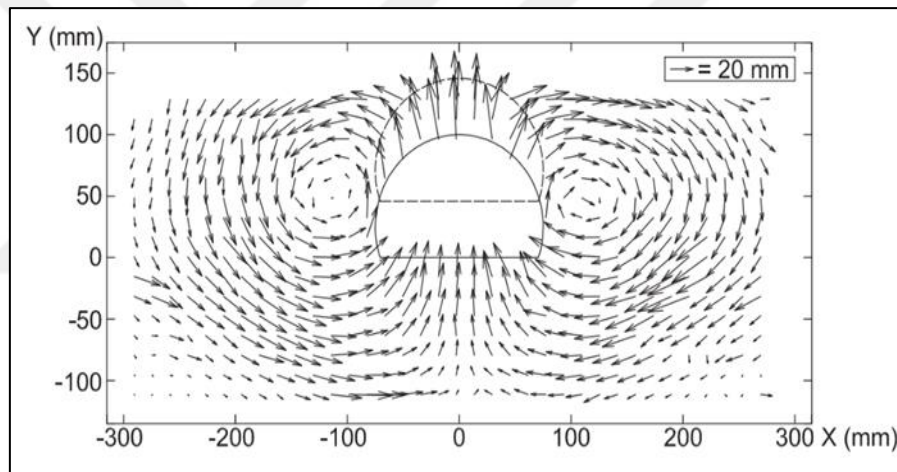


Figure 1.14: The transmission of the pore fluid to the bottom of the underground structure [13].

There are various remediation procedures to avoid liquefaction and to keep the tunnel in a safe side:

- a) Reduce the excess pore water positive pressure beneath the tunnel, this called the soil solidification or the soil densification, by making the groundwater table at a low level and far away from the tunnel, or by dissipation the excess pore water pressure using drain gravel around the tunnel or using a clay soil around the tunnel.
- b) Rise the vertical effective stresses beneath the structures through increase the unit weight of the tunnel or by increase the depth of the tunnel.

- c) Changing the geometry of the tunnel by increase the unit weight of this tunnel, and the unit weight can be increased like using a high thickness for the tunnel's walls.
- d) Resisting the earthquake's shear load by putting members around the tunnel such as sheet piles.

1.3 THE PURPOSE OF THIS THESIS

The purpose of this thesis is trying to simulate the mitigation of the liquefaction-induced uplifts in tunnels, numerically, because this is one of the important topics that must be researched, and there is not much research that talks about this topic. There is some research on the topic using laboratory experiments, but there is little research on that using numerical models, and it was done mostly by using FLAC. But in this thesis, a numerical study on mitigation the liquefaction – induced uplifts in tunnels will conduct by using PLAXIS 2D software.

When liquefaction occurs in the soil, it produces many disasters for the infrastructures and facilities, especially tunnels. This thesis will address this issue and will try to mitigate the uplifting in the tunnels, by making models in PLAXIS 2D software by including mitigation techniques in these models, and then making comparisons between the results from these models to find if these techniques are successful or not.

2. GENERAL INFORMATION

2.1 STUDIES ON THE SOIL LIQUEFACTION IN GENERAL

Karray et al., (2015) applied the approach that based on the energy to determine r_u value from the relationship of stress-strain hysteresis as in the formula 2.1:

$$r_u = \alpha_1 (W_s^{0.5}/a)^{\beta_1} \quad (2.1)$$

where (W_s) is the dissipated energy per unit volume of soil split on the initial effective confining pressure which can be found by calculate the integrating area inside the hysteresis loops of the stress-strain relationship, (a) is a inconstant parameter rely on type of the soil , (α_1) and (β_1) are constant parameters rely on soil characteristics, and these constant parameters can be evaluated in the laboratory through the cyclic loading experimental tests such as the triaxial simple shear tests.

The formation of a pore pressure increases quickly in the initial cycles of loading, and then increases slowly until the pore water pressure becomes equal to the applied confining pressure at the soil, and the soil liquefaction occurs at the point where the pore pressure equals the confining pressure, and after this point where the liquefaction occurs there will be no increase in pore water pressure as in Figure 2.1 [2].

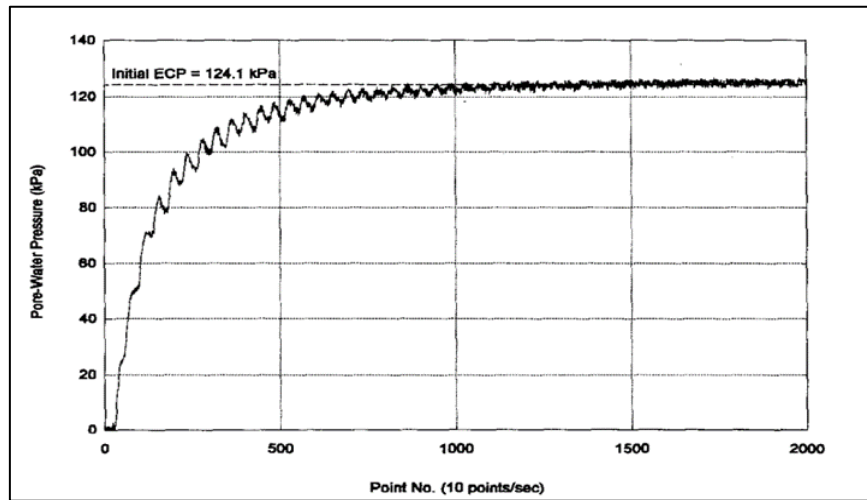


Figure 2.1: The variation in the formation of pore water pressure in torsional shear test [2].

At the beginning of shaking, the pore water pressure increases and after that remains about a constant value when it reaches the maximum, that was noticed by many researchers such as (Azadi and Hosseini (2010)); (Madabhushi and Madabhushi (2015)).

The torsional resistance (shear stress) of the soil decays and reaches to a roughly constant value after occurrence the liquefaction in the soil, and the inherent friction prevents the torsional resistance from being completely zero as in Figure 2.2 [2].

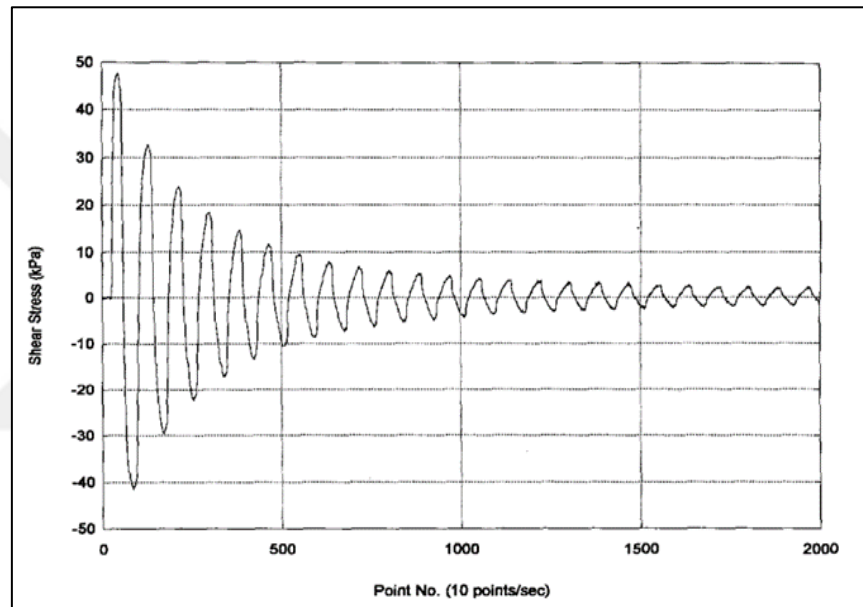


Figure 2.2: The decaying of the torsional resistance (Shear Stress) in torsional shear test [2].

The shear modulus of the soil decreases as the amount of pore water pressure approaches toward the value of effective confining pressure in the soil, and the shear modulus falls to an extremely low value (about zero) at the end, and the soil with a higher relative density has a higher shear modulus as in Figure 2.3, so liquefying a soil with a higher relative density requires a higher amount of energy per unit volume, and the soil at a higher shear-strain amplitude has a lower shear modulus as in Figure 2.4, there is no obvious tendency to the amount of energy per unit volume at a various shear strain amplitude, and the shear modulus is observed higher in a soil under a higher effective confining pressure as in Figure 2.5, so a high initial effective confining pressure in the soil causes dissipation a high amounts of energy per unit volume [2].

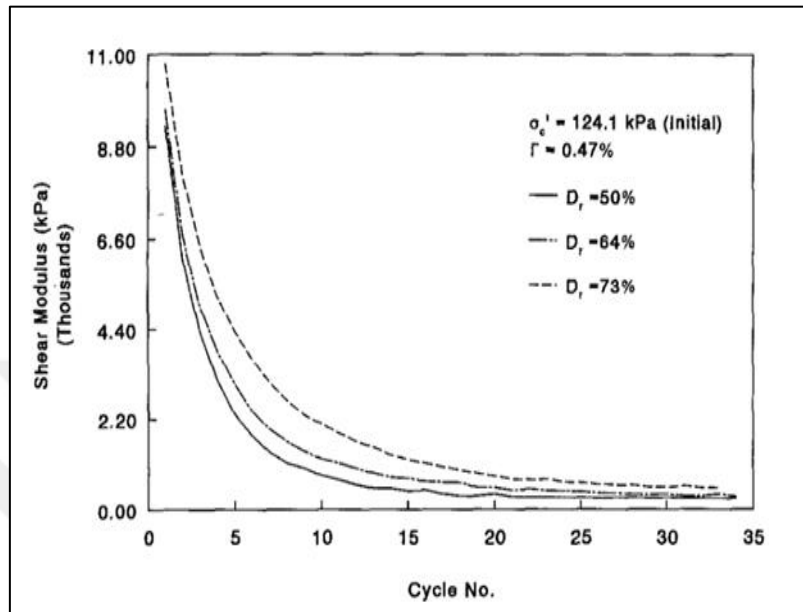


Figure 2.3: The variation in the shear modulus with the number of torsional cycles under different relative density of the soil [2].

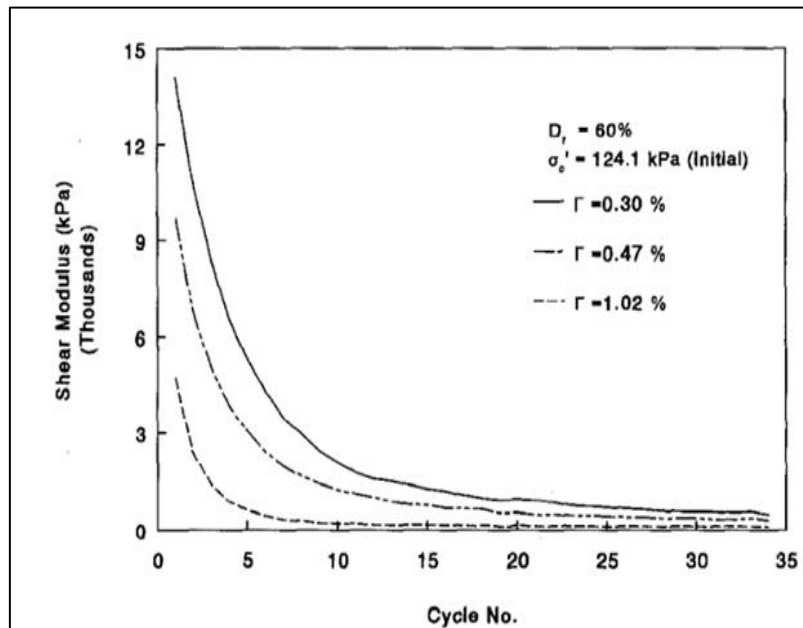


Figure 2.4: The variation in the shear modulus with the number of torsional cycles under different shear strain amplitude [2].

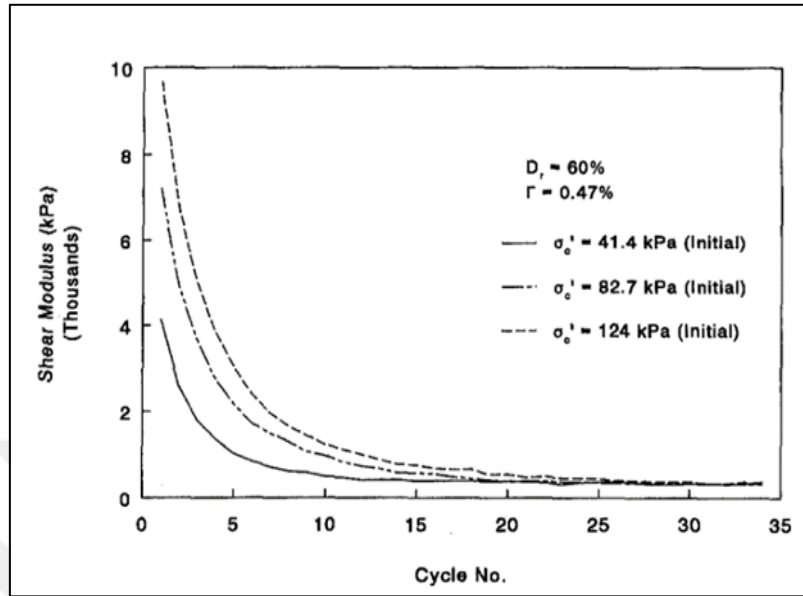


Figure 2.5: The variation in the shear modulus with the number of torsional cycles under different effective confining pressure [2].

The laboratory shows a degree of uncertainty for evaluation of the profile layout of the soil, the input motion and the properties (such as shear modulus, density and shear wave velocity) [3].

Evidence of liquefaction usually shows that the liquefaction occurring through a depth of less than 15 m from the surface, because the youngest or most recent deposits occur at a shallower depth, and for that reason these deposits are most prone to liquefaction, but if these deposits become old, the deposits become more resistant [1].

The behavior of liquefiable soils under seismic load can be obtained by the dynamic analysis of soil-water couple in synchronism with the constitutive relationships of cyclic elastoplastic of the soils, by depending on the computational geomechanics (Zienkiewicz et al., (1999)).

2.2 STUDIES ON LIQUEFACTION MITIGATION

Theoretically, liquefaction mitigation philosophies can be summarized in two fundamental concepts which are prevention the build-up of excess pore pressure and reduction the deformations induced from liquefaction [4].

In general, the typical manners of the seismic response are precisely reproduced by using numerical simulation, and the suggested numerical method should therefore take the essential aspects of the investigated problems and makes results for design objectives [4].

There are some studies that conducted to mitigate the liquefaction, including controlling the pore pressure build-up (Seed and Booker (1976)), increase the stiffness of material (Gallagher and Mitchell (2002)); (Ozener et al., (2015)), increase the soil confining pressure (Funahara et al., (2015)), and increase the soil density (Hatanaka et al., (1987)); (Olarate et al., (2018)).

Ground improvement techniques and structural treatment measures have been implemented to prevent or reduce the soil deformation and related damage caused by liquefaction, such as cementing, drainage, reinforcement, and compaction [4].

Although there is very limited research on the successfulness of soil improvement techniques for mitigate the liquefaction (Zheng et al., (1996)); (Adalier et al., (2004)), a lot of the designs for mitigate the liquefaction have been depended on simplified analysis or conceptual understanding, and there is a large need for dependable numerical methods to evaluate the mitigation of soil deformations caused by liquefaction to allowable levels for structures [4].

The essential reason that leads to liquefy the soil is the formation of excess pore water pressure in the soil during the cyclic loading in undrained conditions, and there are numerical and experimental research on the resistance of partially saturated sands against liquefaction [5].

There are some techniques aimed to countermeasure of liquefaction from occurring by rise the pore fluid compressibility through generation gas in the pores of the fully saturated sand, and this process converts the sand from fully saturated state to partially saturated state, and the studies have proven that the partially saturated sand reaches liquefaction after a lot of cycles more than the fully saturated sand [5].

Chemical materials have been added to the saturated soil to generation gas in the soil and to increase the pore fluid compressibility by some researchers such as (Yegian et al., (2007)), whereas other researchers inject the air into the saturated soil to convert it to partially saturated soil to resist the soil liquefaction such as (Marasini and Okamura (2015a)).

Hatanaka and Masuda (2009) noticed that the decrease in the degree of saturation (S_r) or the B -value can significantly lead to increase the resistance of soil liquefaction known as the ratio of cyclic stress CSR.

The developed peak average stress ratio or the strength versus the soil liquefaction is decreased by rise the maximum excess pore pressure ratio ($r_{u,max}$), and increase the weight of the foundation leads to reduce the excess pore water pressure ratio through increase the soil effective stress, so the liquefaction doesn't appear in the saturated soils beneath the heavy foundations (Marasini and Okamura (2015a,b)).

2.3 STUDIES ON MITIGATION OF LIQUEFACTION INDUCED UPLIFTS OF THE UNDERGROUND STRUCTURE, ESPECIALLY TUNNELS

When the buried structure is built in a liquefiable soil, the lower submerged total density for this structure compared to the neighboring soil, in conjunction with the buoyancy force, leads to uplift this structure (Zhuang et al., (2016)), (Otsubo et al., (2016)), (Zhang and Chian (2019)), (Haiyang et al., (2019)).

Zhuang et al., (2016) found that when the partial liquefaction occurs for the soil layer neighboring the buried structure, the cumulative deformations of the subway station are much larger than the deformations in the non-liquefied soil or in the completely liquefied soil, and that because of the great lateral unrecoverable deformation of the soils which prone to partial liquefaction.

Otsubo et al., (2016) conducted some shaking table tests to confirm the effectiveness of the suggested mitigation methods like the grout injection partially, the mechanical constraints, the drainpipes installation, then they conclude that the suggested mitigation methods give the acceptable results in mitigation the liquefaction - induced the deterioration to the underground structures such as sewage pipelines.

The resistance of friction between the soil and the underground structure interface was ignored in a lot of studies, because the magnitude of this friction is small after the formation of the excess pore pressure. Zhang and Chian (2019) determined the friction between the manhole's interface

and the soil, and they found it performs a considerable portion of residual resistance that leads to resist the uplifting in manholes even with the low magnitude comparing to the other forces like seepage force, buoyancy and manhole's self-weight at steady condition, so they conducted some centrifuge tests through using various sidewall friction from sandpaper to prove the contribution of the friction in resistance the uplifting of manholes during the liquefaction of soil, and they concluded that the uplifting was decreased remarkably through the rough sidewall of the manhole.

In the past, numerous cases have been reported on the liquefaction-induced uplift of the underground structures, the beginning studies on this problem have been conducted by (Koseki et al., (1997)), and in their study they conducted several shake tables tests on different underground structures. Then several studies have been conducted on different underground structures involving the lifeline structures through two approaches, the first one is numerical approach by (Azadi and Hosseini (2010)); (Chian et al., (2014)); (Madabhushi and Madabhushi (2015)), and the second one is experimental approach by (Orense et al., (2003)); (Tobita et al., (2011)); (Otsubo et al., (2016)). All the studies noticed that the liquefaction-induced uplift of structure occurs at most due to the following three phenomena: the soil lateral deformation, the movement of pore fluid and the soil reconsolidation.

A lot of these studies confirmed that the main cause of liquefaction-induced uplift of the underground structures is the build-up of pore water pressure at the bottom of the structure and there is a significant pore pressure leads the structure to get uplift (Orense et al., (2003)). The buried structure uplifts as soon as the uplifting force overruns the resisting force (Koseki et al., (1997)).

Koseki et al., (1997) conducted different model tests to check the behavior of the tunnel during the time of the liquefaction in the soil, and they found that the uplift of buried structure was happened due to the instantaneous deformation of the neighboring subsoils and the pore fluid transition to the bottom of the structure, whereas the liquefied soil reconsolidation leads to settle the structure, and in their tests noticed that there are some various effects of the structure type on the behavior of the uplifting.

Mahmoud et al., (2015) provide an information about the failure processes of the buried structures that resulting from the earthquake loads, and they found that the ground movement during the seismic loading leads to a critical damage to the underground structures based on the data from the experiments , the investigations after the earthquake and the numerical modelling of the buried structures that have been done using a computer code such as FLAC, and this critical failure is performed in two principal kinds of failure has been happened, the first one is the stress-strain failure of the buried structure due to the increasing in stress and deformation that resulting from the soil movement during the seismic loading, and the second one is the status of the stress failure of the soil causes an uplift of the buried structures and breakdown of the neighboring soil.

Mahmoud et al., (2015) concluded that the rise in the CSR beneath the structure with reduce the CRR at the middle third of soil beneath the structure leads to rise the possibility of occurring the liquefaction in the soil.

Mahmoud and Kenawi (2021) showed the behavior of tunnels under the seismic loading and the uplift of the tunnel sections at different seismic motions, in different soil stiffnesses, with various soil thicknesses, with various levels of water table and with various depths of the structure, and they analyzed the ground response of a tunnel by using the computer code FLAC that relying on the energy-based model, and they concluded that the rise in the level of water table, the reduce in the depth of the structure , the decrease in shear wave velocity, the increase in the thickness of the soil layer , and the increase in the PGA of earthquake cause an increase in the probability of the liquefaction in the soil , so the tunnel will be uplift.

Figure 2.6 shows that the increase of the peak ground acceleration (PGA) of the earthquake leads to increase the excess pore water pressure, and this leads to increase the uplifting in the tunnel as in Figure 2.7, and the increase of the buried depth of the structure leads to increase the excess pore water pressure as in Figure 2.8, and that because the soil effective stress beneath the structure increases with the increase in the depth of the structure, but the uplifting in the underground structure increases with decrease the buried depth of the structure as in Figure 2.9,

and that because when the structure is at a shallow depth that means a low soil weight above the structure and in this case the structure uplifts easily [9].

The speed of the shear wave (V_s) is in positive relationship with the soil's shear modulus or the rigidity of the soil, so it was seen that any decrease in the soil's shear stiffness (the speed of the shear wave V_s) leads to increase the pore water pressure as in Figure 2.10, and any decrease in the V_s of the soil leads to increase the uplifting in the buried structure as in Figure 2.11, and the uplifting in the buried structure decreases with the increase in the depth of water table level, and it was seen that the uplifting in the buried structure is approximately zero if the ground water level is under the base of the structure as in Figure 2.12 [9].

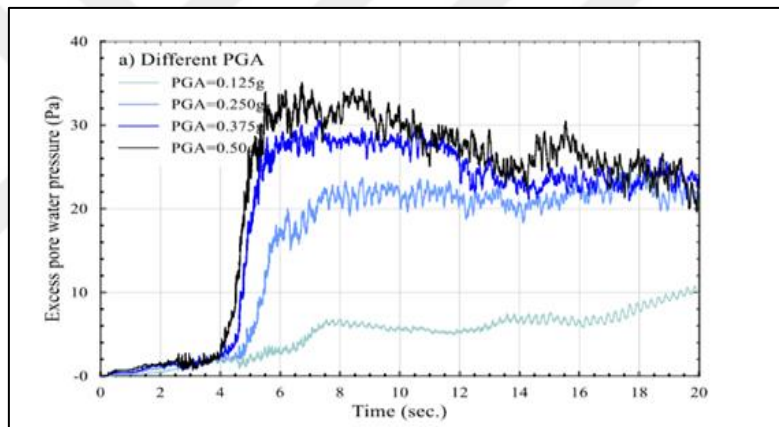


Figure 2.6: The increase of the peak ground acceleration (PGA) of the earthquake leads to increase the excess pore water pressure [9].

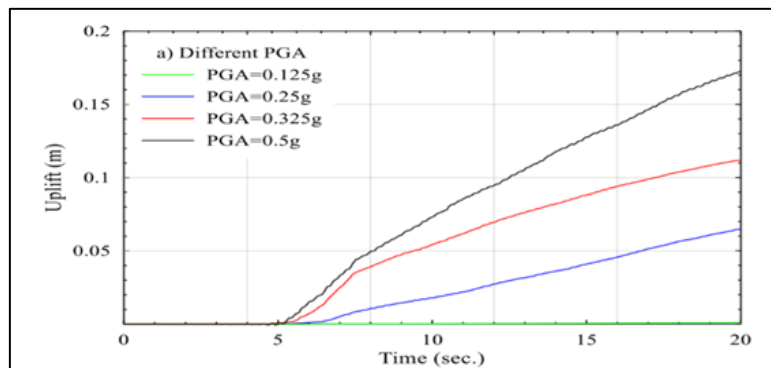


Figure 2.7: The increase of the peak ground acceleration (PGA) of the earthquake leads to increase the uplifting in the tunnel [9].

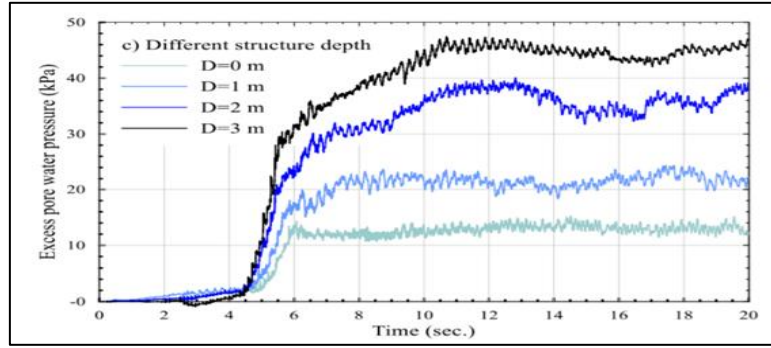


Figure 2.8: The increase of the buried depth of the structure leads to increase the excess pore water pressure [9].

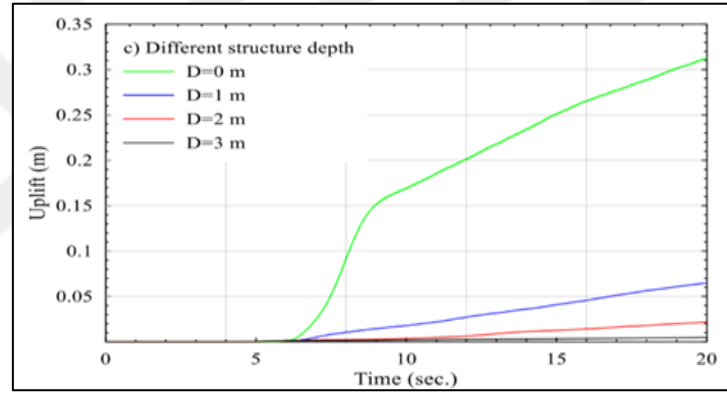


Figure 2.9: The uplifting in the underground structure increases with decrease the buried depth of the structure [9].

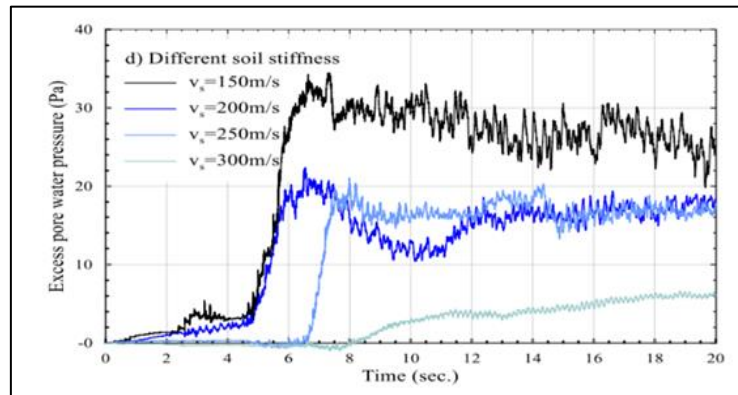


Figure 2.10: Any decrease in the soil's shear stiffness (the speed of the shear wave V_s) leads to increase the excess pore water pressure [9].

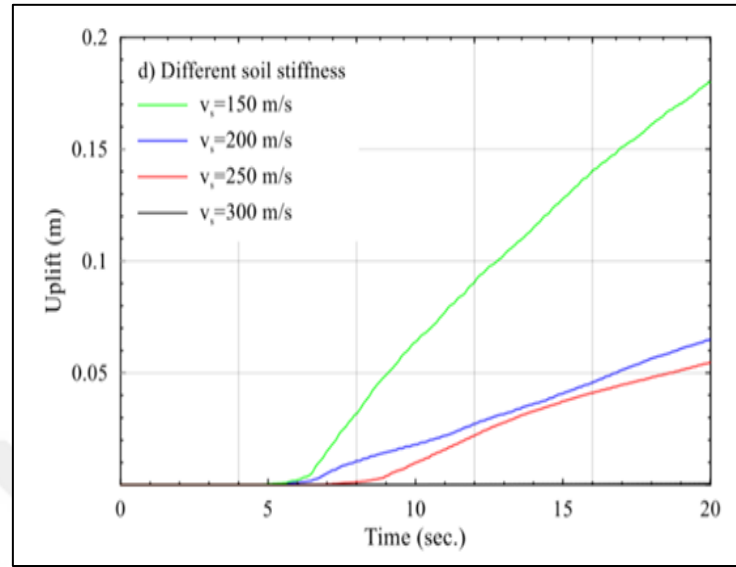


Figure 2.11: Any decrease in the speed of the shear wave (V_s) of the soil leads to increase the uplifting in the buried structure [9].

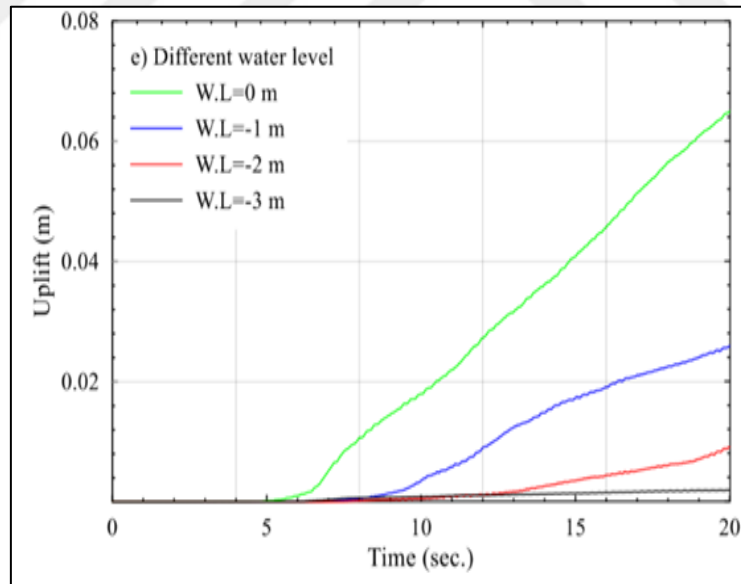


Figure 2.12: The uplifting in the buried structure decreases with the increase in the depth of water table level [9].

Chian et al., (2014) conducted numerical modelling with the centrifuge tests on the buried structure such as circular tunnel to realize the increase in the pore pressure, and the numerical modelling was done through using the computer software FLAC2D, and they investigate the

effect of the buried depth of the underground structure and the seismic input acceleration on the uplift response of the structure, and they noticed that the experimental and the numerical analyses gives similar results in the excess pore water pressure of the soil and the uplift response of the buried structure, and also the experimental and the numerical analyses noticed that there are need of high seismic input accelerations and high excess pore water pressures for the development of the structure's uplift.

The uplift of the buried structure increases when the magnitude of the earthquake increases, and the uplift of the structure stopped when the shaking stopped, and the shallower underground structure have a higher uplift due to the smaller shear resistance and smaller weight given by the soil which exist above the structure [35].

Sasaki and Taniguchi (1982) found the efficacy of gravel drains at the bottom of the buried structure in minimizing the uplift of the structure, and they focused on a partially buried structure in their study.

Adalier et al., (2003) noticed that the gravel drains at both sides of the buried structure is appropriate mitigation method to minimize the uplift of the underground structure through accelerating the drainage around the structure and make some confinement of the buried structure.

Otsubo et al., (2016) studied the influence of drainpipes to cease the uplift of the underground structure during the liquefaction of the soil, and they noticed a further decrease in the uplift of the structure when the exist of a surface layer from gravel soil.

It is necessary to find mitigation methods to minimize the uplift of these underground structures, and it is important to apply these methods in the field without any difficulties. however, the numerical research and studies about the earthquake response of a buried structure and its mitigation technique are limited [6].

Sudevan et al., (2021) conducted a numerical study using a finite difference program FLAC3D, in their research they studied the mitigation of the uplift that occurs at the buried structure

through accelerating the dissipation of the pore water pressure from the neighbourhood soil around the structure by change the soil with another sand which have a more permeability.

This numerical modelling by (Sudevan et al., (2021)) about the uplift of a buried structure that set inside a saturated soil was done through using FINN-Byrne model and its results were compared with the results of a centrifuge test that done by (Chian and Madabhushi (2013)), the comparison was about the excess pore water pressure and the maximum uplift of the buried structure, and the Figure 2.13 and Figure 2.14 illustrate this comparison, and they found that the excess pore water pressure response that obtained from their numerical study is approximately matching with the response obtained by the experimental studies that done by (Chian and Madabhushi (2013)), and the uplift response of the buried structure at a saturated sand that have a relative density RD 52% is approximately matching with the uplift obtained from the experimental studies that done by (Chian and Madabhushi (2013)).

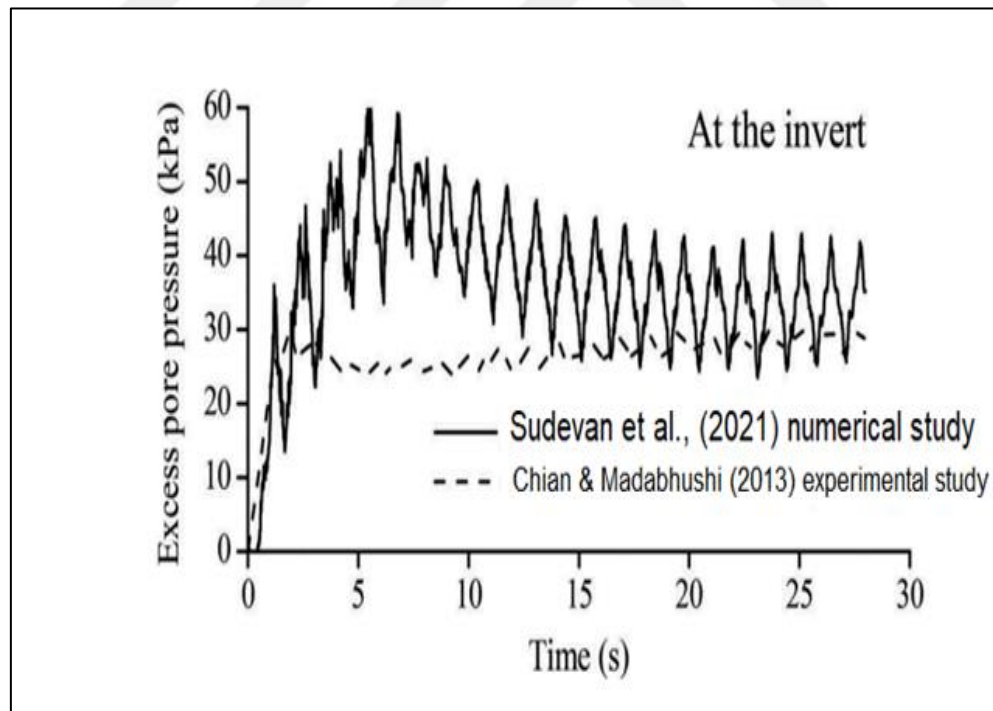


Figure 2.13: The excess pore pressure at the invert of the buried structure by the numerical study and the centrifuge test [7].

