A

PROJECT REPORT

On

"SAFETY AND DEFORMATION ASSESSMENT OF UNDERGROUND METRO TUNNEL"

SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE

OF

Master of Technology

IN

GEOTECHNICAL ENGINEERING

Submitted by:

Vinit Kumar

(2K19/GTE/17)

Under the Supervision of

PROF. ANIL KUMAR SAHU

DEPARTMENT OF CIVIL ENGINEERING

DELHI TECHNOLOGICAL UNIVERSITY

(Formerly Delhi College of Engineering)

Bawana Road, Delhi-110042

AUGUST 2021

DELHI TECHNOLOGICAL UNIVERSITY

(Formerly Delhi College of Engineering)

Bawana Road, Delhi-110042

CANDIDATE'S DECLARATION

I, Vinit Kumar, Roll No. 2K19/GTE/17 of M.Tech (Geotechnical Engineering), hereby declare that the work embodied in this dissertation entitled **"Safety and Deformation Assessment of Underground Metro Tunnel"** which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. An authentic record of my work carried out under the supervision of Prof. Anil Kumar Sahu. This work is not previously formed the basis for the award of any other Degree or Diploma Associateship. Responsibility for any plagiarism-related issue stands solely with me.

Place: Delhi

Vinit Kumar

Date: 11/08/2021 **(VINIT KUMAR)**

DELHI TECHNOLOGICAL UNIVERSITY

(Formerly Delhi College of Engineering)

Bawana Road, Delhi-110042

CERTIFICATE

I hereby certify that the Project Dissertation titled "**Safety and Deformation Assessment of Underground Metro Tunnel"** which is submitted by Vinit Kumar, Roll No. 2K19/GTE/17, Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the student under the supervision. To the best of my knowledge this work has not been submitted in part or full for any Degree or Diploma to this University or elsewhere.

Place: Delhi **ANIL KUMAR SAHU**

Date: 11/08/2021 **(PROFESSOR & SUPERVISOR)**

ACKNOWLEDGEMENT

Foremost, I would like to express my sincere gratitude and thanks to **Prof Yogesh Singh**, Vice-Chancellor, DTU, **Prof V.K. Minocha,** Head of Department of Civil Engineering, and my supervisor **Prof. Anil Kumar Sahu**, Department of Civil Engineering, Delhi Technological University, Delhi for the continuous support of my M.Tech study and research, for his patience, motivation, enthusiasm, and immense knowledge. His guidance helped me at all the time of research and writing of this thesis.

I am also thankful to **Dr. Raju Sarkar**, coordinator, Geotechnical Engineering, all faculties, and support staff of the Department of Civil Engineering, Delhi Technological University for constant help and for extending the departmental facilities for my project work.

Vinit Kumar

Date- 11/08/2021 (**Vinit Kumar)**

2K19/GTE/17

ABSTRACT

Geological conditions such as space-time effect, tunnel dimensions, construction methods, tunneling method, and effect of groundwater on soil/rock are some of the factors influencing the movements of the ground. The magnitude of maximum surface settlement and the shape of the surface settlement curve are greatly influenced by these characteristics. This project is focused **on the safety and deformation assessment of underground metro tunnels**. The work presents a safety and deformation assessment of the underground twin metro tunnel and also evaluates risk measures related to rock-soil mass concerning different stress-strain states. Empirical and numerical both methods are carried out in this analysis. The numerical analysis of the tunnel deformation is carried out using the finite element method, and an empirical equation is used to validate the results. The Hoek–Brown failure criterion and Mohr-Coulomb criteria were used for numerical calculation of stresses and to find out physical parameters such as cohesion value and angle of friction. The rock-soil mass parameter for this calculation was taken from the **Mumbai metro rail project**. The worst category of the rock mass was considered in the analysis that may likely encounter TBM tunneling to have a conservative approach for evaluating settlements due to excavation of TBM tunnel. The findings showed that the distance between tunnels, tunnel lining, and the most critical component in twin tunnel-induced surface settling was discovered to be the deformation modulus of the geo-materials surrounding the tunnel.

TABLE OF CONTENTS

LIST OF THE TABLE

LIST OF FIGURE

CHAPTER-1

INTRODUCTION

1.1) General

In civil engineering, Investigation of underground space and underground construction is one of the most rigorous challenges which are faced throughout the world. Due to the booming population and day by day increasing traffic activities underground construction comes as a boon for providing a viable and resourceful method of construction to provide various types of amenities to the people such as underground canals, metros, subway, etc.

Knowledge about deformation profiles is important and crucial to check, secure, and minimized effects on the nearby existing environment. The settlement of the surrounding surface area around tunneling must be investigated as this will influence the superstructures and surrounding utilities. Many techniques are used to find movements of ground instigate by tunneling which comprises many methods like analytical, empirical, and practical modeling methods. The vertical overburden pressure exerts the primary stresses at the tunnel periphery, whereas lateral pressure is exerted mostly by the rest condition of the neighboring soil-rock mass. It's suggested that the vertical overburden pressure is caused by the soil-rock mass that covers the underground space and the ground level surcharge applied along the tunnel's periphery within the influence zone.

Another factor to consider is tunnel stability. Tunnel stability is primarily determined by the tunnel's relative position and the construction method used. To determine the stresses and deformation in a tunnel, various theoretical formulations have been suggested over the years. The site under consideration is the **Mumbai metro rail project** which is also the one of its kind completely underground construction project in India and it passes through the most densely populated area in Mumbai. The physical and mechanical properties of rock and soil mass are studied using borehole data. By borehole data, it is found that the rock type in that area is **Breccia.**

The following are the primary assumptions that were employed in the analysis method-

1. The shape of the tunnel is circular.
2. In situ stress field is hydrostatic
3. The rock mass is homogeneous and isotropic. Major structural discontinuities do not govern failure.
4. The support response is considered to be elastic-perfectly plastic.

1.2) Methods of Tunneling:-

A tunnel is an underground passage that runs beneath the earth's surface. Extensive excavation of earth, rock, and other materials is required for tunnel construction. Because of the availability of specialized technology, excavation and backfilling have become easier.

According to lithology and geology different tunnel construction methods are adopted, as mentioned below-

1.2.1) Cut and Cover Method-

"Shallow tunnels are commonly built using this technology. Essentially, this approach entails digging a trench in the soil, installing tunnel box pieces, and then covering them with a load-bearing support mechanism to restore the surface. Cut and cover construction is used when the tunnel profile is shallow and the excavation from the surface is possible, economical, and acceptable. Cut and cover construction is used for underpasses, the approach sections to mined tunnels and for tunnels in flat terrain or where it is advantageous to construct the tunnel at a shallow depth" (Mohammed, 2015).

Cut and cover tunnels are built using two different methods.

i. Bottom-up ii. Top down

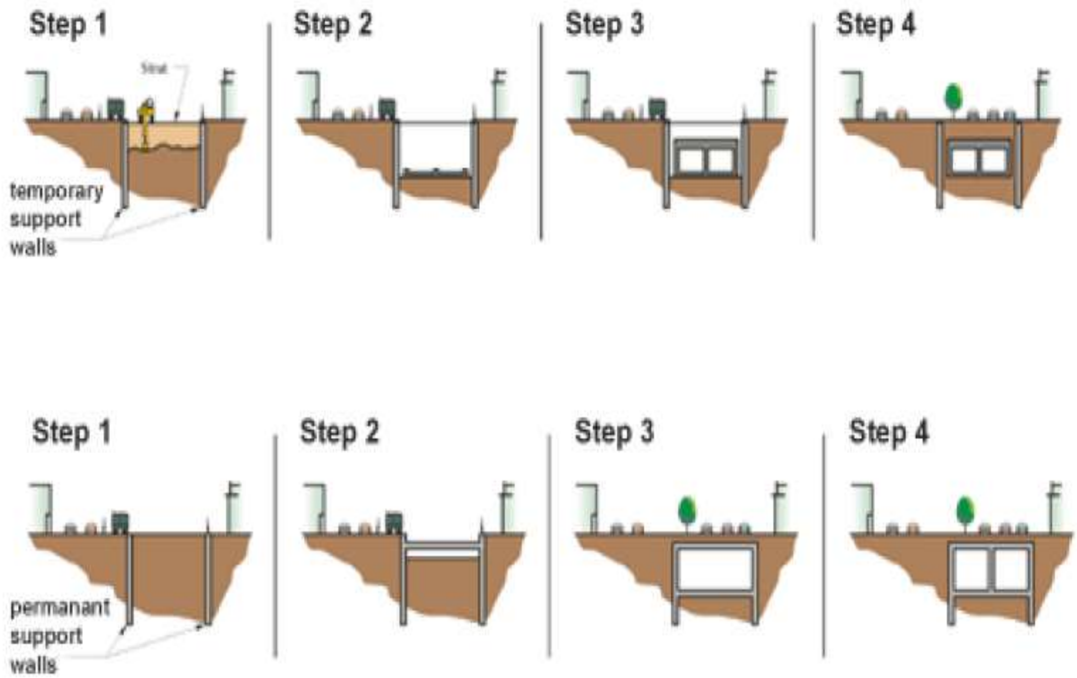


Figure 1: Bottom-Up & Top-Down construction sequence (Mohammed, 2015)

"Because each project is unique and has a variety of limitations and factors to consider when choosing a building technique, it is difficult to generalize the use of a certain construction method. The conditions that may make one construction approach more appealing than the other are outlined in the following summary. To make a final selection of the construction method to be employed, this summary should be used in conjunction with a detailed review of all elements related to a project" (Mohammed, 2015).

Conditions that favour the bottom-up construction include-

- There are no restrictions on right-of-way.
- Sidewall deflections are not required to be limited.

- There is no need to restore the surface permanently.

Conditions that favour the top down construction includes-

- Right-of-way width is restricted.
- To safeguard nearby features, side wall deflections must be restrained.
- As soon as feasible, the surface must restore its original state as usable condition as before.

Procedure of cut and cover tunnel construction is given below as diagram representation,

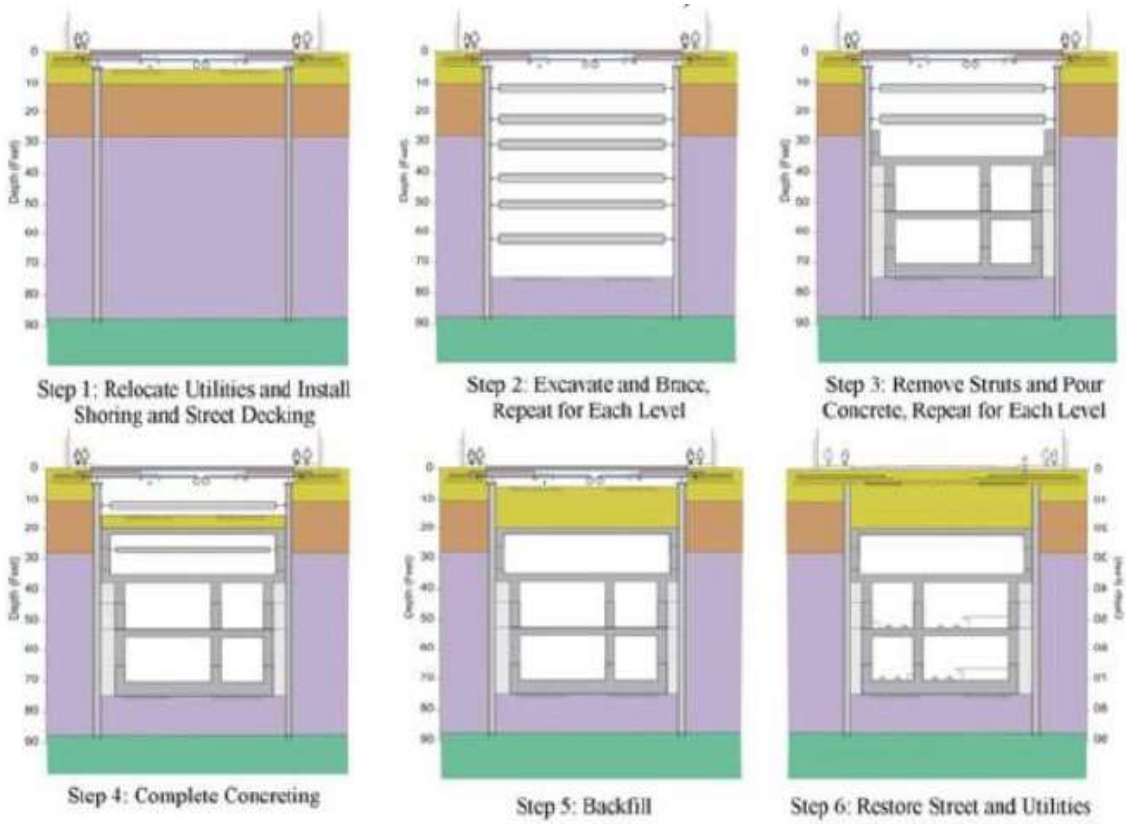


Figure 2: Procedure of cut and cover tunnel construction (Mohammed, 2015)

1.2.2) Excavation Tunneling Methods-

Drill and blast method-

Drill and blast excavation is the most common method used all around the world. The technology applies to all types of rocks and has a lower starting cost than a mechanical method such that TBM. Explosives are used in this tunneling technique. When compared to boring tunnels with a Tunnel Boring Machine, blasting produces higher vibration levels for longer periods. TBM's excavation rate is likewise lower (usually 3 to 5m a day). The basic method entails drilling a series of small craters, pile them with explosives, and then detonating the explosives, resulting in an opening in the rock. The blasted and shattered rock (muck) is then removed, and the rock surface is supported, allowing the procedure to be repeated as many times as needed to progress the desired opening in the rock.

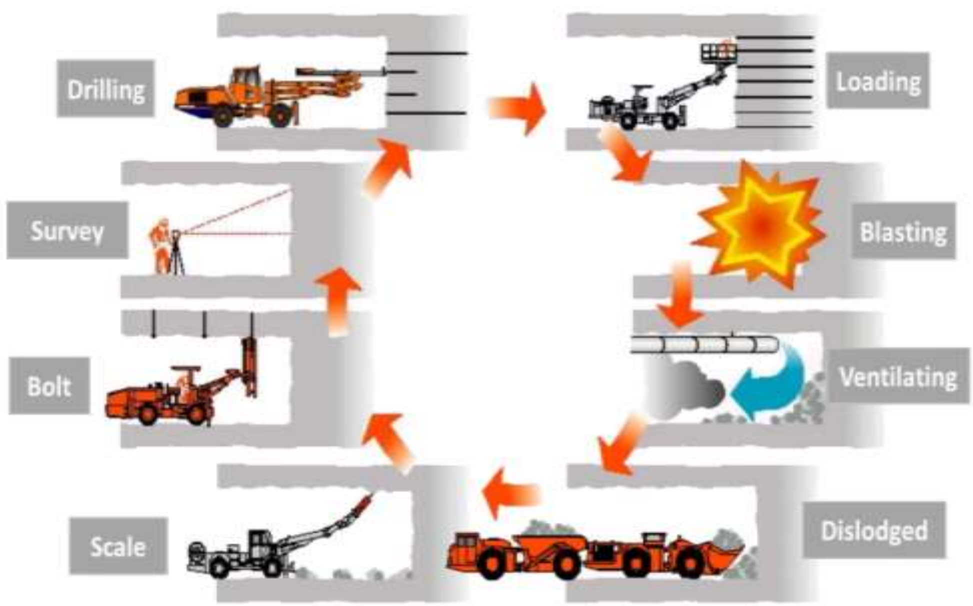


Figure 3: Drill and Blast Construction Method (Sandvik, 1999)

Tunnel Boring Machine (TBM) Method-

Mechanized tunneling machines, also known as tunnel boring machines, have been proposed for over a century but have never been realized. That started to change in the 1960s when the oil field drilling technique was attempted. Some progress was made, but it was slow since the machines sought to crush the rock away rather than digging it because the physics was incorrect. With the invention of the disc cutter in the late 1960s, all of that changed.

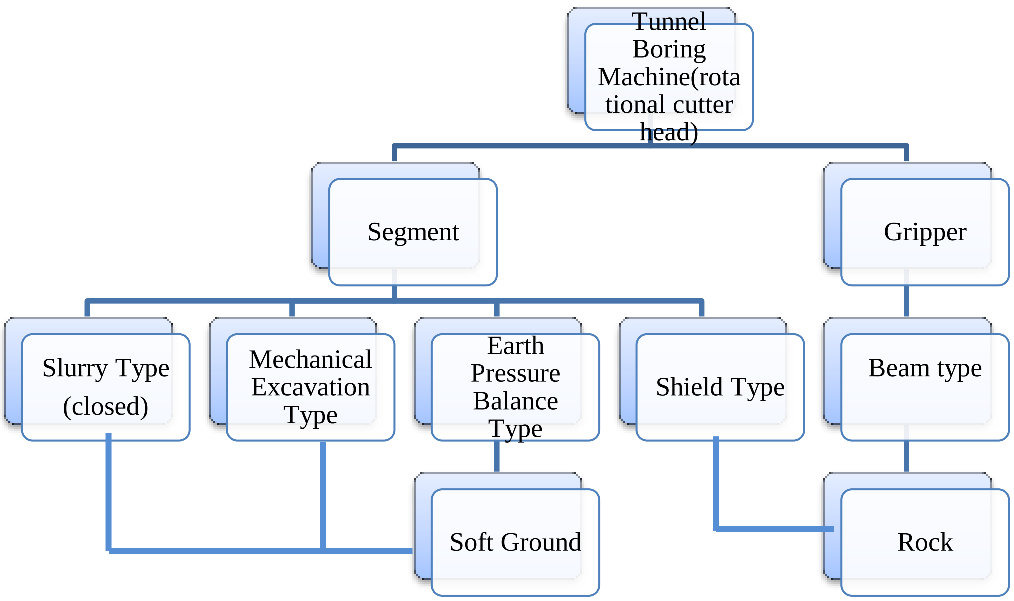


Figure 4: Classification of Tunnel Excavation Machine

The shield is used to excavate ground at the tunnel's front face, and excavated waste is removed via machinery as slurry, depending on the TBM type. The TBM is propelled forward using hydraulic jacks. To build tunnel lining, an erector is used to pick up precast concrete segments and position them in the right places.