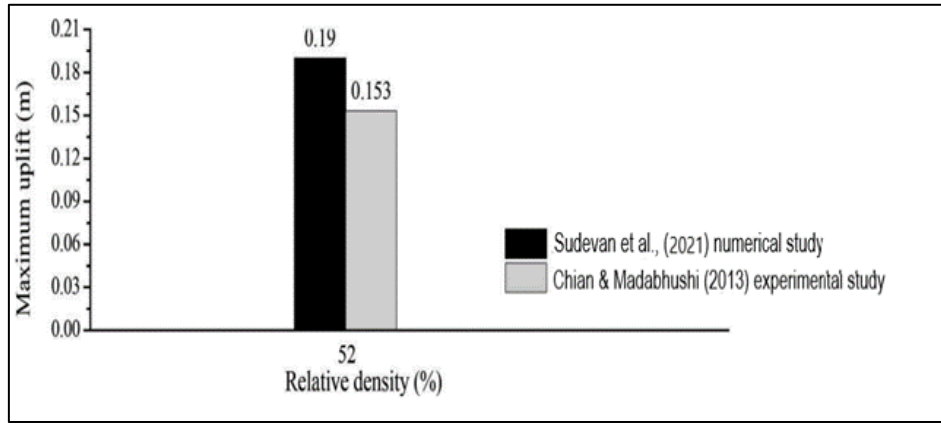


Figure 2.14: The maximum uplift of the buried structure obtained by the numerical study and the centrifuge test [7].

Sudevan et al., (2021) studied four cases of mitigation the uplift of the structure during the soil liquefaction by replacing the soil around the structure with another soil that have a high permeability, it was noticed that the lowering in the uplift of the buried structure is about 85% when replacing the soil around the structure compared to the case without any replacement, and they noticed that the case of replacing the soil around the structure in a width of $2D$ led to more reduction in the uplift of the structure compared to the case of replacing the soil in a width D (D represent the diameter of the structure), and the Figure 2.15 and Figure 2.16 illustrate the mitigation cases and the result of these case in reducing the uplift of the structure.

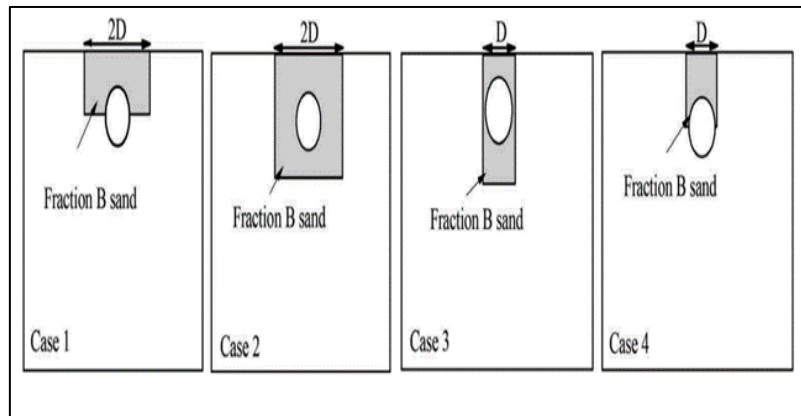


Figure 2.15: The four replacement cases of mitigation the uplift of the structure that used in (Sudevan et al., (2021)) study.

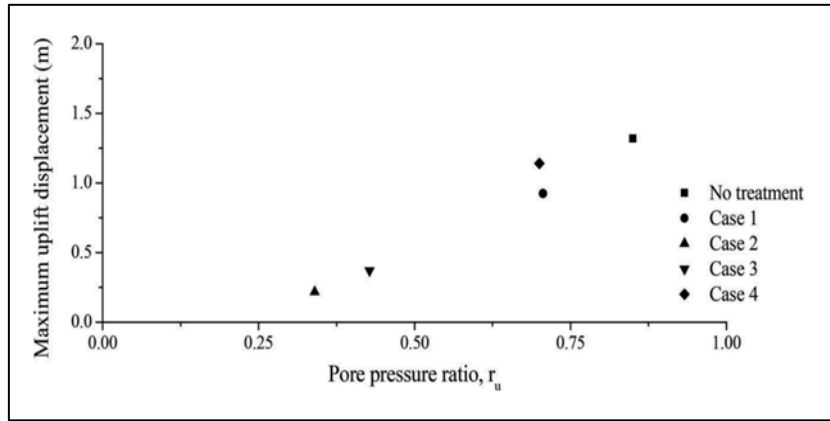


Figure 2.16: The maximum uplift displacement in each replacement cases of mitigation the uplift of the structure that used in (Sudevan et al., (2021)) study.

A few researchers talked about the mitigation techniques which use the accelerated drain of the excess pore water pressure around the buried structure. Sasaki and Taniguchi (1982) conducted a shake table test and found the efficiency in lowering the displacement and the uplift of a partially buried structure during the soil liquefaction through using gravel drains beneath the buried structure.

Orense et al., (2003) and Otsubo et al., (2016) conducted study to try using drain material from the different wastes, and they proposed that the crushed stones are appropriate material and can be selected to drain the pore water pressure from the surround of the structure.

To study the response of an underground tunnel during the soil liquefaction, a fluid flow analysis is conducted using a three-dimensional finite difference formulation FLAC3D, because the FLAC3D can be used for large strains and displacement [6].

The input motion characteristics effect on the uplifting of the tunnel during the soil liquefaction, and the technique of accelerated drainage of the pore water pressure at the surround of the buried structure is used to mitigate the uplift of the tunnel during the soil liquefaction [6].

Mahmoud et al., (2020) studied the behaviour of buried structures in sandy soil during the seismic load through the finite differences program (FLAC) with focusing on the failures of the structures during the actual earthquakes, and in their study they combined the numerical code

as a constitutive model of the soil with a new liquefaction model (energy-based approach) which illustrates the seismic behaviour of the materials and evaluates the formation of the pore water pressure in the soil, so they provided some details about the impacts of the new combined mitigation technique on the performance of the structure.

The numerical modelling showed that gravel drain is a successful way to dissipate the excess pore water pressure under the buried structure, and also increase the structure's depth and putting an impermeable layer beneath the buried structure would raise the vertical effective stress, and that would reduce the capability of excess pore pressure to uplift the buried structure, and it was noticed that gathering the gravel drains around the buried structure in addition to an impermeable layer under it would minimize the uplift of the buried structure more than any other method which uses a one technique [8].

So, using two or more techniques at the same time leads to various mitigation effects and more effective than a unique technique in reducing the excess pore water pressure, increase the effective vertical stress or stopping the earthquake's shear wave [8].

Sudevan et al., (2019) studied the numerical modelling of the uplift occurring of a tunnel at a saturated soil through using FINN-Byrne model, and they studied the effect of the gravel drains surrounding the tunnel in reduction the pore water pressure at the invert of the tunnel, and they found that the uplift that happens of the tunnel by a low-frequency input motion happens early unlike to what happens in a high-frequency input motion, and the highest uplift for the tunnel happens at a highest earthquake motion (highest PGA).

There is a poor understanding about the use of mitigation techniques to minimize the uplift of the underground structures. So, there is a necessity to find a mitigation technique to reduce the uplift of the buried structure, and it must apply in the field through an easy way without any difficulties. Also, most of the research about this topic are conducted by using a centrifuge test or shake table test, but there are a little of studies have been conducted by using a numerical modelling to analysis the seismic response of a buried structure and its mitigation method.

3. METHODOLOGY

This project was carried out using PLAXIS 2D software, because this program conducts finite element analysis (FEA) in the field of geotechnical engineering. Input procedures enable improved output aperients to give a detailed representation of calculation results. it's easy to handle with this software, the new users need just a few hours of training to start working with it.

By using PLAXIS 2D software, a general model of liquefiable sand (RD = 40%) with a tunnel was designed, then the model was subjected to earthquake cyclic loading such as the earthquake of Kocaeli, Kobe and Northridge, after that the uplifting of the tunnel and the excess pore water pressure have been seen, then some models were made as mitigation techniques to see if the excess pore water pressure and the uplifting of the tunnel will reduce or not.

3.1 MODELING

A layer of fully saturated sand of relative density RD 40% was defined as 120 m width and 24 m depth, and free field soil at both side of this layer as 3 m width for each side, and under all these there is a bed rock layer with depth 1 m, and the water table level at the surface. There is also general tunnel with radius 2.5 m, the centre of this tunnel located at the middle of the sand layer in the horizontal direction, and at depth 7.5 m in the vertical direction, and the water table at the ground surface, and this model didn't have any mitigation technique for the soil liquefaction, and Figure 3.1 shows the model that was used.

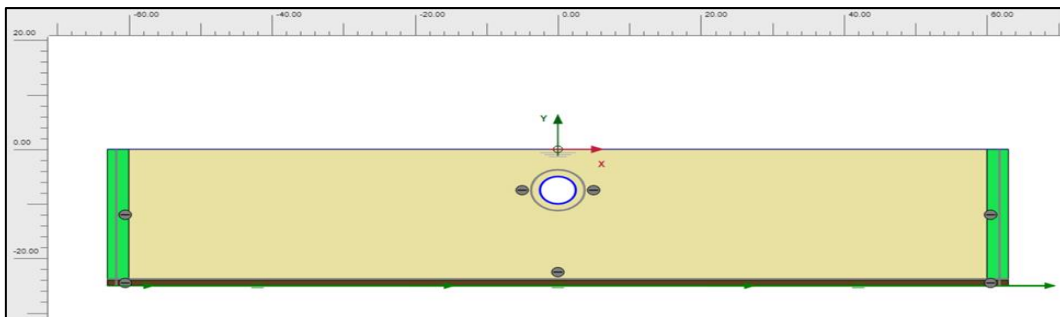


Figure 3.1: Modelling of a tunnel in a fully saturated sand of relative density RD 40% without using any mitigation techniques for the soil liquefaction.

Then other models have been designed as a mitigation technique to minimize the uplift of the tunnel that happened in the previous model. These models can be defined as the following:

- a) Changing the relative density of the sand, so the previous model of sand RD 40% was changed with other models by increasing the sand's relative density such as 55%, 65% and 75%, or by decreasing the sand's relative density such as 35%.
- b) Replacing the sand around the tunnel by higher relative density sand, and this method was done by using the sand RD 40% but replacing the sand around the tunnel by higher relative density such as 75%, and the replacing was 4.5 m from both side (right and left) of the tunnel, and 3 m from the bottom of the tunnel as in Figure 3.2.

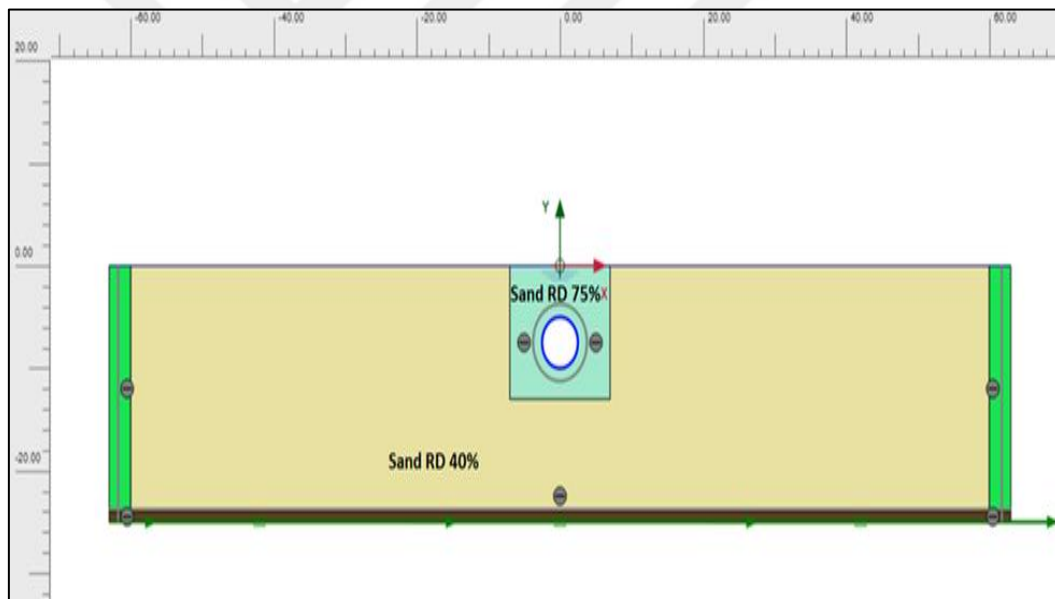


Figure 3.2: Modelling of a tunnel in a fully saturated sand of relative density RD 40% with replacing the sand around the tunnel by a sand of relative density RD 75%.

- c) Replacing the sand around the tunnel by clay soil, and this method was done by using the sand RD 40% but replacing the sand around the tunnel by clay soil, and the replacing was 4.5 m from both side (right and left) of the tunnel, and 3 m from the bottom of the tunnel as in Figure 3.3.

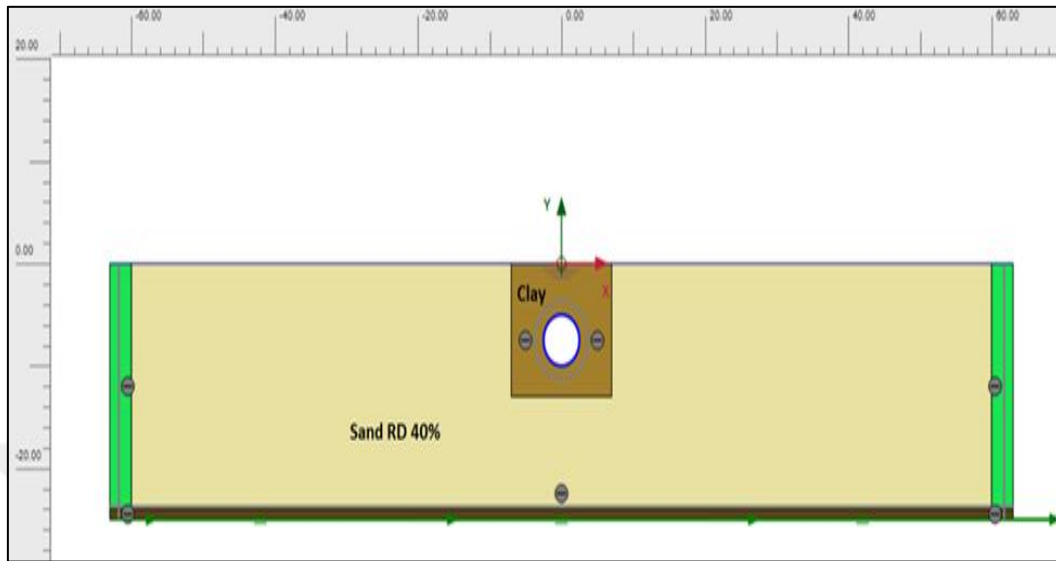


Figure 3.3: Modelling of a tunnel in a fully saturated sand of relative density RD 40% with replacing the sand around the tunnel by a clay soil.

- d) Replacing the sand around the tunnel by gravel soil, and this method was done by using the sand RD 40% but replacing the sand around the tunnel by gravel soil, and the replacing was 4.5 m from both side (right and left) of the tunnel, and 3 m from the bottom of the tunnel as in Figure 3.4.

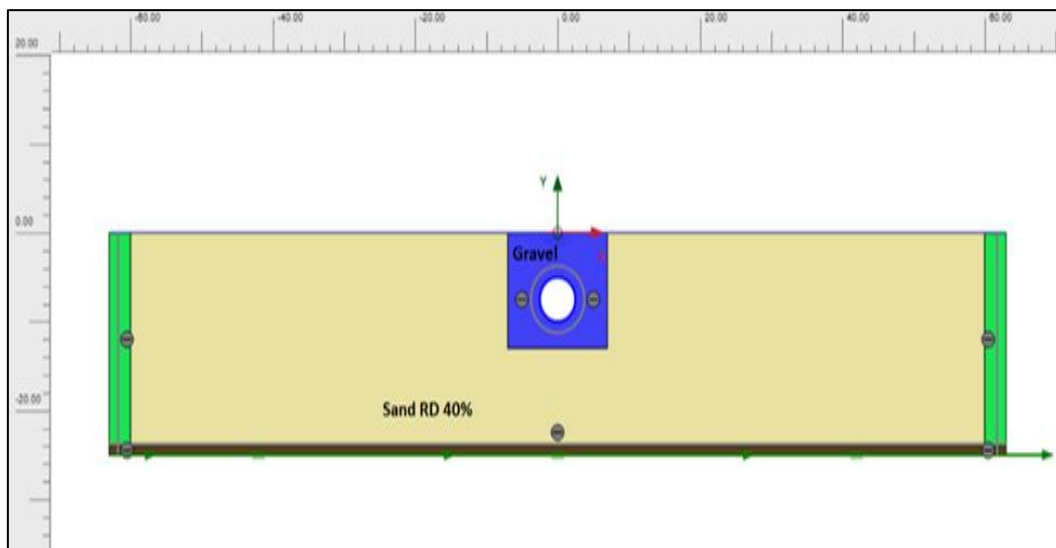


Figure 3.4: Modelling of a tunnel in a fully saturated sand of relative density RD 40% with replacing the sand around the tunnel by a gravel soil.

3.2 SOIL'S PARAMETERS

To model the sand, a material model (HS Small) was used in the statics state and a material model (UBC3D-PLM) was used in dynamics state. And to model the clay and the gravel soil, a material model (Mohr-Coulomb) was used in the statics state and a material model (HS small) was used in the dynamics state. And a material model (Linear Elastic) was used for modelling the bed rock.

Before start calculating the parameters of each model, the unit weights of sand can be calculated by using the relative density in the formulas 3.1 and 3.2 [10]:

$$\gamma_{unsat} = 15 + 4.0(RD/100) \quad (3.1)$$

$$\gamma_{sat} = 15 + 1.6(RD/100) \quad (3.2)$$

where (γ_{unsat}) [kN/m^3] is the unit weights of the sand in unsaturation state and (γ_{sat}) [kN/m^3] is the unit weights of the sand in saturation state, and RD [%] is the relative density of the sand.

3.2.1 HS Small Parameters

The HS small material model consists of four various stiffness parameters (E_{50}^{ref} , $E_{\text{oed}}^{\text{ref}}$, $E_{\text{ur}}^{\text{ref}}$, G_0^{ref}), each of those parameters giving the reference stiffness in a certain stress path for a specified reference stress level, P_{ref} [14]. Where E_{50}^{ref} [kN/m^2] is the secant stiffness in standard drained triaxial test, and $E_{\text{oed}}^{\text{ref}}$ [kN/m^2] is the tangent stiffness for primary oedometer loading, and $E_{\text{ur}}^{\text{ref}}$ [kN/m^2] is the unloading and reloading stiffness from drained triaxial test and G_0^{ref} [kN/m^2] is the reference shear modulus at very small strains such $\epsilon < 10^{-6}$.

In relation of the relative density, some formulas are suggested for calculating the reference stiffness parameters, as in the formulas from 3.3 to 3.6, taking into consideration $P_{\text{ref}} = 100 \text{ kN/m}^2$ [10]:

$$E_{50}^{ref} = 60000(RD/100) \quad (3.3)$$

$$E_{oed}^{ref} = 60000(RD/100) \quad (3.4)$$

$$E_{ur}^{ref} = 180000(RD/100) \quad (3.5)$$

$$G_0^{ref} = 60000 + 68000(RD/100) \quad (3.6)$$

There is a formula that suggested for the rate of stress dependency or the power for stress-level dependency of stiffness (m), as in the formula 3.7 [10]:

$$m = 0.7 - (RD/320) \quad (3.7)$$

The parameter ($\gamma_{0.7}$) represents the modulus decreasing curve to the cyclic shear strain level, and it is the threshold shear strain when $G_s = 0.722 (G_0)$, it can be calculated as in the formula 3.8 [10]:

$$\gamma_{0.7} = (0.7 - (RD/100)) \cdot 10^{-4} \quad (3.8)$$

The strength-related properties (e.g., ϕ' [°], ψ [°], R_f [-]) can be computed by the formulas from 3.9 to 3.11 [10]. Whereas ϕ' is the friction angle, ψ is the dilatancy angle and R_f is the failure ratio.

$$\phi' = 28 + 12.5(RD/100) \quad (3.9)$$

$$\psi = -2 + 12.5(RD/100) \quad (3.10)$$

$$R_f = 1 - (RD/800) \quad (3.11)$$

In Table 3.1 below there are the parameters of (HS Small) model for different relative densities that used to describe the sand in statics state. And in table 3.2 there are the parameter of (HS Small) model for each of the clay from and the gravel from in the dynamics state as in [47] and [46].

Table 3.1: Table of (HS Small) parameters for the sand in statics state.

RD (%)	35	40	55	65	75
Drainage type	Drained	Drained	Drained	Drained	Drained
γ_{unsat} [kN/m ³]	16.40	16.60	17.20	17.60	18.00
γ_{sat} [kN/m ³]	19.56	19.64	19.88	20.04	20.20
p^{ref} [kN/m ²]	100	100	100	100	100
E_{50}^{ref} [kN/m ²]	21000	24000	33000	39000	45000
$E_{\text{oed}}^{\text{ref}}$ [kN/m ²]	21000	24000	33000	39000	45000
$E_{\text{ur}}^{\text{ref}}$ [kN/m ²]	63000	72000	99000	117000	135000
G_0^{ref} [kN/m ²]	83800	87200	97400	104200	111000
m [-]	0.5906	0.5750	0.5281	0.4968	0.4656
$\gamma_{0.7}$ [-]	$1.65e^{-4}$	$1.60e^{-4}$	$1.45e^{-4}$	$1.35e^{-4}$	$1.25e^{-4}$
ϕ' [°]	32.375	33.000	34.875	36.125	37.375
Ψ [°]	2.375	3.000	4.875	6.125	7.375
R_f [-]	0.9562	0.9500	0.9312	0.9187	0.9062

Table 3.2: Table of (HS Small) parameters for the clay and the gravel (in the dynamics state) [47], [46].

HS small parameters	Clay	Gravel
Drainage type	Undrained (A)	Undrained (A)
γ_{unsat} [kN/m ³]	16	16
γ_{sat} [kN/m ³]	20	20

Table 3.2: Table of (HS Small) parameters for the clay and the gravel (in the dynamics state) [47], [46]
“tables continued”.

p^{ref} [kN/m ²]	100	100
E_{50}^{ref} [kN/m ²]	20000	200000
$E_{\text{oed}}^{\text{ref}}$ [kN/m ²]	25610	200000
$E_{\text{ur}}^{\text{ref}}$ [kN/m ²]	94840	600000
G_0^{ref} [kN/m ²]	270000	735000
m [-]	0.5	0.5
$\gamma_{0.7}$ [-]	0.00012	0.00012
ϕ' [°]	18	39.6
Ψ [°]	0	9.6
C'_{ref} [kN/m ²]	10	0
v'_{ur}	0.20	0.25

3.2.2 UBC3D-PLM Parameters

The model which can simulate the behaviour of liquefaction in sands and silty sands during seismic load is the UBC3D-PLM model, and this model is an effective stress elasto-plastic model [14].

The evaluation of the liquefaction possibilities in a soil can be done by conducting an analysis of a dynamic site response. To carry out of this analysis, the following steps are needed:

- Determining the geotechnical model of soil such as the soil layer distribution, the level of groundwater table, suitable dynamic boundary states and soil mechanical characteristics to describe the soil behaviour during static and cyclic load.

- b) Applying the earthquake input motion (seismic loading).
- c) Determination of the numerical model to carry out the analysis containing the suitable dynamic boundary states.

A cyclic direct simple shear test and a cyclic triaxial test are the suitable methods to bring the whole parameters for the UBC3D-PLM model. However, in a lot of cases just data from drained triaxial test (CD TxC) and in situ test (standard penetration test SPT) are obtainable. So, the UBC3D-PLM model executes a certain formulation by using the input parameters from these tests [14].

In this project the data from these tests are not available. So, the corrected SPT value $(N1)_{60}$ [-] can be calculated depending on the soil's relative density RD (in %), as in the formula 3.12 [14]:

$$(N1)_{60} = RD^2 / 15^2 \quad (3.12)$$

The elastic shear modulus factor (K_G^{*e}) [-] can be calculated as in the formula 3.13 [14]:

$$K_G^{*e} = 21.7(20)((N1)_{60})^{0.3333} \quad (3.13)$$

The elastic bulk modulus factor (K_B^{*e}) [-] can be calculated as in the formula 3.14 [14]:

$$K_B^{*e} = 0.7(K_G^{*e}) \quad (3.14)$$

The plastic shear modulus factor (K_G^{*p}) [-] can be calculated as in the formula 3.15 [14]:

$$K_G^{*p} = K_G^{*e}((N1)_{60})^2(0.003) + 100 \quad (3.15)$$

The parameters m_e [-], n_e [-] and n_p [-] are called the index parameters, and these parameters must be calibrated by curve fitting. The range of these parameters are between 0 and 1. The proposed default value for each of (m_e) and (n_e) is 0.5, and for (n_p) it can be the relative density of the soil in decimal [14]. Whereas the parameter (m_e) is the rate of stress-dependency of elastic bulk modulus, and (n_e) is the rate of stress-dependency of elastic bulk modulus, and (n_p) is the

rate of stress-dependency of plastic shear modulus. The cohesion (C) has a value 0 kN/m² for the cases of granular soils such as sandy soil without the fine component [14].

The parameter φ_p [°] is called a peak friction angle, and this parameter can be computed from SPT test as in the formula 3.16 [14]:

$$\varphi_p = \varphi_{CV} + ((N1)_{60}/10) + \max(0; ((N1)_{60} - 15)/5) \quad (3.16)$$

The parameter φ_{CV} is called a constant volume friction angle, and its value ranges from 32° to 36.5° [15]. in this project the parameter φ_{CV} was taken as 33°.

The parameter R_f [-] is called a failure ratio, and this parameter can be calculated as in the formula 3.17 [14]:

$$R_f \approx 1.1((N1)_{60})^{-0.15} < 0.99 \quad (3.17)$$

The factor (f_{dens}) is called a densification factor, and this factor controls on the volume of plastic shear modulus factor under the secondary load. The reasonable range is between 0 and 1, when this factor under 1 that means the parameter K_G^{*P} becomes lower and the behaviour becomes more flexible [14].

It is suggested to use the value 1.0 for the factor (f_{dens}) because the difference of the densification does not largely influence on the liquefaction triggering, and the parameter (f_{Epost}) to regulate the post-liquefaction behaviour. The reasonable range of the parameter (f_{Epost}) is between 0.1 and 1, and in this project the value of (f_{Epost}) was taken as 1 [14]. In Table 3.3 below there are the parameters of UBC3D-PLM model for different relative densities that used to describe the sand in dynamics state.

Table 3.3: Table of (UBC3D-PLM) parameters for the sand in dynamics state.

RD (%)	35	40	55	65	75
Dranige type	Undrained (A)	Undrained (A)	Undrained (A)	Undrained (A)	Undrained (A)
γ_{unsat} [kN/m ³]	16.40	16.60	17.20	17.60	18.00

Table 3.3: Table of (UBC3D-PLM) parameters for the sand in dynamics state “tables continued”.

γ_{sat} [kN/m ³]	19.56	19.64	19.88	20.04	20.20
$(N1)_{60}$ [-]	5.44	7.11	13.44	18.77	25.00
K_G^{*e} [-]	763.45	834.52	1031.88	1153.44	1268.88
K_B^{*e} [-]	534.41	584.16	722.32	807.40	888.22
K_G^{*p} [-]	167.89	226.60	659.55	1320.12	2479.16
m_e [-]	0.5	0.5	0.5	0.5	0.5
n_e [-]	0.5	0.5	0.5	0.5	0.5
n_p [-]	0.35	0.40	0.55	0.65	0.75
ϕ_{CV} [°]	33.00	33.00	33.00	33.00	33.00
ϕ_p [°]	33.54	33.71	34.34	35.63	37.50
R_f [-]	0.8530	0.8196	0.7449	0.7085	0.6787

3.2.3 Mohr-Coulomb Parameters

Mohr-Coulomb model is a simple linear elastic perfectly plastic model, gives an initial approximation of the soil behaviour. The linear elastic part of this model depends on Hooke's law of isotropic elasticity. but the perfectly plastic part depends on the Mohr-Coulomb failure standard, and this part prepared in a non-associated plasticity scope [14].

In this project the Mohr-Coulomb model was used to describe and model the clay and gravel soil at the statics state. In Table 3.4 below there are the parameters of (Mohr-Coulomb) model for each of clay and gravel soil that used in this project to model it in statics state.

The values of these parameters are hypothetical values were estimated as the mean values of the parameters of clay and gravel that were found in the previous literature. In this project the

parameters of Mohr-Coulomb model haven't been calculated and there is no need to calculate it, because there are no comparisons between different types of clay and gravel soils, only one type of clay and gravel soil is needed.

Table 3.4: Table of (Mohr-Coulomb) parameters for each of clay and gravel (in statics state).

Mohr-Coulomb parameters	Clay	Gravel
Drainage type	Drained	Drained
γ_{unsat} [kN/m ³]	16.00	16.00
γ_{sat} [kN/m ³]	20.00	20.00
Young's modulus at reference level (E') [kN/m ²]	3400	450000
Cohesion (C'_{ref}) [kN/m ²]	10	0
Friction angle (ϕ') [°]	18	39.6
Dilatancy angle (ψ) [°]	0	9.6
Undrained shear strength at reference level ($S_{u,\text{ref}}$) [kN/m ²]	5.00	-
Poisson's ratio (ν') [-]	0.33	0.30
Young's modulus inc. (E'_{inc}) [kN/m ² /m]	400.0	0
Reference level (y_{ref}) [m]	3.00	0
Undrained shear strength inc. ($S_{u,\text{inc}}$) [kN/m ² /m]	2.00	-
Cohesion inc. (C'_{inc}) [kN/m ²]	-	0
Horizontal permeability k_x [m/s]	0.0001	0.25
Vertical permeability k_y [m/s]	0.0001	0.25

3.2.4 Linear Elastic Parameters

In this project the Linear Elastic model was used to describe and model the bed rock. This model depends on Hooke's law, and it includes elastic parameters such as Poisson's ratio (ν) and Young's modulus (E), and this model is not appropriate to model and describe soil, but it is appropriate to model stiff materials inside the soil such as concrete or bed rock [14]. In Table 3.5 below there are the parameters of Linear Elastic model that used in this research to model the bed rock. The values of these parameters are hypothetical values were estimated as the mean values of the parameters of bed rock layers that were found in the previous literature. In this project the parameters of Linear Elastic model haven't been calculated and there is no need to calculate it, because there are no comparisons between different types of bed rocks, only one type of bed rock is needed.

Table 3.5: Table of (Linear Elastic) parameters for the bed rock.

Soil type	Bed rock
Drainage type	Non-porous
γ_{unsat} [kN/m ³]	22.00
γ_{sat} [kN/m ³]	22.00
Young's modulus (E) [kN/m ²]	3400000
Poisson's ratio (ν') [-]	0.20
Shear modulus (G) [kN/m ²]	1417000
Oedometer modulus (E_{oed}) [kN/m ²]	3778000
Shear wave velocity (V_s) [m/s]	794.8
Compression wave velocity (V_p) [m/s]	1298

3.3 MODELING OF THE TUNNEL

The material of the tunnel that was used in this project is lining material, and the type of this lining is Elastic-Isotropic, the normal stiffness (EA) is 8.2×10^6 kN/m, the flexural rigidity (EI) is 30×10^3 kNm²/m, the weight (w) is 8 kN/m/m and the Poisson's ratio (ν) is 0.15. These values are hypothetical values were estimated as the mean values of the tunnel models that were found in the previous literature.

3.4 DESCRIPTION OF EARTHQUAKE

In this project, three different earthquakes have been used as a cyclic loading to obtain the liquefaction in the soil, these earthquakes are Kocaeli, Kobe and Northridge. The time history of these earthquakes has been taken from (SeismoSignal) software. Table 3.6 below illustrates the summary data of each earthquake [43].

3.4.1 Kocaeli Earthquake 1999

It is called also İzmit earthquake or Gölcük earthquake, this earthquake was happened in İzmit-Turkey on August 17, 1999 at 00:01:38 UTC time. The time history of Kocaeli earthquake is shown in Figure 3.5.

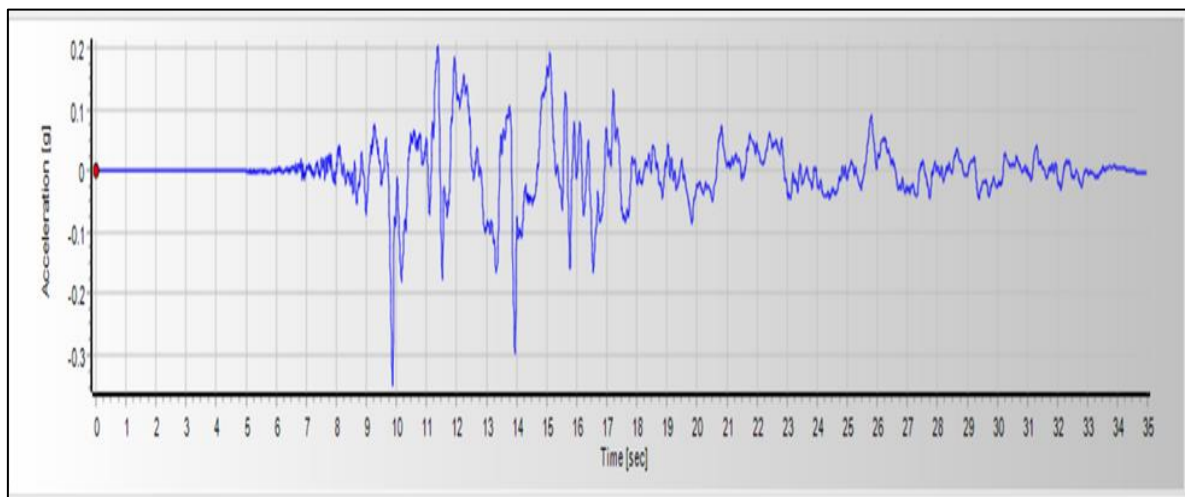


Figure 3.5: The time history of the Kocaeli earthquake from (SeismoSignal) software [44].

3.4.2 Kobe Earthquake 1995

It is called also Great Hanshin earthquake, this earthquake was happened in the south of Hyogo-Japan on January 16, 1995 at 20:46:52 UTC time. The time history of Kobe earthquake is shown in Figure 3.6.

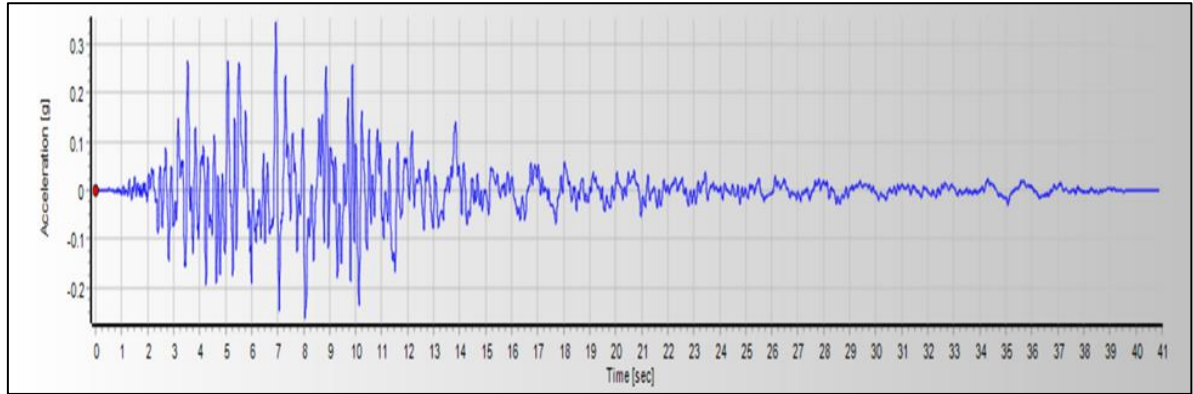


Figure 3.6: The time history of the Kobe earthquake from (SeismoSignal) software [44].

3.4.3 Northridge Earthquake 1994

This earthquake was happened in North America - Los Angeles city at the San Fernando Valley zone on January 17, 1994 at 12:30:55 UTC time. The time history of Northridge earthquake is shown in Figure 3.7.

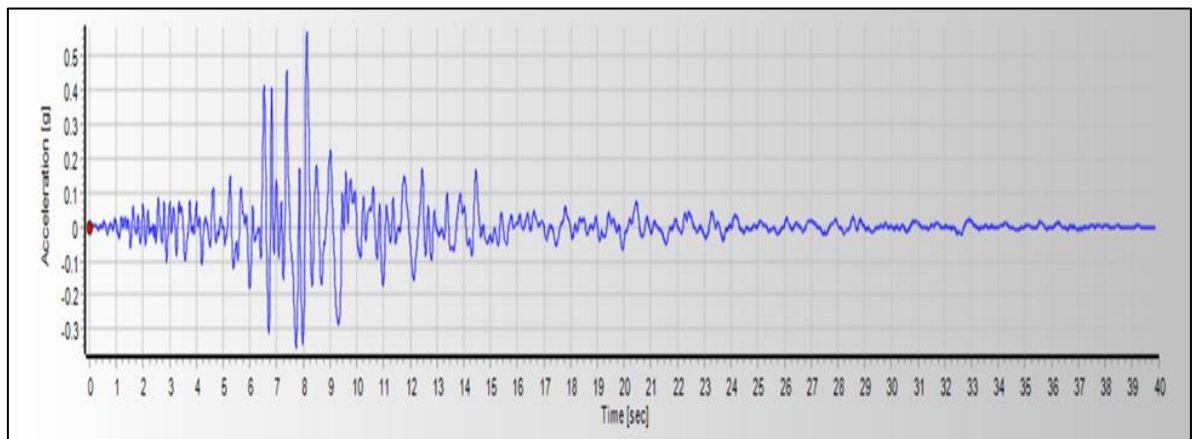


Figure 3.7: The time history of the Northridge earthquake from (SeismoSignal) software [44].

Table 3.6: The data of Kocaeli, Kobe and Northridge earthquakes [43].

Earthquake	Kocaeli	Kobe	Northridge
Latitude	40.7270	34.5948	34.2057
Longitude	29.9900	135.0121	-118.5539
Depth	15.0 km	17.9 km	17.5 km
Mechanism	Strike-slip	Strike-slip	Reverse
Strike	272	230	122
Dip	88	85	40
Rake	180	180	104
M _w	7.4	6.9	6.7
M _s	7.8	-	6.8
PGA	0.3g – 0.4g	0.91g	1.82g
Intensity	X (Extreme)	XI (Extreme)	IX (Violent)

3.5 DYNAMICS CONDITIONS

In the earthquake phase, the following dynamic boundaries have been considered, and these boundaries are important conditions to model the soil in the earthquake loading.

3.5.1 Free Field

Free field boundary was considered at the left side and the right side of the model (boundaries X_{min} and X_{max}). This boundary just available on the lateral sides of the model, and the free field boundary consists of a combination of viscous boundary and load history, this combination permits for the input of earthquake motion, and it absorbs the incoming waves, so this boundary is preferred for earthquake modelling.

3.5.2 Compliant Base

Compliant base boundary was considered at the bottom of the model (boundary Y_{min}). The compliant base boundary consists of a combination of viscous boundary and line prescribed displacement, this combination permits for the input of earthquake motion, and it absorbs the incoming waves, so this boundary is preferred for earthquake modelling on the base of the model. This boundary absorbs the outgoing earthquake waves to the infinite space beneath the base of the model.

3.6 DEFORMATIONS CONDITIONS

For the deformation conditions, horizontally fixed boundary was considered for the lateral sides of the model (boundaries X_{min} and X_{max}), and fully fixed boundary at the base of the model (boundary Y_{min}), and free boundary at the upper face of the model (boundary Y_{max}).

3.7 INTERFACES

The soil-structure interfaces are very important in the modelling to get realistic solutions of many geotechnical problems, and the interface elements have been used in this project to simulate the interaction between the tunnel and the soil. without this interface, there will not be any relative displacement between the tunnel and the soil. the behaviour of interfaces has large influence in the response of the overall structural, especially during the dynamic loads.

4. RESULTS

Figure 4.1 shows the stress-strain relationship (Hysteresis Loop) for the sand of relative density (40%) at number of cycles equal (15) and under confining stress (100 kN/m^2).

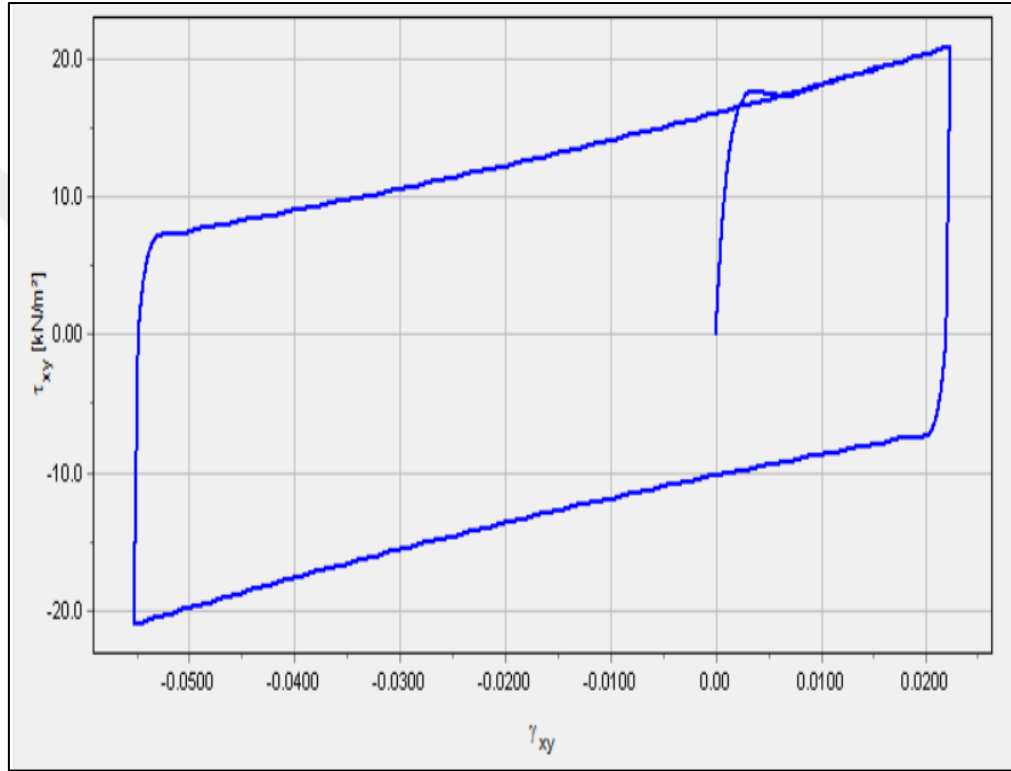


Figure 4.1: The stress-strain relationship for the sand of relative density (40%) at number of cycles equal (15) and under confining stress (100 kN/m^2).

4.1 UNDER KOCAELI EARTHQUAKE

The research was started with a sand of relative density 40% without any mitigation techniques to see the effect of the earthquake on the liquefaction and the uplifting of the tunnel. When the model has a sand of relative density 40%, the built up of the pore water pressure and the uplift of the tunnel were observed as in Figure 4.2 and Figure 4.3 at the invert of the tunnel.

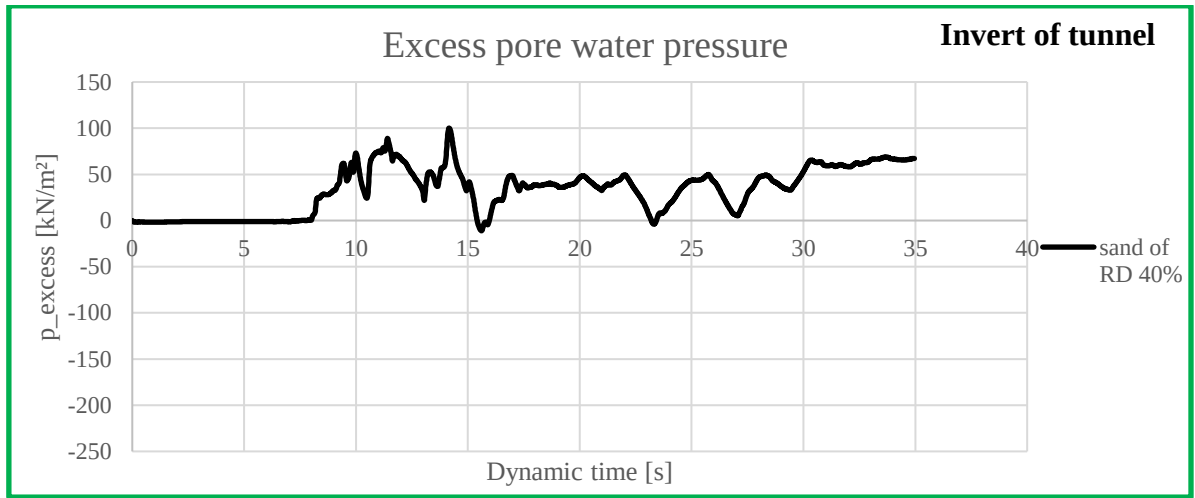


Figure 4.2: The excess pore water pressure at the invert of the tunnel for the sand of relative density 40% without any mitigation techniques (under Kocaeli earthquake).

From Figure 4.2, the excess pore water pressure increases through the dynamic time of the earthquake loading. The peak value of the excess pore water pressure is 100 kN/m², but the excess pore water pressure is 67.3 kN/m² at the end of the earthquake.

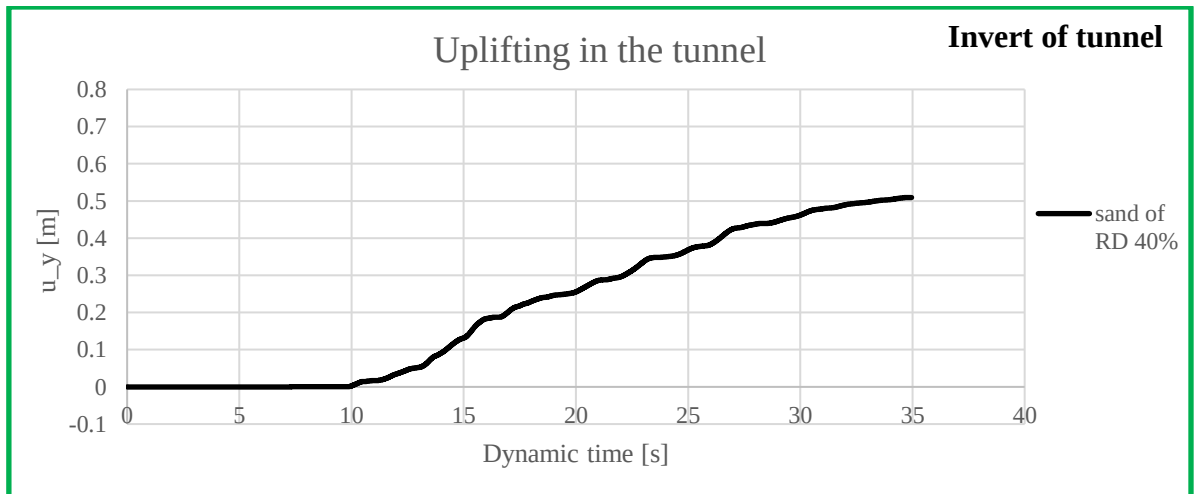


Figure 4.3: The uplifting at the invert of the tunnel for the sand of relative density 40% without any mitigation techniques (under Kocaeli earthquake).

From Figure 4.3, the uplifting in the tunnel increases through the dynamic time of the earthquake loading. the uplifting in the tunnel is 0.509 m at the end of the earthquake.

At the crown of the tunnel, the built up of the pore water pressure and the uplift of the tunnel were observed as in Figure 4.4 and Figure 4.5.

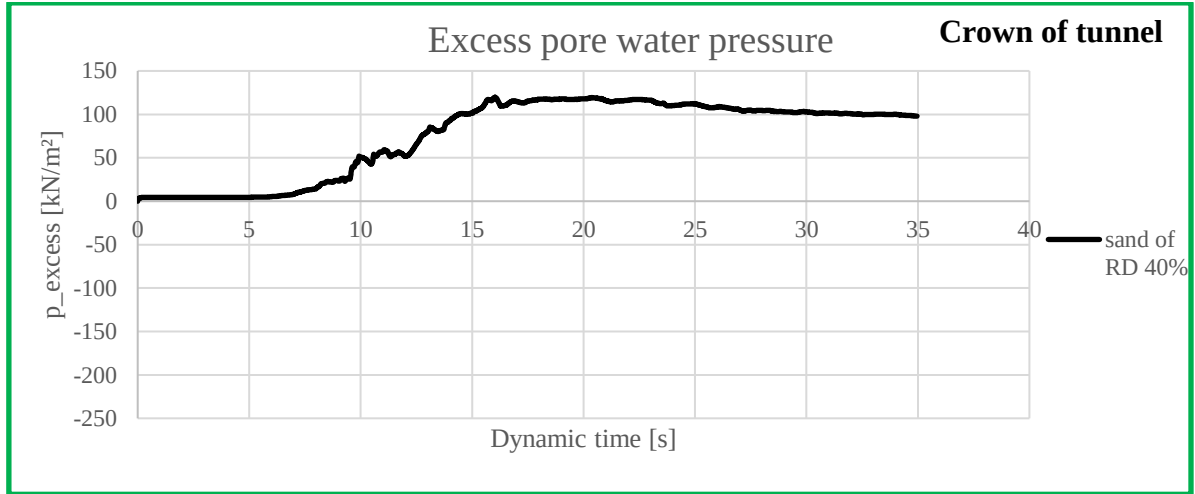


Figure 4.4: The excess pore water pressure at the crown of the tunnel for the sand of relative density 40% without any mitigation techniques (under Kocaeli earthquake).

From Figure 4.4, the excess pore water pressure increases through the dynamic time of the earthquake loading. The peak value of the excess pore water pressure is 120 kN/m^2 , but the excess pore water pressure is 98.1 kN/m^2 at the end of the earthquake.

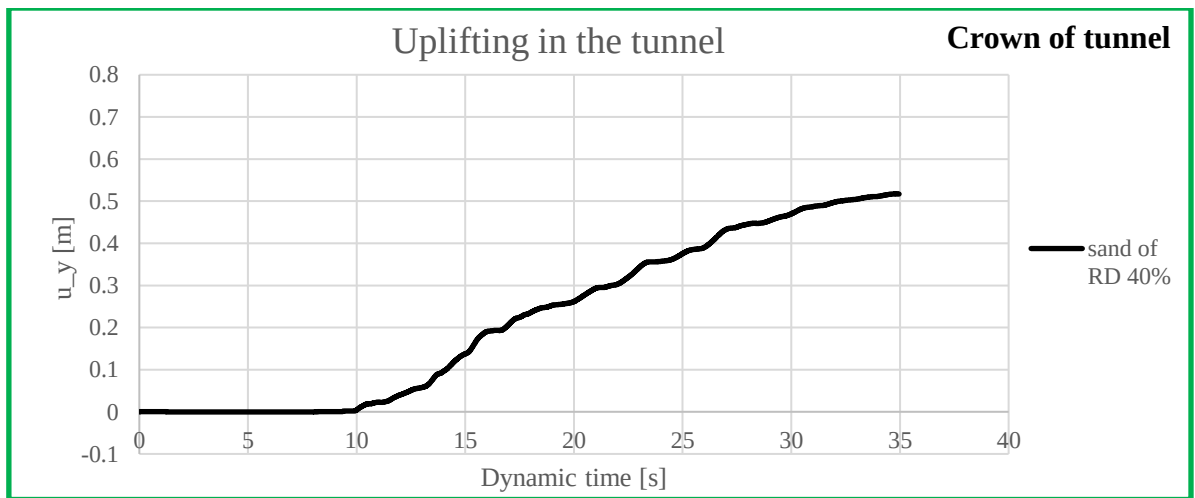


Figure 4.5: The uplifting at the crown of the tunnel for the sand of relative density 40% without any mitigation techniques (under Kocaeli earthquake).

From Figure 4.5, the uplifting in the tunnel increases through the dynamic time of the earthquake loading. the uplifting in the tunnel is 0.517 m at the end of the earthquake.

Figure 4.6 and Figure 4.7 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of relative density 40% without any mitigation techniques.

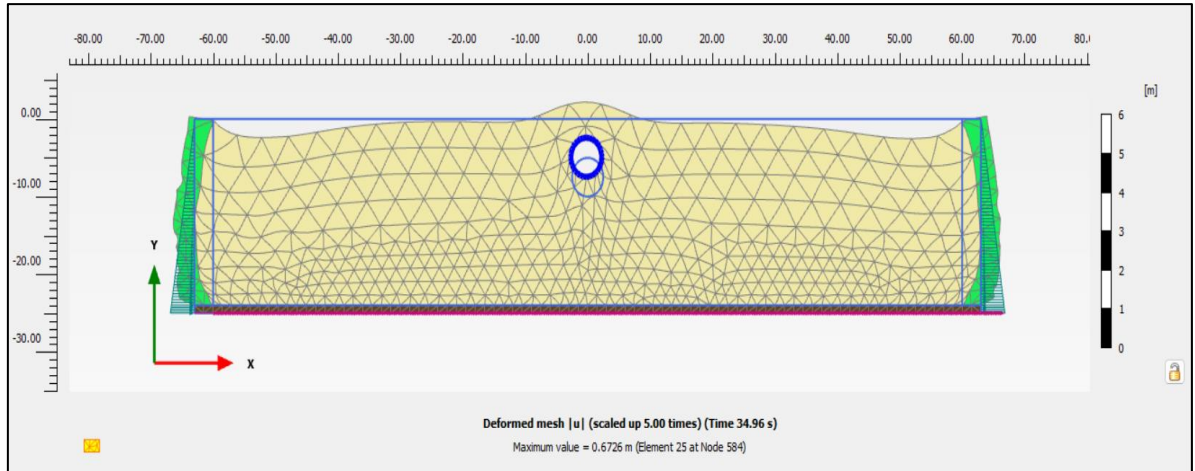


Figure 4.6: Deformation shape that occurs in a sand of relative density 40% without any mitigation techniques (under Kocaeli earthquake).

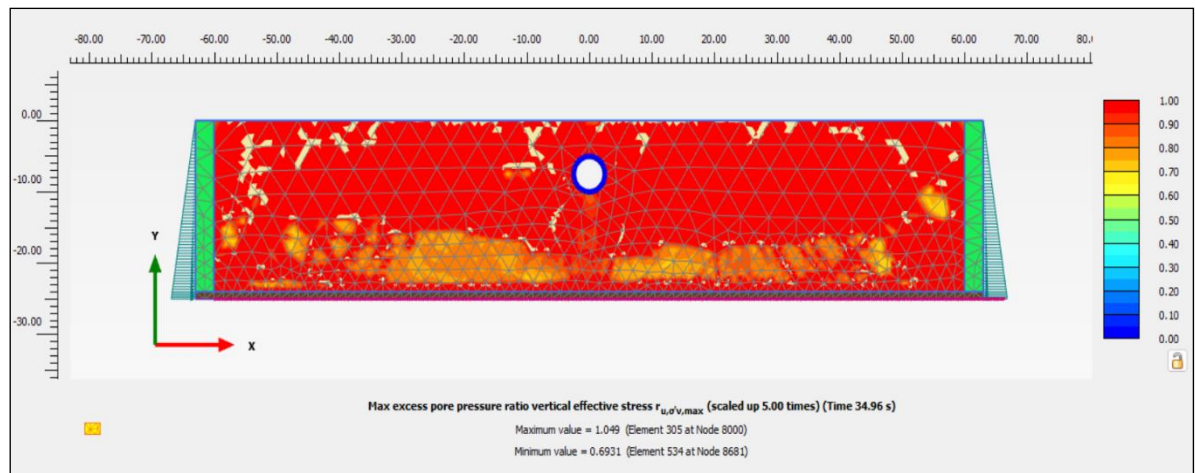


Figure 4.7: The shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of relative density 40% without any mitigation techniques (under Kocaeli earthquake).

In Figure 4.7, it was noticed that the excess pore water pressure ratio at the soil in the ground surface or in shallow depth is higher than the excess pore pressure ratio in the deep depth, that due to the effective confining stresses at the deep soil are higher, so the liquefaction has a higher severity at the shallow depths more than deep depths.

So, in the next steps are trying to mitigate the value of excess pore water pressure and the uplifting in the tunnel, by making some models with using mitigation techniques and comparing the value of these models with the value of previous model of sand RD 40% to see if these mitigation methods or techniques are effective or not, and to find which technique is the best.

4.1.1 Changing The Relative Density Of The Sand

The previous model of sand RD 40% was compared with other models by increasing the sand's relative density such as 55%, 65% and 75%, or by decreasing the sand's relative density such as 35%. the built up of the pore water pressure and the uplift of the tunnel were seen as in the following curves for both invert and crown of the tunnel.

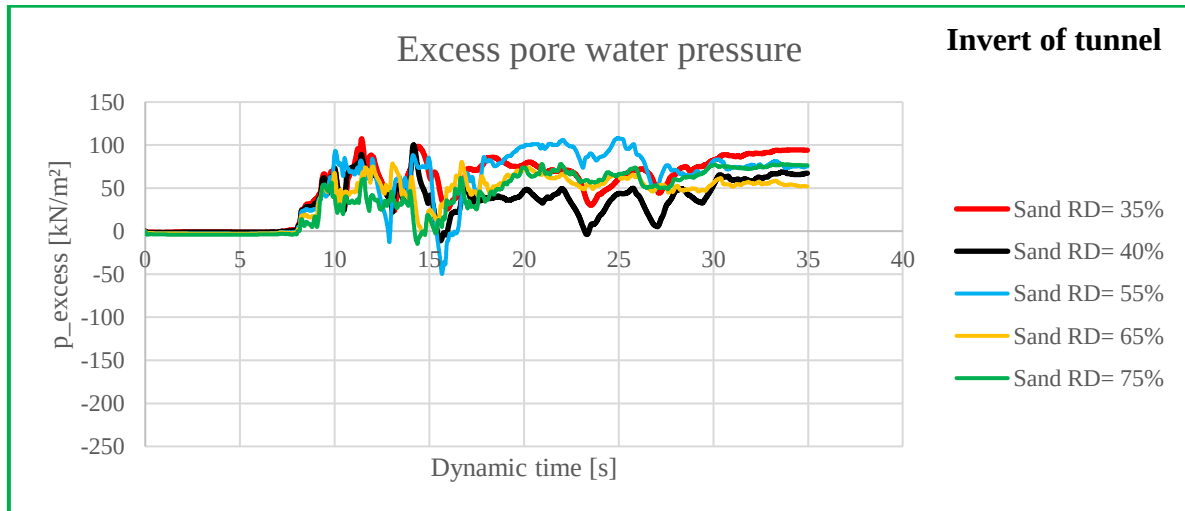


Figure 4.8: The excess pore water pressure at the invert of the tunnel for different relative densities of the sand (under Kocaeli earthquake).

From Figure 4.8, the excess pore water pressure increases through the dynamic time of the earthquake loading. And on the peak of the earthquake (between 10 s and 15 s) the excess pore

water pressure is higher at the lower relative density of the sand, and it is lower at the higher relative density of the sand, because the excess pore water pressure occurs in the loose sand more than in the dense sand. So, the excess pore water pressure can be mitigated by increasing the relative density of the sand.

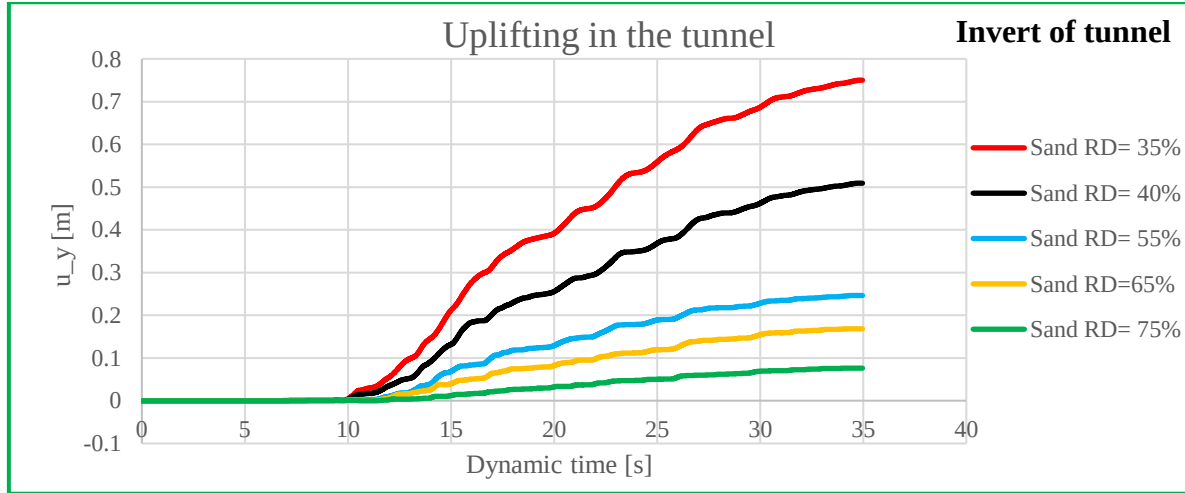


Figure 4.9: The uplifting at the invert of the tunnel for different relative densities of the sand (under Kocaeli earthquake).

From Figure 4.9, the uplifting in the tunnel increases through the dynamic time of the earthquake loading. And the uplifting in the tunnel is higher at the lower relative density of the sand, and it is lower at the higher relative density of the sand, because the uplifting in the tunnel occurs in the loose sand more than in the dense sand. So, the uplifting in the tunnel can be mitigated by increasing the relative density of the sand.

When comparing the uplifting in the tunnel at a sand of RD 40% with the sand of RD 35% and 75%, the uplifting in the tunnel is 0.509 m at sand of RD 40%, but it is 0.751 m when decrease the RD to 35%, and it is 0.076 m when increase the RD to 75%.

So, increase the relative density of the sand is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.10 and Figure 4.11.

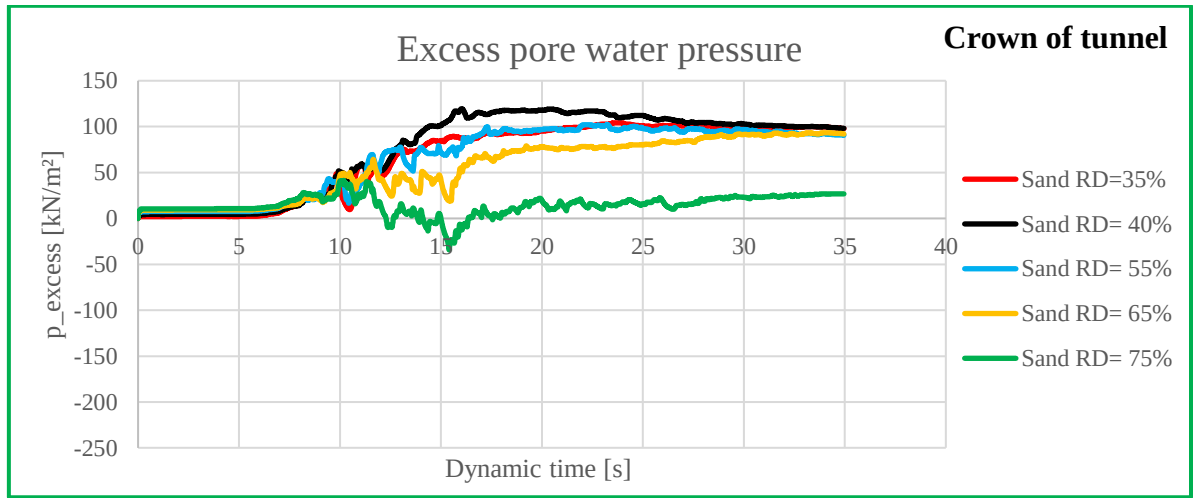


Figure 4.10: The excess pore water pressure at the crown of the tunnel for different relative densities of the sand (under Kocaeli earthquake).

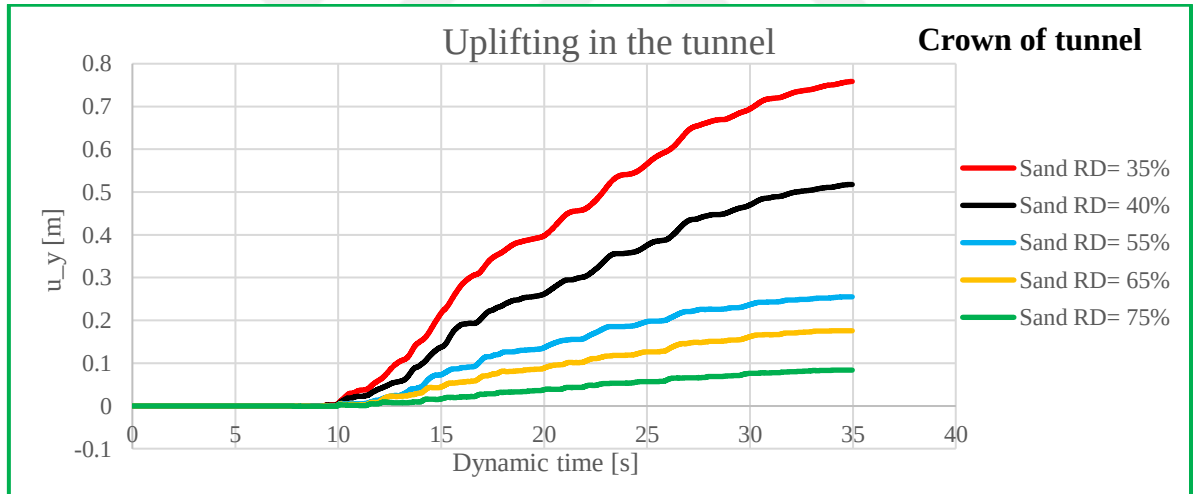


Figure 4.11: The uplifting at the crown of the tunnel for different relative densities of the sand (under Kocaeli earthquake).

4.1.2 Replacing The Sand Around The Tunnel By Higher Relative Density Sand

This method was done by using the sand RD 40% but replacing the sand around the tunnel by higher relative density such as 75%, and then comparing the results between the sand with replacing and without replacing.

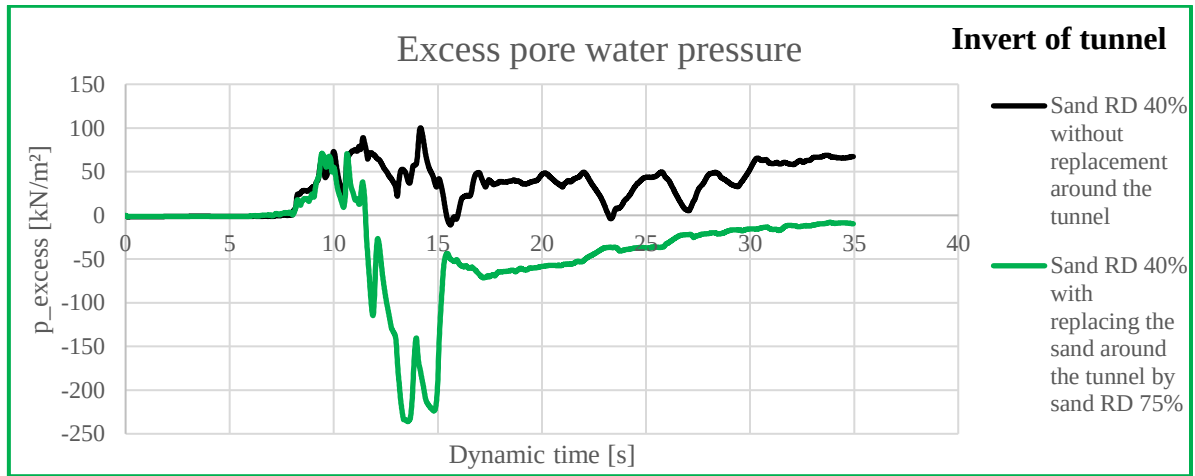


Figure 4.12: The excess pore water pressure at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kocaeli earthquake).

From Figure 4.12, the excess pore water pressure was decreased when replacing the sand around the tunnel by a higher relative density sand, such as replacing the sand of RD 40% by a sand of RD 75% around the tunnel.

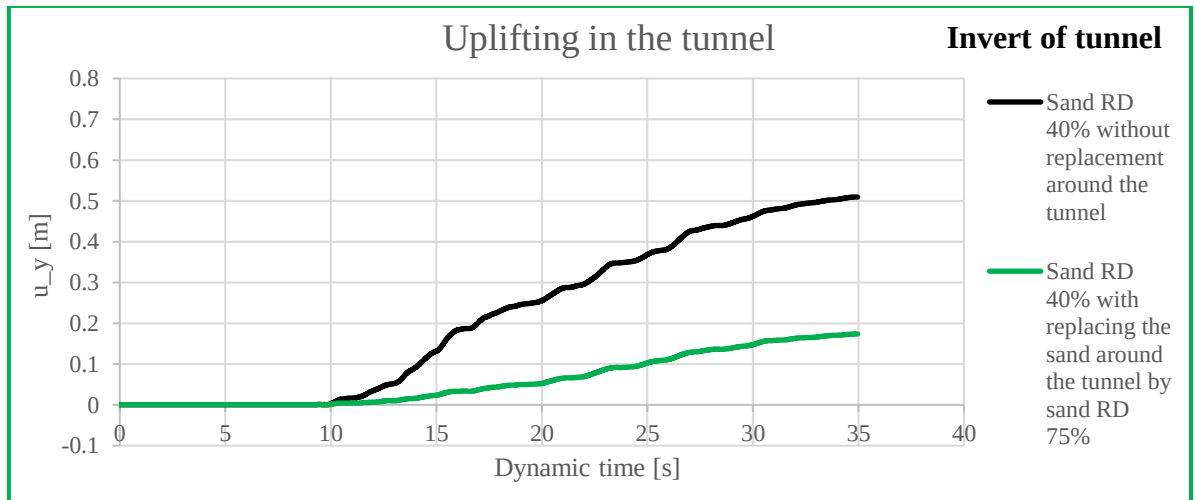


Figure 4.13: The uplifting at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kocaeli earthquake).

From Figure 4.13, the uplifting in the tunnel was decreased when replacing the sand around the tunnel by a higher relative density sand, such as replacing the sand of relative density 40% by a sand of relative density 75% around the tunnel.

The maximum uplifting in the tunnel is 0.509 m in a sand of RD 40% without replacement, but it is 0.174 m in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75%.

So, replacing of the sand around the tunnel by a higher relative density sand is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.14 and Figure 4.15.