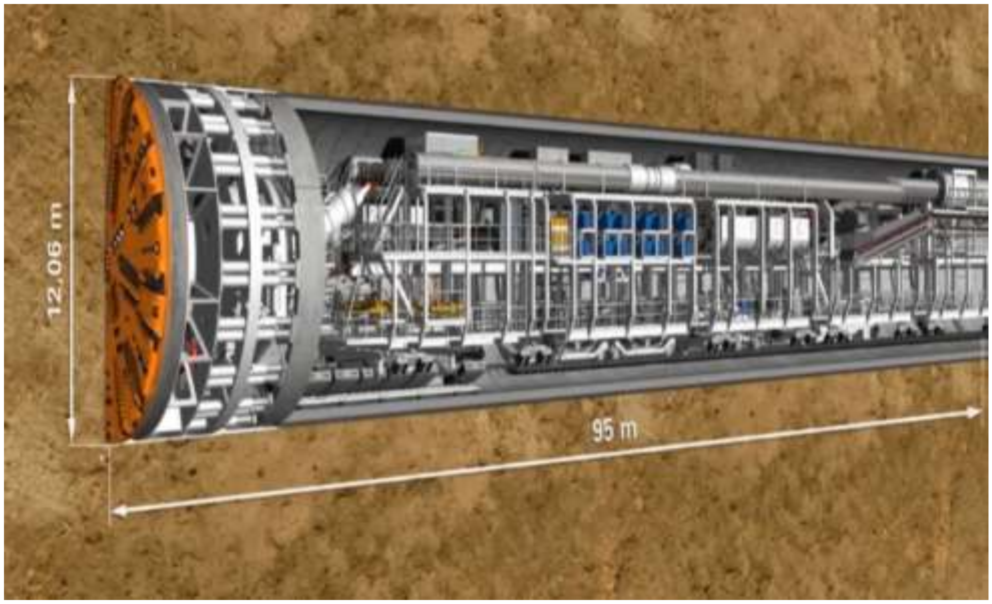


Figure 5: Typical TBM below ground surface (Sandvik, 1999)

Figure 6: Tunnel after TBM excavation (Mumbai Metro line-3)

Roadheader-

The roadheader is another method for cutting an opening closer to a required portion. A massive milling head positioned on a boom, which is mounted on rails or within a shield, is the fundamental cutting tool for a roadheader. The corners of the milling head must be cut to match its curvature, while the majority of the walls, crown, and invert can be cut to nearly any form. A single roadheader, unlike a TBM, can cut varied or unusual forms that would normally need TBM excavation combined with drill and blast or drill and blast alone. Roadheaders are also the preferred option for relatively short tunnels, because of their flexibility, availability, and reduced cost.

Figure 7: AM 105Roadheader, Australia (Mohammed, 2015)

On the contrary, roadheaders are inefficient over longer distances and in hard rock. The roadheader picks are about 10% as successful at removing rock as TBM discs, but they must be replaced regularly and may not be useful in rock with a UCS higher than 140 MPa.

New Austrian Tunneling Method (NATM)-

NATM was first developed for use in rock tunneling in the early 1960s, but it was later used for soft ground in urban tunneling in the late 1960s, and it has since gained widespread international use in both rural and urban environments. A huge number of tunnels have been constructed around the world employing a construction method known as NATM. In its most basic form, the NATM aims to maximize the ground's self-supporting capabilities, resulting in ground support cost savings.

"The NATM is founded on the idea that the ground around the tunnel should be considered as an integral part of the tunnel support system. The initial lining of flexible shotcrete provides for a controlled ground deflection to mobilise the ground's natural shear strength and commence load redistribution." (Mohammed, 2015)

Tunnels typically subjected to three different loading conditions, all of which must be considered while constructing the lining material and allowing for proper settlement as per guidelines

- The static load is caused by Rock or Soil overburden mass.
- Dynamic load generated due to movement of heavy vehicle and metro rail.
- Blast and earthquake-induced impact load

The nature of these forces ultimately depends purely on the geometry, deformation & stress at the periphery of the tunnel. In some circumstances, excessive deformation can cause accidents or even super-structure settlement, and structures can eventually collapse.

Hence, in this paper, the Assessment of maximum deformation is carried out in different soil-rock mass conditions to rule out the possibility of a tunnel collapse in any possible way.

1.3) OBJECTIVE:-

- To validate the empirical relation proposed by Herzog.
- To determine the shear parameters for each rock mass using borehole data and Hock-Brown & Mohr-Coulomb failure criterion.
- To analyse the settlement behavior with respect to tunnel spacing.
- To perform a comparative study for deformations of the circular tunnel for various soil-rock masses.
- To understand the settlement behavior with lining and without lining.
- To understand the behavior of settlement profile in case of homogeneous soil-rock mass and mixed ground condition respectively.

CHAPTER- 2

LITERATURE REVIEW

2.1) Introduction

Tunnel analysis can be divided into three types: physical or experimental, numerical, and analytical. Because each project is unique, tunnel construction generally entails selecting a method that is appropriate for that project. This strategy is chosen based on the project's characteristics. It should also properly serve the function for which the tunnel is being built, making efficient use of the available space above and below ground while limiting the negative effects of the tunnel's construction, such as excessive settlements or ground movement. Various construction approaches are utilised depending on whether it is a metro or a highway. Tunnel stability is primarily determined by the tunnel's relative position and the construction method used. To determine the stresses and deformation in a tunnel, various theoretical formulations have been suggested over the years**.**

2.2) Literature Review

Deformation analysis of tunnel has been investigated by many researchers. Literature reviews are presented below:

Islam & Iskander (2021) did analysis of twin tunneling-induced ground settlement. The interaction phenomena connected to surface settlement, a large amount of information was gathered, analysed, and analogize to one another. In this paper field observations that have been published, laboratory testing, and finite element analyses were used to compile this information. A synopsis of available methodologies for calculating ground movement produced by a new tunnel excavation in the presence of an existing tunnel is also presented in the study. Finally, the report reviews what is known about ground settling caused by different twin tunneling arrangements.

Zhang et al. (2021), a discrete element numerical simulation is used to investigate the face stability of a shallow shield cross-river tunnel over silty fine sand. The micro mechanism of silty fine sand is investigated using direct shear testing, and the micro parameters are scaled. The shallow shield cross-river tunnel model is then run, with the water pressure and buoyancy force taken into account. The findings show that a wedge model is first produced in front of the tunnel face, and that the area of the failure zone in front of the tunnel face extends as the displacement of the piston movement rises.

Soomro et al. (2020) presents a numerical parametric analysis in three dimensions to better understand the settlement and load transfer mechanisms in single piles induced by twin stacked tunnelling in stiff saturated clay utilising alternate construction sequences. In contrast, the change in tunnel construction order had no noticeable influence on the weight transmission mechanism in the pile.

Ağbay & Topal (2020) developed a technique for calculating a modification factor that takes into account the impacts of a pre-support system and the rock mass quality and may be utilized as the ratio of reduction in twin tunnel-induced surface settlement prediction methods. A parametric analysis was carried out with different distances between pipes in pre-support systems, yielding a statistical formula that illustrates the pre-support system's diminishing effect on maximum surface settlement. The most critical component in twin tunnel-induced surface settlement is the deformation modulus of the ground surrounding the tunnel. A novel formula for forecasting of twin tunnel ground deformation is given as a modification factor to Herzog's equation.

Forsat et al. (2021) investigated the results of a three-dimensional modelling study of twin and single metro tunnels respectively for the Tehran metro line. Initially, the simulation was based on a comparison of ground movements in single and twin tunnels. The impact of EPB-TBM effective parameters on surface settlements during excavation was then simulated. The preliminary findings indicated that the ground settlement behaviour was appropriate. The shield tail's end had the most settlements, while the single tunnel had the most.

Zhou et al. (2021) studied three types of tunnel support conditions. The stress emancipating coefficient, rheological model, and added radial body force of bolts are all taken into account. The displacement analytical formulas for the primary support and secondary lining are also calculated. The estimated support structure displacements and loads are largely compatible with field measurements, demonstrating the theoretical method's validity.

Heidary et al. (2021) the effect of surrounding soil layers and lens quality on vibrations produced by railway twin tunnels was compared. When compared to the homogeneous soil layer, the layered region surrounding the single and twin tunnels increases vibration levels by up to 10 decibels. The impact of thicker soil lenses is also worth mentioning.

Lai et al. (2020) investigated the safety of the metro tunneling beneath an existing glass structure, in which different techniques used such as long pipe roofs, pre-grouting, and parameter optimization. Monitoring the settling of the structure and surface during tunneling confirmed their impact. In addition, a new approach for controlling settlements in time was used to divide settlement monitoring according to processes.

Fuyong et al. (2019) described a technique that has been simplified to calculate the crossing tunnel's collapse chances by considering the properties of spatial variability of the rock mass. Based on the reliability result Sensitivity analysis has been done considering the geophysical property of rock mass and also determining the critical design factor.

Li and Wang (2019) investigate the issue related to construction of metro stations in which the pile-beam-arch method is used. The major construction is divided into three phases: the first is the excavation of preparative tunnels, the second is the establishment of the load-bearing system, and the third is the removal of earth from within the metro station. In most cases, a 30 mm settlement was indicated in the results, which is also an allowed settlement, although the highest number of observed settlements is significantly more.

Poovizhi et al. (2020) suggested that for 'risk & safety management'. a new type of model in metro rail projects is required because of the natural uncertainties of the ground conditions so it is not designed based on the surface conditions. The existing risk & safety management system is analyzed and recognizes the risks that happen during the time of construction.

Golshani et al. (2019) presented a case study of a twin tunnel built using the New Austrian Tunneling Method. Due to dimension and obstruction constraints, the twin tunnel is designed with connected initial linings and separate ending linings**.** To reduce settlement, three excavation scenarios were numerically modelled. The first scenario entails complete excavation and the sequential installation of the initial lining for each area. The second scenario models the effect of maintaining the Center Cross Diaphragm in place throughout excavation until the initial lining is completed. The third scenario assesses the impact of final lining construction for one section prior to the completion of initial stabilisation for another. According to numerical data, the third scenario reduces surface settling by 44 percent when compared to the first, whereas the second has no effect.

Nematollahi & Dias (2019) investigated the pile-tunnel interaction using a three-dimensional numerical model. A finite difference technique was used to model all of the mechanised excavation phases of an EPB-TBM as well as the segmental lining. It should be emphasised that, based on the obtained results, the MC model is unable to effectively predict ground settlements and forces in tunnels and piles, necessitating the use of advanced constitutive models such as CY soil for design purposes.

Shivaei et al. (2020) investigated the interaction techniques between mechanised twin tunnels construction and groundwater using three-dimensional coupled Finite Element Analyses (FEA) at Shiraz metro line. Following that, in the coupled FEA of twin tunnels, parametric studies are conducted to examine the effects of influencing components such as the grout layer drainage border condition, groundwater level position, and the rest interval between twin tunnel excavations. The numerical results reveal that the grout layer's drainage quality has a big influence on ground surface settlements and internal strains in existing segmental lining.

Jin (2018) studied tunnelling technique that used to excavate a stretch at Shangmeilin station on Shenzhen metro line 9 of closely spaced twin tunnels. A portion of this stretch runs parallel to the tunnel that serves Metro line 4. Due to surface limits, an in-tunnel grouting protection method was used in conjunction with the shield mechanism to manage the excess settlement of the existing tunnel in the area. When constructing the twin tunnels underground, this strategy considerably decreased the impact on the existing tunnels. The existing tunnels were properly monitored for settlement and stress during the construction of the new tunnels.

Camós and Molins (2015) suggested a new equation to determine the horizontal surface strain. These equations deviate from the classical profile of Gaussian settlement. The modeling of the tunnel advance including the equivalent beam method in 3D is allowed through this new equation. Wall position with respect to the tunnel axis provides a significant variation of estimated damage in the results.

The paper presents certain relevant aspects of damage prediction of building, as of the areas which are influenced by settlement and influence of ground horizontal strain to damage reduction. A constant evaluation is also executed along the building wall to create a nonlinear regression model to obtain max. tensile strain regarding the input values of geological condition, tunnel geometries, and positioning of walls.

Fang et al. (2015) studied the case of closely spaced twin tunnels excavated beneath other similarly spaced existing twin tunnels in Beijing, China. The shield method was used to construct the existing twin tunnels, while the shallow tunnelling method was used to create the new twin tunnels. The settlements of existing tunnels as well as the ground surfaces linked with the building of new tunnels were all meticulously monitored.

Osman (2010) gave solutions to the problem of twin tunnel excavation stability in soft ground. To idealise ground deformations around shallow, unlined twin tunnels embedded inside an undrained clay layer, a suitable displacement field has been constructed. For measuring the influence of interaction between nearby tunnels on their stability, the principle of superposition is applied. The stability numbers are reduced by up to 35 percent in the case of narrowly spaced tunnels at shallow depths (C/D = 1).

O'Reilly, New (1982) suggested that the settlements can be an issue with soft ground tunneling in metropolitan areas, putting buildings, both contemporary and old, in jeopardy; services, too, can be jeopardised, and it has been required to divert services before tunneling can begin in several cases. As a result of these environmental concerns, a significant amount of research has been devoted to the study of settlements induced by tunneling through the soft ground; much of this research has been done either directly or under contract for the Transport and Road

Research Laboratory. The results of settlement and ground movement measurements taken on tunneling projects that are mostly in populated areas are examined. The information gathered from these case studies is used to develop simple analytical methods that allow for a more accurate prediction of the magnitude of settlements and ground movements produced by tunneling through soft ground.

Atkinson and Mair (1993) stated that a variety of calculations dealing with the stability of tunnels and tunnel headings, as well as settlements generated by tunneling, largely as a result of research conducted at Cambridge University. In accordance with the basic principles of soil mechanics, these calculations usually consider drained and undrained instances separately, but it is not always evident which calculation is suitable for a particular actual tunneling problem. This work covers the stresses and pore pressures in soft ground induced by tunneling, as well as the calculations needed to estimate the tunnel's stability and its heading, as well as the settlements caused by tunneling.

Hamza et al. (1999) evaluated the settlement prediction approaches that was used to estimate the surface settlements concerned with the building of the Greater Cairo Metro Line 2. Cut-and-Cover underground stations and bored tunneling were used in the construction of the Cairo Metro. The top-down building method was used to construct a standard underground station. This study is the initial step toward improving settlement prediction processes and assessing potential damages to overlying structures and utilities in preparation for the future construction of the twin road tunnels in Cairo's historic Al Azhar neighborhood and Khan El Khalily market.

Atkinson & Mair (1981) investigated that the same basic concepts of soil mechanics that apply to retaining walls, slopes, and foundations were also apply to the stability of tunnels in soft ground and the settlements induced by tunneling.

Tunnel engineers, on the other hand, use terms like ground loss, squeeze, and stand-up time, to describe specific components of the tunneling process. This work covers the stresses and pore pressures in soft ground induced by tunneling, as well as the calculations needed to estimate the tunnel's stability and heading, as well as the settlements caused by tunneling.

Herzog et al. (1985) a simple and transparent conceptual model is developed based on measurements from modern underground and road tunnel construction that allows for the accurate prediction of projected settlement troughs over shallow tunnels without the use of finite elements or higher mathematics.

Hoek et al. (2002) described the Hoek-Brown rock mass failure criteria which are generally acknowledged and have been used in a variety of projects throughout the world. While it has been proven to be adequate in general, it has significant uncertainties and errors that make it difficult to implement and incorporate into numerical models and limit equilibrium algorithms. The failure criterion given by Hoek-Brown provides a way for inputting the data for the analysis required for the design of the underground excavation in hard rock. The criterion provides a graph between the major and minor principal stresses after considering the uniaxial compressive strength, intact model parameter, elastic modulus, GSI, model parameter, and distribution factor.

Rankin (1988) offered advice on calculating the impact of 'soft ground' tunneling in metropolitan locations on existing structures and services. The size and distribution of surface movements are assessed using case history data, and several empirical techniques to defining the surface settlement zone are presented. A tentative risk categorization based on settlement and maximum slope criteria is presented, which would allow for faster route optimization and, as a result, identification of buildings that are particularly vulnerable and require further investigation.

CHAPTER- 3

MATERIALS AND METHODS

3.1) Empirical Approach for Surface Deformation due to Tunnelling-

The assumptions for estimation of tunneling deformation using empirical approach are as follows:

Most of the tunnels are excavated in areas where surface constructions and underground cables and pipes are present already. To find the influence of tunneling on already constructed structures, we should know about the settlement troughs developed by Surface and underground structures as proposed by R.B. Peck (1969). observations from the field were analysed and concluded that the Gaussian functions satisfy reasonably the trough of surface settlement and also analyzed the observations from the field and concluded that the Gaussian functions satisfy reasonably the trough of surface settlement. There are two parameters that are included in Gaussian Function, these are:

1. The Maximum ground Settlement is denoted by Smax.

2. The width coefficient of settlement trough which is denoted by i.

Settlement trough conforming to a Gaussian distribution curve (the approach adopted for movements due to soft ground tunneling is based on relation proposed by (R.B.Peck, 1969); (New, 1982) and (O'Reilly, 1991)) is used in this assessment;

$$S=S_{max}\ e^{-\frac{y^2}{2i^2}} \tag{3.1}$$

Where,

S = vertical ground surface settlement

S_{max}= Maximum settlement at y = 0

ΔV = Volume of the settlement trough

= Ground loss Factor (GL) × tunnel face area (πR^2)

Where R = Tunnel Radius

K = Trough parameter

Z = Vertical distance between ground level and tunnel axis

i = Location of maximum settlement gradient or point of inflection i = K·z

Combining the above formulae for a circular tunnel (radius R), gives:

$$S_{max}=\frac{\Delta V}{\sqrt{2\pi}i}=\frac{GL\times\pi R^2}{\sqrt{2\pi}Kz}=1.2533\frac{R^2(GL)}{Kz} \tag{3.2}$$

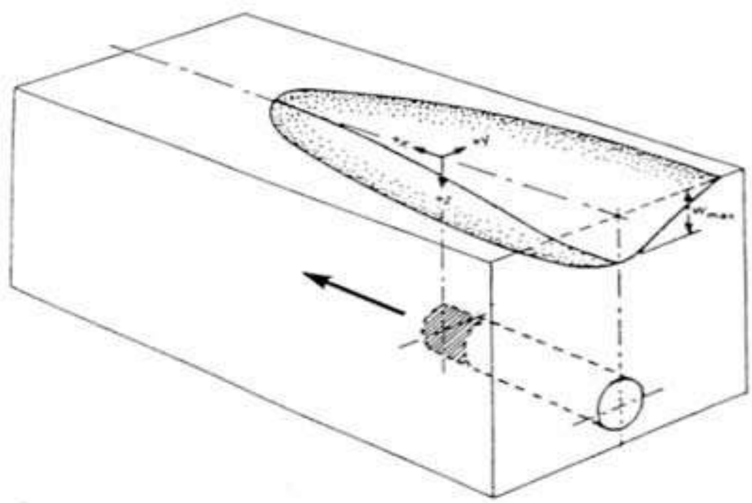


Figure 8**:** Settlements with “green field” conditions (R.B.Peck, 1969)

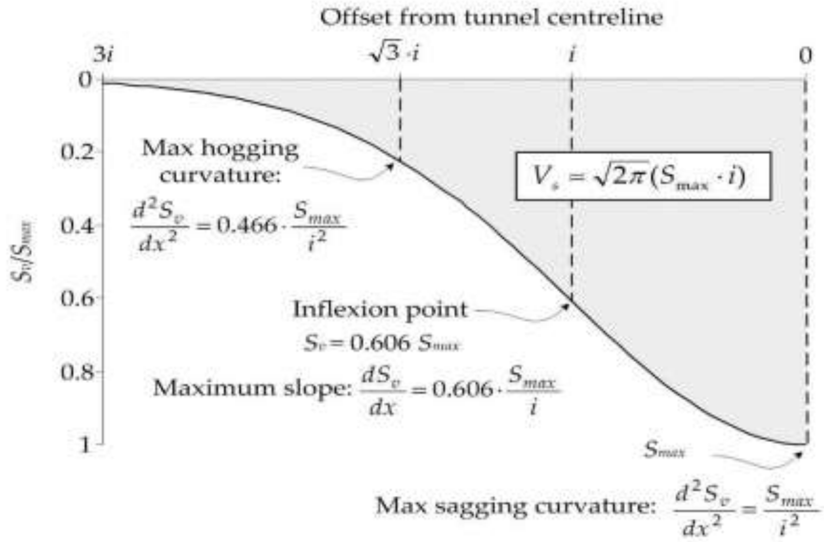


Figure 9**:** Settlements curve, Peck 1969

For the twin-tube tunnels, the superimposed settlement is given by the sum of each displacement curve of the two single tunnels. TBM tunnels are located in good to fairground conditions (rock, basalts & breccia) therefore the low value of volume loss is expected. Based on the geological description, the tunnels are generally driven through “discontinuous rock mass and weak rock” and the overburden would be of discontinuous weathered rock, or mixed conditions (rock mass and soil).

There are various technique for determining the point of inflexion (i) that have been proposed. In analysis, i value is estimated based on the average of different empirical methodologies presented in Equations, ((Arioglu, 1992), (New, 1982), (Hamza, 1999), (Mair, 1981)):

$$i = \frac{i_1+i_2+i_3+i_4}{4} \quad (3.3)$$

$$i_1 = 0.386Z_\circ + 21.84 \quad (3.4)$$

$$i_2 = 0.5Z_\circ \quad (3.5)$$

$$i_3 = 0.9R\left(\frac{Z_\circ}{2R}\right)^{.88} \tag{3.6}$$

$$i_4 = 0.43Z_\circ + 1.1 \tag{3.7}$$

Where, $Z_\circ$ the depth (m) of the tunnel axis and D is the diameter of the tunnel. Despite several studies on the calculation of twin tunnel interaction, there are few empirical relationships for determining the volume and shape of surface settlement curves caused by twin tunnel excavation. (Herzog, (1985)) provided an equation for estimating the maximum vertical surface settling in twin tunnels. The formula is as follows:

$$S_{max} = 4.71(\gamma Z_\circ + \sigma_s).\left(\frac{D^2}{(3i+a).E}\right) \tag{3.8}$$

Where,

E = modulus of elasticity of formation

γ = natural unit weight

σ_s= surface surcharge

a = spacing between the tunnel axis

$Z_\circ$ = depth of the tunnel.

3.2) Numerical approach for surface Deformation due to Tunnelling-

3.2.1) Design software-

The analysis is carried out using Ansys workbench version 19.2 which is available in DTU CAD Lab. the performed Models are axial-symmetrical, due to the circular shape of the tunnel.

3.2.2) General Modelling method-

The same basic approach was used to construct all of the simulation models. The following event is depicted in chronological order-

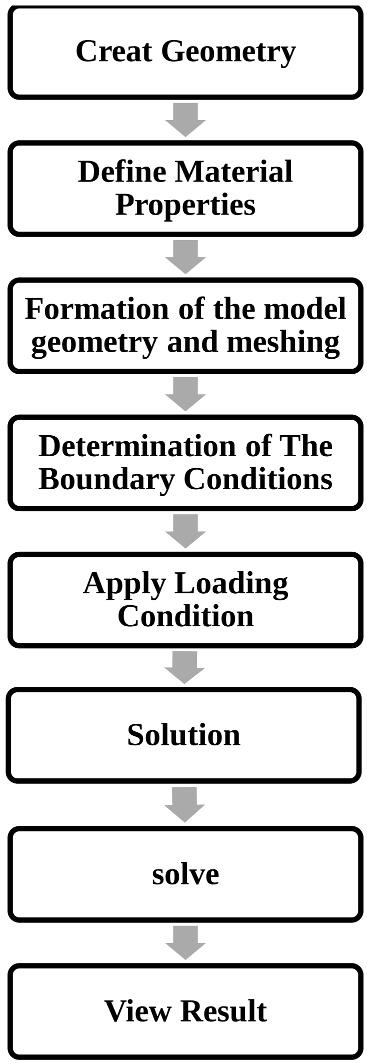


Figure 10: Flow chart for simulation model

The physical conditions of the region to be modeled are referred to model geometry and meshing. The response of a model to a given loading condition is known as model behaviour. The model's physical limits and original conditions are explained using boundary and initial conditions.

3.2.3) Assumptions-

The main focus of this research is on the impact of rock mass characteristics and tunnel spacing on the stability of existing tunnels. As a result, various numerical analysis assumptions are made to make the computation process easier. The following are the fundamental assumptions:

i) The rock mass behavior follows the Hoek-Brown failure criterion.
ii) In the numerical model, the shield tunneling approach is used.
iii) Shell elements support the tunnels, and the tunnel lining is 0.3 meters thick. The shield tunnel segment is made of mix of grade M40.
iv) The adjacent rock creep isn't taken into account.
v) The numerical model does not take into account tunnel boring machine design factors such as face support pressure, thrust force, and grouting pressure, among others.
vi) The value of ψ=0, corresponds to the volume preserving deformation while in shear.

3.2.4) Design Limitations-

The validity of the presented calculations is limited to the boundary conditions and information known and valid at the time of preparing this document. Any subsequent change in these boundary conditions or updating of presently known information, such as ground characteristics, design parameters, etc., will require a check of the applicability of this calculation.

3.2.5) General model description-

The 'static structure' analysis function in Ansys software is being used to fully consider the three-dimensional effects on the Finite Element model of weak rock and the tunnel assembly. For the elastic-plastic behavior of rock, the Mohr-Coulomb model is used in this numerical Investigation. It is assumed that the rock mass is homogenous, isotropic, continuous, and semi-infinite**.**

The following are the model geometry and meshing instructions (Hamid Chakeri, 2014):

i) The longitudinal dimension of this model (in the y-direction) is 74.3 m (approximately 11.3D),

ii) An extension under the tunnel axis (in the z-direction) of 44.77 m (approximately 6.8D)

iii) On the tunnel axis, a lateral expansion (in the +x direction) is at least eight times greater than the cover (94.56m).

iv) The model is subjected to standard boundary conditions, which means that no horizontal deformations are permitted in the vertical boundaries and no vertical deformations are permitted in the model's bottom border.

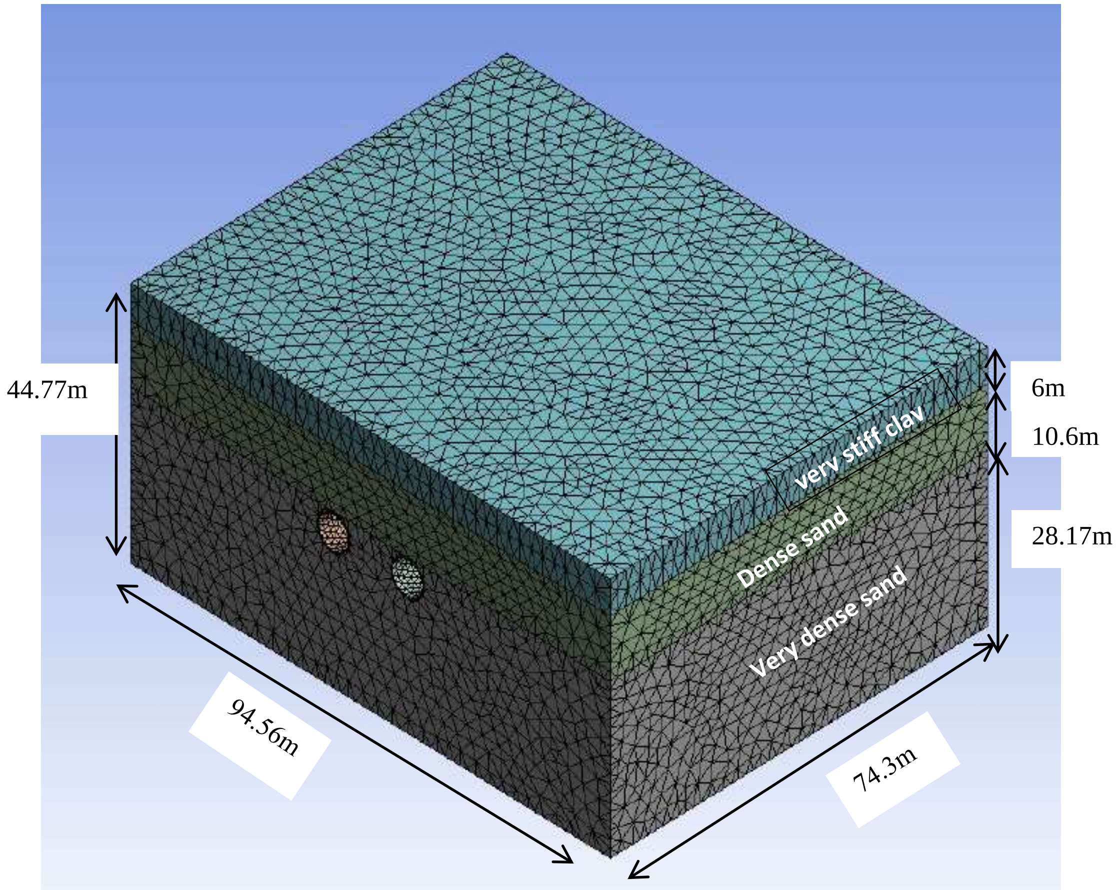


Figure 11: Three-dimensional view of the model with mesh and block property (Hamid Chakeri, 2014)

3.2.6) Soil and rock constitutive material models-

i) Mohr-Coulomb material model-

Most of the geotechnical software uses the Mohr-Coulomb failure criterion so it becomes necessary to determine the cohesive strength and angle of internal friction for each rock mass and strength range**.** The result**s** are given in the form of the equation which involves the angle of friction in degree and cohesion value in MPa and a linear relationship between major and minor principal stresses.

ii) Hoek-Brown criterion-

The failure criterion given by Hoek-Brown provides a way for inputting the data for the analysis required for the design of the underground excavation in hard rock. The criterion provides a graph between the major and minor principal stresses after considering the uniaxial compressive strength, intact model parameter, elastic modulus, geological strength index (GSI), model parameter, and distribution factor.

3.2.7) Rock Mass Properties-

The Hoek-Brown criteria are used in this research, as stated previously. The following equations are used to preliminarily determine the rock mass characteristics for the numerical studies carried out in this study. (Hoek and Brown, 2018), (Hoek, 2002)

$$m_b = m_i exp\left(\frac{GSI-100}{28-14D}\right) \tag{3.9}$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right) \tag{3.10}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \tag{3.11}$$

$$\phi = \sin^{-1}\left[\frac{6am_b(s+m_b\sigma_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\sigma_{3n})^{a-1}}\right] \tag{3.12}$$

$$c = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\sigma_{3n}](s+m_b\sigma_{3n})^{a-1}}{(1+a)(2+a)\sqrt{(1+(6am_b(s+m_b\sigma_{3n})^{a-1})/(1+a)(2+a)}} \tag{3.13}$$

Where,

$$\sigma_{3n} = \sigma_{3max}/\sigma_{ci} \tag{3.14}$$

$$\frac{\sigma_{3max}}{\sigma_{cm}} = .47\left(\frac{\sigma_{cm}}{\gamma H}\right)^{-.94} \tag{3.15}$$

With c and ϕ determined for the stress range $\sigma_t < \sigma_3 < \sigma_{ci}/4$ giving,

$$\sigma_{cm} = \sigma_{ci}.\frac{\left(m_b+4s-a(m_b-8s)\right)\left({}^{m_b}/_4+s\right)^{a-1}}{2(1+a)(2+a)} \tag{3.16}$$

Where GSI is geological strength index, D is disturbance factor which is set to 0 in shield tunneling, m_i is the material constant of intact rock, m_b is a reduced value of the material constantm_i, s and a are constants for the rock mass, σ_{ci} is the uniaxial compressive strength of the intact rock material, σ_{3max} is the top limit of confining stress at which the Hoek-Brown and Mohr-Coulomb criteria are considered. σ_{cm} is the rock mass strength, γ is the unit weight of rock mass and H is the depth of the tunnel below the surface.

3.3) Geology, Geotechnical Parameters-

The tunnels will mainly pass through volcanic Breccia GIII and Breccia GIV. So for analysis, it is considered that the tunnel is passing through Breccia GIV. The geological profile around the tunnel area is shown in lithology.

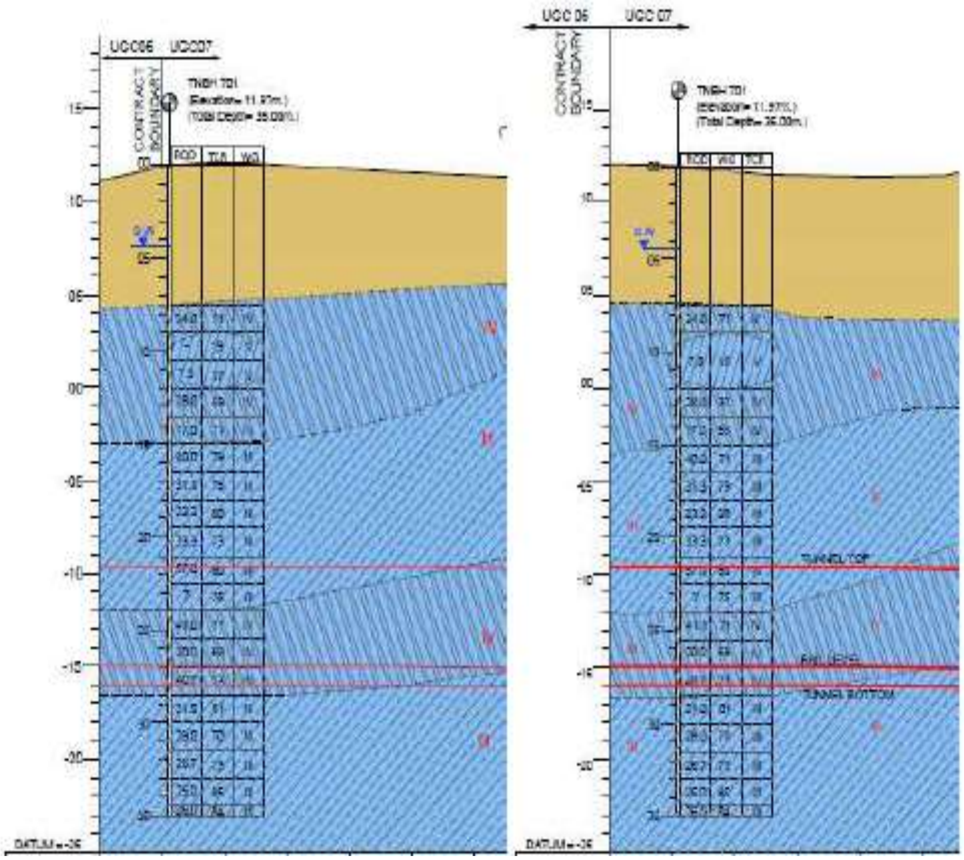


Figure 12: Geological profile around the tunnel (N.R., 2019)

Table 1: Lithology considered in Ansys Model

Mumbai metro rail project (For analysis)	Strata depth (in meters)	(Hamid Chakeri, 2014) (For validation)
Residual soil	6.00	Very stiff clay
Breccia 3	10.60	Dense sand
Breccia 4	24.89	Very dense sand

3.4) Geotechnical Design Parameters-

For validation-

Table 2: Design parameter of soil (Hamid Chakeri, 2014)

Parameter	Name	Unit	Very stiff clay	Dense sand	Very dense sand
Material Model	Model	-	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Soil unit weight	γ	KN/m^3	18.2	19	19.5
Young's modulus	E	KN/m^2	51×10^3	24×10^3	30×10^3
Poisson ratio	μ	-	0.35	0.25	0.30
Cohesion	c	KN/m^2	20	1	1
Friction angle	ϕ	$degree$	9	35	35

For Analysis (Soil Material) –

Table 3: Design parameter of soil (Mumbai metro project (N.R., 2019))

Parameter	Name	Unit	Residual soil
Material Model	Model	-	Mohr-Coulomb
Soil unit weight	γ	KN/m^3	19
Young's modulus	E	KN/m^2	21×10^3
Poisson ratio	μ	-	0.3
Cohesion	c	KN/m^2	50
Friction angle	ϕ	$degree$	28
Dilantance angle	ψ	$degree$	0