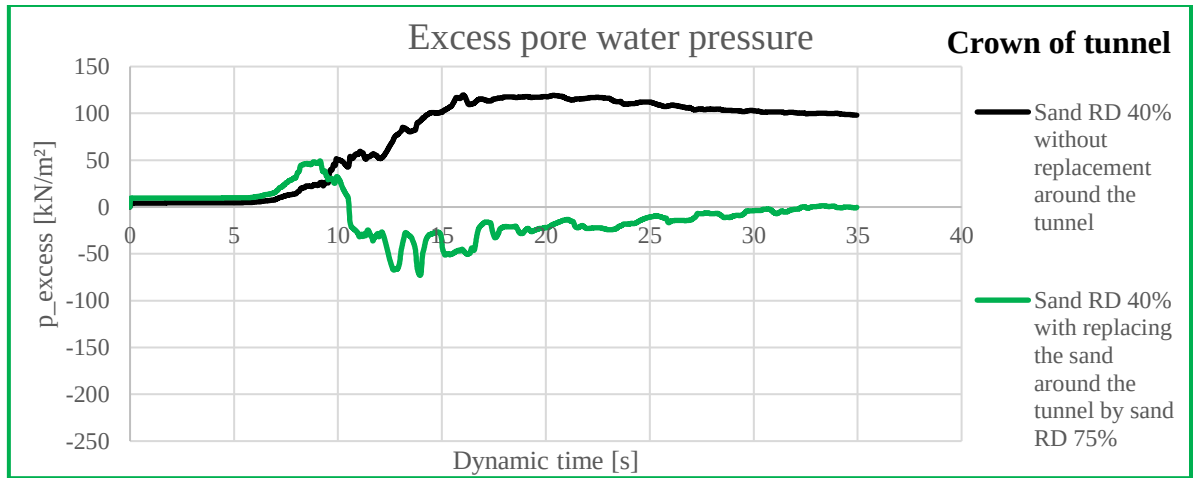


Figure 4.14: The excess pore water pressure at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kocaeli earthquake).

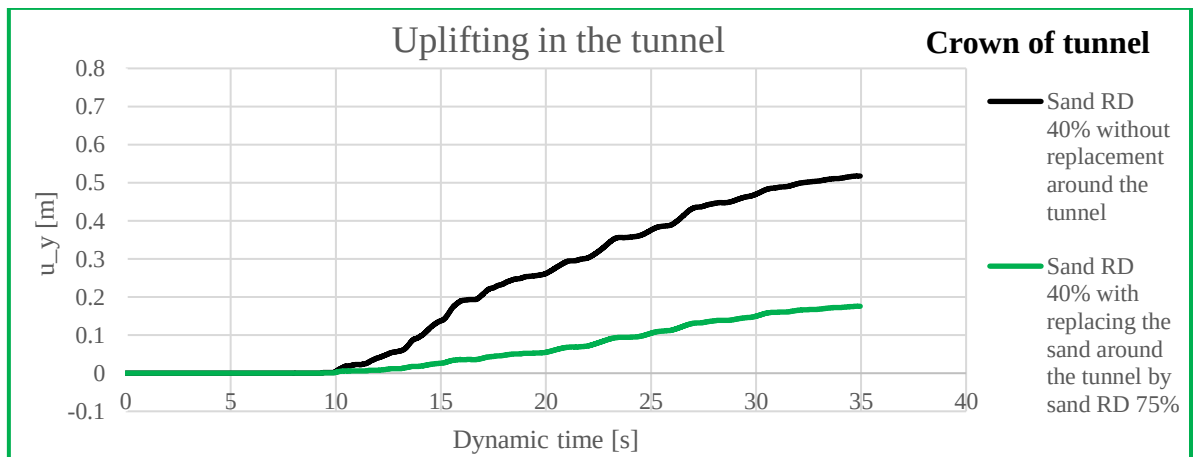


Figure 4.15: The uplifting at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kocaeli earthquake).

Figure 4.16 and Figure 4.17 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75%.

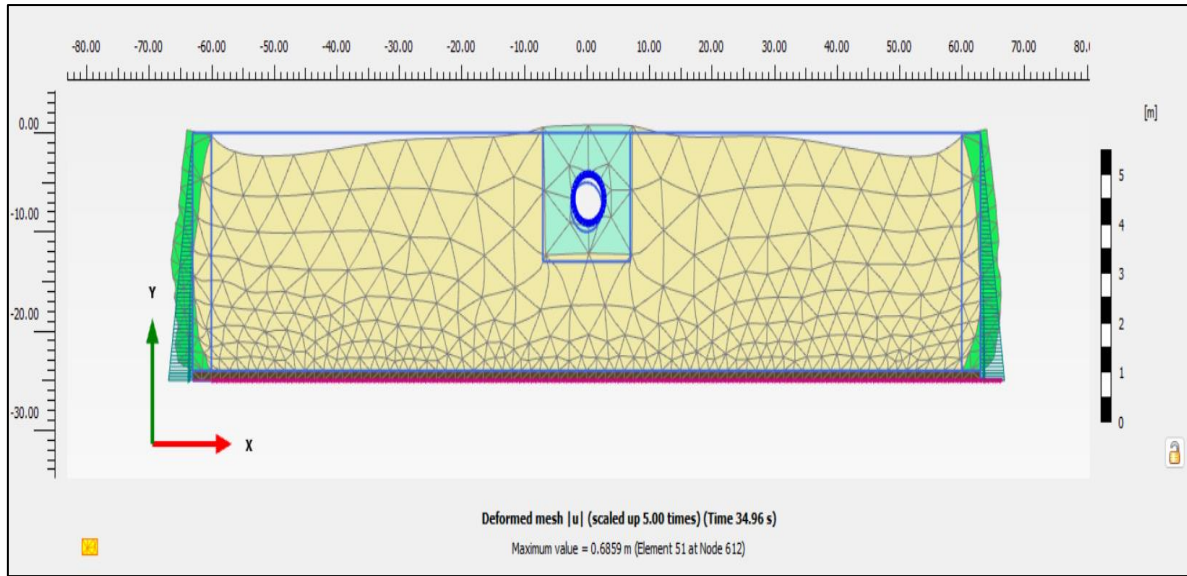


Figure 4.16: Deformation shape that occurs in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kocaeli earthquake).

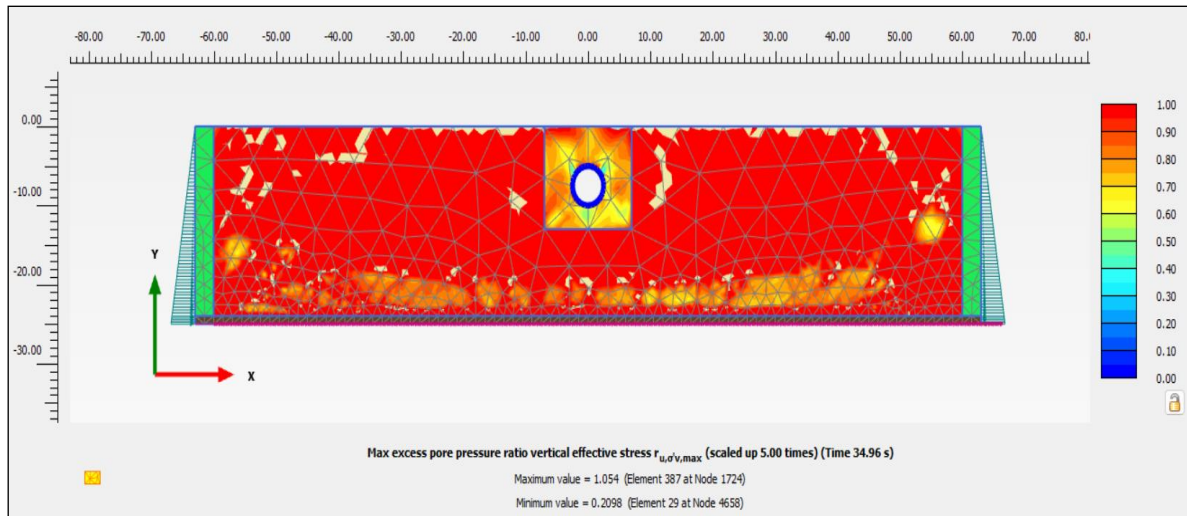


Figure 4.17: The shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kocaeli earthquake).

4.1.3 Replacing The Sand Around The Tunnel By Clay Soil

This method was done by using the sand RD 40% but replacing the sand around the tunnel by clay soil, and then comparing the results between the sand with replacing and without replacing.

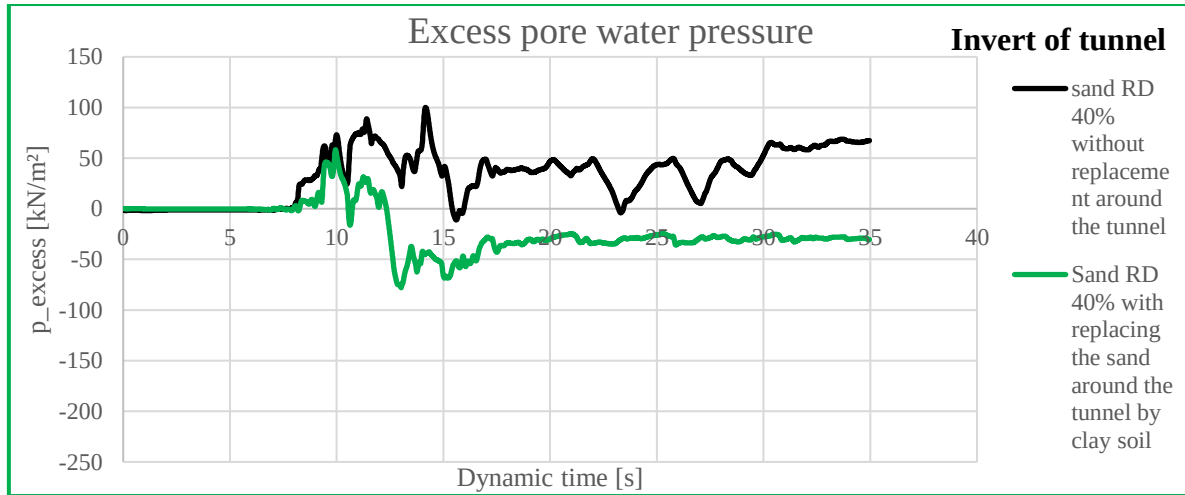


Figure 4.18: The excess pore water pressure at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kocaeli earthquake).

from Figure 4.18, the excess pore water pressure was decreased when replacing the sand around the tunnel by clay soil, because the liquefaction doesn't occur in the clay soil.

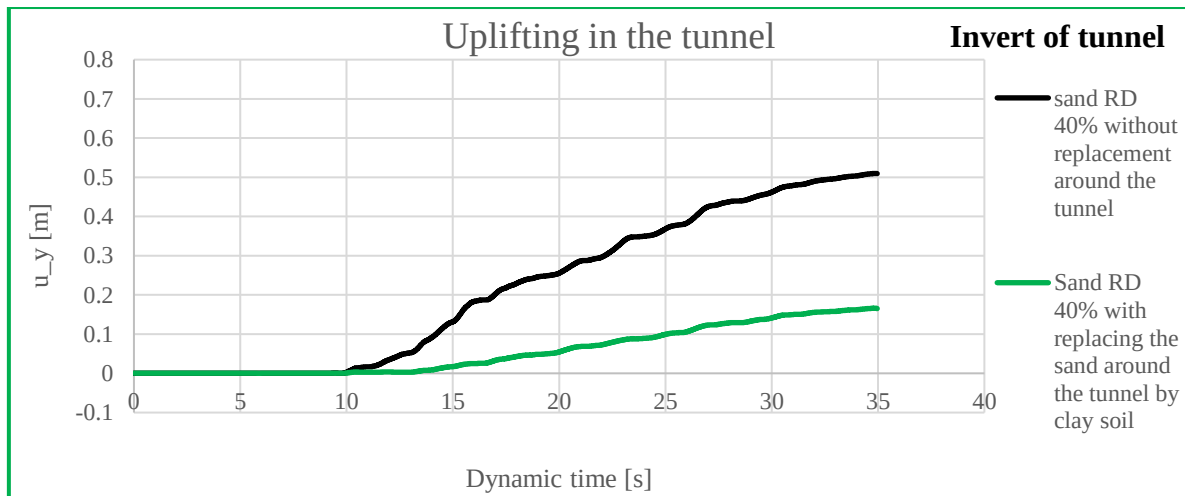


Figure 4.19: The uplifting at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kocaeli earthquake).

From Figure 4.19, the uplifting in the tunnel was decreased when replacing the sand around the tunnel by a clay soil, because the liquefaction doesn't occur in the clay soil.

The maximum uplifting in the tunnel is 0.509 m in a sand of RD 40% without replacement, but it is 0.166 m in a sand of RD 40% with replacing the sand around the tunnel by clay soil.

So, replacing of the sand around the tunnel by a clay soil is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.20 and Figure 4.21.

Figure 4.22 and Figure 4.23 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by clay soil. It was observed that the excess pore water pressure doesn't appear on the clay soil as in Figure 4.23.

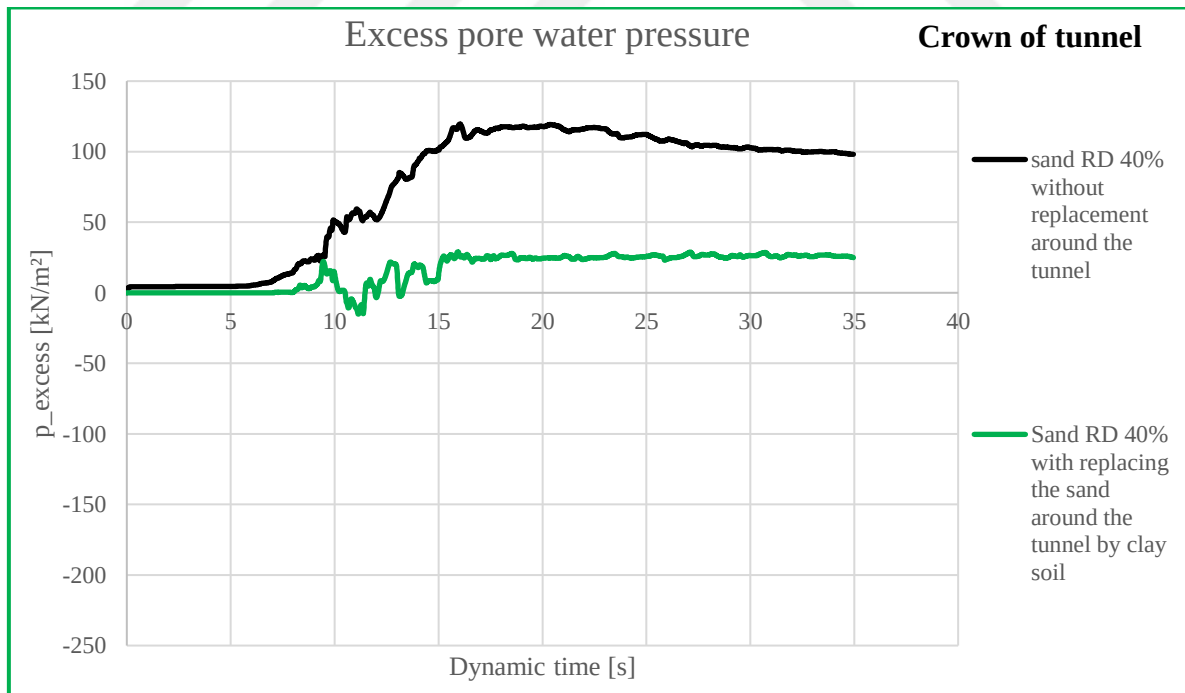


Figure 4.20: The excess pore water pressure at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kocaeli earthquake).

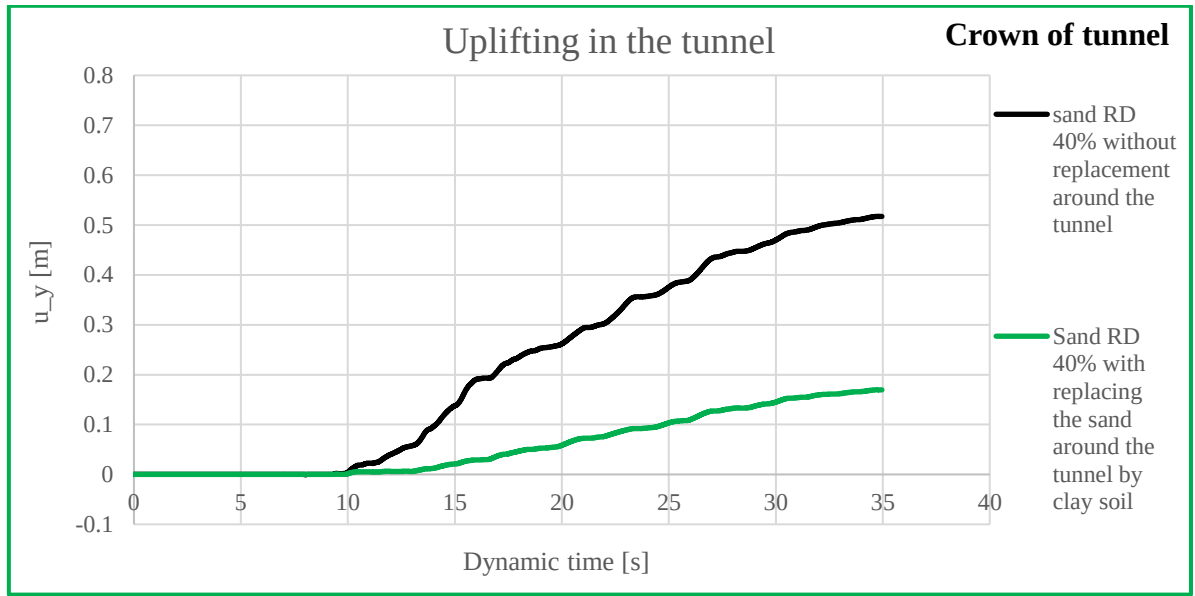


Figure 4.21: The uplifting at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kocaeli earthquake).

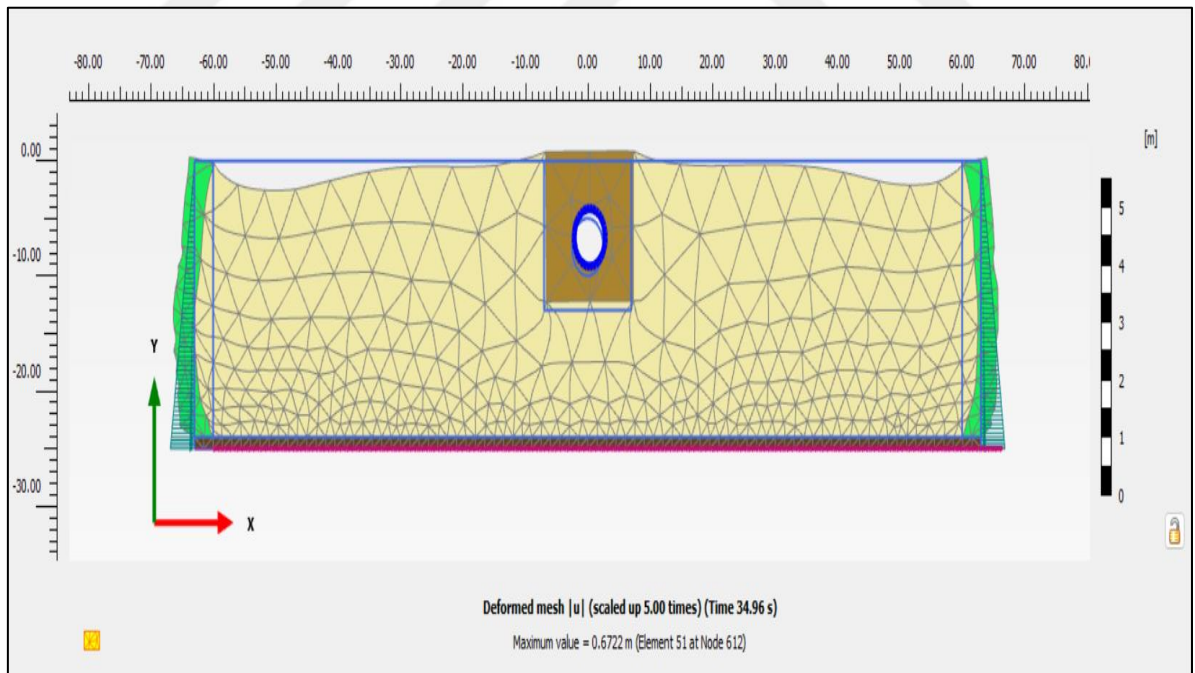


Figure 4.22: Deformation shape that occurs in a sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kocaeli earthquake).

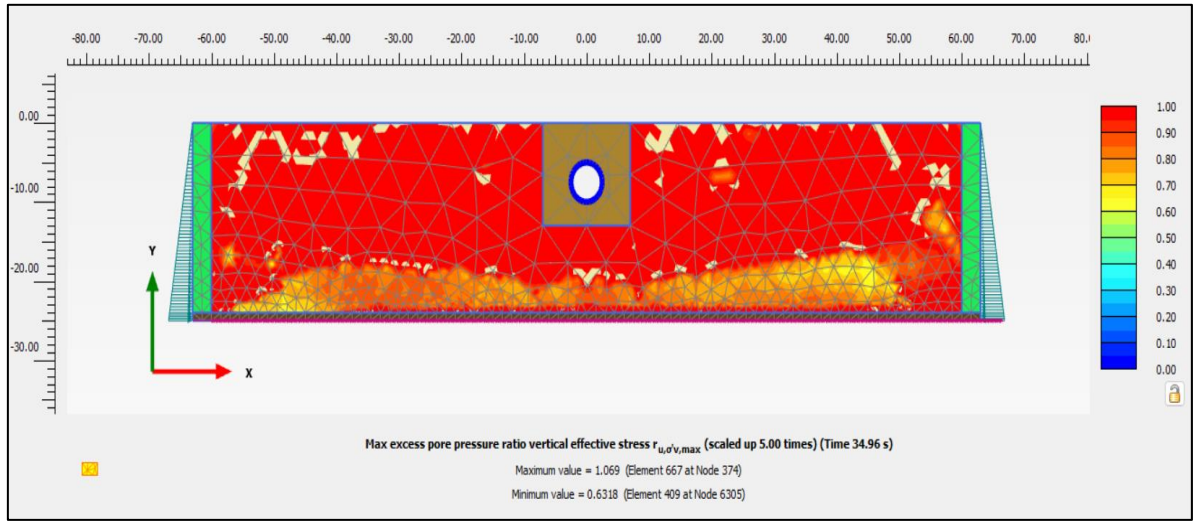


Figure 4.23: The shape of max excess pore pressure ratio ($r_{u, \sigma'v, \max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kocaeli earthquake).

4.1.4 Replacing The Sand Around The Tunnel By Gravel Soil

This method was done by using the sand RD 40% but replacing the sand around the tunnel by gravel soil, and then comparing the results between the sand with replacing and without replacing.

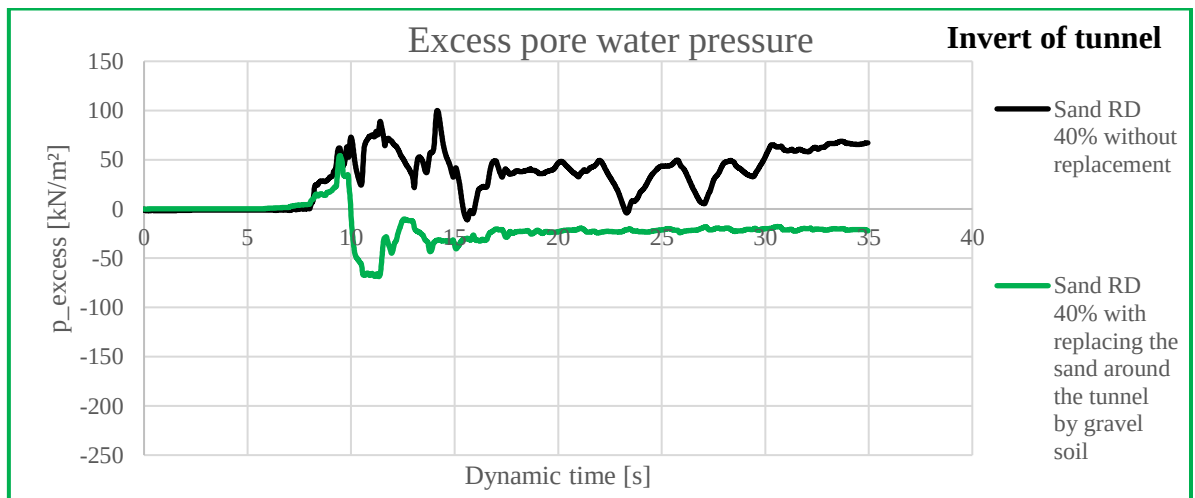


Figure 4.24: The excess pore water pressure at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kocaeli earthquake).

From Figure 4.24, the excess pore water pressure was decreased when replacing the sand around the tunnel by gravel soil, because the liquefaction doesn't occur in the gravel soil.

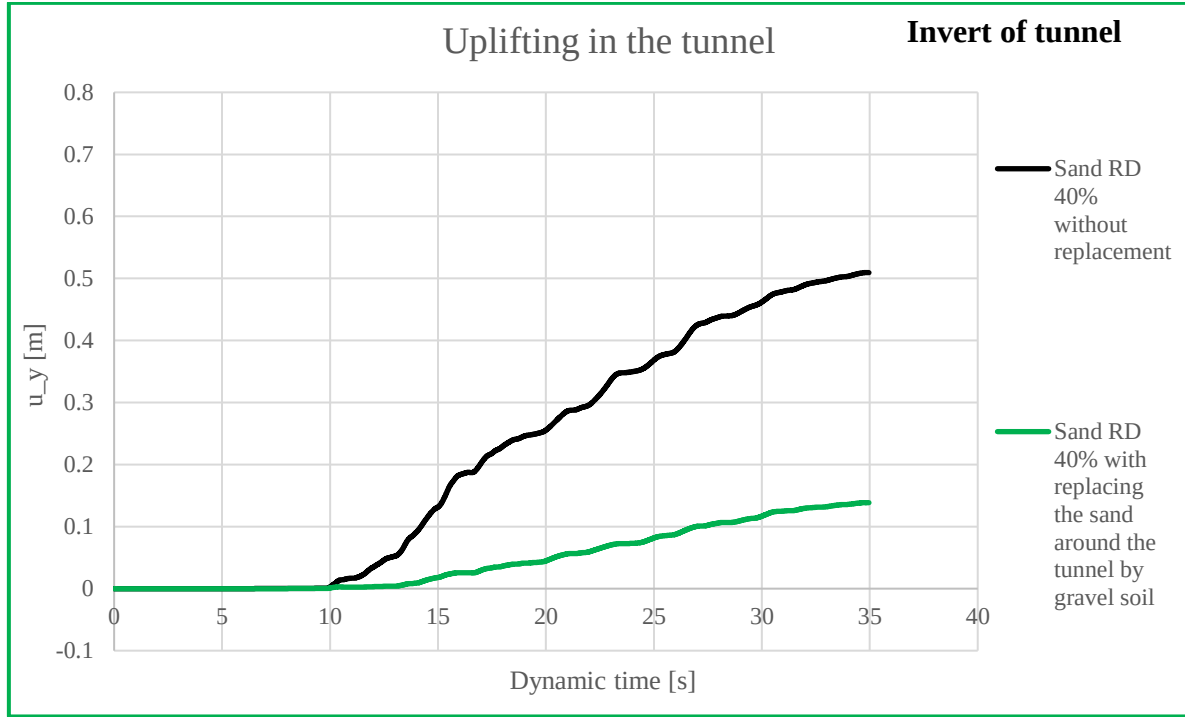


Figure 4.25: The uplifting at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kocaeli earthquake).

From Figure 4.25, the uplifting in the tunnel was decreased when replacing the sand around the tunnel by a gravel soil, because the liquefaction doesn't occur in the gravel soil.

The maximum uplifting in the tunnel is 0.509 m in a sand of RD 40% without replacement, but it is 0.139 m in a sand of RD 40% with replacing the sand around the tunnel by gravel soil.

So, the replacing of the sand around the tunnel by a gravel soil is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.26 and Figure 4.27.

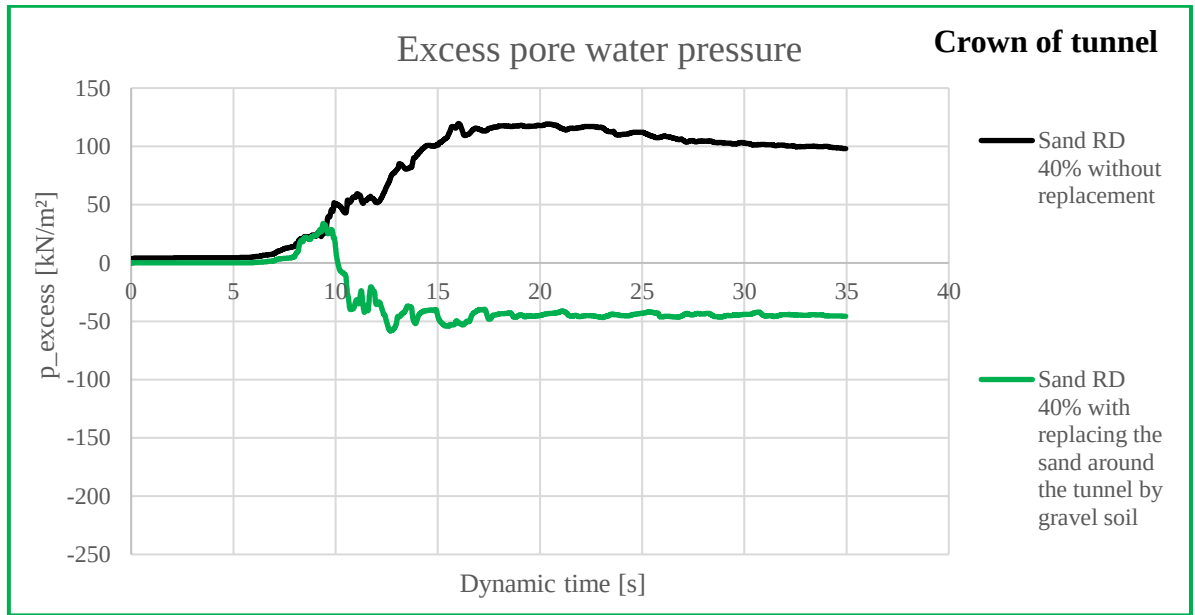


Figure 4.26: The excess pore water pressure at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kocaeli earthquake).

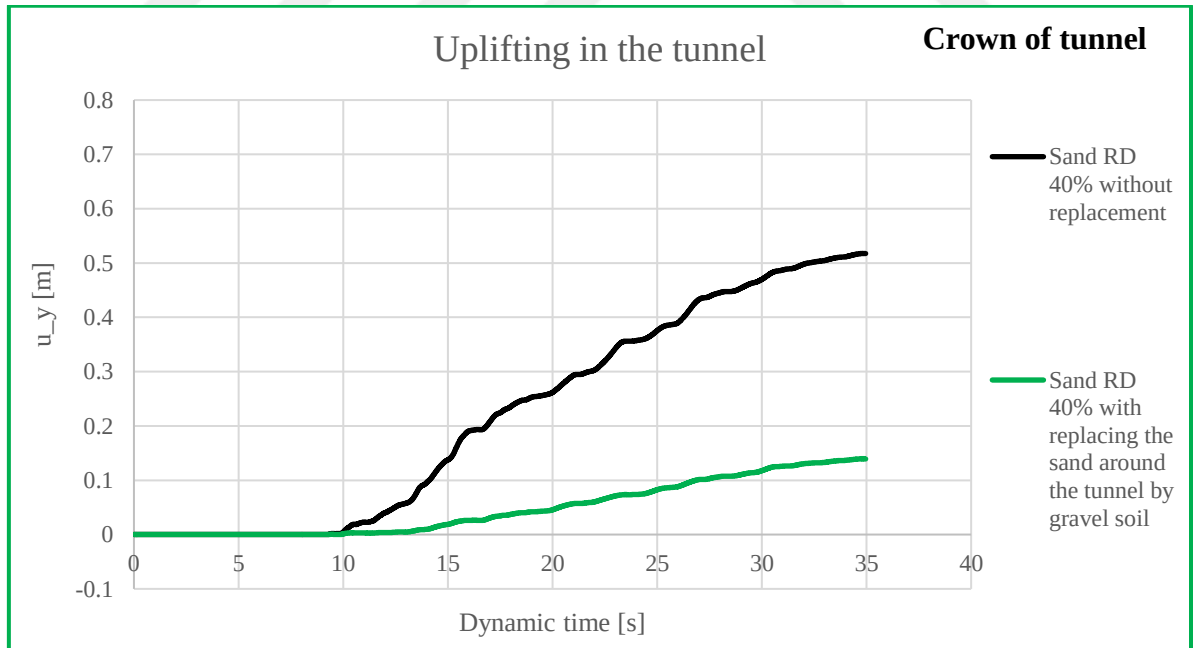


Figure 4.27: The uplifting at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kocaeli earthquake).

Figure 4.28 and Figure 4.29 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by gravel soil. It was observed that the excess pore water pressure doesn't appear on the gravel soil as in Figure 4.29.

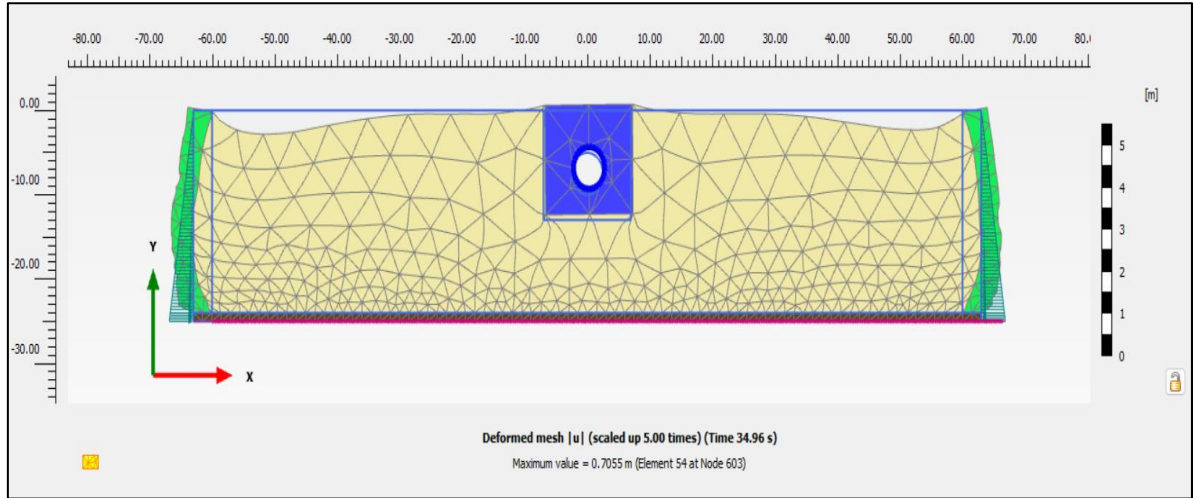


Figure 4.28: Deformation shape that occurs in a sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kocaeli earthquake).

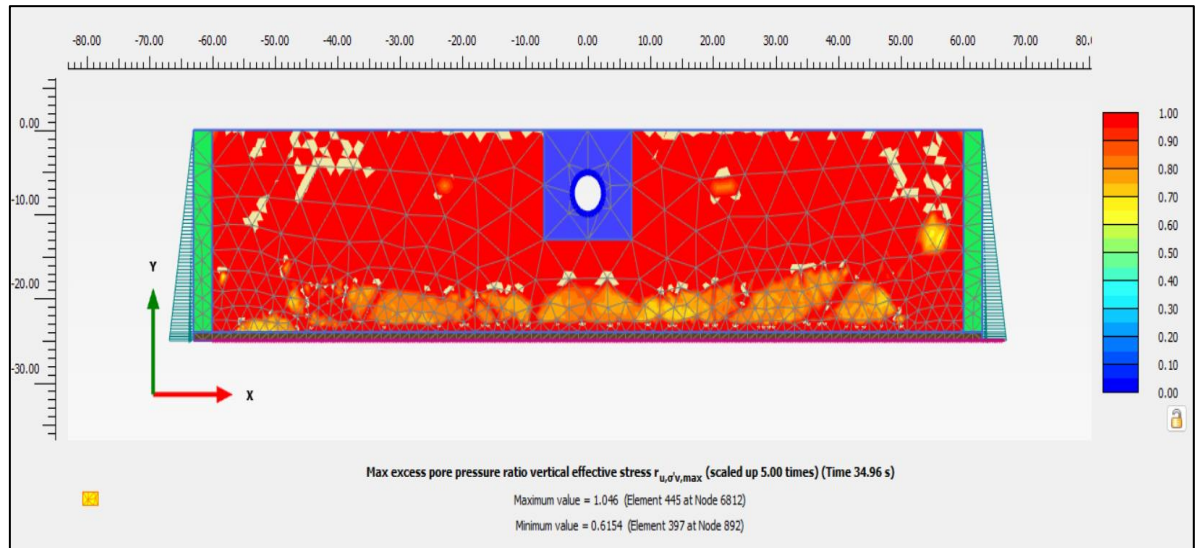


Figure 4.29: The shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kocaeli earthquake).