Rock material-

Table 4: Design parameter of rock (Mumbai metro project, (N.R., 2019))

Parameter	Name	Unit	Breccia GIV	Breccia GIII
Material Model	Model	-	Hoek-Brown	Hoek-Brown
Rock unit weight	γ	KN/m^3	23.5	23.5
Young's modulus	E	KN/m^2	400×10^3	1.67×10^6
Poisson ratio	μ	-	0.25	0.25
Uniaxial compressive strength	σ_{ci}	KN/m^2	18×10^3	25×10^3
Material constant for intact rock	m_i	-	19	19
Geological strength index	GSI	-	18	36
Disturbance factor	D	-	0	0
Dilantance angle	ψ	$degree$	0	5
Cohesion	c^*	KN/m^2	75.65	138.69
Friction angle	$\phi*$	$degree$	48.247	56.81

[$\boldsymbol{c}^*$, $\phi*$ value calculated by using equations **{(3.9) to (3.16)}** (Hoek, 2002)]

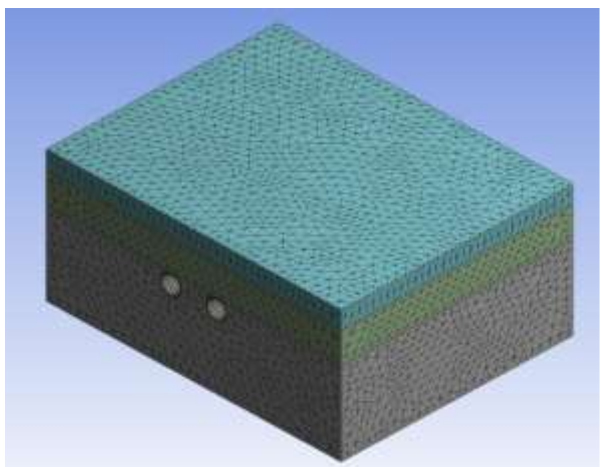
Material properties for tunnel lining-

Table 5: Design parameter of the lining (Hamid Chakeri, 2014)

Parameter	Name	Unit	Concrete
Material Model	Model	-	Linear elastic
unit weight	γ	KN/m^3	23.54
Young's modulus	E	KN/m^2	3×10^9
Poisson ratio	μ	-	0.18
Thickness	-	meter	0.3

3.5) Details of Meshing-

The most crucial aspect of numerical modelling is meshing. Meshing size is considered for the analysis is based on the mesh convergence study. For the model, meshing size is adopted as 1500mm and for the lining it is adopted as 1000mm. The total numbers of nodes and elements are 129456 and 70092 respectively. Model and concrete lining have tetrahedron meshing applied on them. Tetrahedron mesh geometry is similar to that of a triangular pyramid, with six straight edges, four triangular faces, and four vertex corners. Tetrahedron meshing is used for complex geometry, such as the one employed in this study, whereas hexahedron meshing is used for simple geometry.



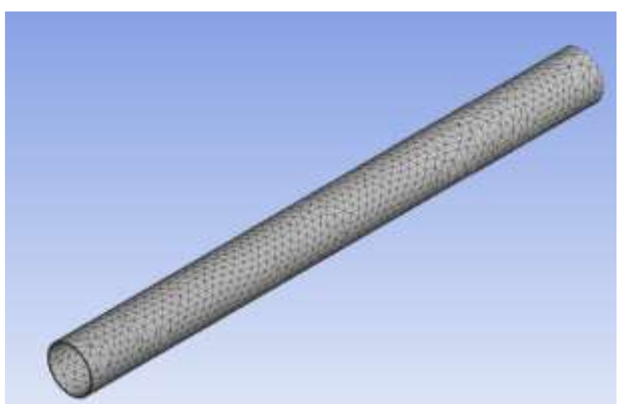


Figure 13: Representation of model and lining meshing

3.6) Loading and boundary condition-

Load is applied on the top surface of model. For study it is assumed that the 40 kPa surcharge is applied on the top surface of model and it is uniformly distributed on the surface. The model is subjected to standard boundary conditions, which means that no horizontal deformations are permitted in the vertical boundaries and no vertical and horizontal deformations are permitted in the model's bottom border.

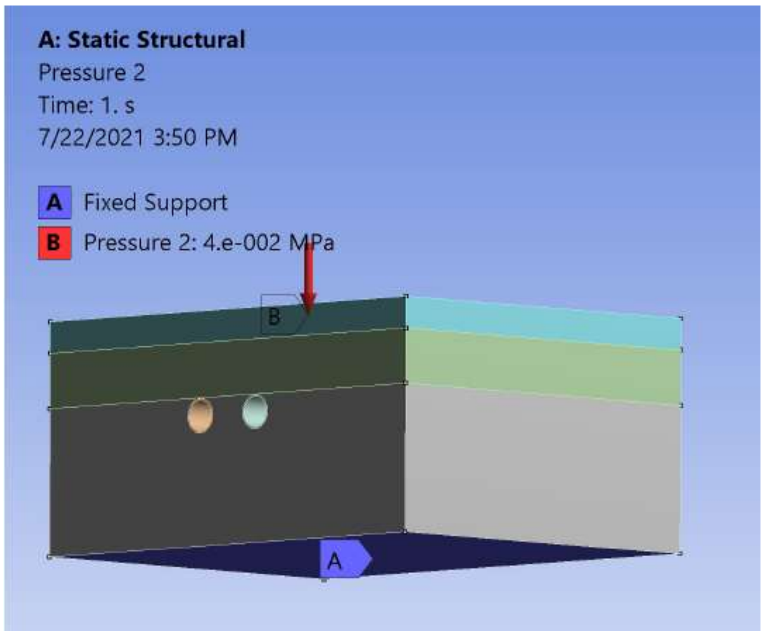


Figure 14: The view of load setup and boundary condition

Assembly of tunnel models

CHAPTER- 4

RESULTS AND DISCUSSION

4.1) Numerical Modeling Validation-

The most essential aspect in evaluating numerical modeling results is determining whether the numerical findings are correct or not. The empirical equation presented by Herzog(1985) for the twin tunnel is used to evaluate and analyse the validity and applicability of the Ansys programme for geotechnical condition modeling. The equation is given as,

$$S_{max} = 4.71(\gamma Z_\circ + \sigma_s).\left(\frac{D^2}{(3i+a).E}\right)$$

D is tunnel diameter which is 6.56m in this study; E is the weightage average modulus of elasticity of the formation for the entire three layers which is calculated as 35000 kPa, ‘a’ is the distance between the axis of the tunnel which is 14.56m in this study, γ is the weightage average natural unit weight for all the three layers which is 18.9 kN/m^3; σ_s is total surcharge pressure which is assumed to be 40 kPa in this study; ‘i’ is the horizontal distance between the tunnel centreline and the point of inflexion on surface settlement trough which is calculated based on equation {(3.3) to (3.7)}((Arioglu, 1992), (New, 1982), (Hamza, 1999), (Mair, 1981)) which is 10.84m in this study and $Z_\circ$ is the tunnel depth which is 19.88m in this study. All the design parameters are given in table 2.

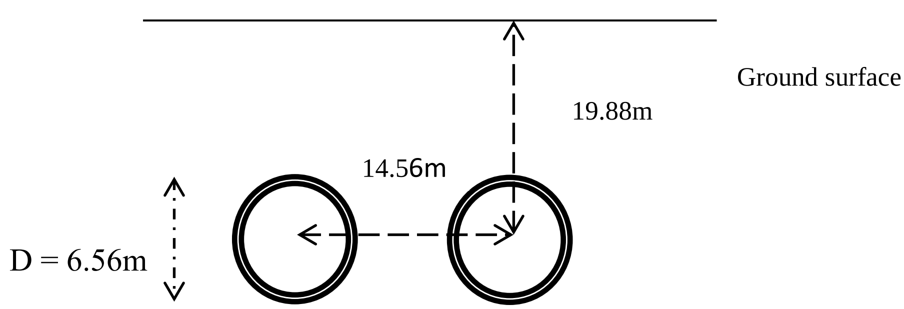


Table 6: Settlement value based on empirical and numerical data

σ_s (kPa)	$Z_\circ$(m)	i (m) Herzog	S_{max}(mm) Herzog	S_{max}(mm) Ansys
40	19.88	10.84	51	42.661

The results obtained from numerical analysis using ANSYS software has a lower deformation value than the result obtained from the empirical equation given by (Herzog, (1985)). deformation obtained from ANSYS is about 16% less with respect to the result obtained from the empirical equation, which is under the permissible limit. Previous researchers (Ercelebi, 2011) (Hamid Chakeri, 2014) result show that the result obtained from the Herzog equation generally shows a higher settlement value than the observed one.

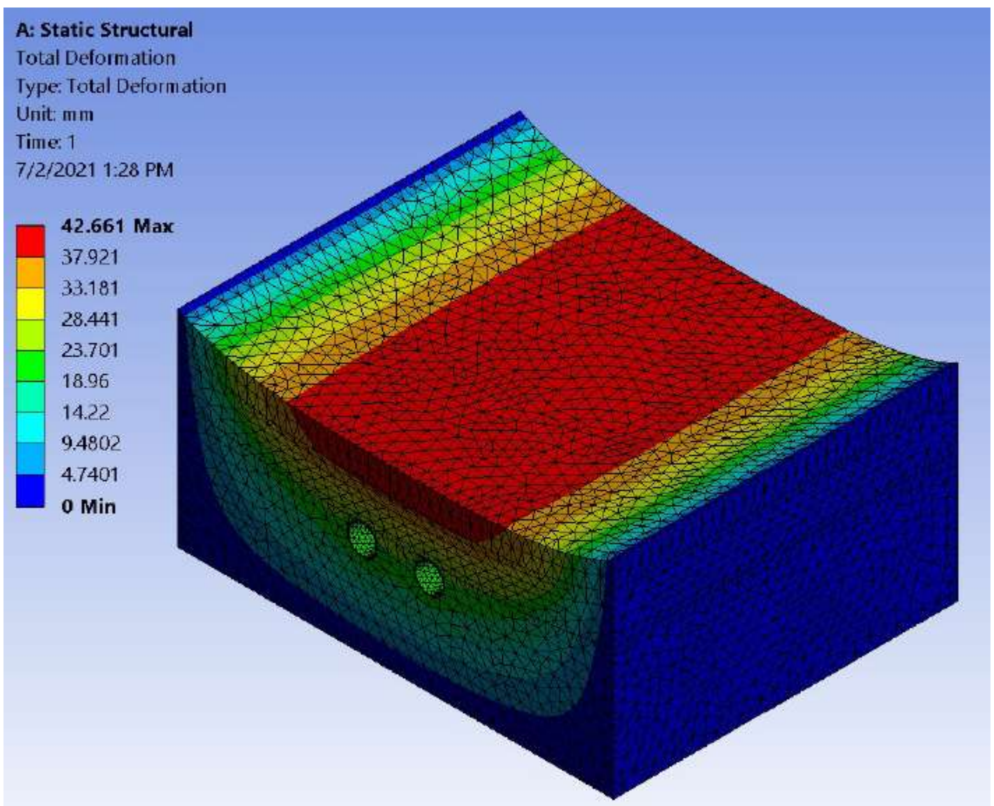


Figure15: Deformation profile for validation result

4.2) Results obtained from numerical modeling -

For safety assessments, in every case two analysis has been performed, one for the tunnel with lining and other is for tunnel without lining to see the effect of lining on deformation and safety point of view.

The following cases have been subjected to numerical analysis:

Case 1: Mixed ground (soil-rock) condition

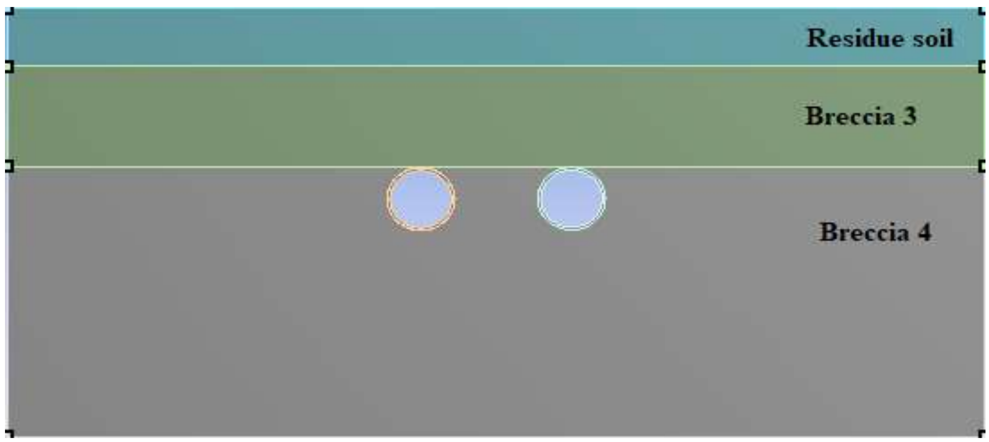
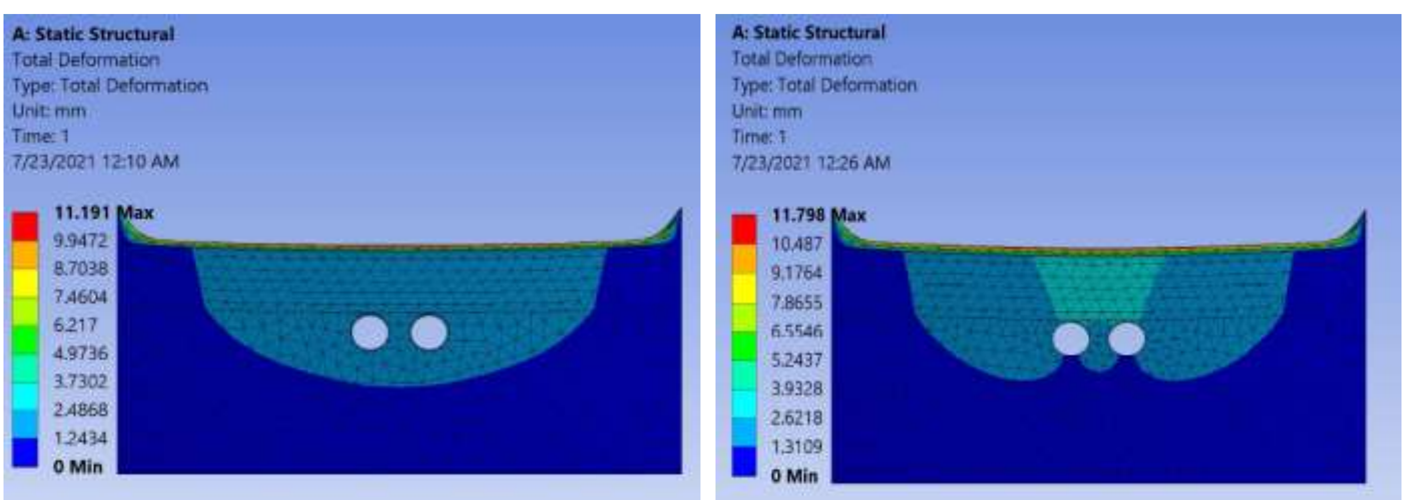
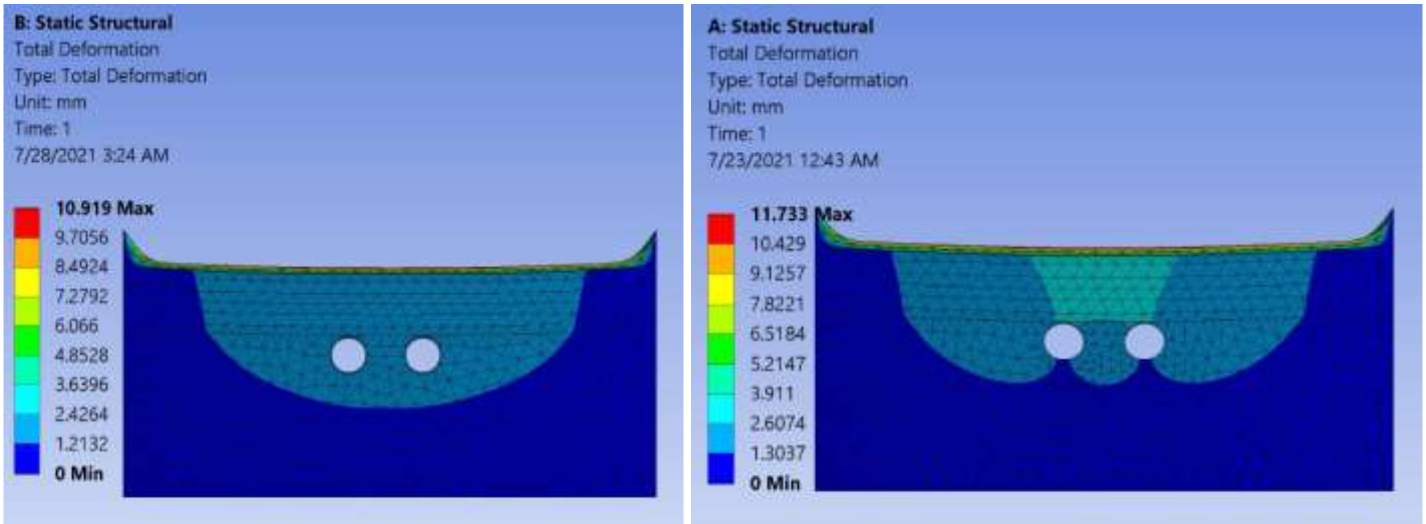