Figure 4.30 and Figure 4.31 illustrate the comparison between the previous mitigation techniques, and it was noticed that the technique of increase the relative density of the sand to 75% has advantageous for mitigation the soil liquefaction and for mitigation the uplift of the tunnel, and it was noticed that replacing the sand around the tunnel by gravel soil is better than replacing it by clay soil, because the gravel soil is better for drainage the water and dissipating the excess pore water pressure at the soil.

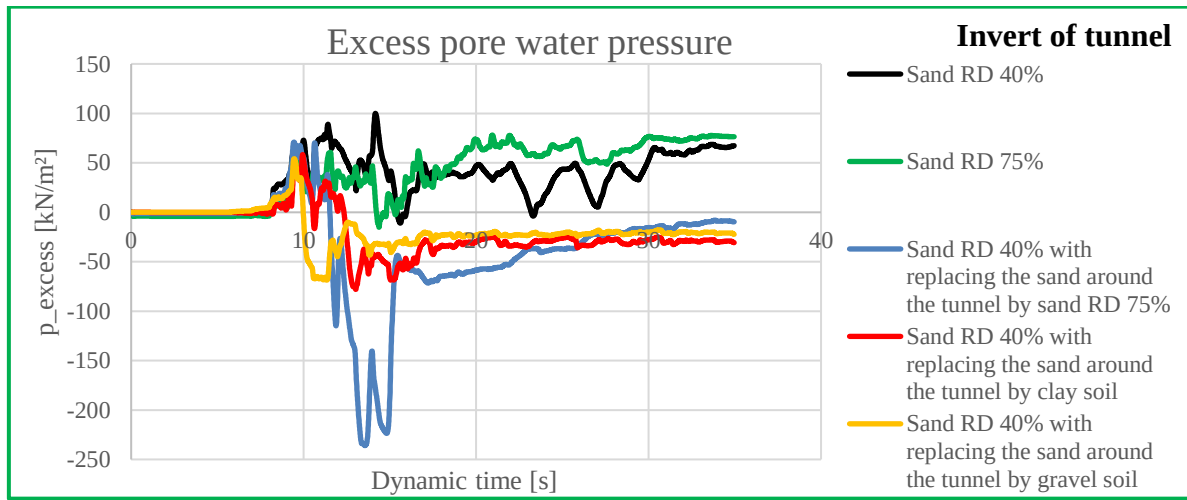


Figure 4.30: The comparison of the excess pore water pressure between the techniques that were used for mitigation the soil liquefaction and the uplift of the tunnel (under Kocaeli earthquake).

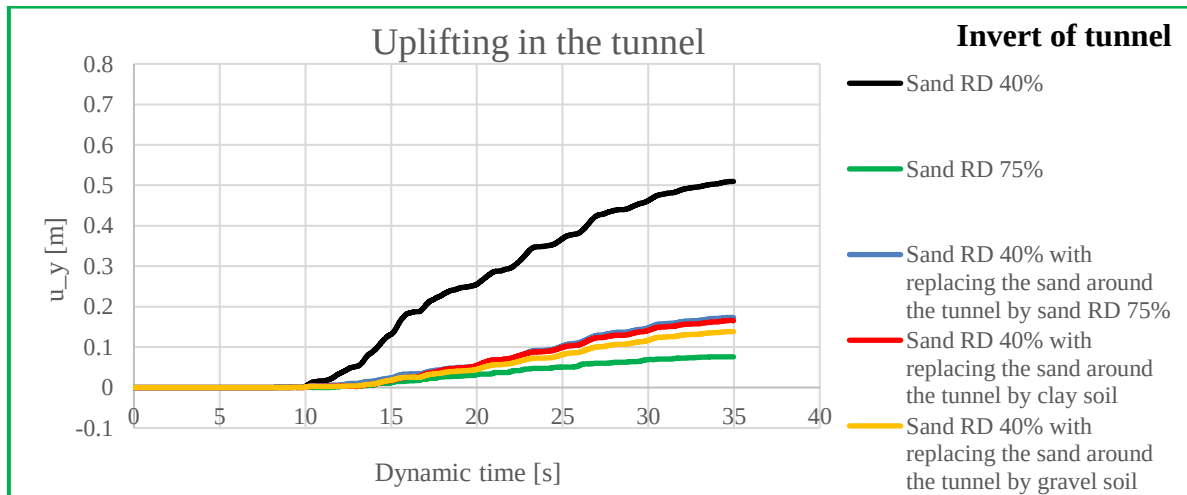


Figure 4.31: The comparison of the uplift of the tunnel between the techniques that were used for mitigation the soil liquefaction and the uplift of the tunnel (under Kocaeli earthquake).

4.2 UNDER KOBE EARTHQUAKE

When the model has a sand with relative density 40%, the built up of the pore water pressure and the uplift of the tunnel were seen as in the following curves for both invert and crown of the tunnel.

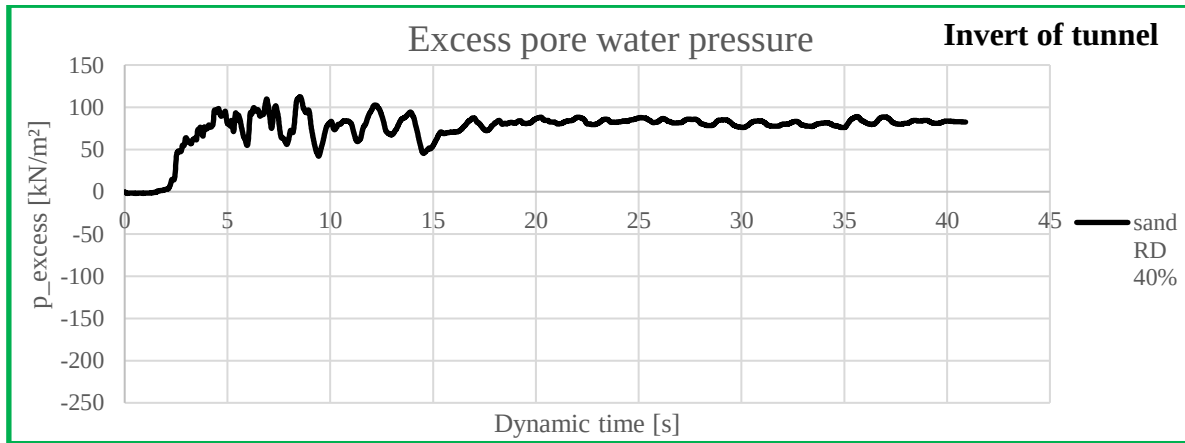


Figure 4.32: The excess pore water pressure at the invert of the tunnel for the sand of relative density 40% without any mitigation techniques (under Kobe earthquake).

From Figure 4.32, the excess pore water pressure increases through the dynamic time of the earthquake loading. The peak value of the excess pore water pressure is 112 kN/m^2 , but the excess pore water pressure is 82.7 kN/m^2 at the end of the earthquake.

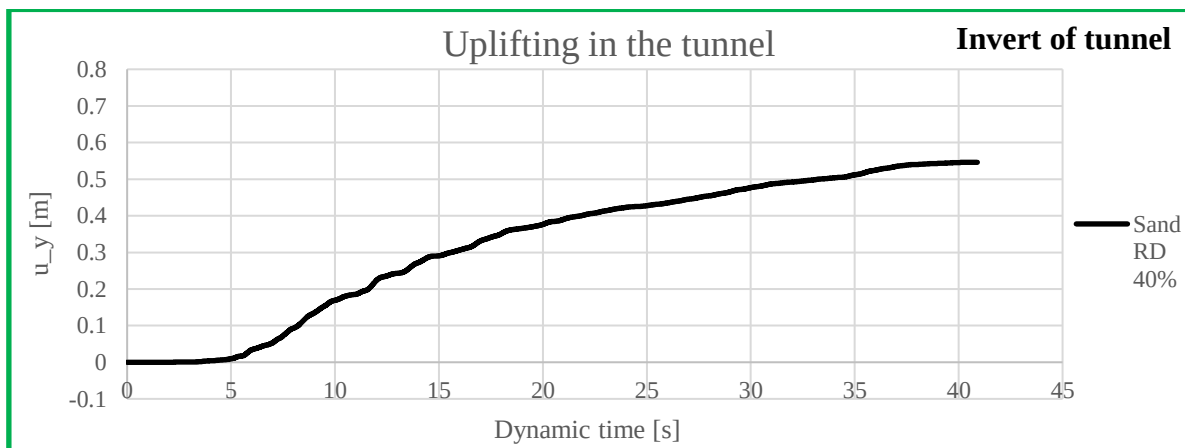


Figure 4.33: The uplifting at the invert of the tunnel for the sand of relative density 40% without any mitigation techniques (under Kobe earthquake).

From Figure 4.33, the uplifting in the tunnel increases through the dynamic time of the earthquake loading. The maximum uplifting in the tunnel is 0.546 m at the end of the earthquake.

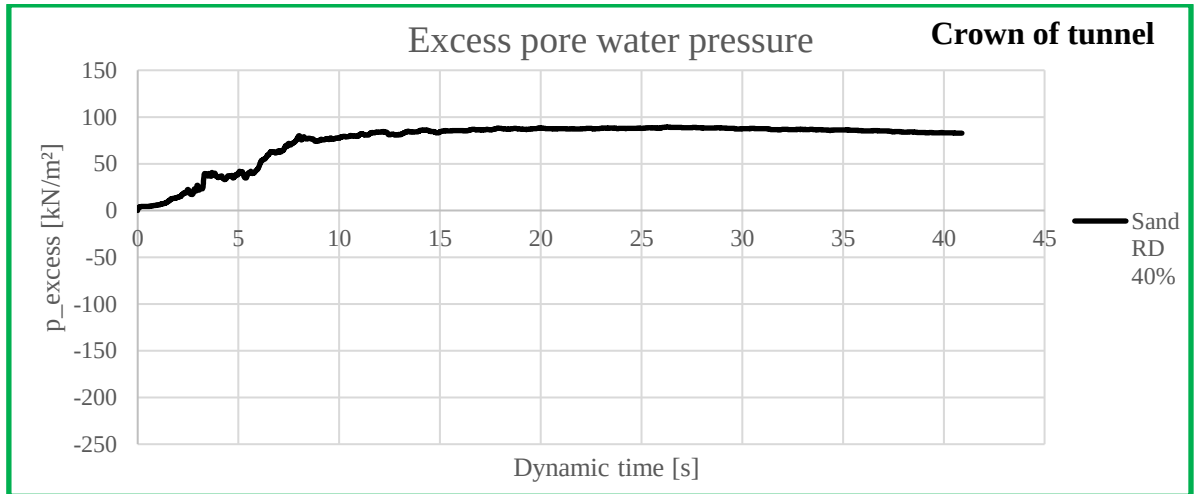


Figure 4.34: The excess pore water pressure at the crown of the tunnel for the sand of relative density 40% without any mitigation techniques (under Kobe earthquake).

From Figure 4.34, the excess pore water pressure increases through the dynamic time of the earthquake loading. The peak value of the excess pore water pressure is 89.1 kN/m^2 , but the excess pore water pressure is 82.7 kN/m^2 at the end of the earthquake.

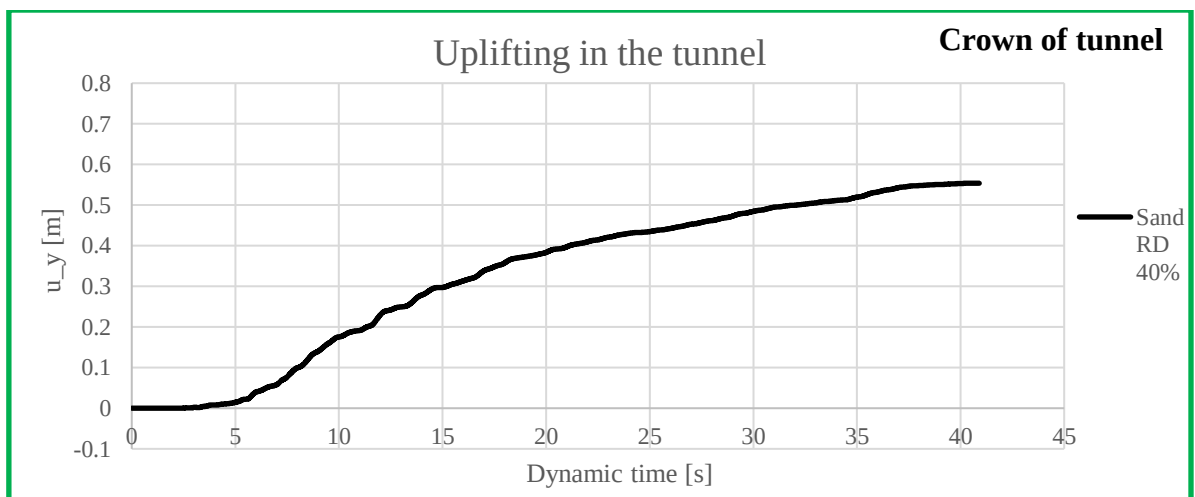


Figure 4.35: The uplifting at the crown of the tunnel for the sand of relative density 40% without any mitigation techniques (under Kobe earthquake).

From Figure 4.35, the uplifting in the tunnel increases through the dynamic time of the earthquake loading. The maximum uplifting in the tunnel is 0.554 m at the end of the earthquake.

Figure 4.36 and Figure 4.37 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of relative density 40% without any mitigation techniques.

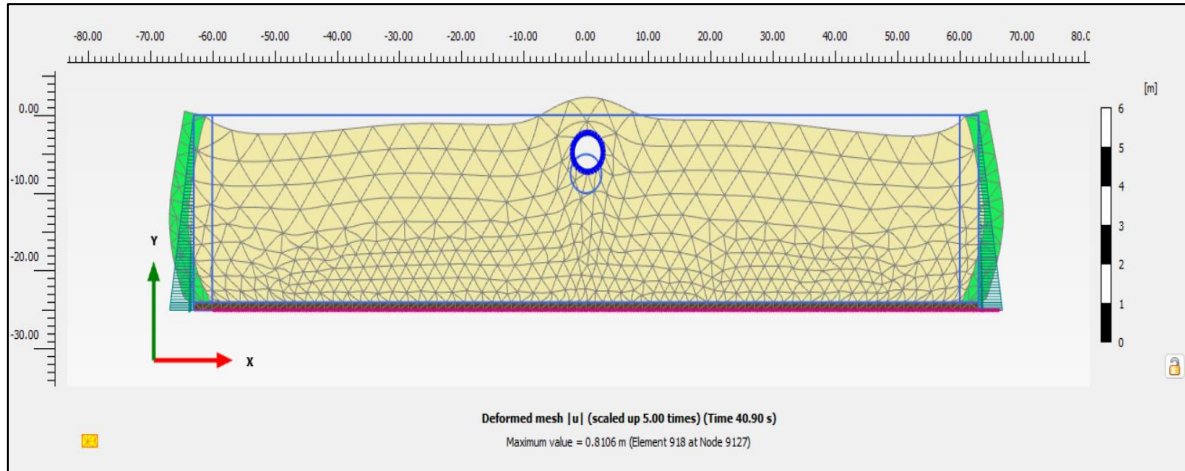


Figure 4.36: Deformation shape that occurs in a sand of relative density 40% without any mitigation techniques (under Kobe earthquake).

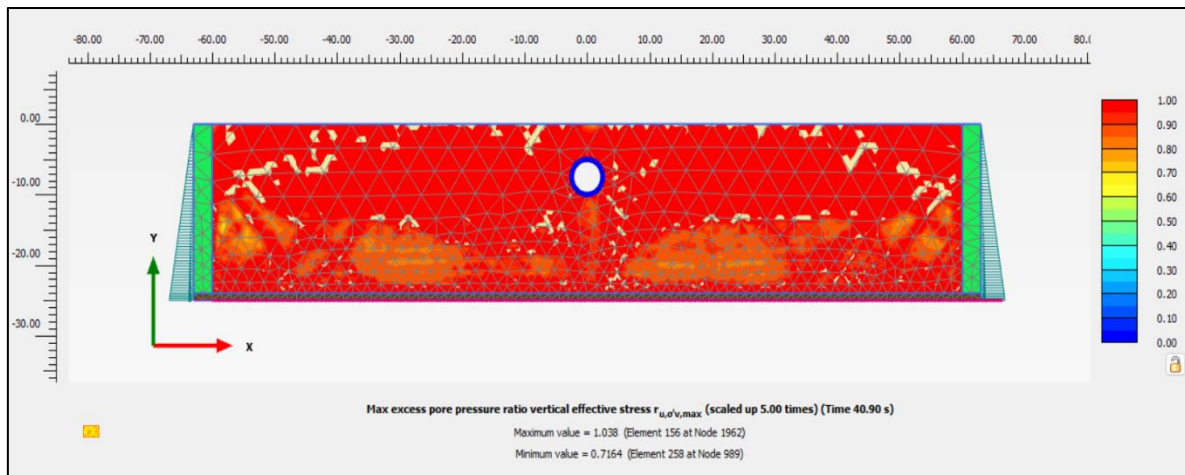


Figure 4.37: The shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of relative density 40% without any mitigation techniques (under Kobe earthquake).

From Figure 4.37, it was noticed that the excess pore water pressure ratio at the soil in the ground surface or in shallow depth is higher than the excess pore pressure ratio in the deep depth, that due to the effective confining stresses at the deep soil are higher, so the liquefaction has a higher severity at the shallow depths more than deep depths.

So, in the next steps are trying to mitigate the value of excess pore water pressure and the uplifting in the tunnel, by making some models with using mitigation techniques and comparing the value of these models with the value of previous model of sand RD 40% to see if these mitigation methods or techniques are effective or not, and to find which technique is the best.

4.2.1 Changing The Relative Density Of The Sand

The previous model of sand RD 40% was compared with other models by increasing the sand's relative density such as 55%, 65% and 75%, or by decreasing the sand's relative density such as 35%. The built up of the pore water pressure and the uplift of the tunnel were seen as in the following curves for both invert and crown of the tunnel.

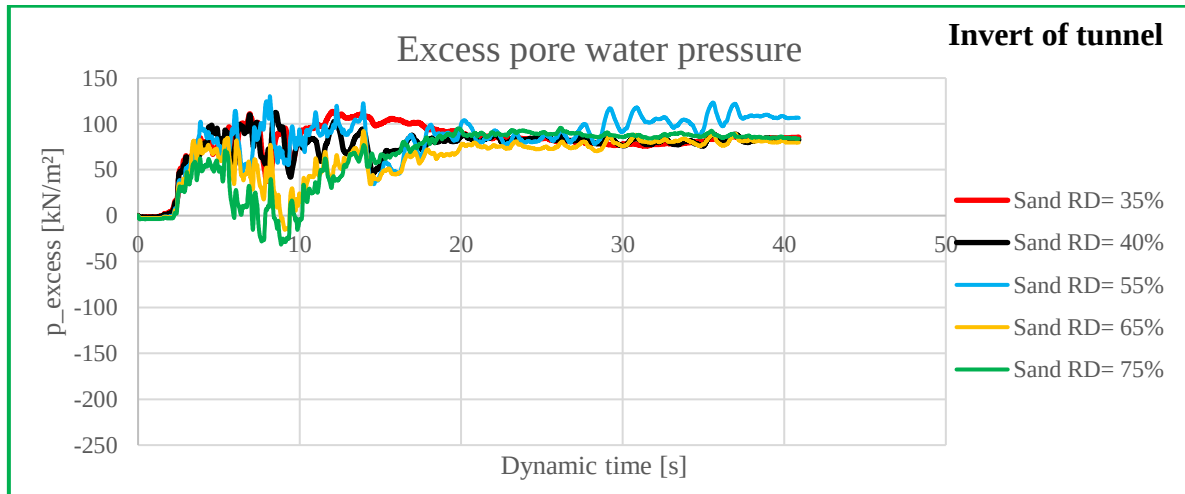


Figure 4.38: The excess pore water pressure at the invert of the tunnel for different relative densities of the sand (under Kobe earthquake).

From Figure 4.38, the excess pore water pressure increases through the dynamic time of the earthquake loading. And on the peak of the earthquake (between 5 s and 10 s) the excess pore water pressure is higher for the lower relative density, and it is lower for the higher relative

density, because the excess pore water pressure occurs in the loose sand more than in the dense sand. So, the excess pore water pressure can be mitigated by increasing the relative density of the sand.

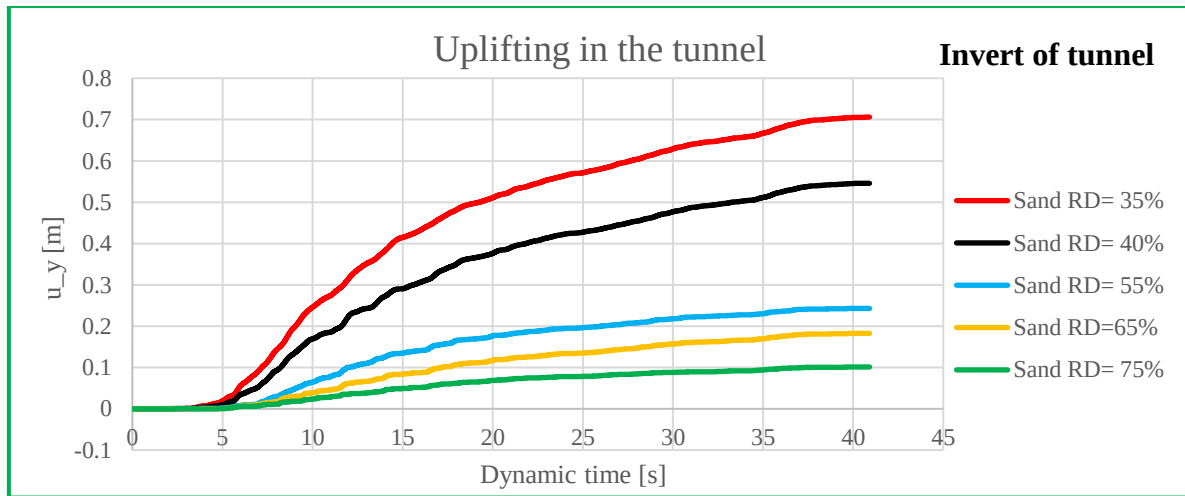


Figure 4.39: The uplifting at the invert of the tunnel for different relative densities of the sand (under Kobe earthquake).

From Figure 4.39, the uplifting in the tunnel increases through the dynamic time of the earthquake loading. And the uplifting in the tunnel is higher for the lower relative density, and it is lower for the higher relative density, because the uplifting in the tunnel occurs in the loose sand more than in the dense sand. So, the uplifting in the tunnel can be mitigated by increasing the relative density of the sand.

When comparing the uplifting in the tunnel at a sand of RD 40% with the sand of RD 35% and 75%, the uplifting in the tunnel is 0.546 m at sand of RD 40%, but it is 0.706 m when decrease the RD to 35%, and it is 0.101 m when increase the RD to 75%.

So, increase the relative density of the sand is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.40 and Figure 4.41.

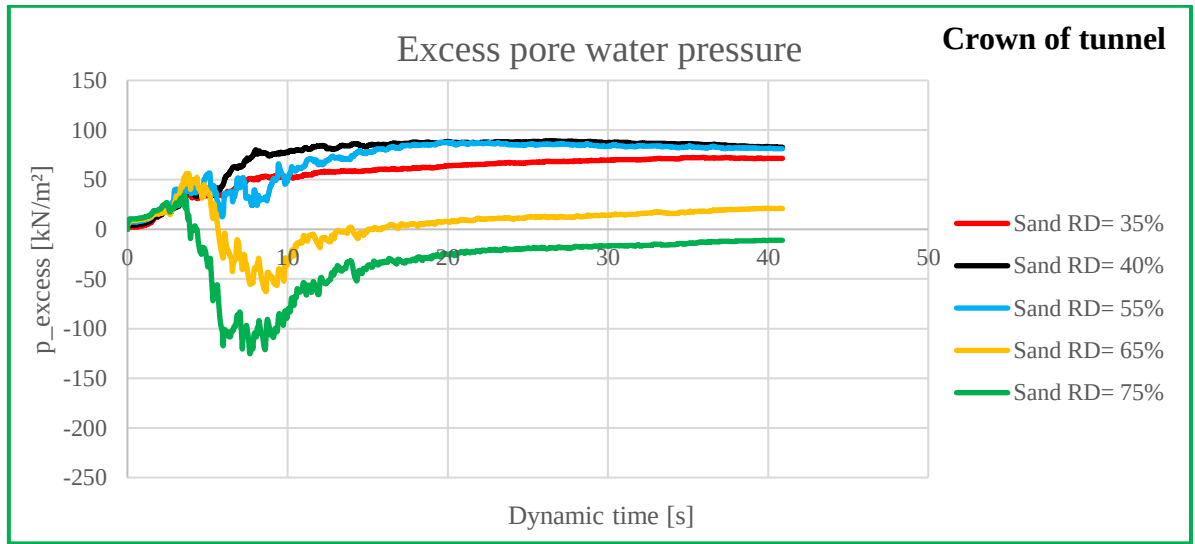


Figure 4.40: The excess pore water pressure at the crown of the tunnel for different relative densities of the sand (under Kobe earthquake).

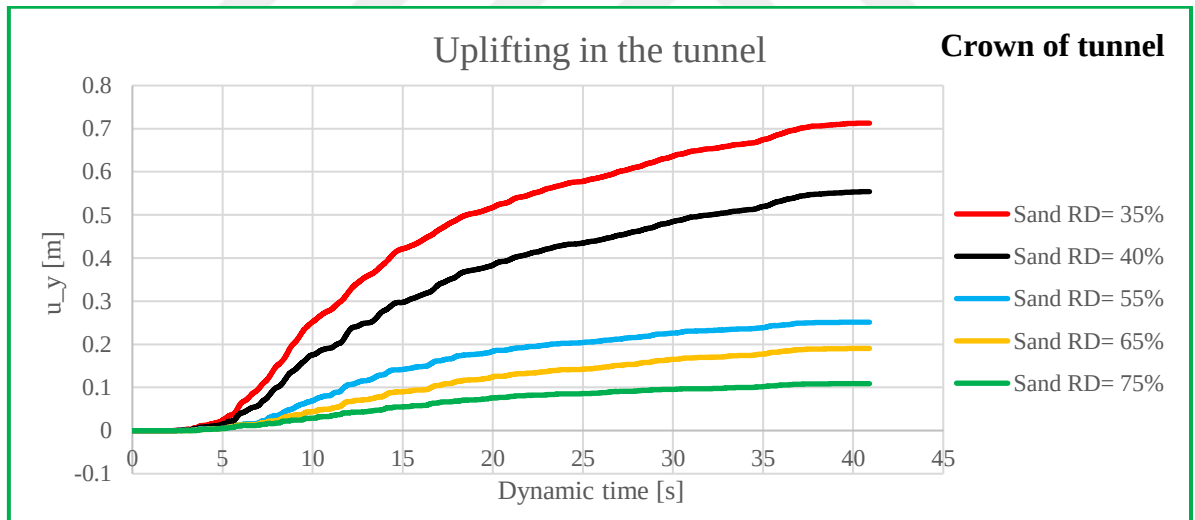


Figure 4.41: The uplifting at the crown of the tunnel for different relative densities of the sand (under Kobe earthquake).

4.2.2 Replacing The Sand Around The Tunnel By Higher Relative Density Sand

This method was done by using the sand RD 40% but replacing the sand around the tunnel by a higher relative density such as 75%, and then comparing the results between the sand with replacing and without replacing.

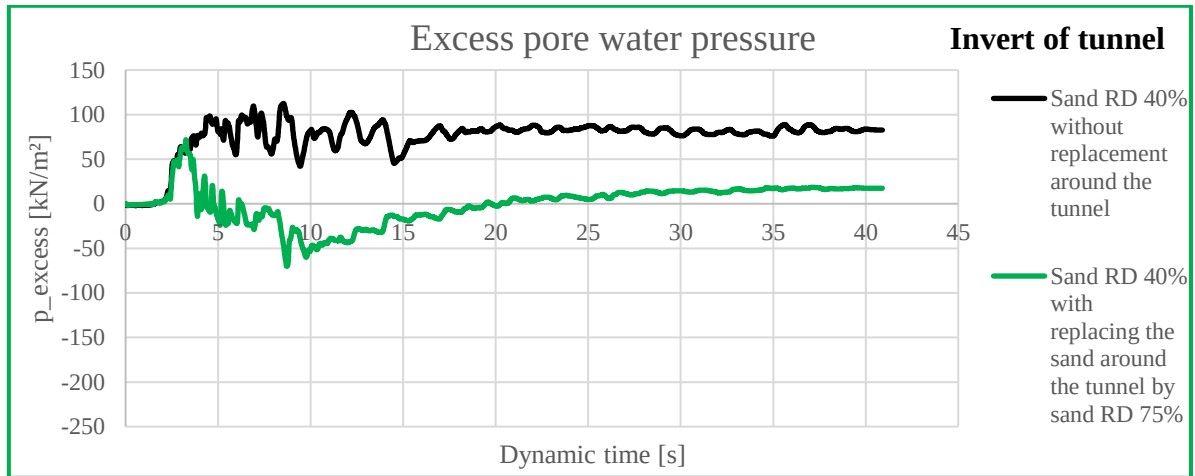


Figure 4.42: The excess pore water pressure at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kobe earthquake).

From Figure 4.42, the excess pore water pressure was decreased when replacing the sand around the tunnel by a higher relative density sand, such as replacing the sand of RD 40% by a sand of RD 75% around the tunnel.

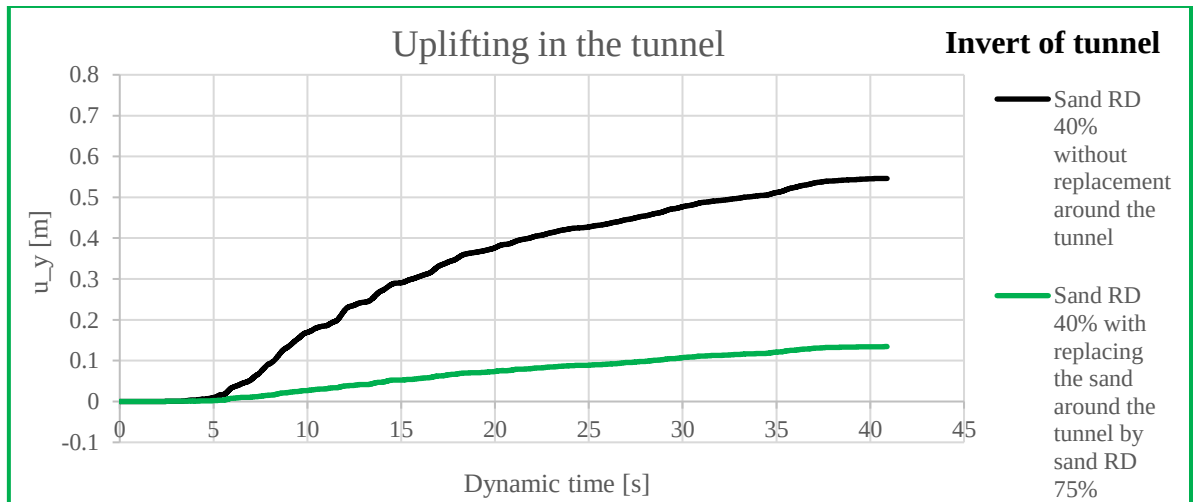


Figure 4.43: The uplifting at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kobe earthquake).

From Figure 4.43, the uplifting in the tunnel was decreased when replacing the sand around the tunnel by a higher relative density sand, such as replacing the sand of RD 40% by a sand of RD 75% around the tunnel.

The maximum uplifting in the tunnel is 0.546 m in a sand of RD 40% without replacement, but it is 0.134 m in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75%.

So, the replacing of the sand around the tunnel by a higher relative density sand is a successful way to mitigate of liquefaction-induced uplifts in tunnel.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.44 and Figure 4.45.

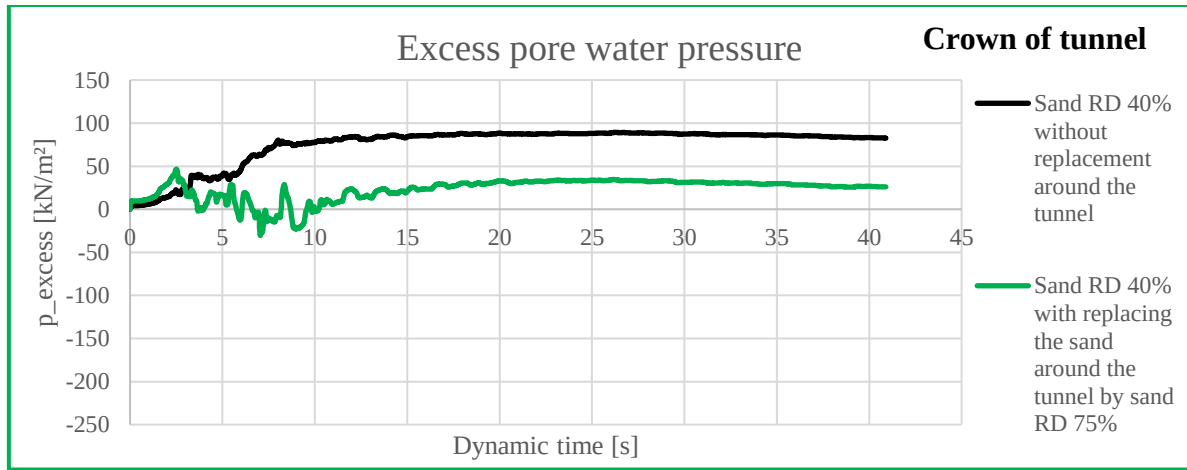


Figure 4.44: The excess pore water pressure at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kobe earthquake).

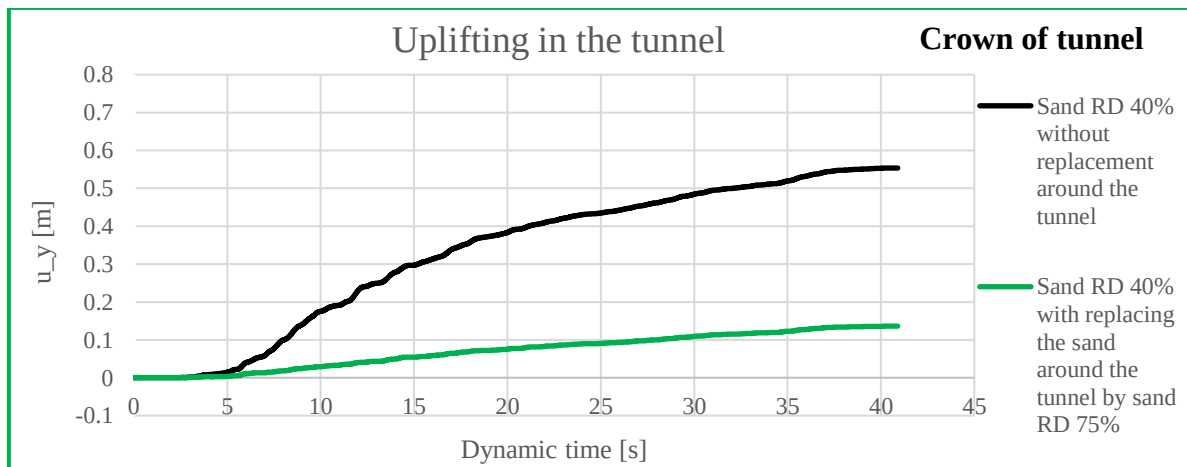


Figure 4.45: The uplifting at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kobe earthquake).