Figure16: View of model with block properties

with lining **without lining**

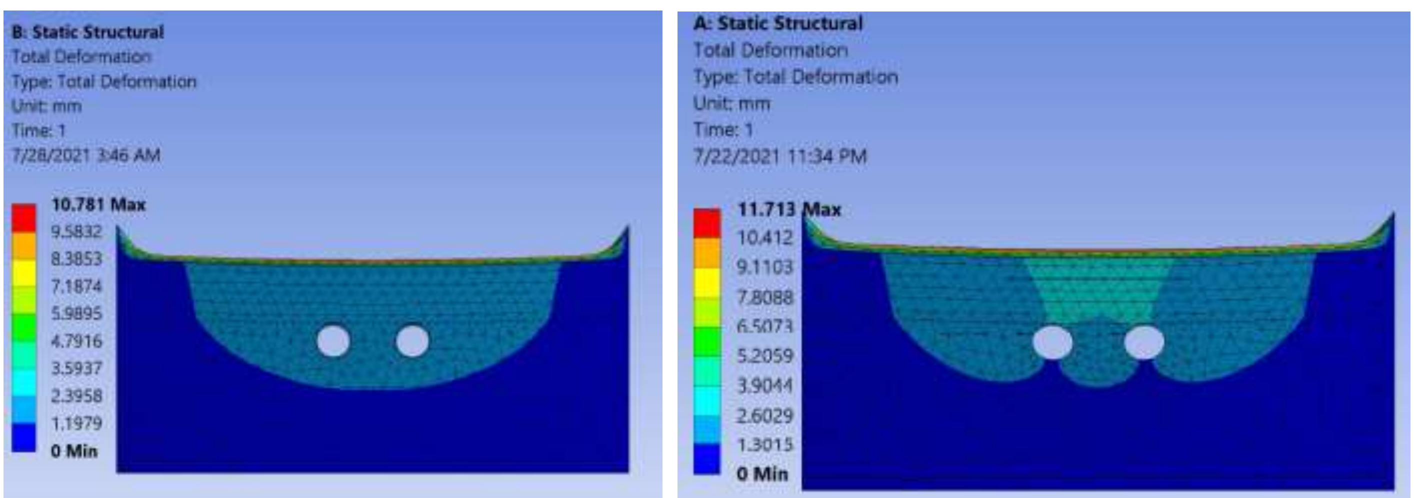


(a) Spacing between tunnels 1.5D (9.84m) with lining without lining



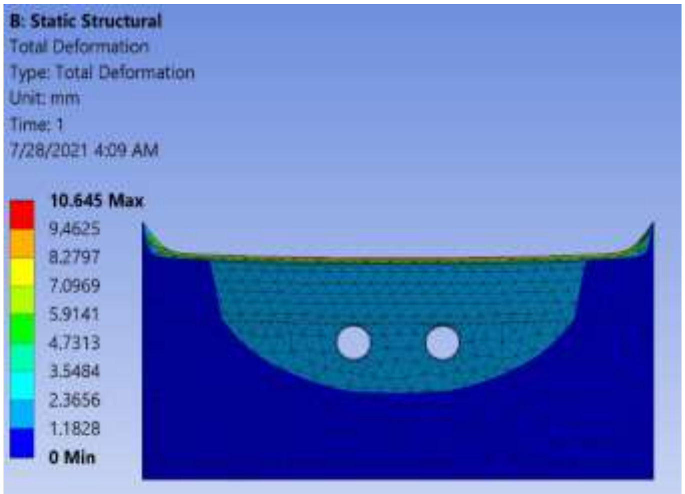
(b) Spacing between tunnels 2D (13.12m)

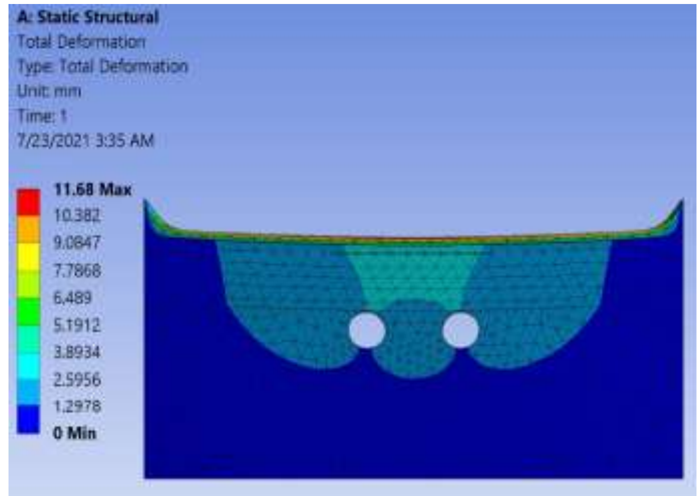
with lining without lining



(c) Spacing between tunnels 2.2D (14.56m)

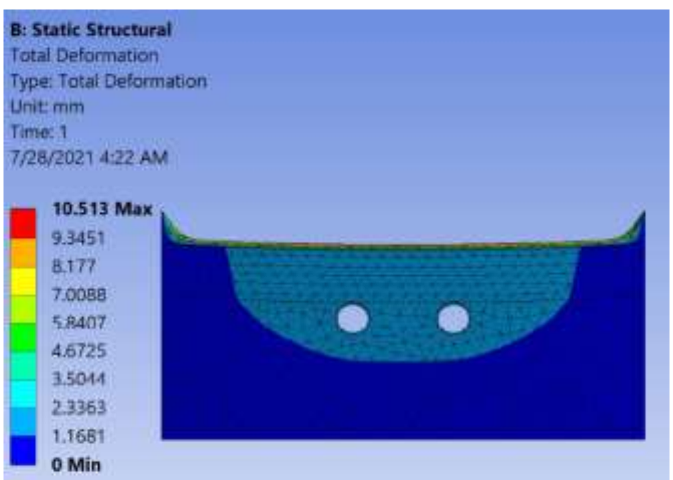
with lining **without lining**

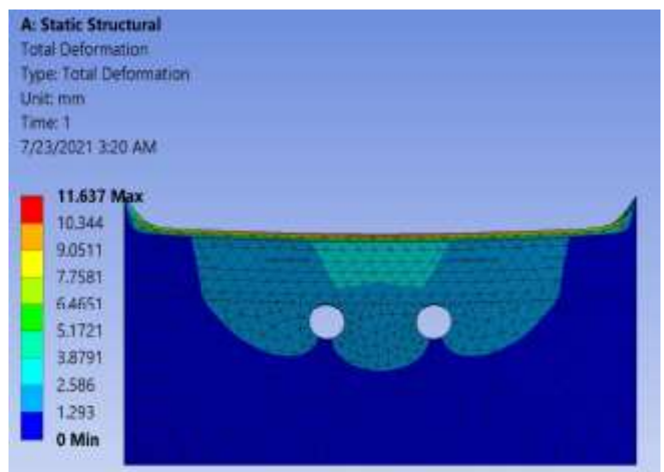




(d) Spacing between tunnels 2.5D (16.4m)

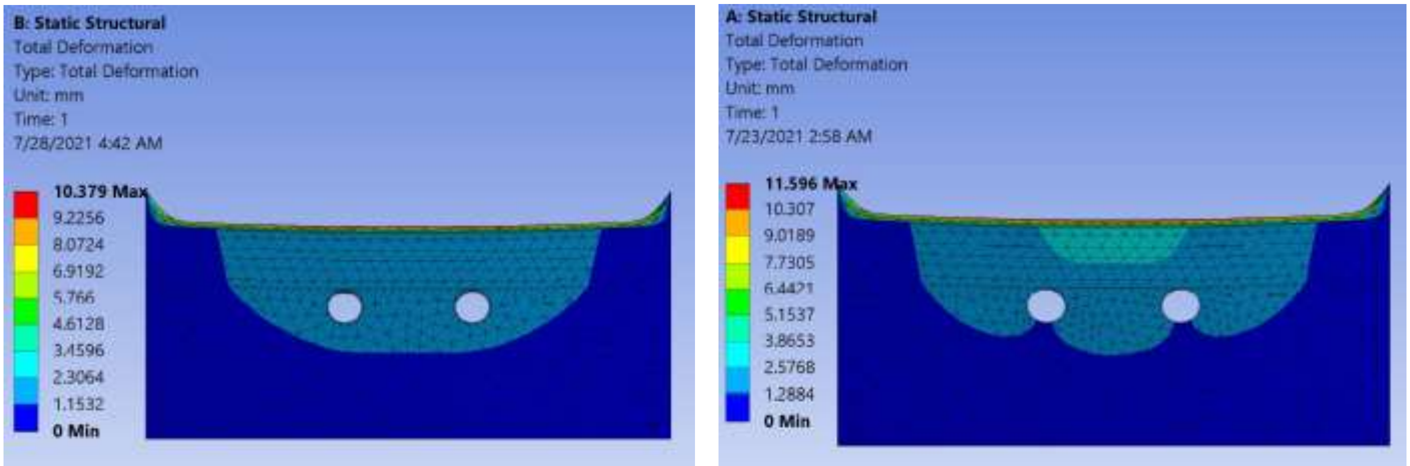
with lining **without lining**





(e)Spacing between tunnels 3D (19.68m)

with lining **without lining**



(f) Spacing between tunnels 3.5D (22.96 m)

Figure 17: Vertical surface settlement contour for six-tunnel spacing in mixed ground condition

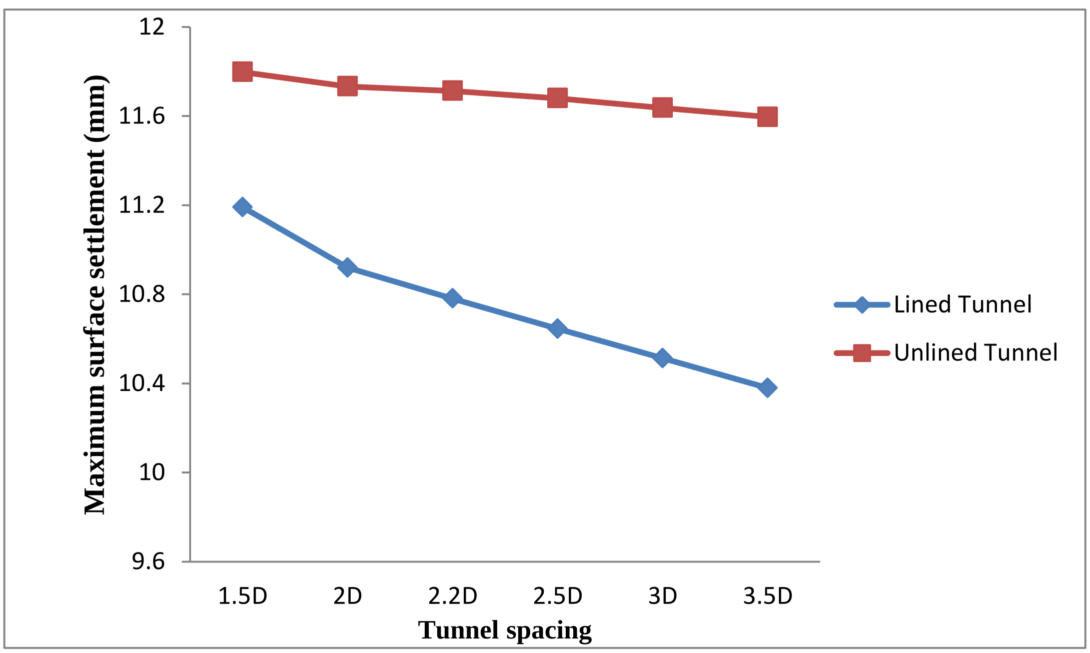
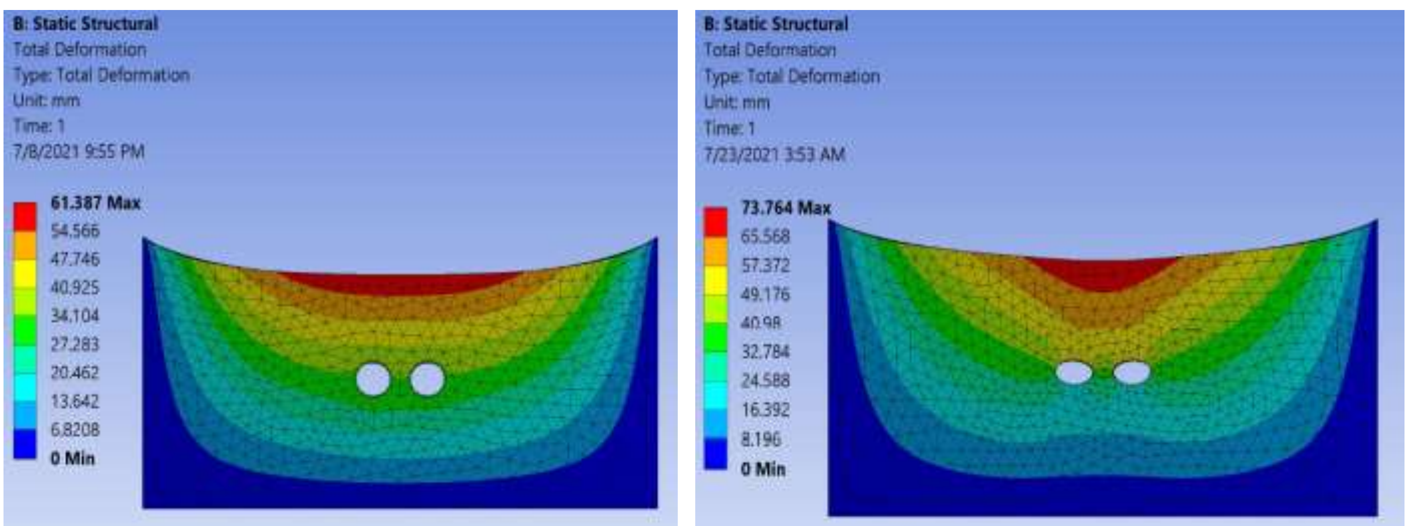


Figure 18: Maximum surface settlement curve for different spacing having mixed ground

For mixed ground conditions, the upper layer consists of residual soil, the middle layer is breccia grade 3 and the last layer is breccia grade 4. Tunnel face material is breccia 4 for this condition. Figure 17 shows the maximum surface settlement curve for tunnel with lining and without lining with respect to tunnel spacing in the mixed ground. In this condition the maximum surface settlement for 1.5D spacing is 5.42% more for unlined tunnel as compare to lined tunnel and for 3.5D spacing maximum surface settlement is 11.725% more for unlined tunnel as compare to lined**.**

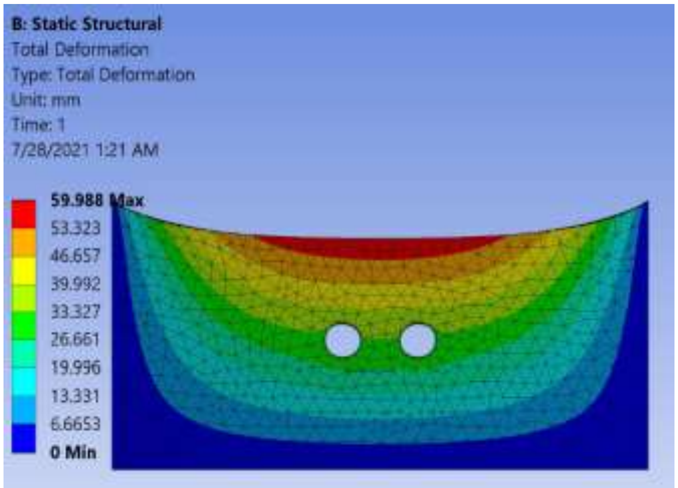
Case 2: Homogeneous ground (soil) condition

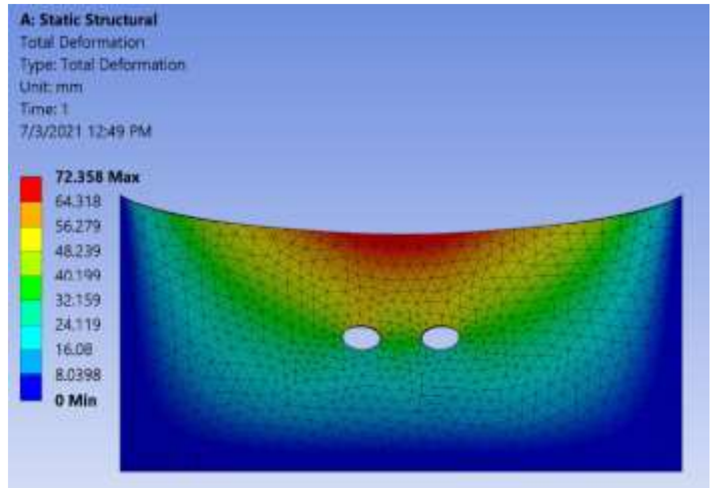
with lining **without lining**



(a) Spacing between tunnels 1.5D (9.84m)

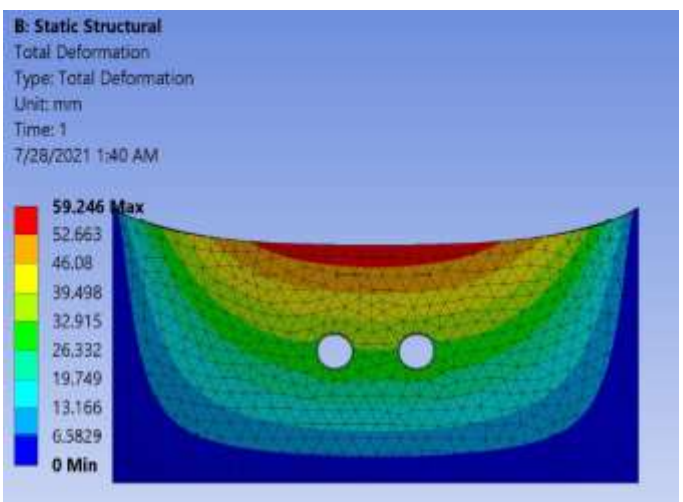
with lining **without lining**

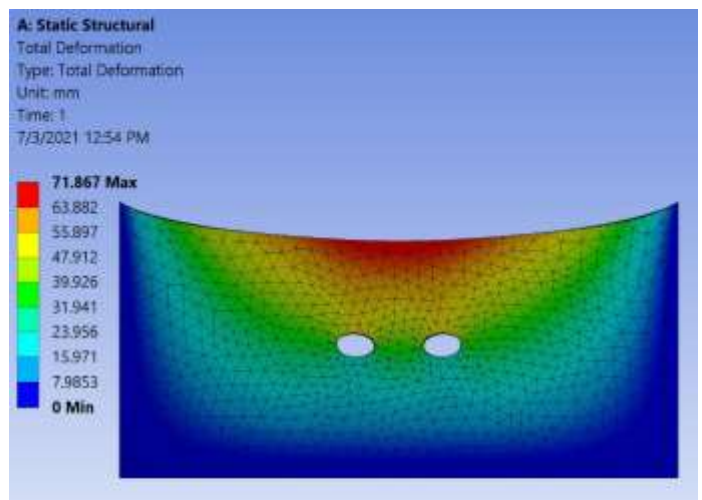




(b) Spacing between tunnels 2D (13.12m)