Figure 4.46 and Figure 4.47 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75%.

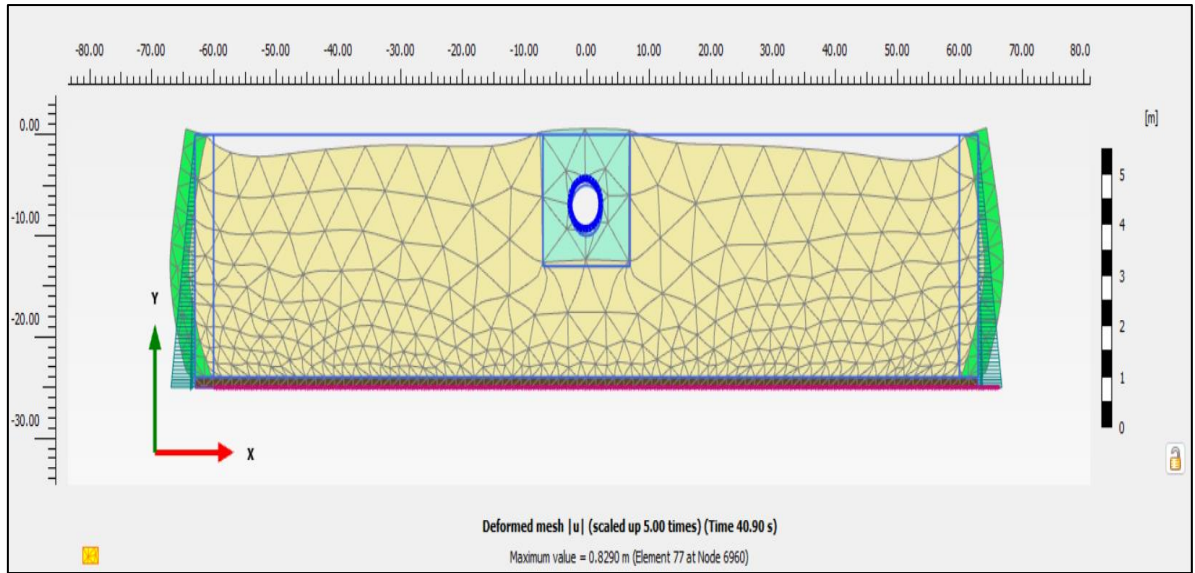


Figure 4.46: Deformation shape that occurs in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kobe earthquake).

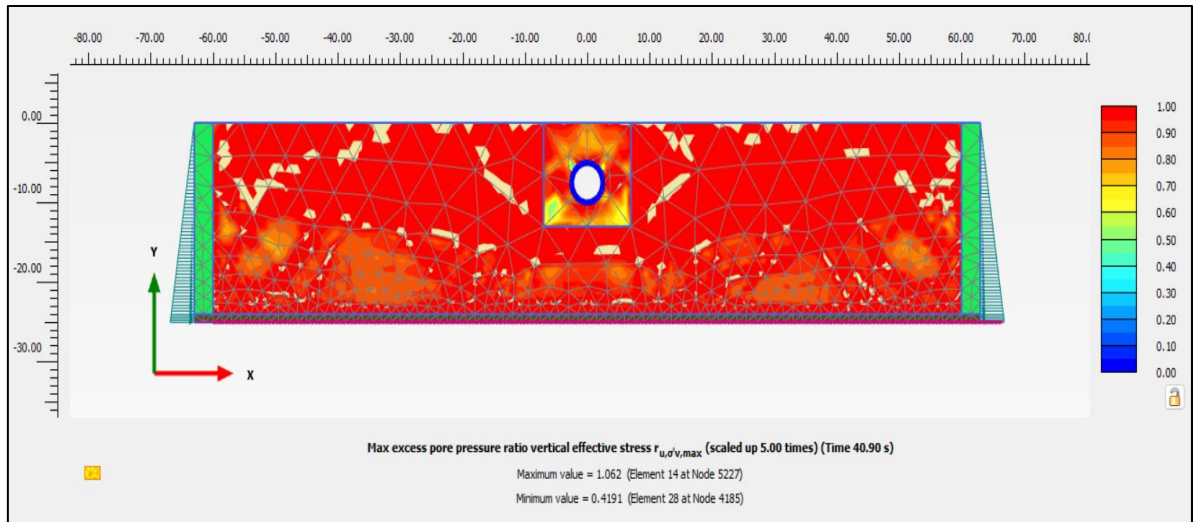


Figure 4.47: The shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Kobe earthquake).

4.2.3 Replacing The Sand Around The Tunnel By Clay Soil

This method was done by using the sand RD 40% but replacing the sand around the tunnel by clay soil, and then comparing the results between the sand with replacing and without replacing.

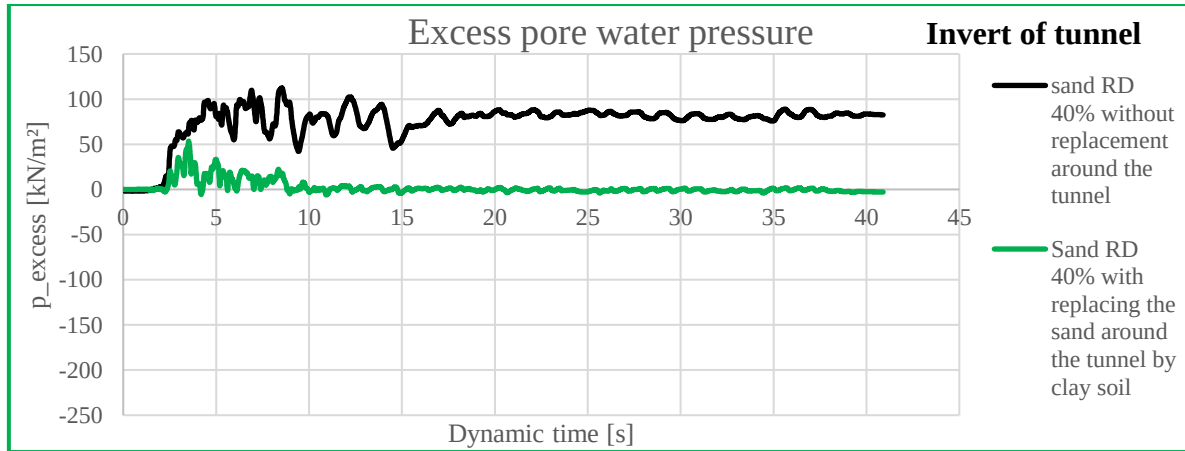


Figure 4.48: The excess pore water pressure at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kobe earthquake).

From Figure 4.48, the excess pore water pressure was decreased when replacing the sand around the tunnel by clay soil, because the liquefaction doesn't occur in the clay soil.

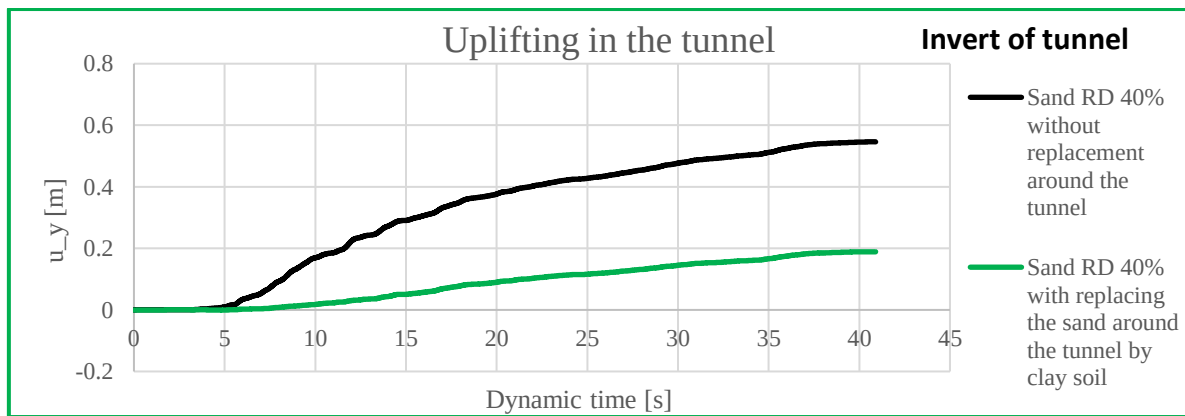


Figure 4.49: The uplifting at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kobe earthquake).

From Figure 4.49, the uplifting in the tunnel was decreased when replacing the sand around the tunnel by a clay soil, because the liquefaction doesn't occur in the clay soil.

The maximum uplifting in the tunnel is 0.546 m in a sand of RD 40% without replacement, but it is 0.189 m in a sand of RD 40% with replacing the sand around the tunnel by clay soil.

So, the replacing of the sand around the tunnel by a clay soil is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.50 and Figure 4.51.

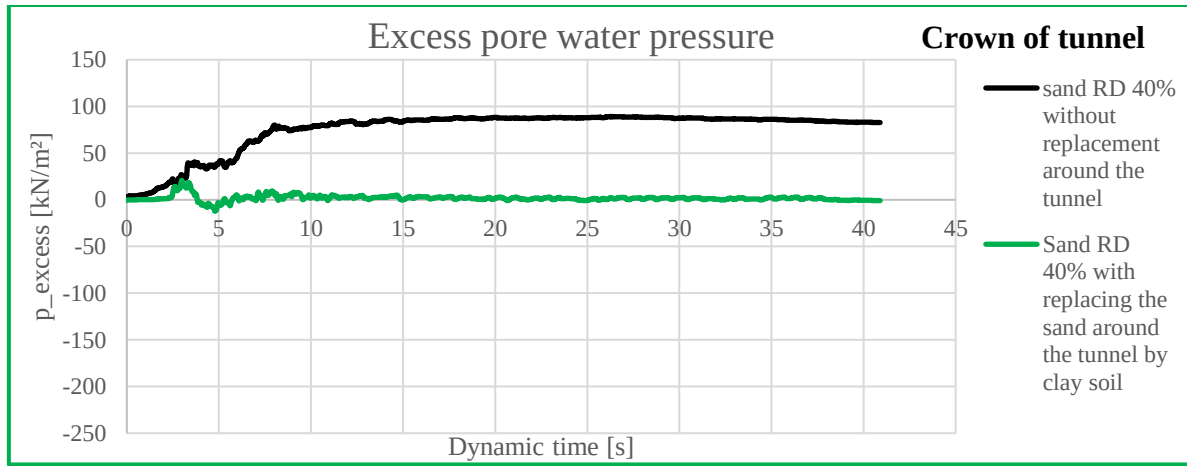


Figure 4.50: The excess pore water pressure at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kobe earthquake).

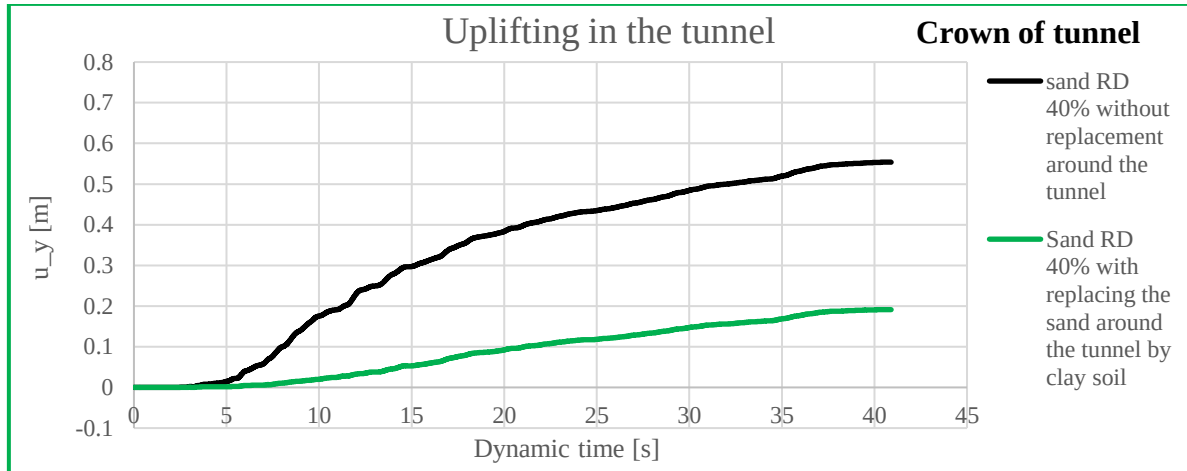


Figure 4.51: The uplifting at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kobe earthquake).

Figure 4.52 and Figure 4.53 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'_v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by clay soil. It was observed that the excess pore water pressure doesn't appear on the clay soil as in Figure 4.53.

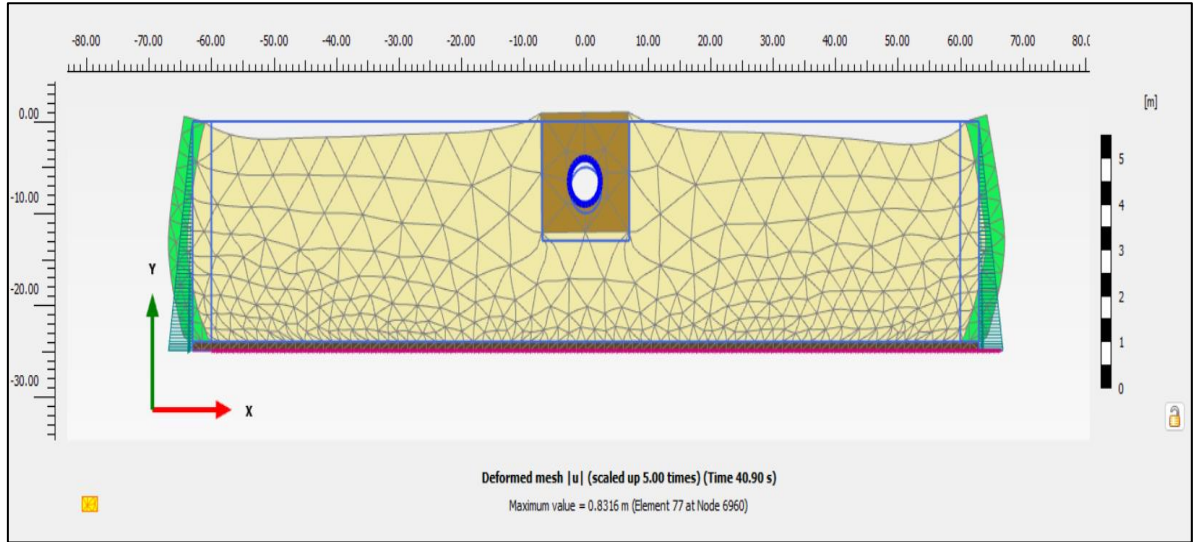


Figure 4.52: Deformation shape that occurs in a sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kobe earthquake).

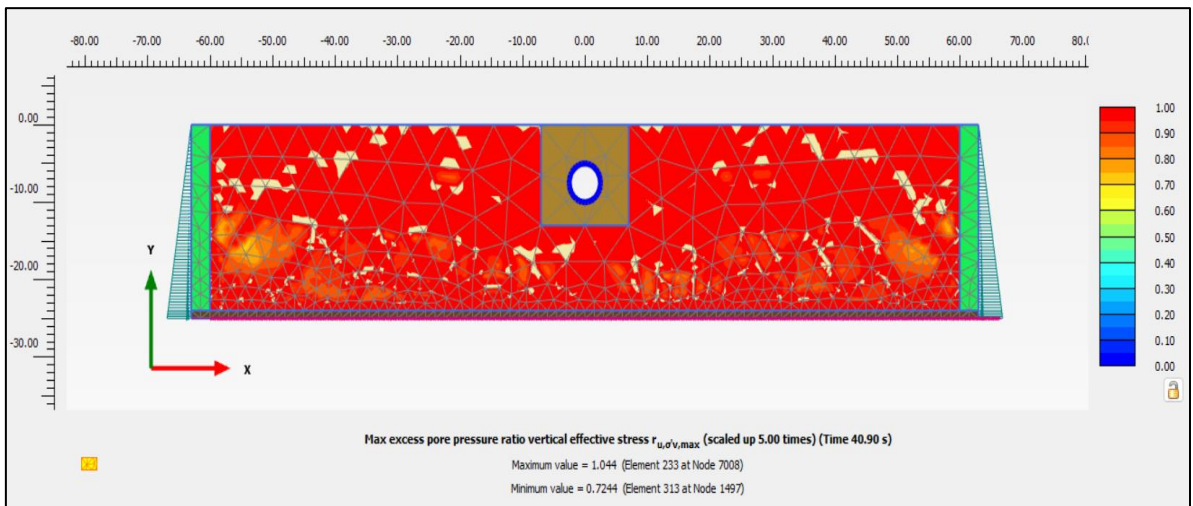


Figure 4.53: The shape of max excess pore pressure ratio ($r_{u,\sigma'_v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by clay soil (under Kobe earthquake).

4.2.4 Replacing The Sand Around The Tunnel By Gravel Soil

This method was done by using the sand RD 40% but replacing the sand around the tunnel by gravel soil, and then comparing the results between the sand with replacing and without replacing.

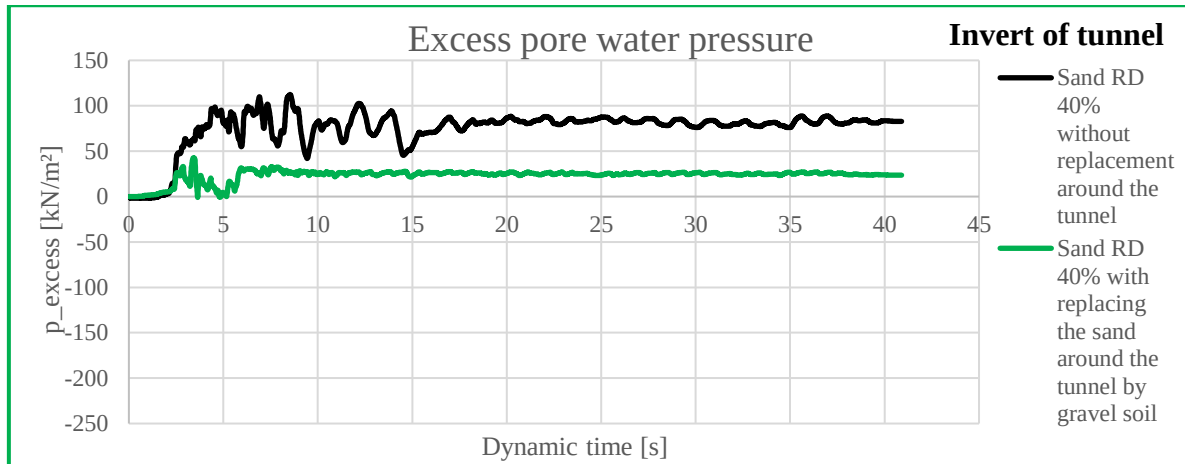


Figure 4.54: The excess pore water pressure at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kobe earthquake).

From Figure 4.54, the excess pore water pressure was decreased when replacing the sand around the tunnel by gravel soil, because the liquefaction doesn't occur in the gravel soil.

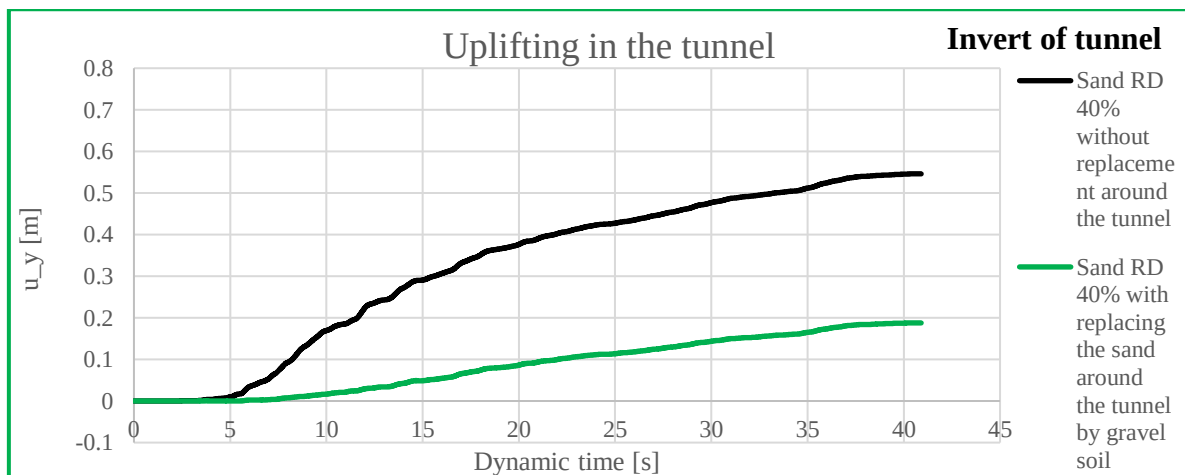


Figure 4.55: The uplifting at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kobe earthquake).

From Figure 4.55, the uplifting in the tunnel was decreased when replacing the sand around the tunnel by a gravel soil, because the liquefaction doesn't occur in the gravel soil. The maximum uplifting in the tunnel is 0.546 m in a sand of RD 40% without replacement, but it is 0.188 m in a sand of RD 40% with replacing the sand around the tunnel by gravel soil. So, the replacing of the sand around the tunnel by a gravel soil is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.56 and Figure 4.57.

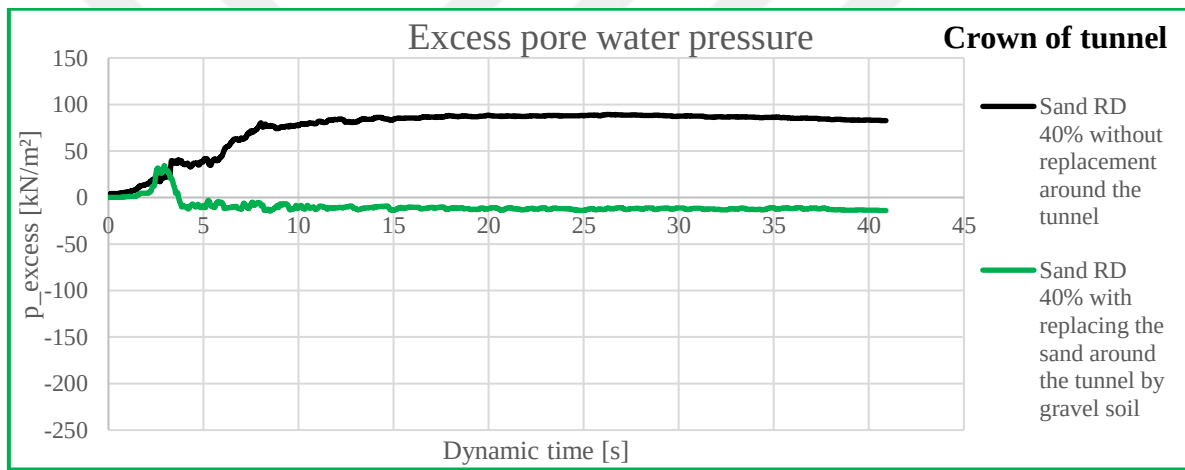


Figure 4.56: The excess pore water pressure at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kobe earthquake).

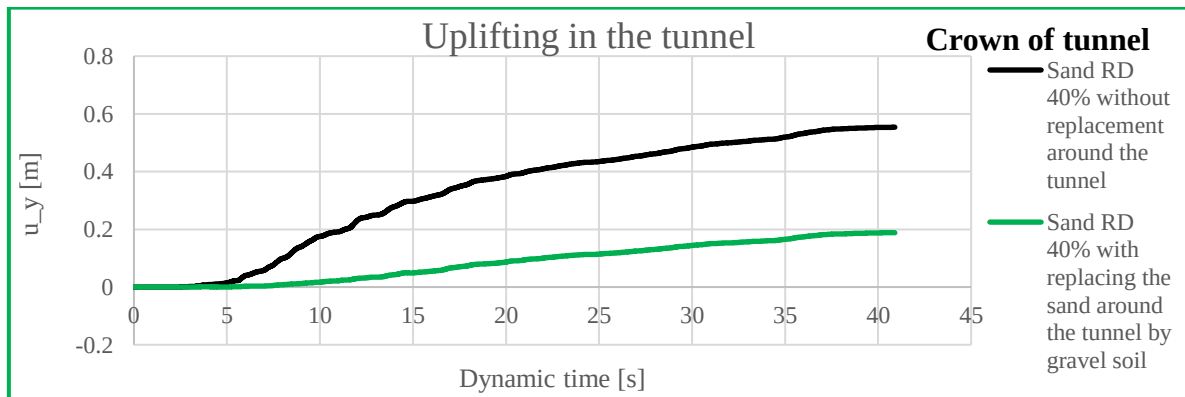


Figure 4.57: The uplifting at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kobe earthquake).

Figure 4.58 and Figure 4.59 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by gravel soil. It was observed that the excess pore water pressure doesn't appear on the gravel soil as in Figure 4.59.

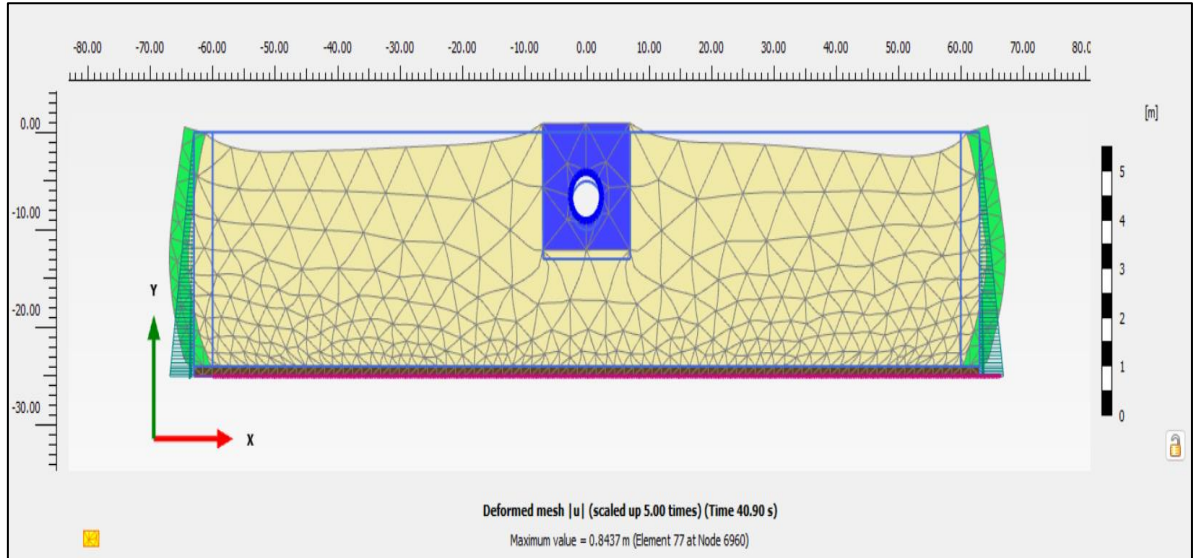


Figure 4.58: Deformation shape that occurs in a sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kobe earthquake).

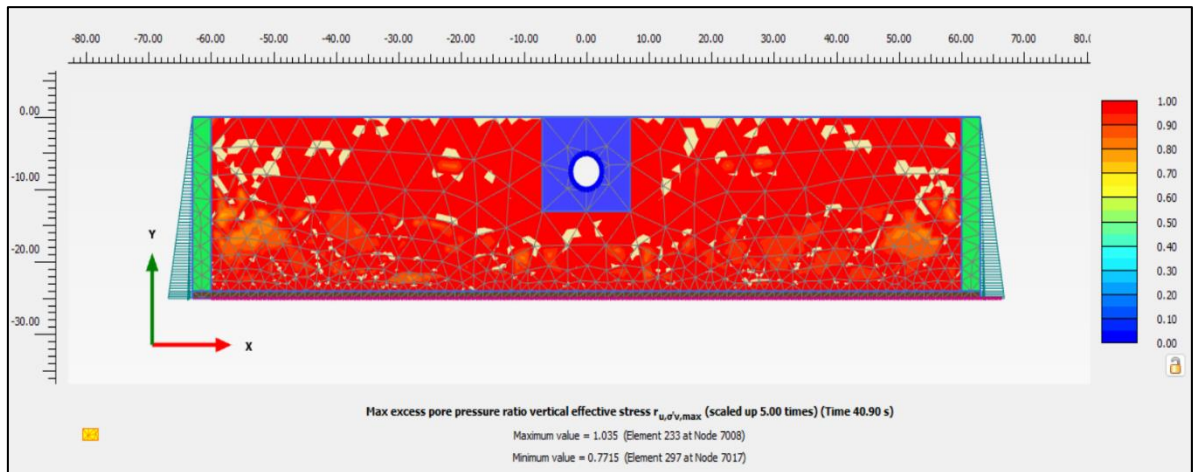


Figure 4.59: The shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Kobe earthquake).

Figure 4.60 and Figure 4.61 illustrate the comparison between the previous mitigation techniques, and it was noticed that the technique of increase the relative density of the sand to 75% has advantageous for mitigation the soil liquefaction and for mitigation the uplift of the tunnel, and it was noticed that replacing the sand around the tunnel by gravel soil is better than replacing it by clay soil, because the gravel soil is better for drainage the water and dissipating the excess pore water pressure at the soil

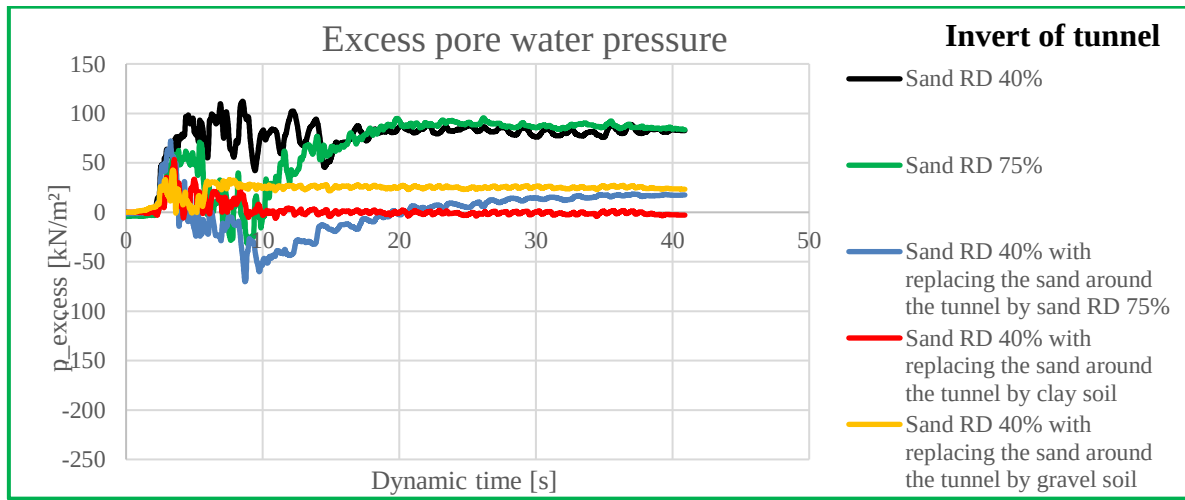


Figure 4.60: The comparison of the excess pore water pressure between the techniques that were used for mitigation the soil liquefaction and the uplift of the tunnel (under Kobe earthquake).

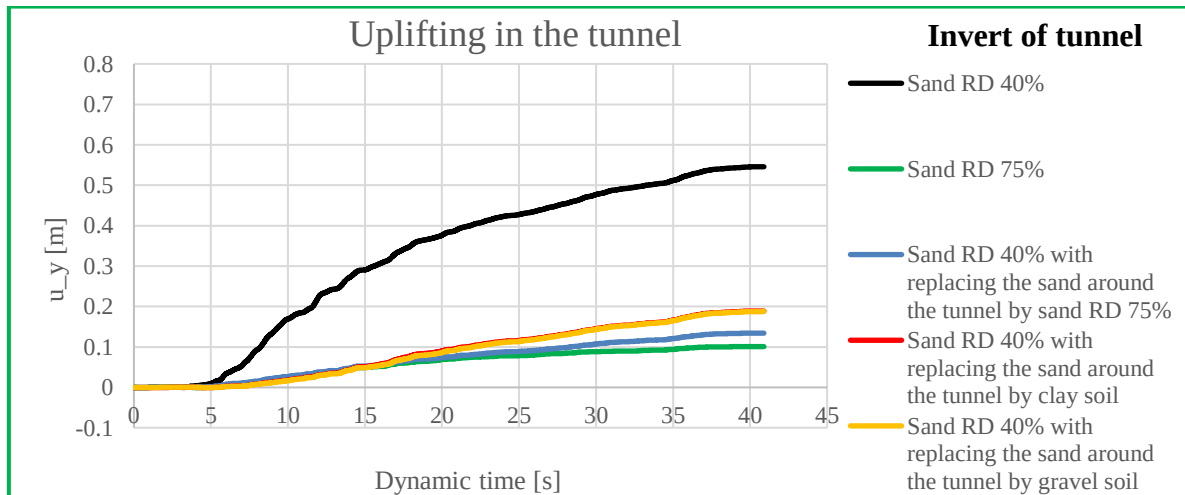


Figure 4.61: The comparison of the uplift of the tunnel between the techniques that were used for mitigation the soil liquefaction and the uplift of the tunnel (under Kobe earthquake).

4.3 UNDER NORTHRIDGE EARTHQUAKE

When the earthquake occurs on a sand of relative density 40%, the built up of the pore water pressure and the uplift of the tunnel were seen as in the following curves for both invert and crown of the tunnel.

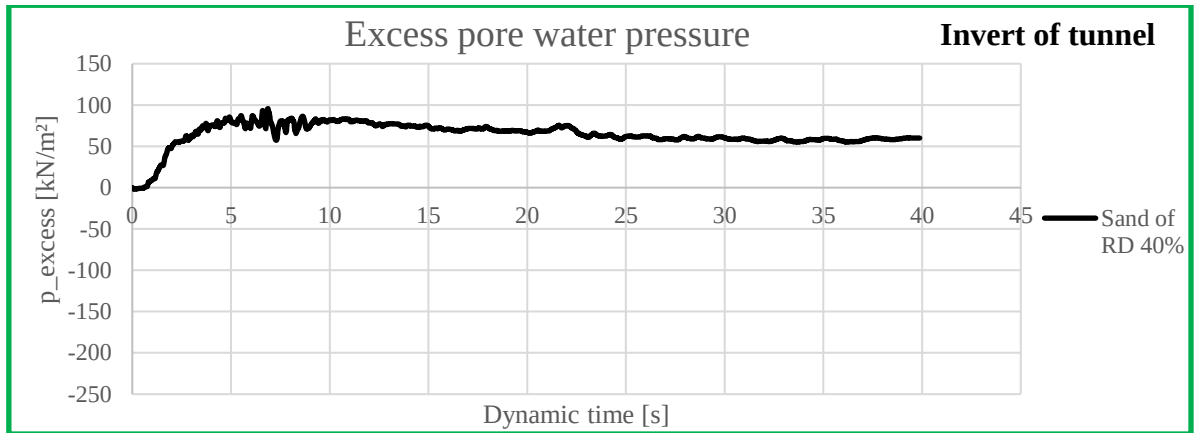


Figure 4.62: The excess pore water pressure at the invert of the tunnel for the sand of relative density 40% without any mitigation techniques (under Northridge earthquake).