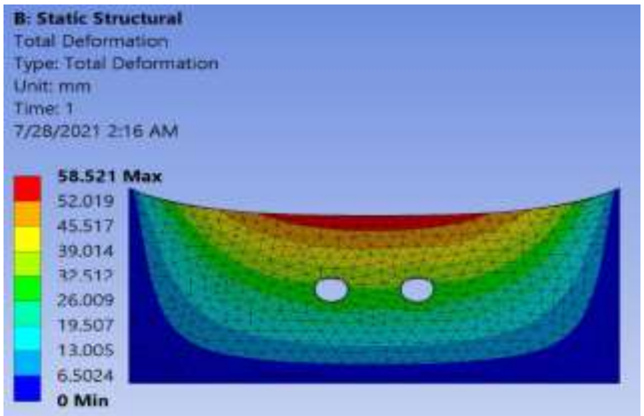
with lining **without lining**

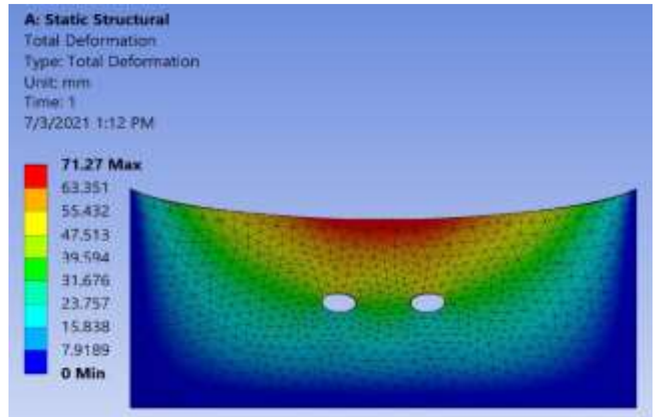




(c)Spacing between tunnels 2.2D (14.56m)

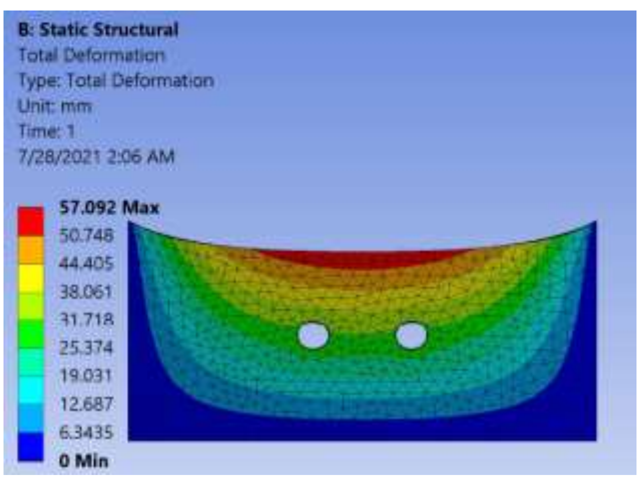
with lining **without lining**

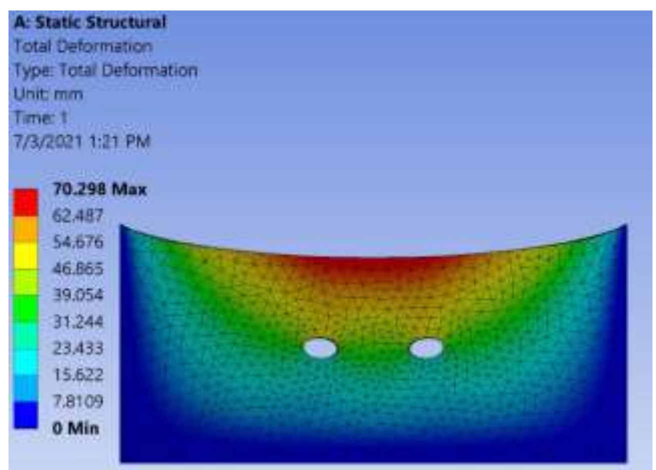




(d)Spacing between tunnels 2.5D (16.4m)

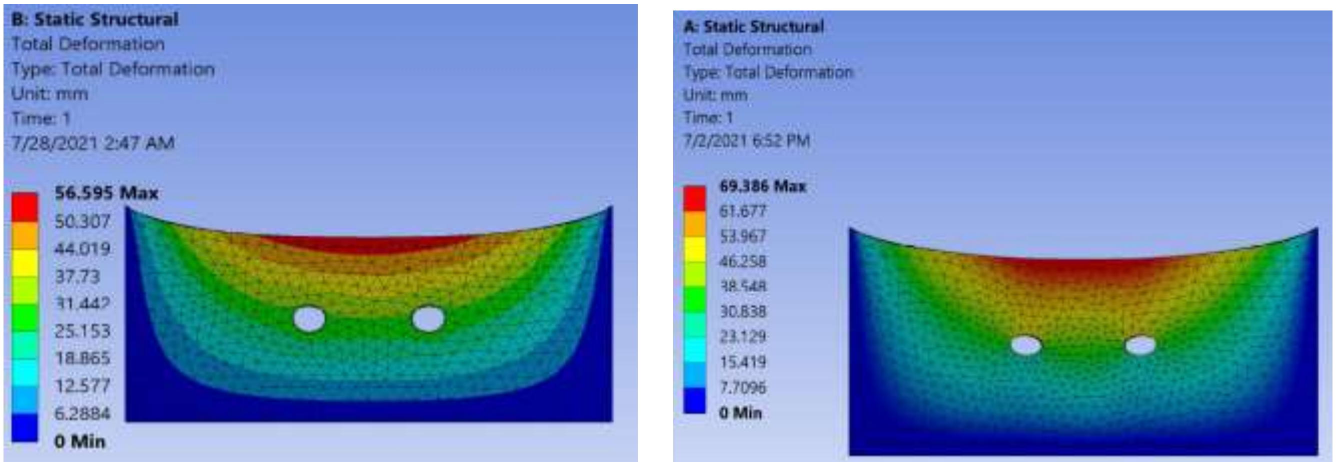
with lining **without lining**





(e)Spacing between tunnels 3D (19.68m)

with lining **without lining**



(f)Spacing between tunnels 3.5D (22.96 m)

Figure 19: Vertical surface settlement contour for six-tunnel spacing in residual soil condition

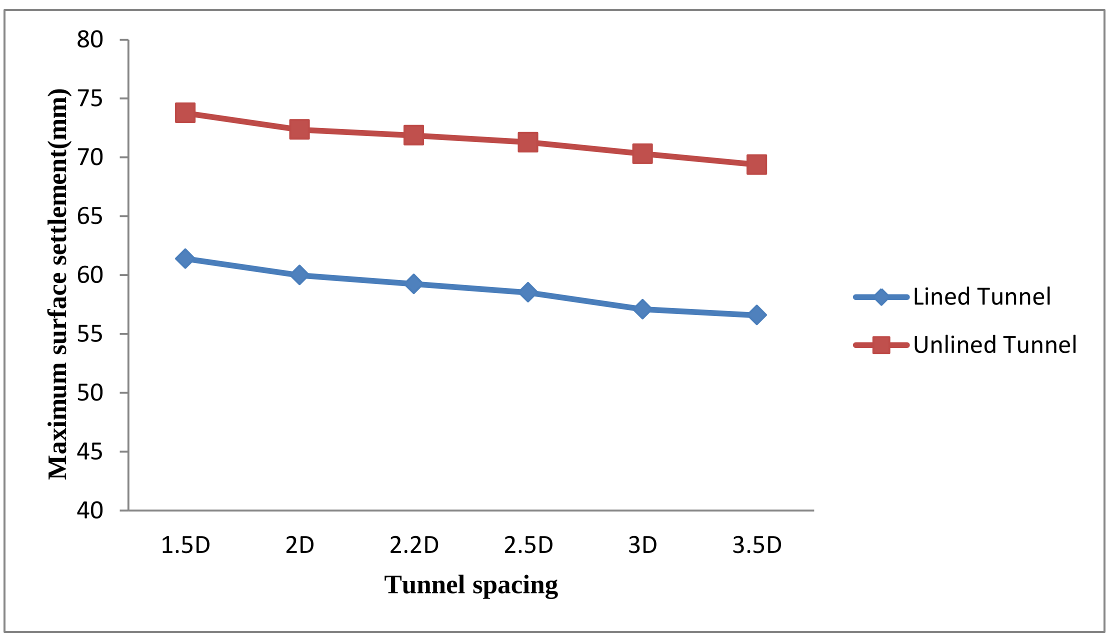
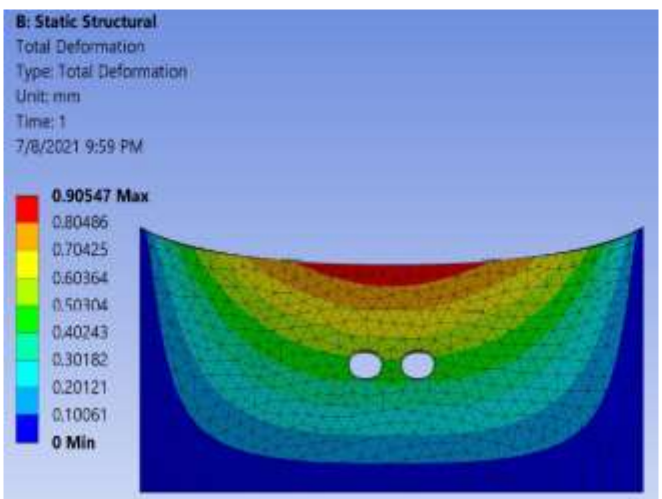


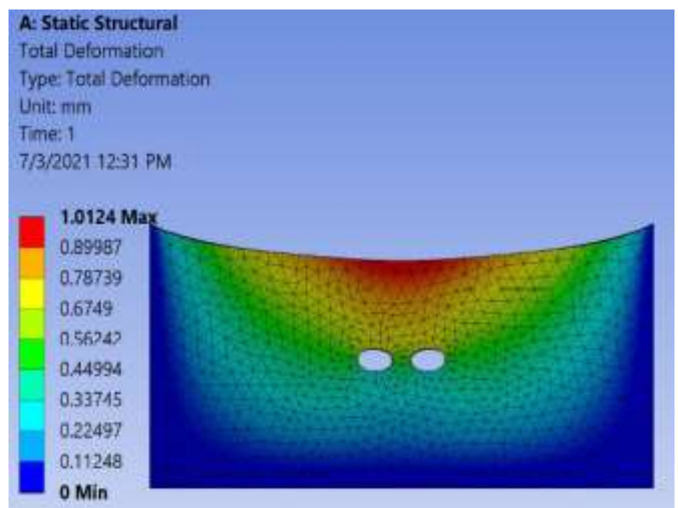
Figure 20: Maximum surface settlement curve for different spacing having residual soil

The tunnel face material is residual soil for this condition. Figure 19 shows the maximum surface settlement curve for the tunnel with lining and without lining with respect to tunnel spacing having residual soil. In this condition, the maximum surface settlement for 1.5D spacing is 20.162% more for the unlined tunnel as compared to the lined tunnel and for 3.5D spacing maximum surface settlement is 22.60% more for the unlined tunnel as compare to lined. The maximum surface settlement also depends on the deformation modulus of the tunnel face material. In this case deformation modulus of residual soil is very less as compared to the other two so more percentage increment in the surface settlement is observed for the lined and unlined tunnel.

Case 3: Homogeneous ground (Breccia 3) condition

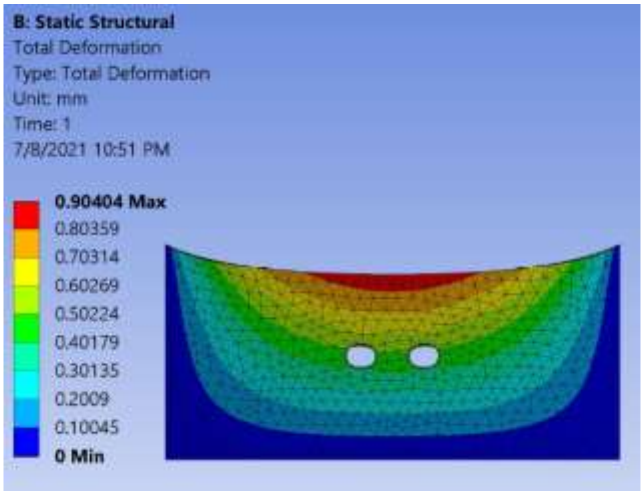
with lining **without lining**

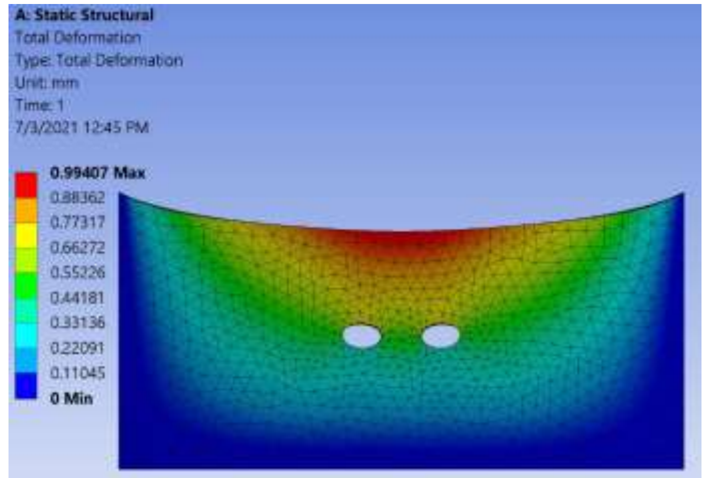




(a)Spacing between tunnels 1.5D (9.84m)

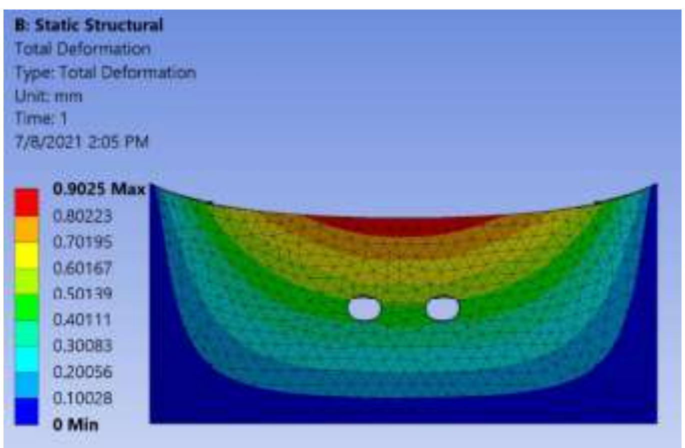
with lining **without lining**

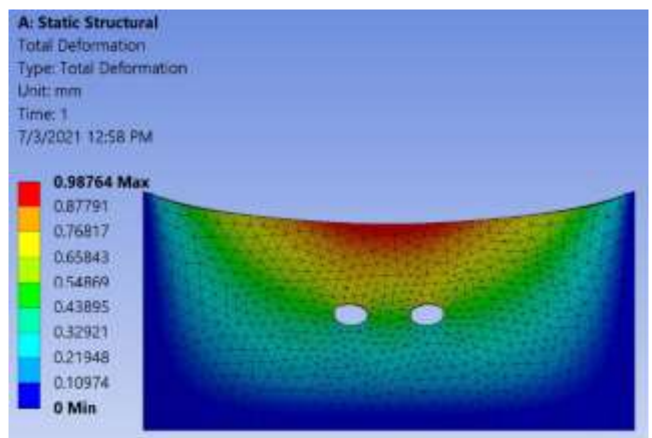




(b)Spacing between tunnels 2D (13.12m)

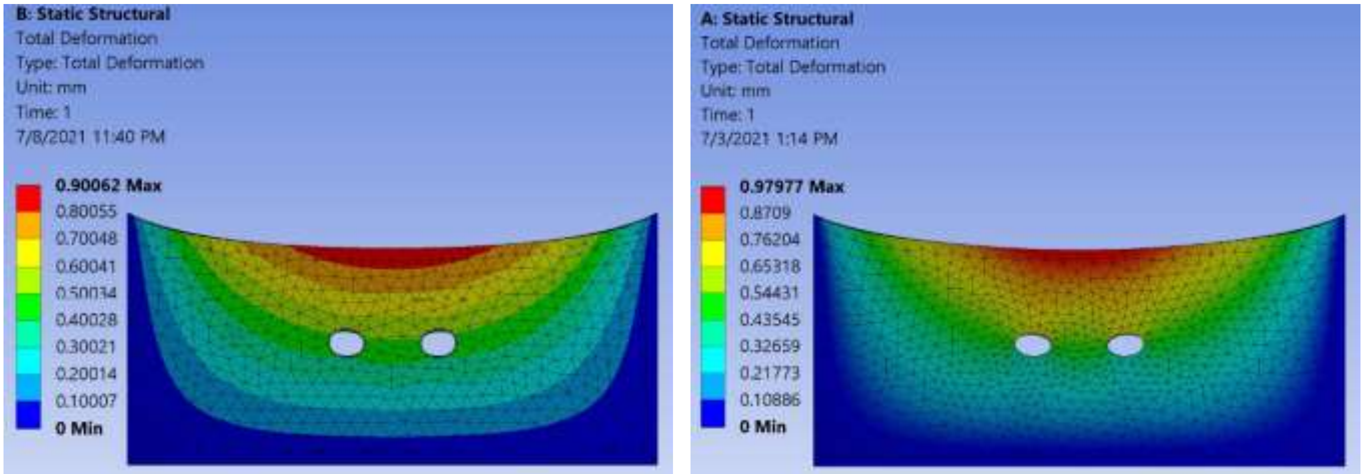
With lining **without lining**





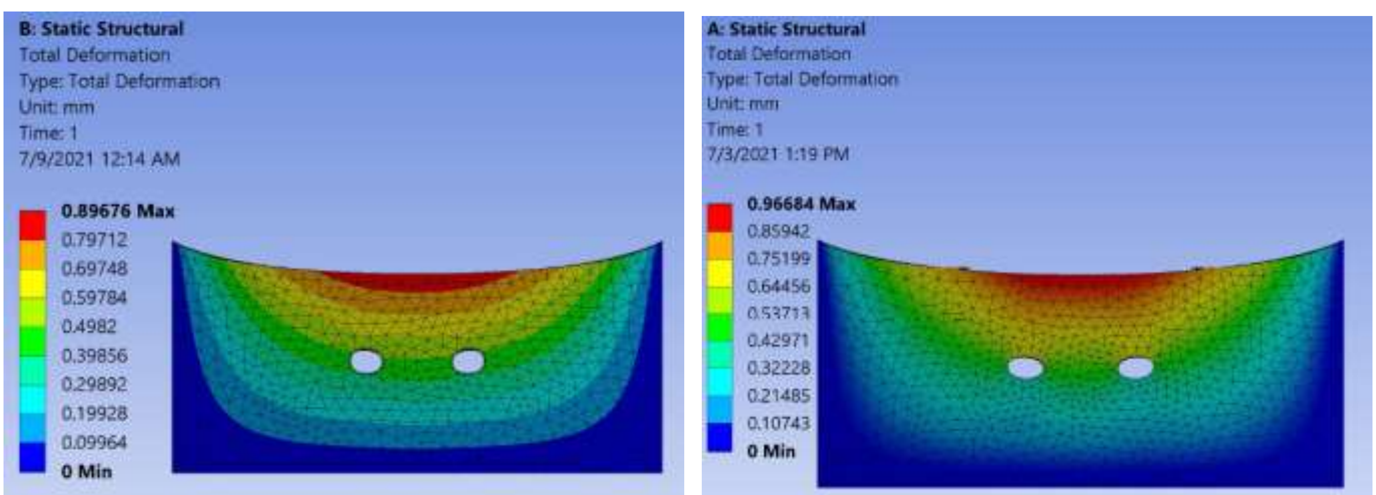
(c)Spacing between tunnels 2.2D (14.56m)

With lining **without lining**



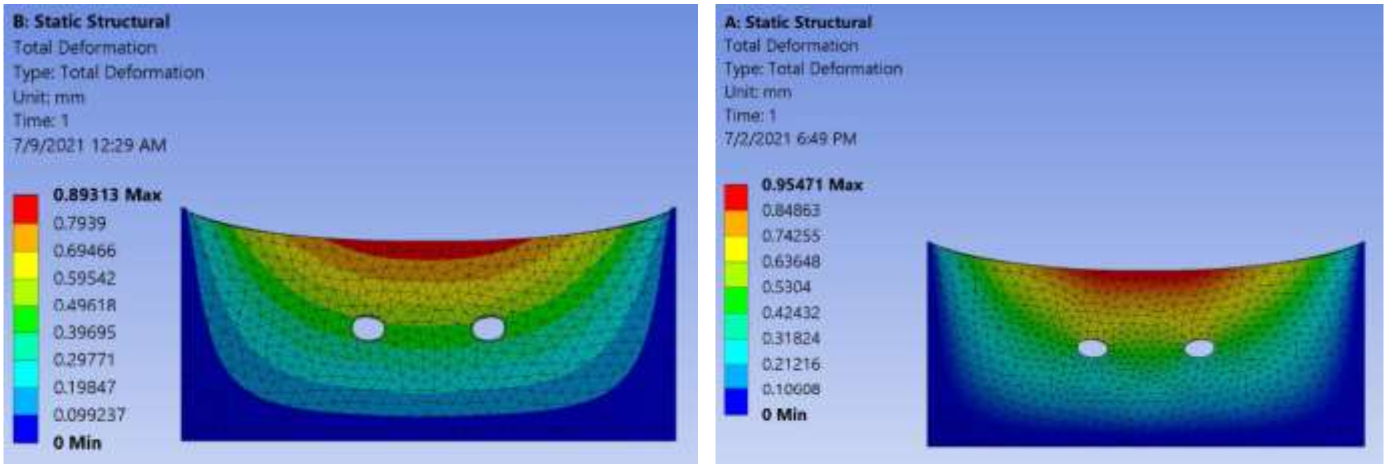
(d)Spacing between tunnels 2.5D (16.4m)

With lining **without lining**



(e)Spacing between tunnels 3D (19.68m)

With lining **without lining**



(f)Spacing between tunnels 3.5D (22.96m)

Figure 21: Vertical surface settlement contour for six-tunnel spacing in Breccia 3 ground condition

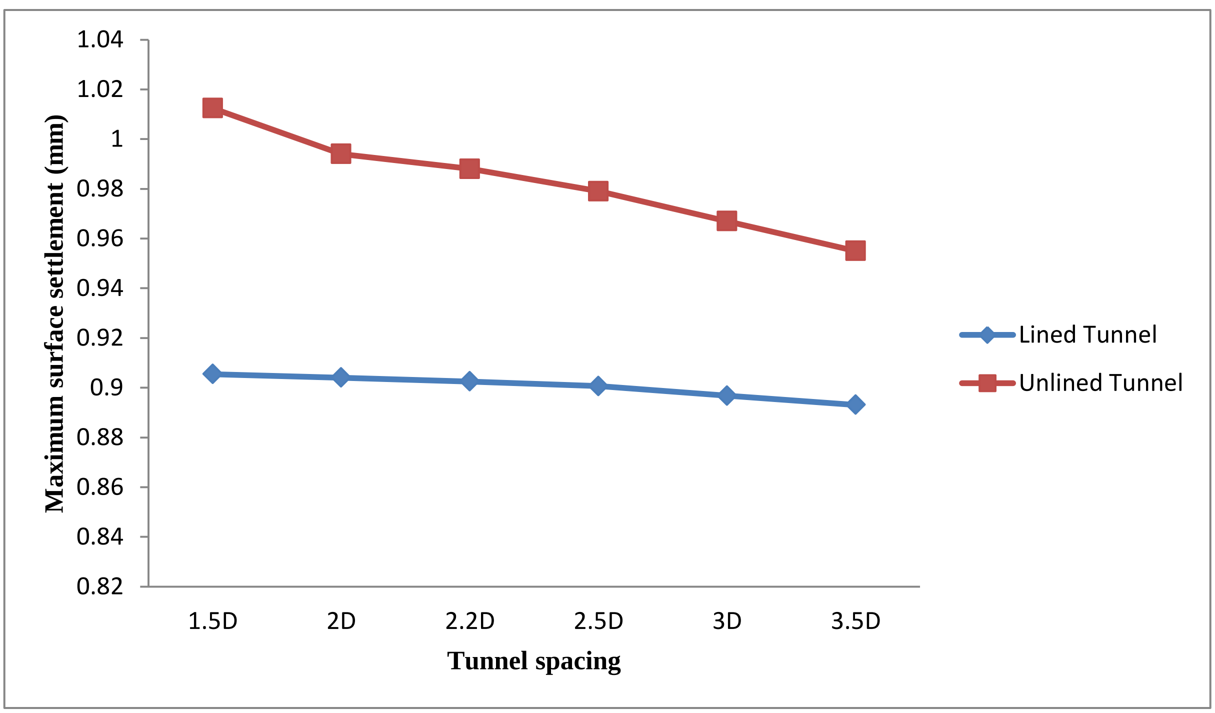
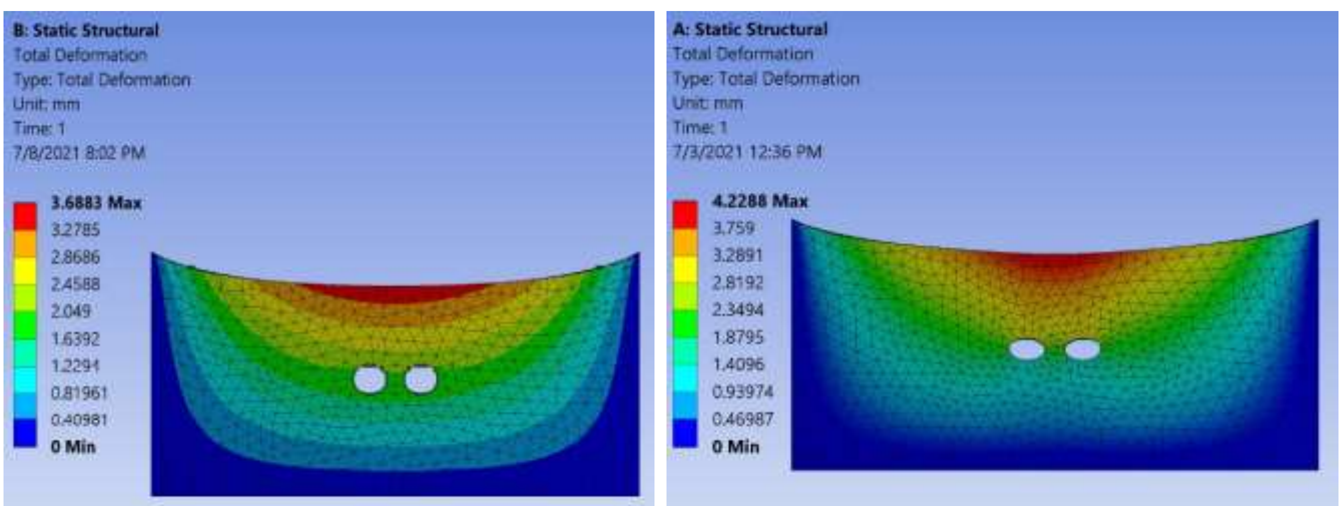


Figure 22: Maximum surface settlement curve for different spacing having Breccia 3

The tunnel face material is breccia 3 in this condition. Figure 21 shows the maximum surface settlement curve for the tunnel with lining and without lining with respect to tunnel spacing having breccia 3. In this condition the maximum surface settlement for 1.5D spacing is 10.562% more for the unlined tunnel as compare to the lined tunnel and for 3.5D spacing maximum surface settlement is 6.449% more for the unlined tunnel as compare to lined. In this case, negligible deformation is observed as compared to residual soil and mixed ground case because of very high deformation modulus of the tunnel face.

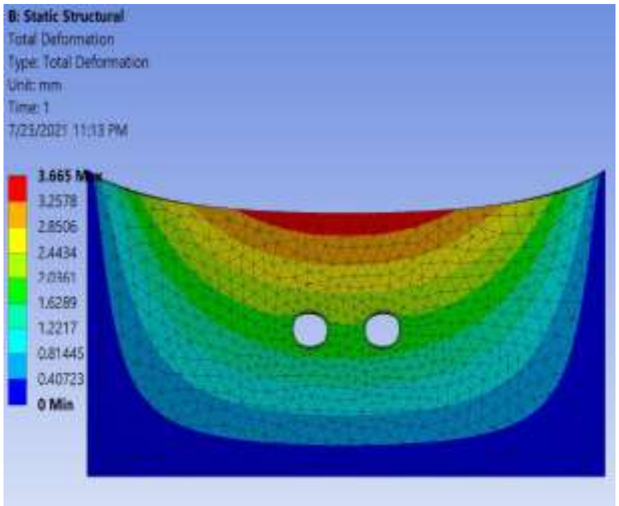
Case 4: Homogeneous ground (Breccia 4) condition

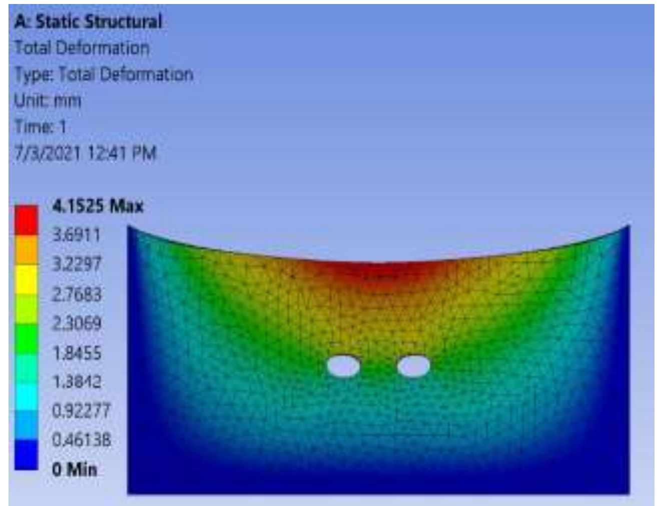
With lining **without lining**



(a)Spacing between tunnels 1.5D (9.84m)

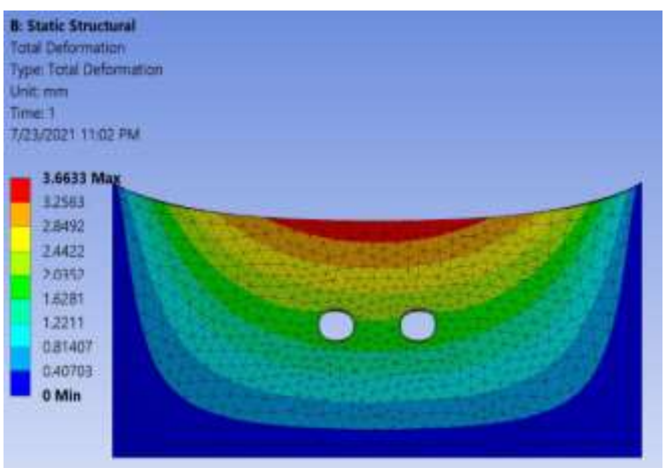
With lining **without lining**

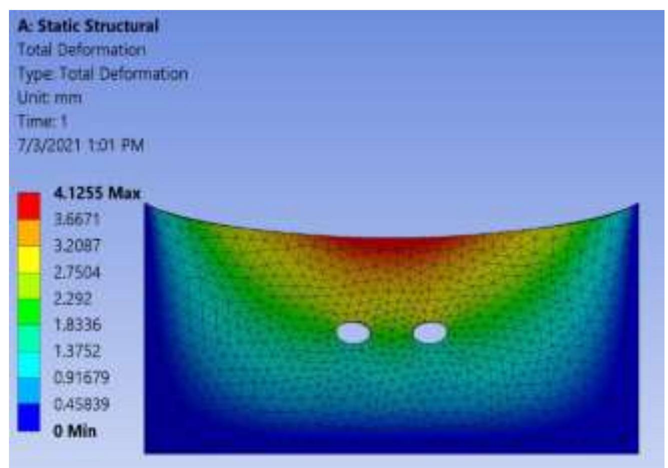




(b)Spacing between tunnels 2D (13.12m)

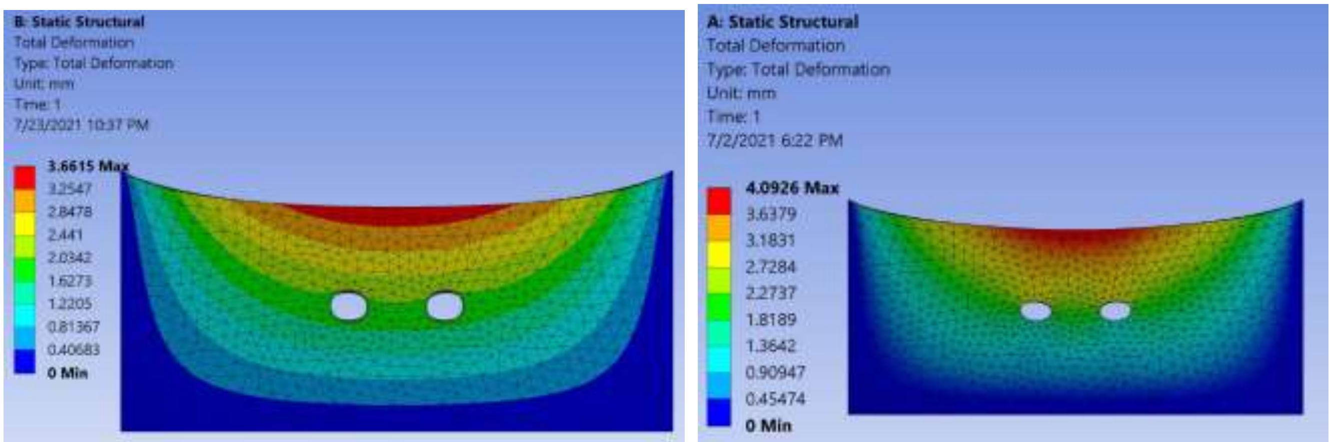
With lining **without lining**





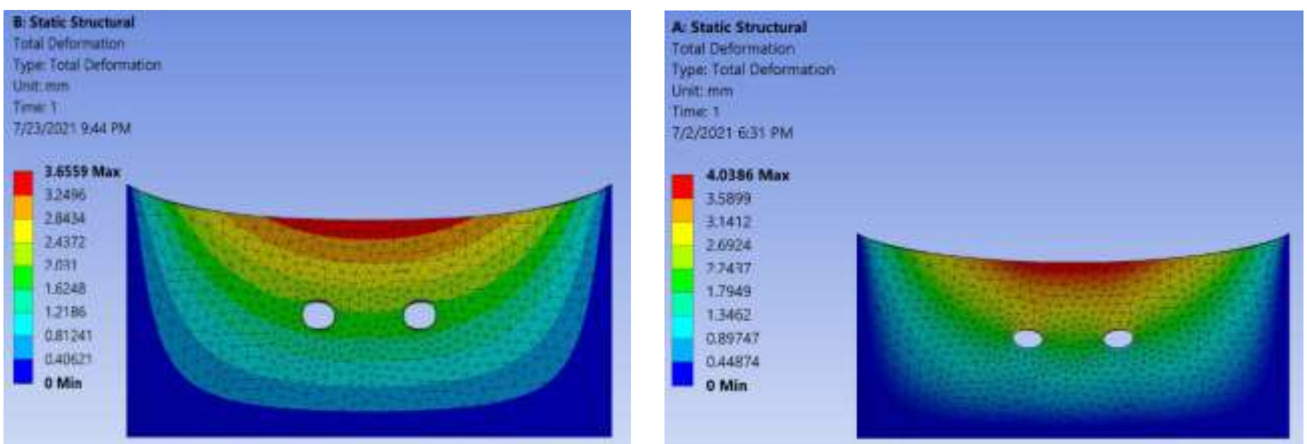
(c)Spacing between tunnels 2.2D (14.56m)