From Figure 4.62, the excess pore water pressure increases through the dynamic time of the earthquake loading. The peak value of the excess pore water pressure is 95.5 kN/m², but the excess pore water pressure is 59.9 kN/m² at the end of the earthquake.

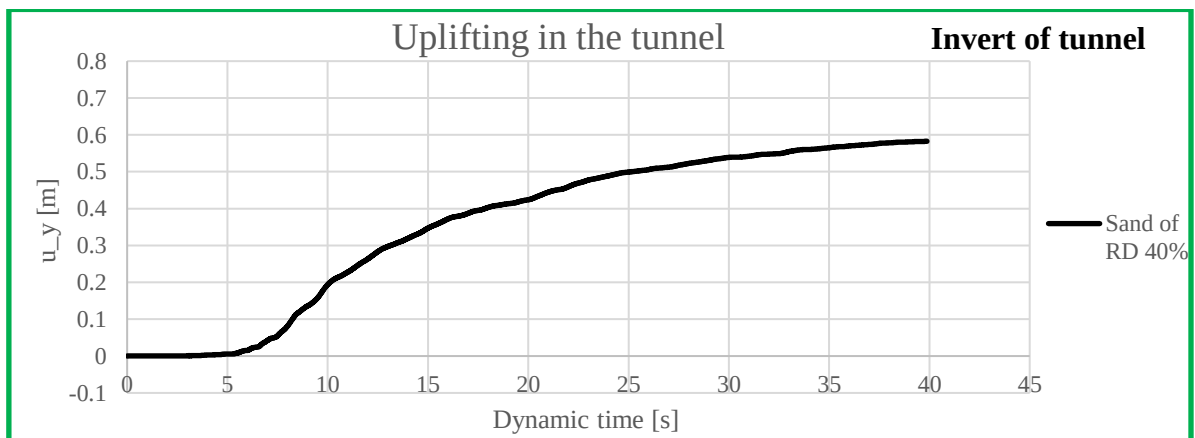


Figure 4.63: The uplifting at the invert of the tunnel for the sand of relative density 40% without any mitigation techniques (under Northridge earthquake).

From figure 4.63, the uplifting in the tunnel increases through the dynamic time of the earthquake loading. The maximum uplifting in the tunnel is 0.583 m at the end of the earthquake.

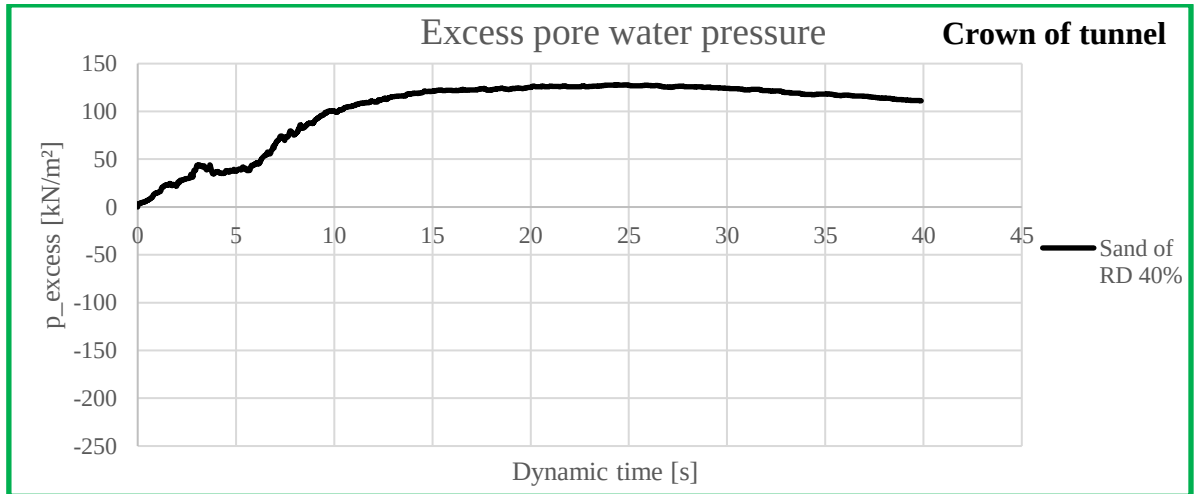


Figure 4.64: The excess pore water pressure at the crown of the tunnel for the sand of relative density 40% without any mitigation techniques (under Northridge earthquake).

From Figure 4.64, the excess pore water pressure increases through the dynamic time of the earthquake loading. The peak value of the excess pore water pressure is 128 kN/m^2 , but the excess pore water pressure is 111 kN/m^2 at the end of the earthquake.

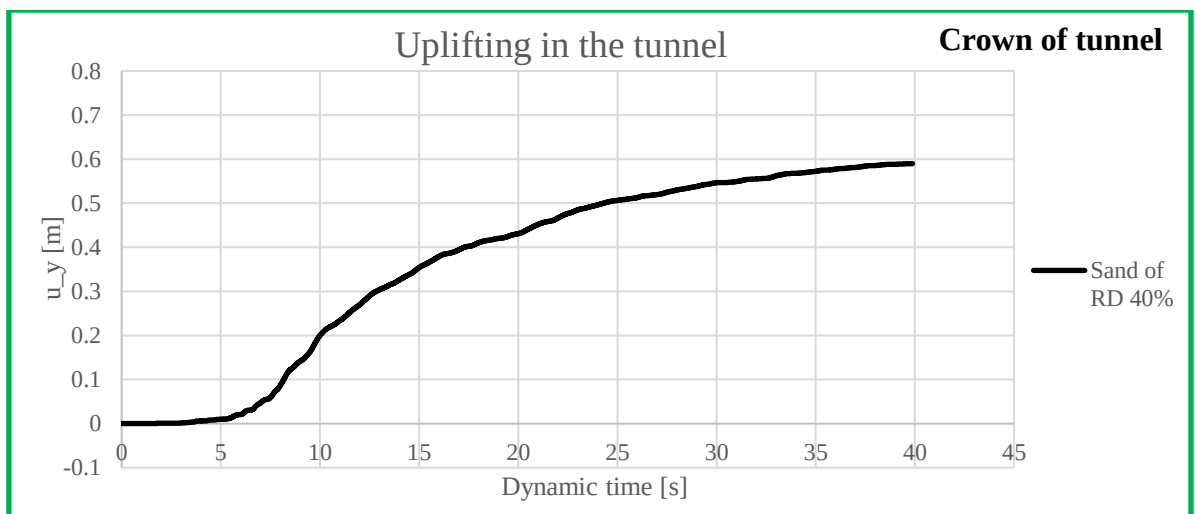


Figure 4.65: The uplifting at the crown of the tunnel for the sand of relative density 40% without any mitigation techniques (under Northridge earthquake).

From Figure 4.65, the uplifting in the tunnel increases through the dynamic time of the earthquake loading. The maximum uplifting in the tunnel is 0.590 m at the end of the earthquake.

Figure 4.66 and Figure 4.67 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of relative density 40% without any mitigation techniques.

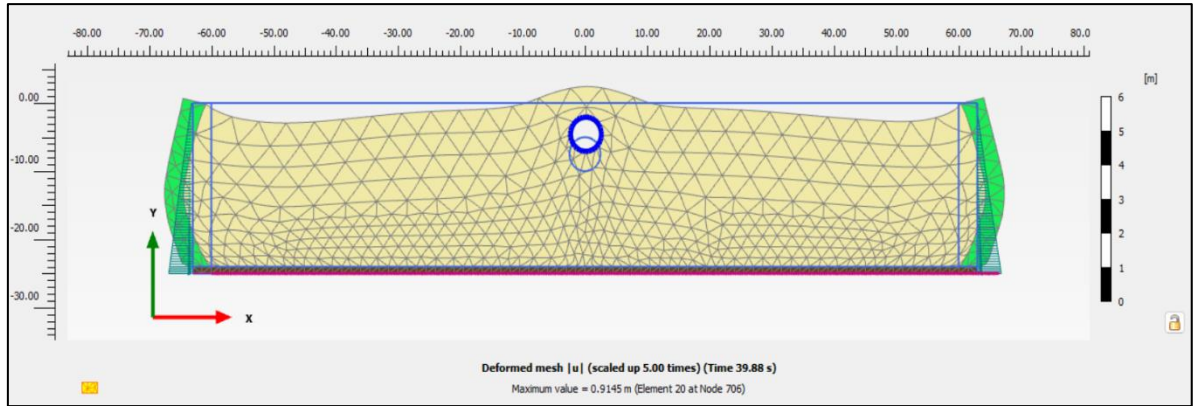


Figure 4.66: The deformation shape that occurs in a sand of relative density 40% without any mitigation techniques (under Northridge earthquake).

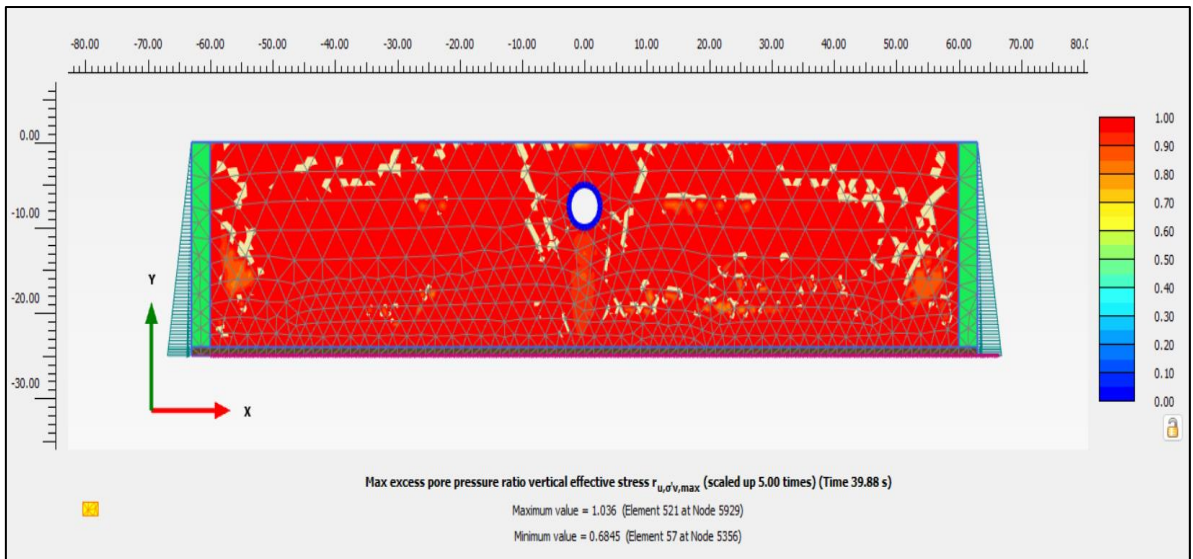


Figure 4.67: The shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of relative density 40% without any mitigation techniques (under Northridge earthquake).

So, in the next steps are trying to mitigate the value of excess pore water pressure and the uplifting in the tunnel, by making some models with using mitigation techniques and comparing the value of these models with the value of previous model of sand RD 40% to see if these mitigation methods or techniques are effective or not, and to find which technique is the best.

4.3.1 Changing The Relative Density Of The Sand

The previous model of sand RD 40% was compared with other models by increasing the sand's relative density such as 55%, 65% and 75%, or by decreasing the sand's relative density such as 35%. the built up of the pore water pressure and the uplift of the tunnel were seen as in the following curves for both invert and crown of the tunnel.

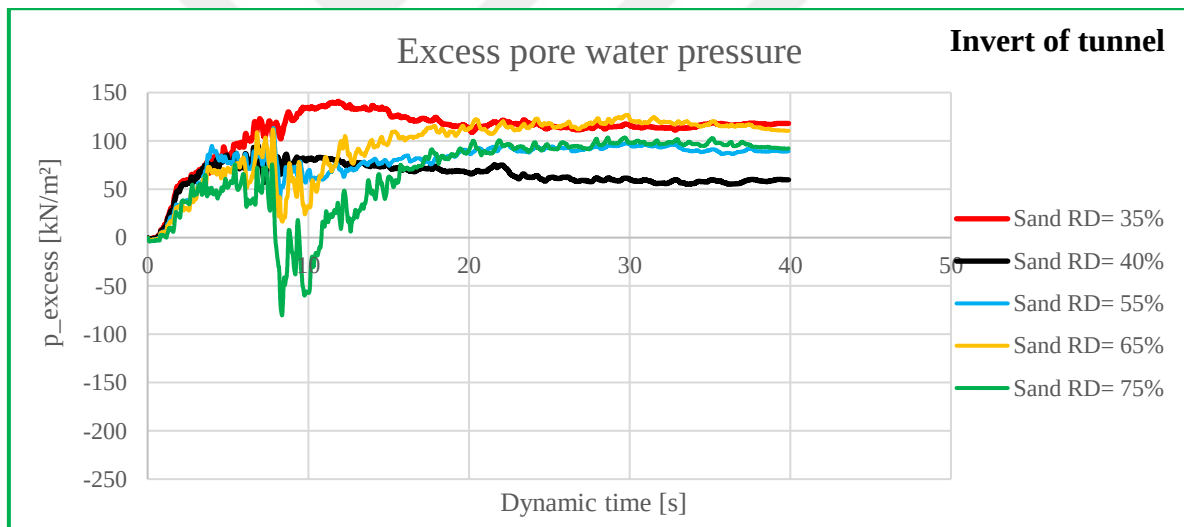


Figure 4.68: The excess pore water pressure at the invert of the tunnel for different relative densities of the sand (under Northridge earthquake).

From Figure 4.68, the excess pore water pressure increases through the dynamic time of the earthquake loading. And on the peak of the earthquake (between 5 s and 15 s) the excess pore water pressure is higher for the lower relative density, and it is lower for the higher relative density, because the excess pore water pressure occurs in the loose sand more than in the dense sand. So, the excess pore water pressure can be mitigated by increasing the relative density of the sand.

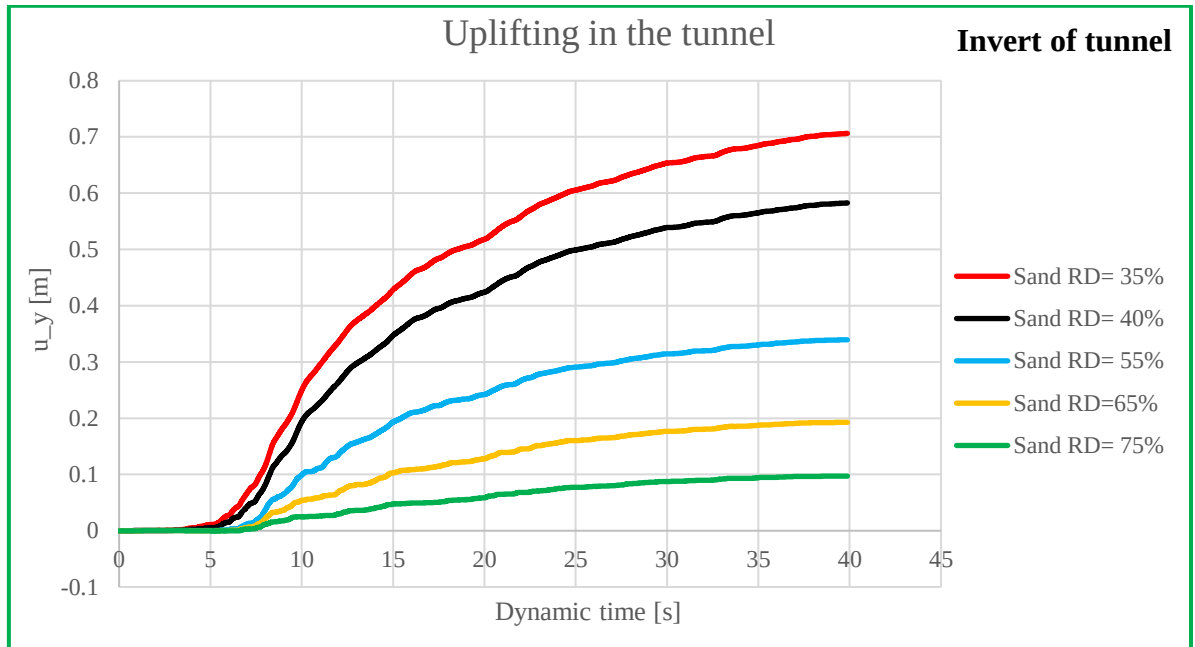


Figure 4.69: The uplifting at the invert of the tunnel for different relative densities of the sand (under Northridge earthquake).

From Figure 4.69, the uplifting in the tunnel increases through the dynamic time of the earthquake loading. And the uplifting in the tunnel is higher for the lower relative density, and it is lower for the higher relative density, because the uplifting in the tunnel occurs in the loose sand more than in the dense sand. So, the uplifting in the tunnel can be mitigated by increasing the relative density of the sand.

When comparing the uplifting in the tunnel at a sand of RD 40% with the sand of RD 35% and 75%, the uplifting in the tunnel is 0.583 m at sand of RD 40%, but it is 0.706 m when decrease the RD to 35%, and it is 0.0969 m when increase the RD to 75%.

So, increase the relative density of the sand is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.70 and Figure 4.71.

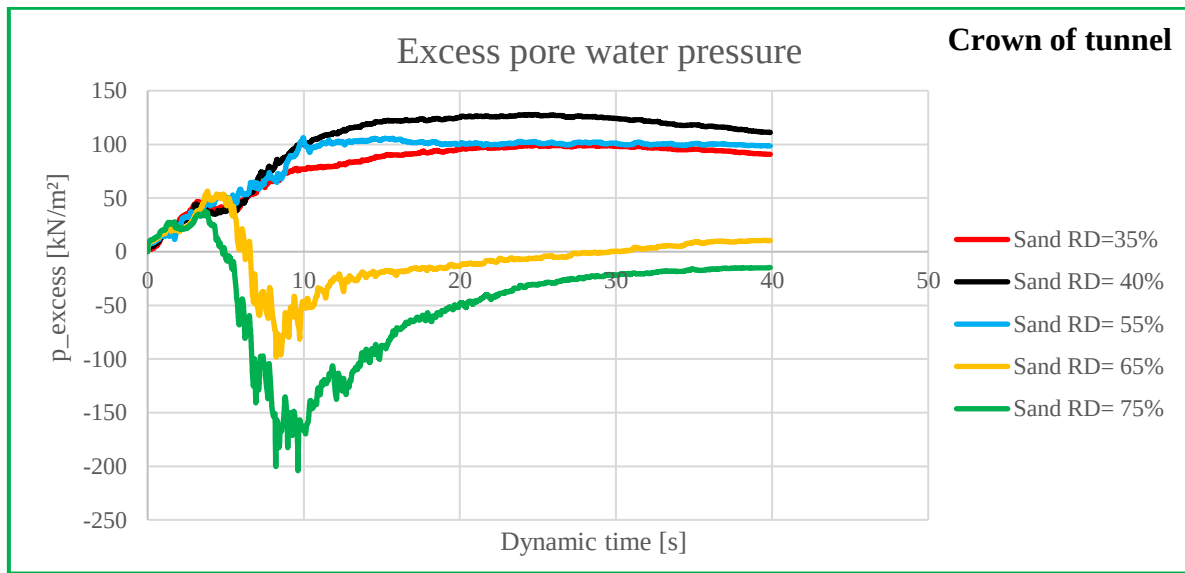


Figure 4.70: The excess pore water pressure at the crown of the tunnel for different relative densities of the sand (under Northridge earthquake).

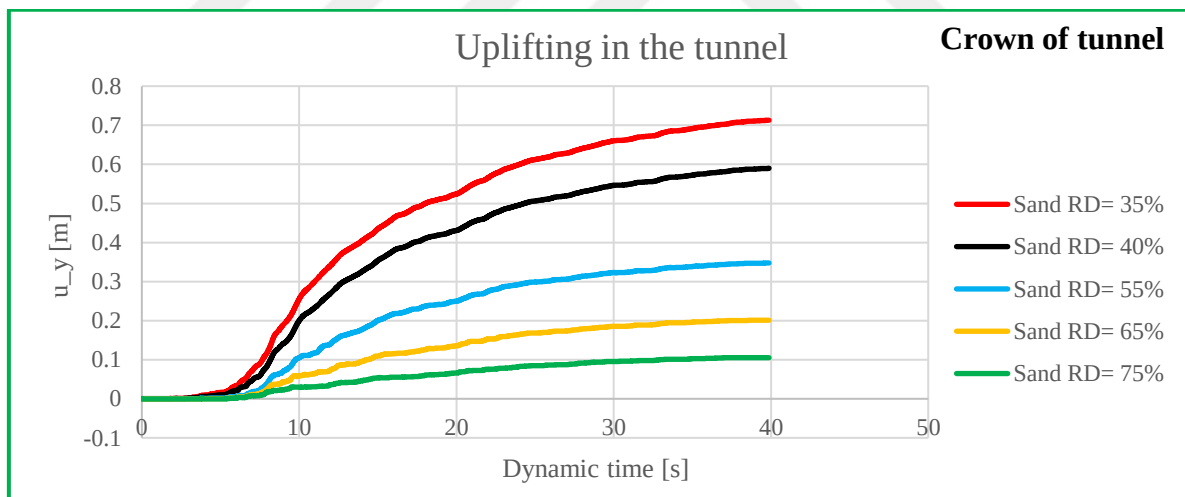


Figure 4.71: The uplifting at the crown of the tunnel for different relative densities of the sand (under Northridge earthquake).

4.3.2 Replacing The Sand Around The Tunnel By A Higher Relative Density Sand

This method was done by using the sand RD 40% but replacing the sand around the tunnel by a higher relative density such as 75%, and then comparing the results between the sand with replacing and without replacing.

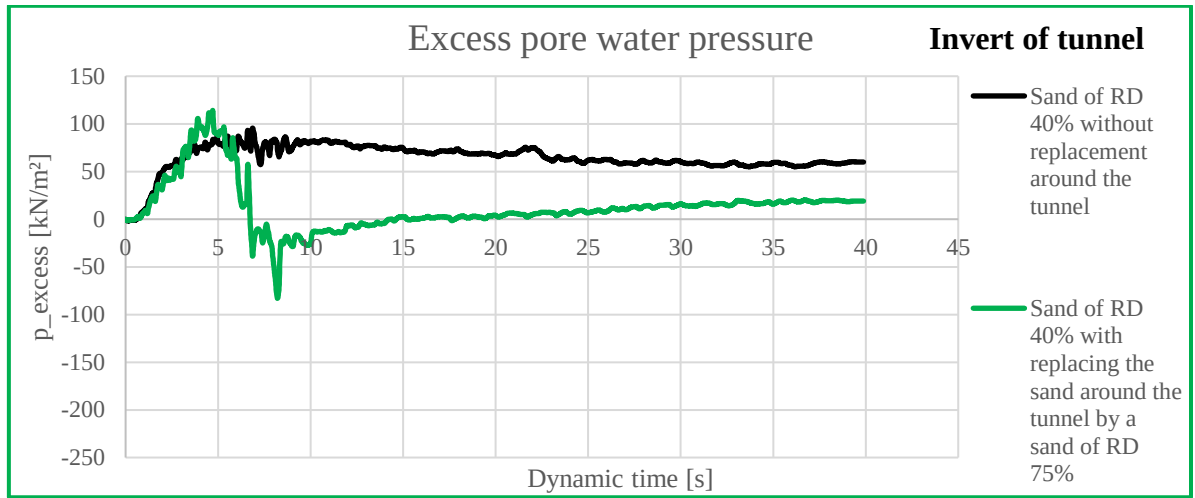


Figure 4.72: The excess pore water pressure at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Northridge earthquake).

From Figure 4.72, the excess pore water pressure was decreased when replacing the sand around the tunnel by a higher relative density sand, such as replacing the sand of RD 40% by a sand of RD 75% around the tunnel.

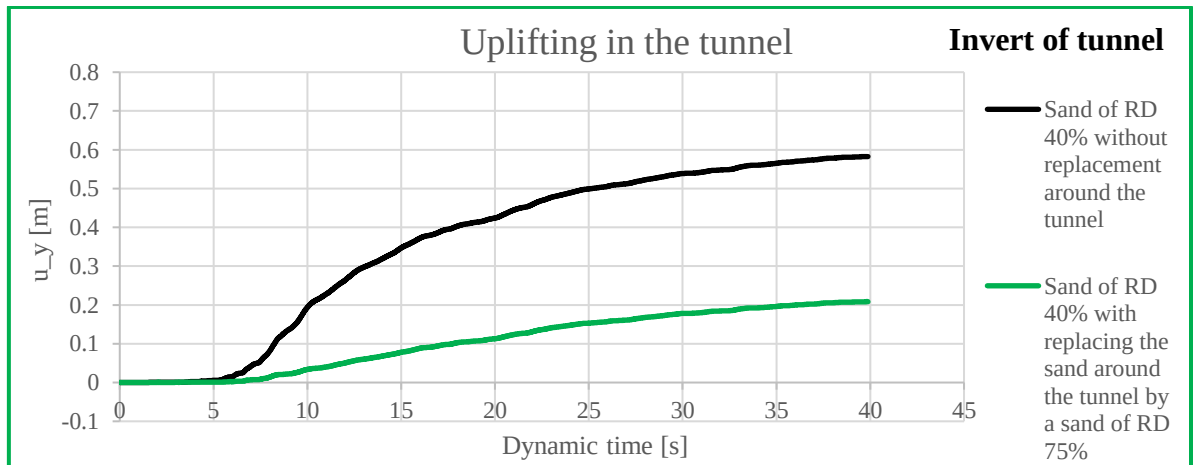


Figure 4.73: The uplifting at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Northridge earthquake).

From Figure 4.73, the uplifting in the tunnel was decreased when replacing the sand around the tunnel by a higher relative density sand, such as replacing the sand of RD 40% by a sand of RD 75% around the tunnel.

The maximum uplifting in the tunnel is 0.583 m in a sand of RD 40% without replacement, but it is 0.208 m in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75%.

So, the replacing of the sand around the tunnel by a higher relative density sand is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.74 and Figure 4.75.

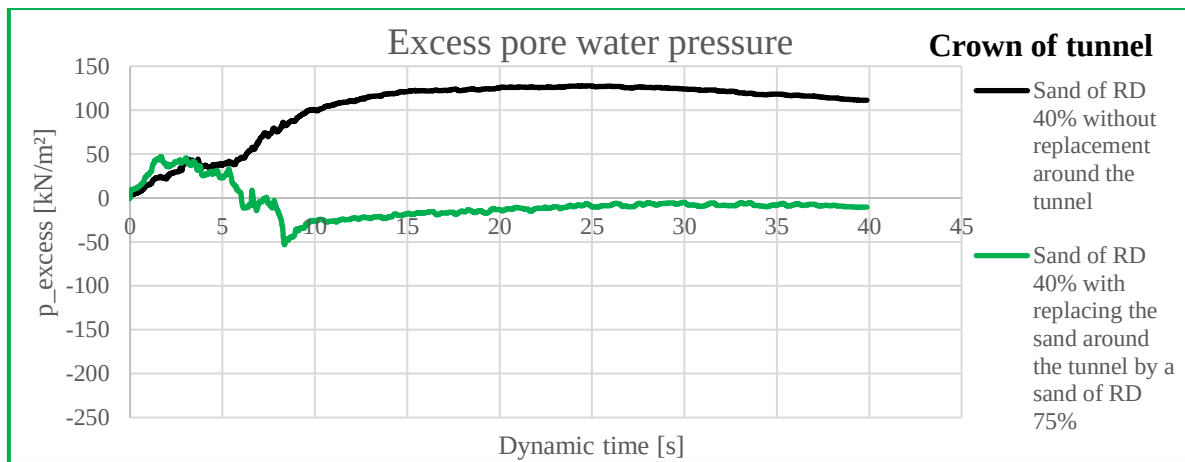


Figure 4.74: The excess pore water pressure at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Northridge earthquake).

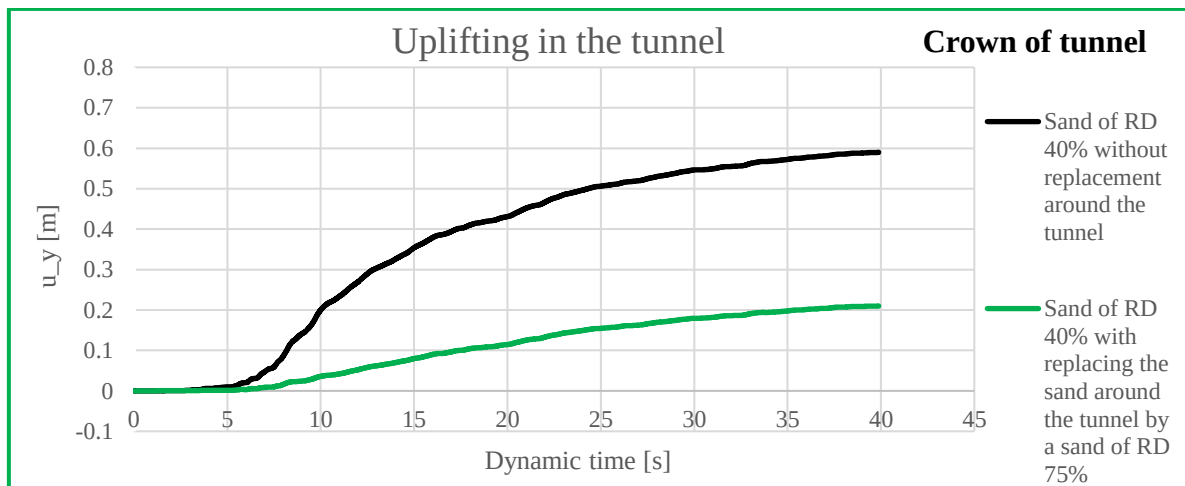


Figure 4.75: The uplifting at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Northridge earthquake).

Figure 4.76 and Figure 4.77 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75%.

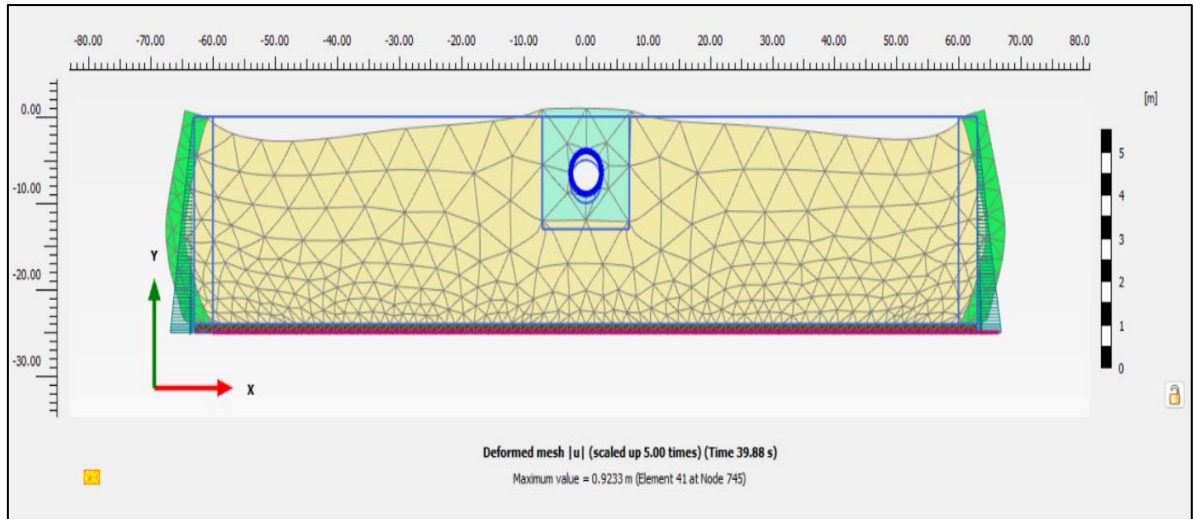


Figure 4.76: The deformation shape that occurs in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Northridge earthquake).

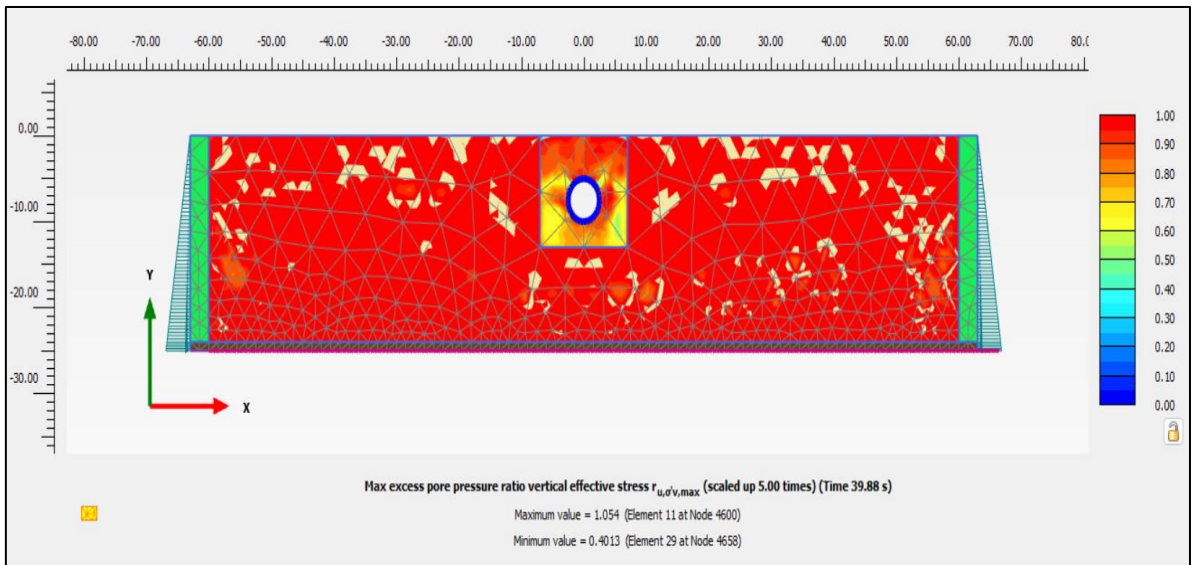


Figure 4.77: The shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by sand of RD 75% (under Northridge earthquake).

4.3.3 Replacing The Sand Around The Tunnel By Clay Soil

This method was done by using the sand RD 40% but replacing the sand around the tunnel by clay soil, and then comparing the results between the sand with replacing and without replacing.

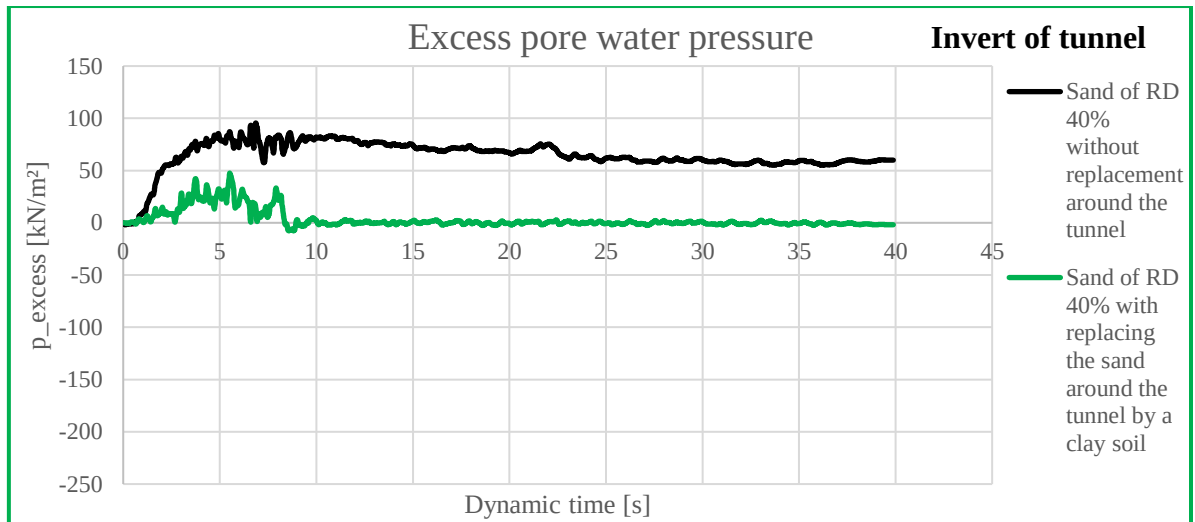


Figure 4.78: The excess pore water pressure at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Northridge earthquake).

From Figure 4.78, the excess pore water pressure was decreased when replacing the sand around the tunnel by clay soil, because the liquefaction doesn't occur in the clay soil.

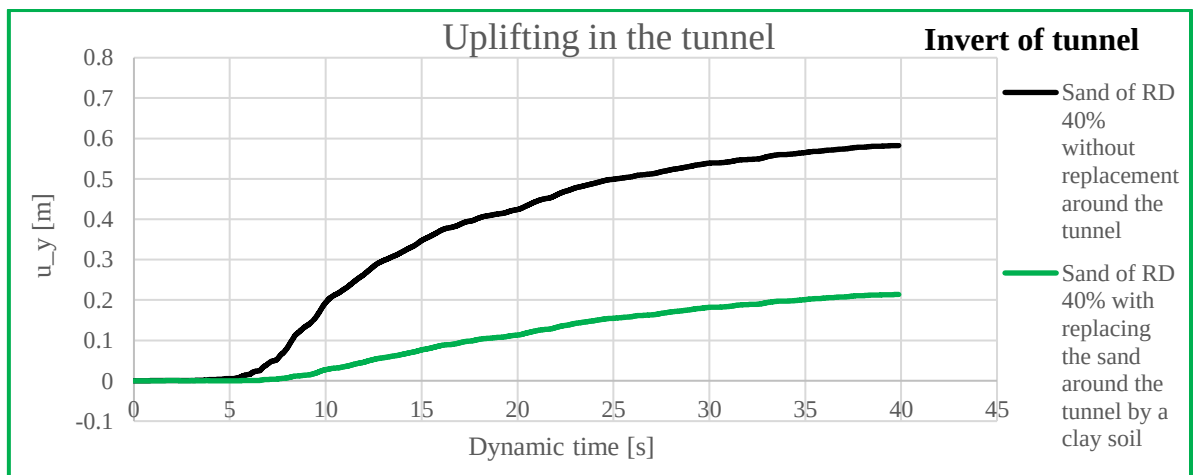


Figure 4.79: The uplifting at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Northridge earthquake).

From Figure 4.79, the uplifting in the tunnel was decreased when replacing the sand around the tunnel by a clay soil, because the liquefaction doesn't occur in the clay soil. The maximum uplifting in the tunnel is 0.583 m in a sand of RD 40% without replacement, but it is 0.214 m in a sand of RD 40% with replacing the sand around the tunnel by clay soil. So, the replacing of the sand around the tunnel by a clay soil is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.80 and Figure 4.81.

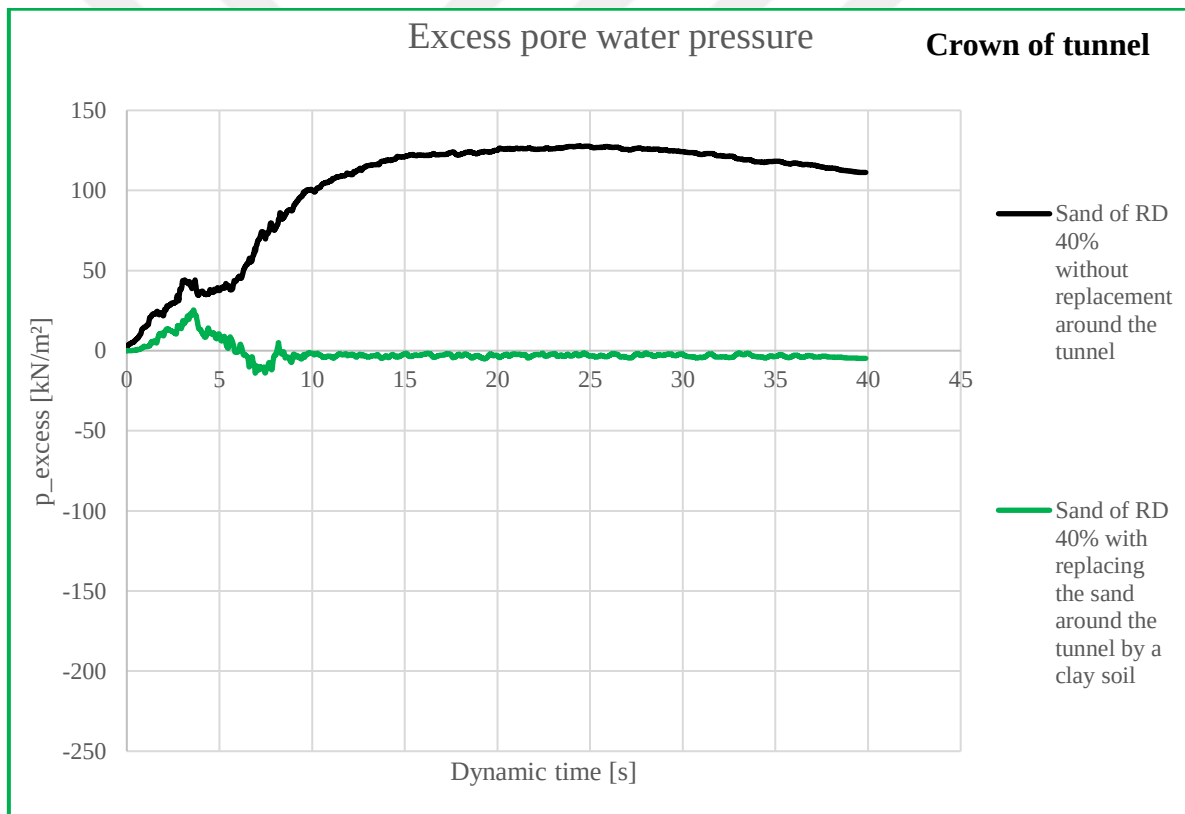


Figure 4.80: The excess pore water pressure at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Northridge earthquake).

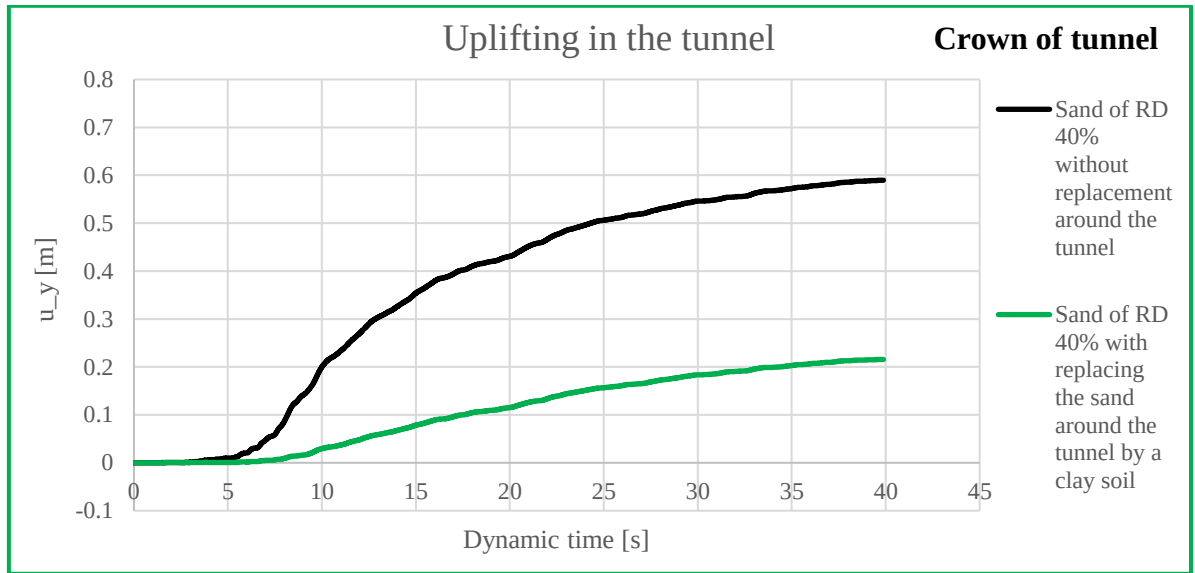


Figure 4.81: The uplifting at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by clay soil (under Northridge earthquake).

Figure 4.82 and Figure 4.83 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by clay soil. It was observed that the excess pore water pressure doesn't appear on the clay soil as in Figure 4.83.

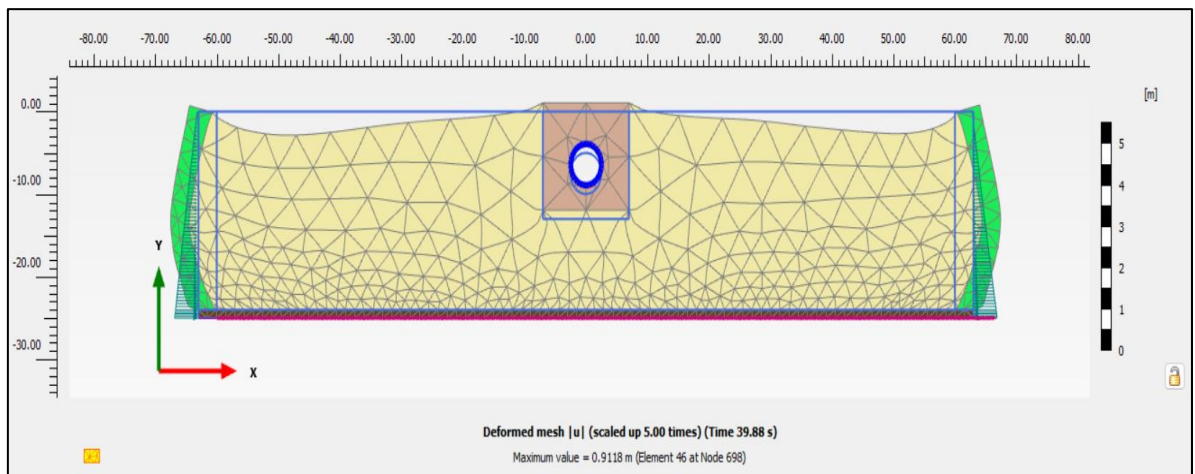


Figure 4.82: The deformation shape that occurs in a sand of RD 40% with replacing the sand around the tunnel by clay soil (under Northridge earthquake).

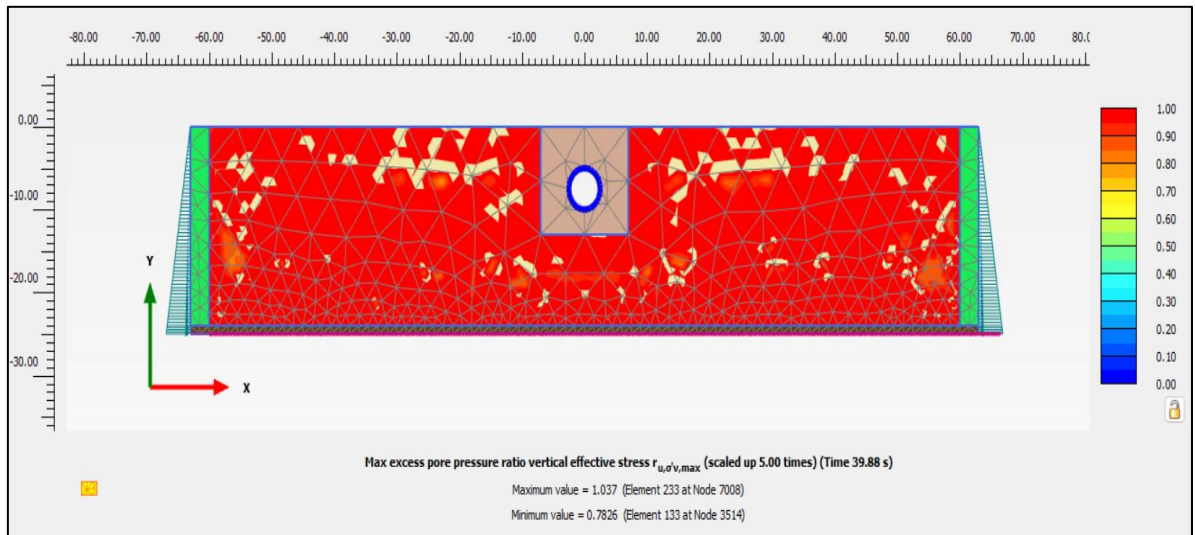


Figure 4.83: The shape of max excess pore pressure ratio ($r_{u, \sigma'_{v, max}}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by clay soil (under Northridge earthquake).

4.3.4 Replacing The Sand Around The Tunnel By Gravel Soil

This method was done by using the sand RD 40% but replacing the sand around the tunnel by gravel soil, and then comparing the results between the sand with replacing and without replacing.

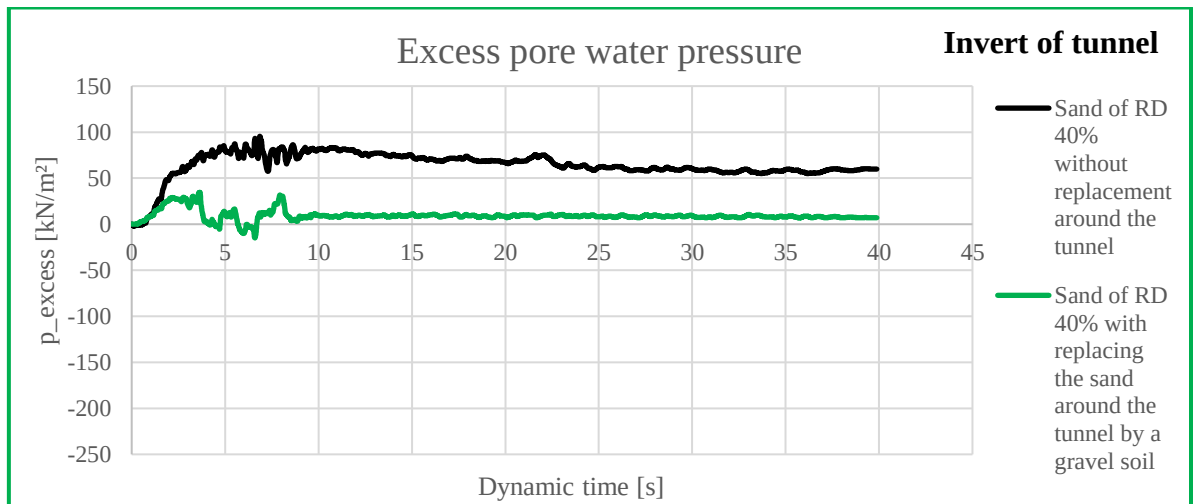


Figure 4.84: The excess pore water pressure at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Northridge earthquake).

From Figure 4.84, the excess pore water pressure was decreased when replacing the sand around the tunnel by gravel soil, because the liquefaction doesn't occur in the gravel soil.

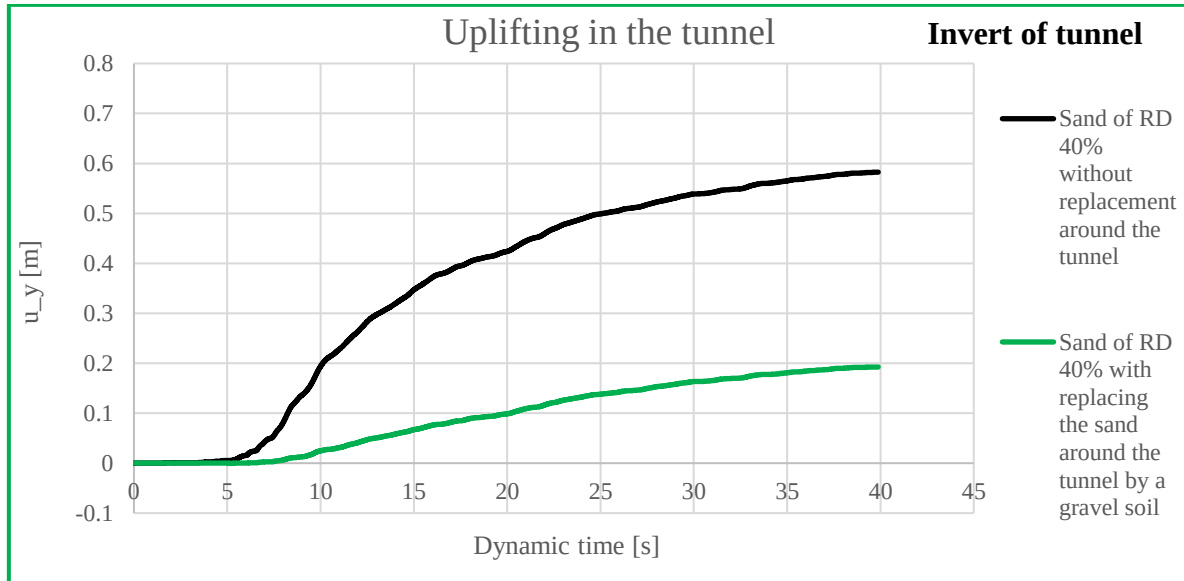


Figure 4.85: The uplifting at the invert of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Northridge earthquake).

From Figure 4.85, the uplifting in the tunnel was decreased when replacing the sand around the tunnel by a gravel soil, because the liquefaction doesn't occur in the gravel soil.

The maximum uplifting in the tunnel is 0.583 m in a sand of RD 40% without replacement, but it is 0.193 m in a sand of RD 40% with replacing the sand around the tunnel by gravel soil.

So, the replacing of the sand around the tunnel by a gravel soil is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

The similar results of the invert of the tunnel will occur in the crown of the tunnel, as in Figure 4.86 and Figure 4.87.

Figure 4.88 and Figure 4.89 illustrate the deformation shape and the shape of max excess pore pressure ratio ($r_{u,\sigma'v,max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by gravel soil. It was observed that the excess pore water pressure doesn't appear on the gravel soil as in Figure 4.89.

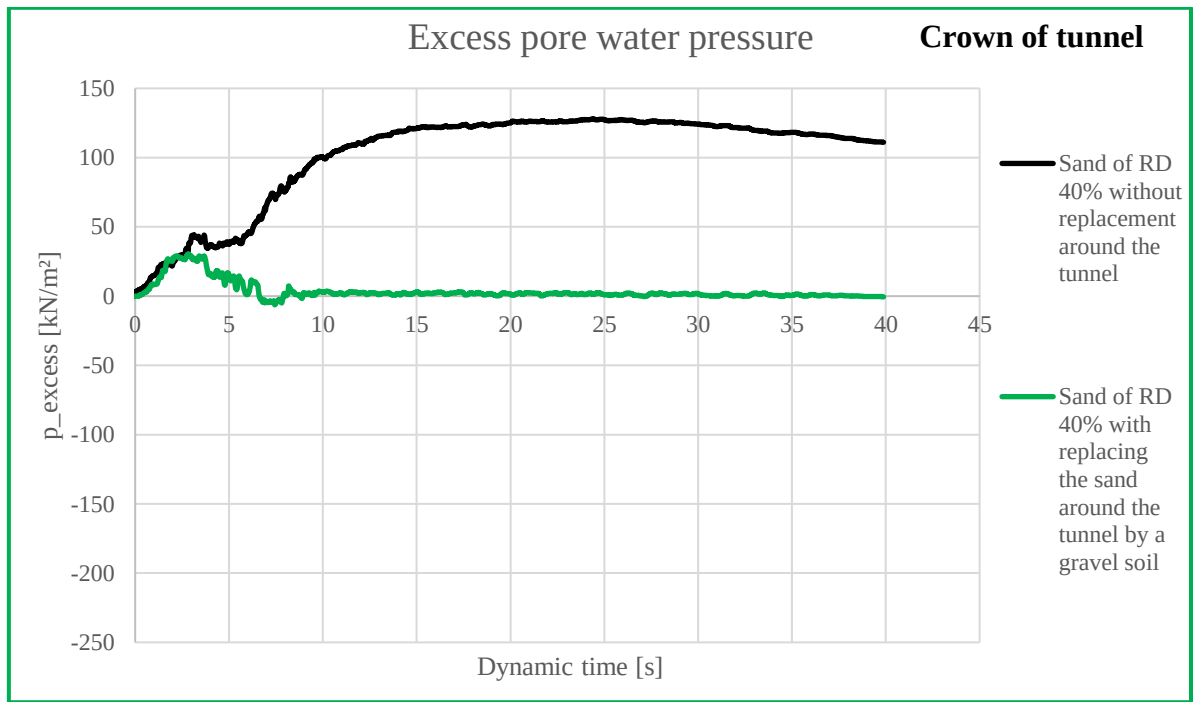


Figure 4.86: The excess pore water pressure at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Northridge earthquake).

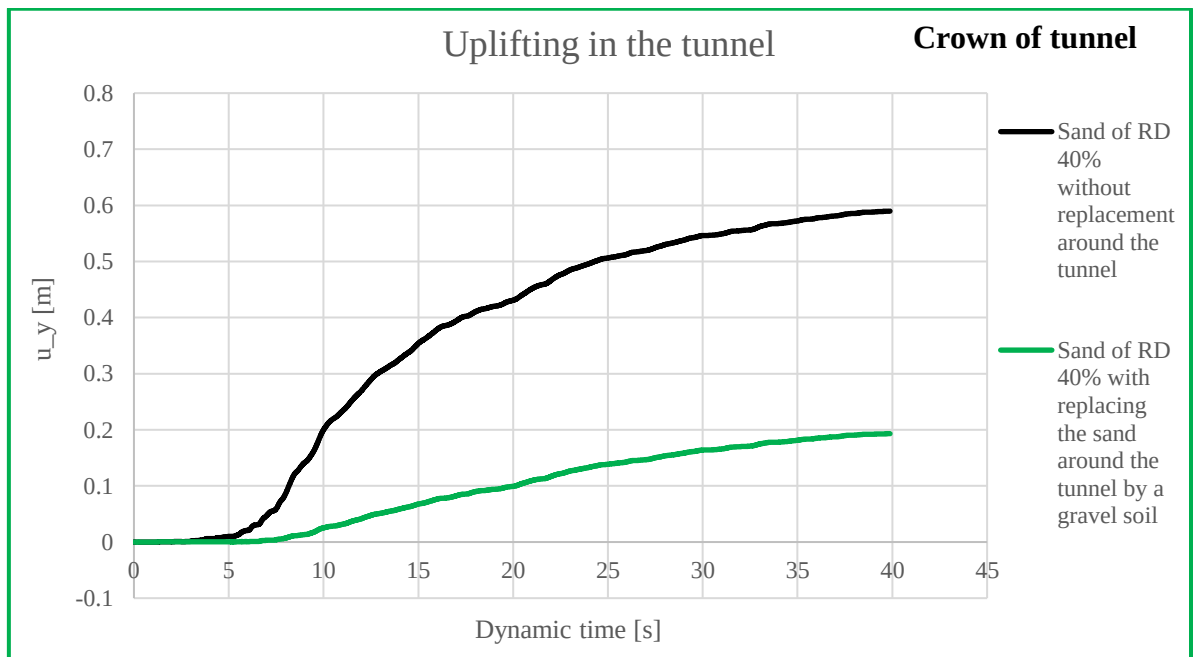


Figure 4.87: The uplifting at the crown of the tunnel for the sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Northridge earthquake).

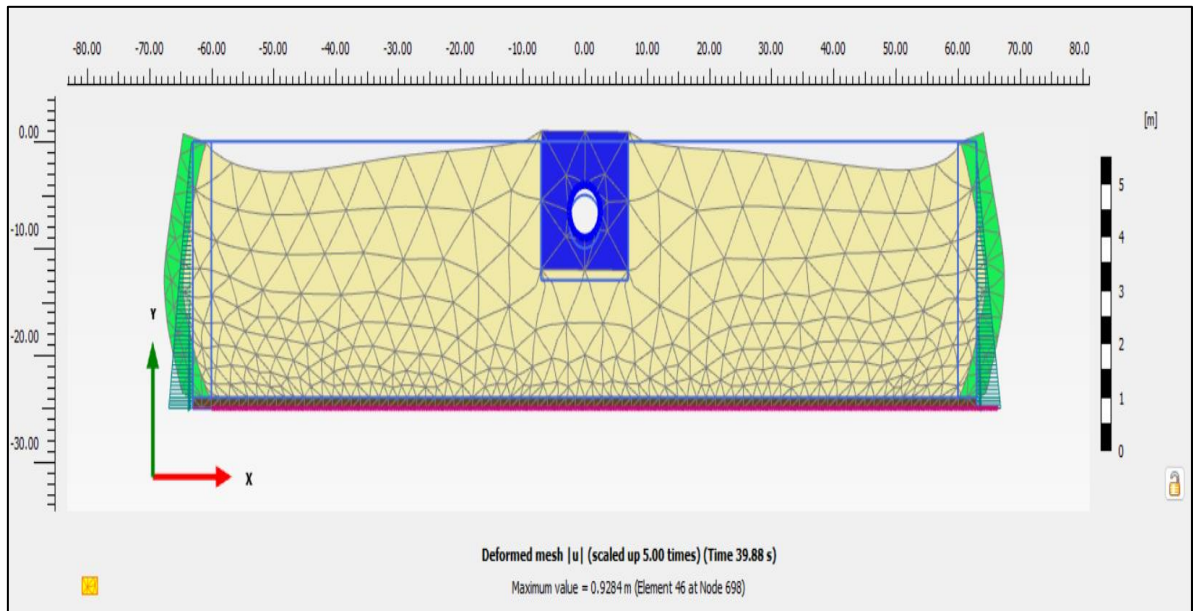


Figure 4.88: The deformation shape that occurs in a sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Northridge earthquake).

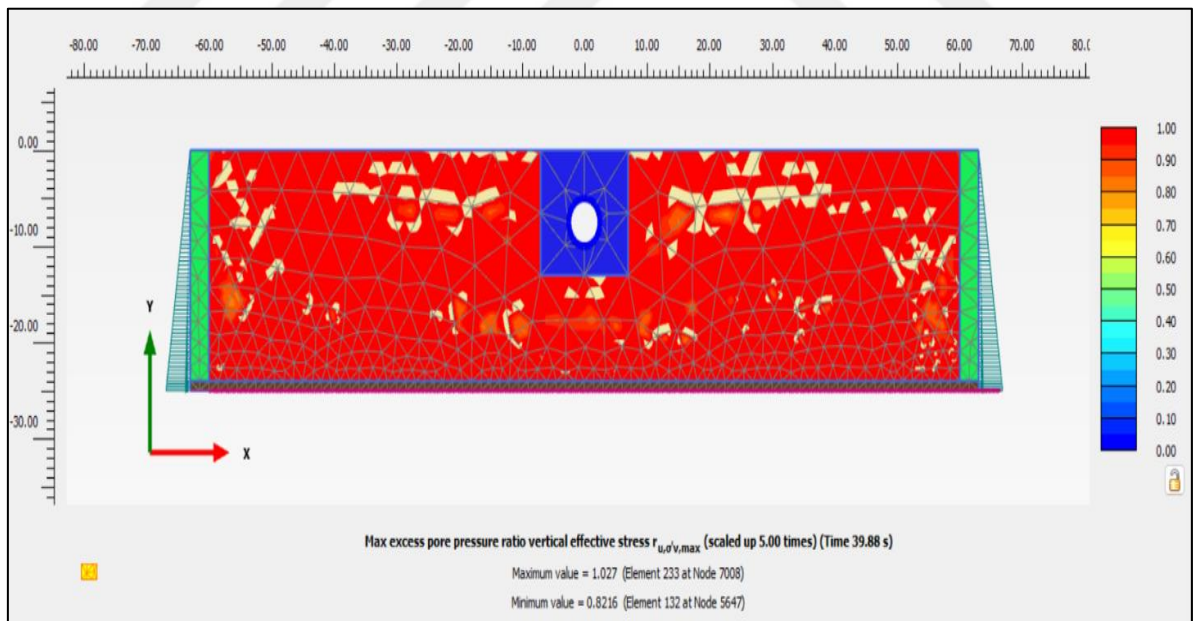


Figure 4.89: The shape of max excess pore pressure ratio ($r_{u, \sigma'v, \max}$) that occurs in a sand of RD 40% with replacing the sand around the tunnel by gravel soil (under Northridge earthquake).

Figure 4.90 and Figure 4.91 illustrate the comparison between the previous mitigation techniques, and it was noticed that the technique of increase the relative density of the sand to 75% has advantageous for mitigation the soil liquefaction and for mitigation the uplift of the tunnel, and it was noticed that replacing the sand around the tunnel by gravel soil is better than replacing it by clay soil, because the gravel soil is better for drainage the water and dissipating the excess pore water pressure at the soil.

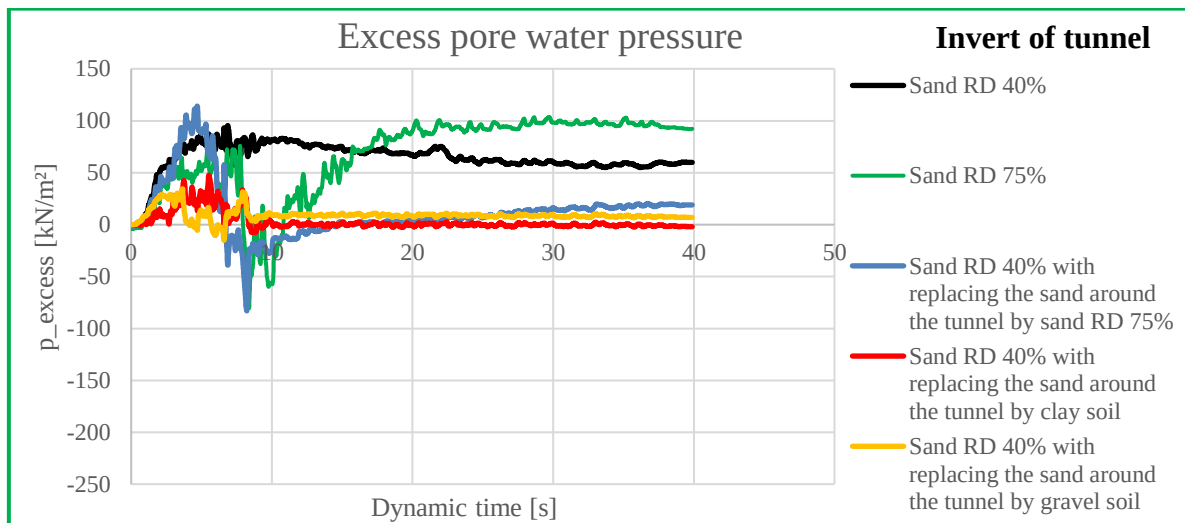


Figure 4.90: The comparison of the excess pore water pressure between the techniques that were used for mitigation the soil liquefaction and the uplift of the tunnel (under Northridge earthquake).

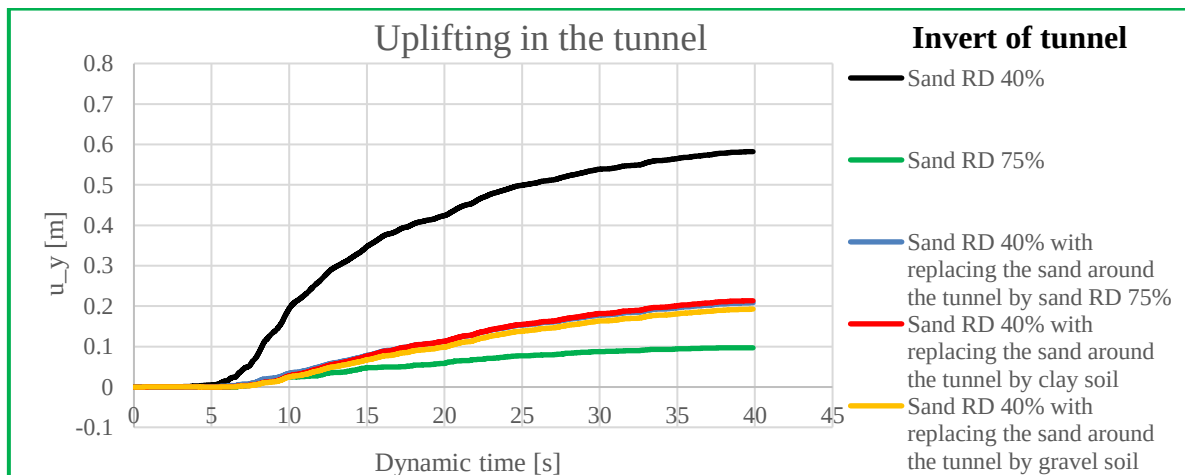


Figure 4.91: The comparison of the uplift of the tunnel between the techniques that were used for mitigation the soil liquefaction and the uplift of the tunnel (under Northridge earthquake).

5. DISCUSSION AND CONCLUSIONS

When an earthquake occurs in saturated soil that can be liquefied, this soil gets liquefied, this liquefaction results in the uplifting of buried structures such as reservoir tanks or tunnels. So, this research tried to mitigate the liquefaction-induced uplifts in tunnels numerically by making some models in PLAXIS 2D software under different earthquakes loading such as Kocaeli, Kobe and Northridge earthquake, and making comparisons between the results from these models.

The research results can be summarized as follows:

- a) The excess pore water pressure increases through the dynamic time of the earthquake loading, so the uplifting in the tunnel increases through the dynamic time of the earthquake loading.
- b) The excess pore water pressure is higher at the lower relative density of the sand, and it is lower at the higher relative density of the sand, because the excess pore water pressure occurs in the loose sand more than in the dense sand, and the uplifting in the tunnel is higher for the lower relative density, and it is lower for the higher relative density. So, the uplifting in the tunnel can be mitigated by increasing the relative density of the sand.
- c) The excess pore water pressure and the uplifting of the tunnel were decreased when replacing the sand around the tunnel by a higher relative density sand. So, the replacing of the sand around the tunnel by a higher relative density sand is a successful way to mitigate of liquefaction-induced uplifts in tunnels.
- d) The excess pore water pressure and the uplifting of the tunnel were decreased when replacing the sand around the tunnel by clay soil, because the liquefaction doesn't occur in the clay soil. So, the replacing of the sand around the tunnel by a clay soil is a successful way to mitigate of liquefaction-induced uplifts in tunnels.
- e) The excess pore water pressure and the uplifting of the tunnel were decreased when replacing the sand around the tunnel by gravel soil, because the liquefaction doesn't occur

in the gravel soil. So, the replacing of the sand around the tunnel by a gravel soil is a successful way to mitigate of liquefaction-induced uplifts in tunnels.

- f) The technique of increase the relative density of the sand to 75% has advantageous for mitigation the soil liquefaction and for mitigation the uplift of the tunnel.
- g) Replacing the sand around the tunnel by gravel soil is better than replacing it by clay soil, because the gravel soil is better for drainage the water and dissipating the excess pore water pressure at the soil.
- h) The excess pore water pressure ratio at the soil in the ground surface or in shallow depth is higher than the excess pore pressure ratio in the deep depth, that due to the effective confining stresses at the deep soil are higher, so the liquefaction has a higher severity at the shallow depths more than deep depths.

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