With lining **without lining**



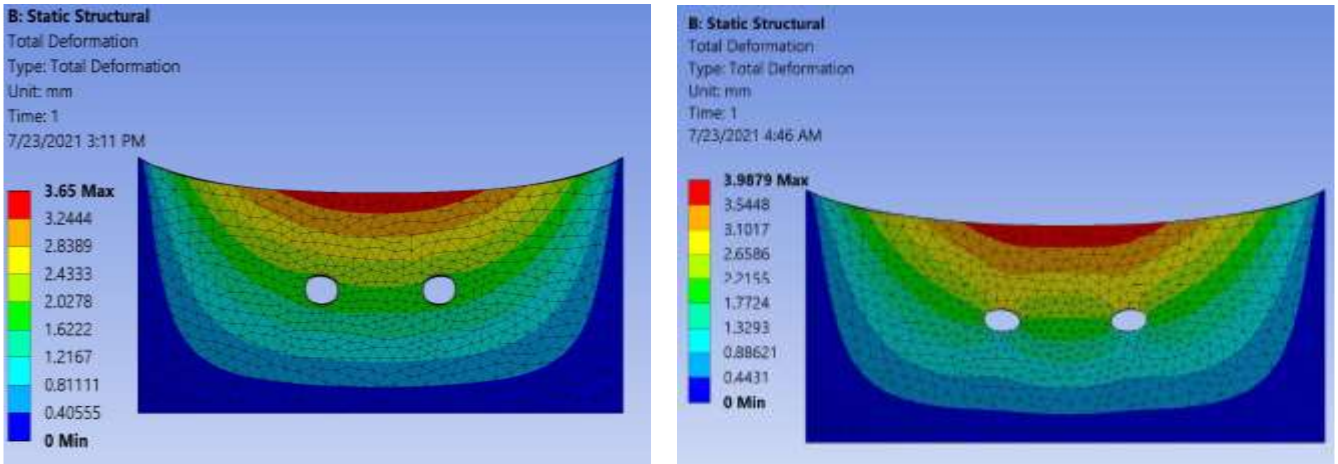
(d)Spacing between tunnels 2.5D (16.4m)

With lining **without lining**



(e) Spacing between tunnels 3D (19.68m)

With lining **without lining**



(f) Spacing between tunnels 3.5D (22.96m)

Figure 22: Vertical surface settlement contour for six-tunnel spacing in Breccia 4 ground condition

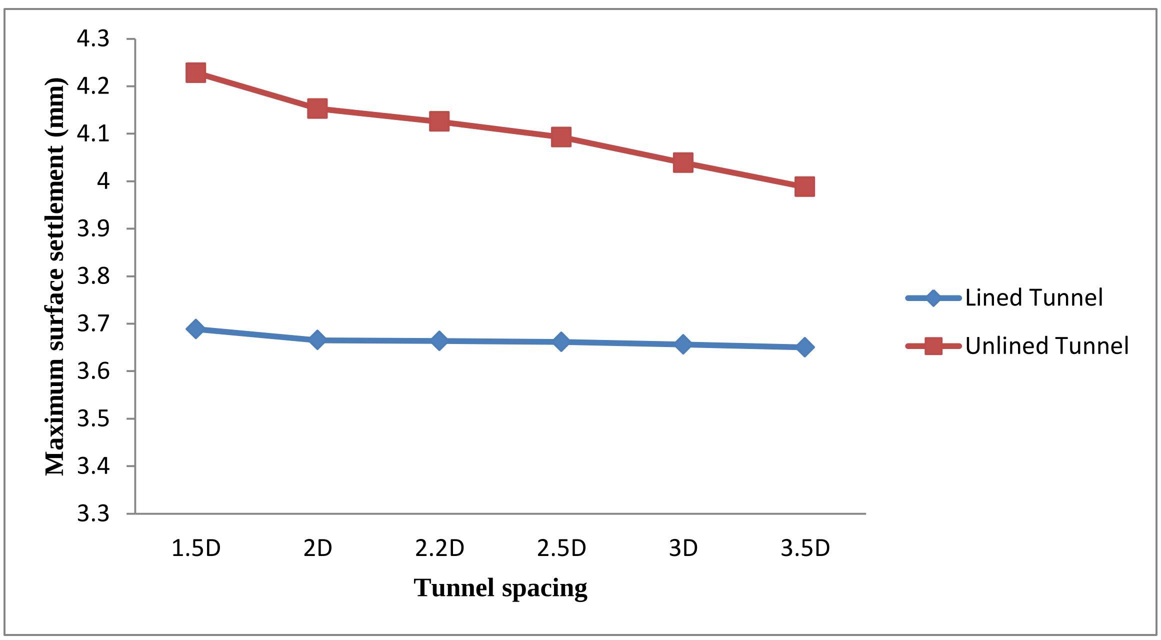


Figure 23: Maximum surface settlement curve for different spacing having Breccia 4

The tunnel face material is breccia 4 in this condition. Figure 23 shows the maximum surface settlement curve for the tunnel with lining and without lining with respect to tunnel spacing having breccia 4. In this condition, the maximum surface settlement for 1.5D spacing is 14.654% more for the unlined tunnel as compared to the lined tunnel and for 3.5D spacing maximum surface settlement is 9.2575% more for the unlined tunnel as compare to lined**.**

Table 7: Maximum surface settlement with spacing of tunnel

Spacing	**Case 1 (in mm)**		**Case 2 (in mm)**		**Case 3 (in mm)**		**Case 4 (in mm)**	
	Lined	**Unlined**	**Lined**	**Unlined**	**Lined**	**Unlined**	**Lined**	**Unlined**
1.5D	11.191	11.798	61.387	73.764	0.90547	1.0124	3.6883	4.2288
2D	10.919	11.733	59.988	72.358	0.90404	0.99407	3.665	4.1525
2.2D	10.781	11.713	59.246	71.867	0.9025	0.98764	3.6633	4.1255
2.5D	10.645	11.68	58.521	71.27	0.90062	0.97977	3.6615	4.0926
3D	10.513	11.637	57.092	70.298	0.89676	0.96684	3.6559	4.0386
3.5D	10.379	11.596	56.595	69.386	0.89313	0.95471	3.650	3.9879

In table 7, it is observed that the maximum surface settlement is decreased when tunnel spacing is increased due to the reduced interaction of both tunnels, because when tunnels interact, soil movement between them increases. It is also observed that for each value of spacing of tunnels, the settlement in case of unlined tunnel is more as compare to the lined tunnel. This may be due to the fact that in case of unlined tunnel the pressure bulb is non-uniform but in case of lined tunnel it is uniform.

Table 8: Percentage increase in settlement of unlined tunnel with respect to lined tunnel

Spacing	Case 1 (%)	Case 2 (%)	Case 3 (%)	Case 4 (%)
1.5D	5.420	20.162	11.809	14.654
2D	7.455	20.620	9.959	13.302
2.2D	8.645	21.302	9.434	12.617
2.5D	9.723	21.850	8.789	11.770
3D	10.690	22.130	7.815	10.468
3.5D	11.725	22.60	6.895	9.257

In table 8, it is observed that tunnel lining plays more important role in case of soil when it compared with rock, because soil undergoes more deformation in plastic stage as compare to rock. In 3.5D spacing condition, it is observed that percentage increase in settlement of unlined tunnel with respect to lined tunnel in case 1 and case 2 is more as compare to case 3 and case 4 because of less cohesion value and deformation modulus respectively. It means more factor of safety required when surface material is soil because of more chances of spalling and popping conditions on soil. When thin slabs of rock fall from the top or walls of a tunnel, they are known as spalling and popping situations, respectively.

Table 9 describes the maximum settlement related to the risk category and damage of the structure. Surface settlement due to Breccia 3 and Breccia 4 signifies negligible risk to the structure and settlement in the mixed ground is in category 2 which means, possible superficial damage which is unlikely to have structural significance but in the case of residual soil condition, it is very like to expected structural damage.

Table 9: Risk categories for structures located inside the settlement trough profile (RANKIN, 1988)

Risk category	Maximum settlement(mm)	Description of probable damage
I	< 10	Negligible
II	10 to 50	Possible superficial damage which is unlikely to have structural significance.
III	50 to 75	Possible damage to structures
IV	>75	Expected structural damage to structure

CHAPTER- 5

CONCLUSION & RECOMMENDATION FOR FUTURE WORK

5.1) CONCLUSION:

The prediction of ground surface settlement in metro tunnels, excavated in metropolitan areas is one of the most difficult tasks due to the interaction between twin tunnels. To estimate ground surface settlement after twin tunnel excavation, there is an empirical method known as Herzog. The crucial factor in surface settlement values in this method is the spacing between twin tunnels. The Findings show that the magnitude of maximum surface settlement and shape of the surface settlement curve are both highly influenced by the distance between tunnels. It is observed that when tunnel spacing increased from lower spacing (i.e. 1.5D) to greater spacing (i.e. 3.5D), the maximum surface settlement will decrease. When tunnel spacing is increased, the deformation of lined tunnels is reduced more as compare to unlined tunnels. It means that when tunnel spacing is increased but within a permissible limit, a lower grade concrete lining can be provided to save cost. In the case of the lined tunnel with 3.5D spacing, it is also observed that in residual soil conditions, the maximum surface settlement is more than the results obtained for Breccia 3 (approximately 70 times), Breccia 4 (about 17 times), and mixed ground (about 5.5 times) as expected, it means Surface settlement increases as the deformation modulus of the tunnel face are reduced. . Lined tunnel and 3.5D spacing is taken because in above all this is the case in which minimum deformation is obtained. Surface settlement due to Breccia 3 and Breccia 4 signifies negligible risk and damage to the structure due to settlement. Tunnel spacing, Tunnel lining, and the deformation modulus of the geo-materials surrounding the tunnel are found to be the most important factor in twin tunnel-induced surface settlement.

5.2) RECOMMENDATION FOR FUTURE WORK

1. In this study, a circular tunnel surrounded by different soil-rock masses is considered for analysis. A similar study can be performed for different shapes of the tunnel and at different vertical positioning of the tunnel with respect to each other.
2. Similar studies can also be performed on jointed rock and fault zone.
3. In this study, the lining is one of the important factors to overcome the excess vertical deformation so different lining materials can be used to see the response against the deformation on the tunnel.
4. Single tunnel of different shapes and in different soil-rock mass conditions can also be used for a similar type of study.

REFERENCES

[1] Ansys R19.2 [Computer Software], Ansys Inc, 2019

[2] Benton, L.J., Phillips, A. (1991). The behavior of two tunnels beneath a building on piled foundations. In: Proceedings of the 10th European Conference on Soil Mechanics and Foundation Engineering, Florence, Italy, May, pp. 665–668.

[3] Chen, F. Y., Wang, L., & Zhang, W. G. (2019). Reliability assessment of stability of tunnelling perpendicularly beneath an existing tunnel considering spatial variabilities of rock mass properties. Tunnelling and Underground Space Technology, 88, 276–289.

[4] Ding, L.Y., Yu, H.L., Li, H., Zhou, C., Wu, X.G., Yu, M.H., 2012. Safety risk identification system for metro construction on the basis of construction drawings. Autom. Constr. 27, 120–137.

[5] Sadaghiani, M.H., Dadizadeh, S., 2010a. Study on the effect of a new construction method for a large span metro underground station in Tabriz-Iran. Tunn. Undergr. Space Technol. 25, 63–69.

[6] Al-Bittar, T., Soubra, A.H., 2017. Bearing capacity of spatially random rock masses obeying Hoek-Brown failure criterion. Georisk: Assess. Manage. Risk Engineered Syst. Geohazards 11 (2), 215–229

[7] Benton, L.J., Phillips, A. (1991). The behavior of two tunnels beneath a building on piled foundations. In: Proceedings of the 10th European Conference on Soil Mechanics and Foundation Engineering, Florence, Italy, May, pp. 665–668.

[8] Chen, F. Y., Wang, L., & Zhang, W. G. (2019). Reliability assessment of stability of tunnelling perpendicularly beneath an existing tunnel considering spatial variabilities of rock mass properties. Tunnelling and Underground Space Technology, 88, 276–289.

[9] Ding, L.Y., Yu, H.L., Li, H., Zhou, C., Wu, X.G., Yu, M.H., 2012. Safety risk identification system for metro construction on the basis of construction drawings. Autom. Constr. 27, 120–137.

[10] Al-Bittar, T., Soubra, A.H., 2017. Bearing capacity of spatially random rock masses obeying Hoek-Brown failure criterion. Georisk: Assess. Manage. Risk Engineered Syst. Geohazards 11 (2), 215–229

[11] Camós, C., Molins, C., Arnau, O., 2014. A case study of damage on masonry buildings produced by tunneling induced settlements. Int. J. Archit. Heritage 8, 602–625.

[12] Arioglu E (1992) Surface movements due to tunnelling activities in urban areas and minimization of building damages (in Turkish). Istanbul Technical University, Mining Engineering Department, Short Course

[13] B. M. New and M. P. O'Reilly, Tunneling-induced ground movements. Prediction of their magnitude and effects, Proceedings of the Fourth International Conference on Ground Movements and Structures, invited review paper, Cardiff, 7 July 1991, Pentech Press, London (1991), pp. 671–697.

[14] Hamza M, Ata A, Roussin A (1999) Ground movements due to construction of cut-and-cover structures and slurry shield tunnel of the Cairo Metro. Tunneling Underground Space Technology 14(3):281–289

[15] Atkinson, JH and Mair, RJ (1981) Soil mechanics aspects of soft ground tunnelling. Ground Engineering, 14. pp. 20-28. ISSN 0017-4653

[16] Herzog M (1985) Die Setsungsmulde Über Seicht Liegenden Tunneln. (Herzog, (1985))Berlin Bautechnik 11:375–377

[17] Hoek, E., Carranza-Torres, C., Corkum, B., 2002. Hoek-Brown failure criterion-2002 edition. Proc. NARMS-Tac 1, 267–273.

[18] Hoek, E., Brown, E.T., 2018. The Hoek-Brown failure criterion and GSI – 2018 edition.

[19] J.Ercelebi S. G, Copur H and Ocak, (2011) Surface settlement predictions for Istanbul Metro tunnels excavated by EPB-TBM, environmental earth science DOI 10.1007/s12665-010-0530-6

[20] Heng Li, Enlin Ma, Jinxing Lai, Lixin Wang, Shuoshuo Xu, Ke Wang, Tong Liu, "Tunnelling-Induced Settlement and Treatment Techniques for a Loess Metro in Xi'an", Advances in Civil Engineering, vol. 2020, Article ID 1854813, 20 pages, 2020. https://doi.org/10.1155/2020/1854813

[21] Ercelebi S. G, Copur H and Ocak, (2011) Surface settlement predictions for Istanbul Metro tunnels excavated by EPB-TBM, environmental earth science DOI 10.1007/s12665-010-0530-6

[22] RANKIN, W J "Ground movements resulting from urban tunnelling: predictions and effects.", Engineering geology of underground movements. Proceedings of the 23rd annual conference of the engineering group of the geological society, nottingham university, september 13-17, 1987

[23] Islam, M.S. and Iskander, M., 2021. Twin tunnelling induced ground settlements: A review. *Tunnelling and Underground Space Technology*, p.103614.

[24] Soomro, M.A., Kumar, M., Xiong, H., Mangnejo, D.A. and Mangi, N., 2020. Investigation of effects of different construction sequences on settlement and load transfer mechanism of the single pile due to twin stacked tunnelling. *Tunnelling and Underground Space Technology*, *96*, p.103171

[25] Ağbay, E. and Topal, T., 2020. Evaluation of twin tunnel-induced surface ground deformation by empirical and numerical analyses (NATM part of Eurasia tunnel, Turkey). *Computers and Geotechnics*, *119*, p.103367.

[26] Forsat, M., Taghipoor, M. and Palassi, M., 2021. 3D FEM Model on the Parameters' Influence of EPB-TBM on Settlements of Single and Twin Metro Tunnels During Construction. *International Journal of Pavement Research and Technology*, pp.1-14.

[27] Zhou, J., Yang, X.A., Ma, M.J. and Li, L.H., 2021. The support load analysis of deep-buried composite lining tunnel in rheological rock mass. *Computers and Geotechnics*, *130*, p.103934.

[28] Zhihua Zhang, Wensheng Xu, Wenting Nie, Lingmin Deng, 2021. DEM and theoretical analyses of the face stability of shallow shield cross-river tunnels in silty fine sand, Computers and Geotechnics, Volume 130,103905, ISSN 0266-352X,

[29] Jin, D., Yuan, D., Li, X. and Zheng, H., 2018. An in-tunnel grouting protection method for excavating twin tunnels beneath an existing tunnel. *Tunnelling and Underground Space Technology*, *71*, pp.27-35.

[30] Golshani, A., Varnusfaderani, M.G. and Poorhashemi, S.M., 2019. Effect of excavation stages and lining sequences on ground settlement in a twin tunnel. In *Tunnels and Underground Cities: Engineering and Innovation meet Archaeology, Architecture and Art* (pp. 5638-5647). CRC Press.

[31] Osman, A.S., 2010. Stability of unlined twin tunnels in undrained clay. *Tunnelling and Underground Space Technology*, *25*(3), pp.290-296.

[32] Heidary, R., Esmaeili, M. and Gharouni Nik, M., 2021. Evaluation of the soil layering and soil lens effect on the ground-borne vibrations induced by twin metro tunnels. *European Journal of Environmental and Civil Engineering*, pp.1-20.

[33] Nematollahi, M. and Dias, D., 2019. Three-dimensional numerical simulation of pile-twin tunnels interaction–Case of the Shiraz subway line. *Tunnelling and Underground Space Technology*, *86*, pp.75-88.

[34] Shivaei, S., Hataf, N. and Pirastehfar, K., 2020. 3D numerical investigation of the coupled interaction behavior between mechanized twin tunnels and groundwater–A case study: Shiraz metro line 2. *Tunnelling and Underground Space Technology*, *103*, p.103458.