DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

AN INVESTIGATION ON THE APPLICATION OF NEW AUSTRIAN TUNNELING METHOD TO THE CONSTRUCTION OF WIDE UNDERGROUND SUBWAY STATIONS

by
Selim AKTAŞ

October, 2023
İZMİR

AN INVESTIGATION ON THE APPLICATION OF NEW AUSTRIAN TUNNELING METHOD TO THE CONSTRUCTION OF WIDE UNDERGROUND SUBWAY STATIONS

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M.Sc. THESIS EXAMINATION RESULT FORM

We have read the thesis entitled **"AN INVESTIGATION ON THE APPLICATION OF NEW AUSTRIAN TUNNELING METHOD TO THE CONSTRUCTION OF WIDE UNDERGROUND SUBWAY STATIONS"** completed by **SELİM AKTAŞ** under supervision of **PROF. DR. GÜRKAN ÖZDEN** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Master of Science.

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AN INVESTIGATION ON THE APPLICATION OF NEW AUSTRIAN TUNNELING METHOD TO THE CONSTRUCTION OF WIDE UNDERGROUND SUBWAY STATIONS

ABSTRACT

Tunneling, as it was in the past, remains an essential construction activity today, serving purposes such as water supply, sewage, drainage, and transportation to meet humanity's basic needs. The increasing population and fundamental requirements, especially in the field of transportation, have necessitated tunnel construction within urban areas. During tunnel excavations, deformations occur within a specific influence area of the tunnel. These deformations can particularly damage structures around the tunnel, and in the event of tunnel collapse, they can even lead to settlement in buildings. To prevent these deformations, it has been emphasized that the geological, hydrological, geotechnical characteristics of the tunnel excavation area, as well as the carefully selected tunnel depth and diameter, need to be thoroughly examined. The anticipated levels of deformation before tunnel excavations were initially predicted using empirical methods proposed by various researchers. These values were then compared with site measurements and analyses conducted using the finite element method. While empirical methods and FE analysis results showed similar values, greater deformations were observed in site measurements. This was attributed to the precision displayed during site excavations. With the existing methods, the anticipated surface settlements were found to be beyond acceptable limits, prompting additional measures and improvements to mitigate these deformations within the scope of this thesis. The conducted work demonstrated that deformations decreased by approximately 36-43 percentage compared to the existing situation.

Keywords: NATM, surface settlement, surface subsidence, building deformation, tunnel, underground subway.

GENİŞ YERALTI METRO İSTASYONLARININ İNŞASINDA YENİ AVUSTURYA TÜNEL İNŞAAT YÖNTEMİNİN UYGULANABİLİRLİĞİNİN İNCELENMESİ

ÖZ

Tünelcilik geçmişte olduğu gibi bugün de insanlığın temel ihtiyaçlarından olan su temini, kanalizasyon, drenaj, ulaşım gibi amaçları karşılamak için tercih edilen bir inşai faaliyettir. Artan nüfus ve insanlığın temel ihtiyaçları özellikle ulaşım alanında şehir içlerinde de tünel açma zorunluluğu doğurmuştur. Tünel kazıları sırasında tünelin belirli bir etki alanı içerisinde deformasyonlar meydana getirmektedir. Bu deformasyonlar özellikle tünel çevresinde bulunan yapılara zarar verebilmekte, hatta tünel göçmesi durumunda yapılarda da göçmelere sebep olabilmektedir. Bu deformasyonların önlenebilmesi için tünel açılacak bölgenin jeolojik, hidrolojik, jeoteknik özelliklerinin ve seçilecek tünel derinliği ve çapının çok hassas bir şekilde incelenmesi gerektiği ortaya koyulmuştur. Tünel kazıları öncesi beklenen deformasyon mertebeleri öncelikle çeşitli araştırmacılar tarafından önerilen ampirik yöntemler ile önden tahmin edilmiş ve bu değerler saha okumaları ve sonlu elemanlar metodu ile yapılan analiz sonuçları ile karşılaştırılmıştır. Ampirik yöntemler ve sonlu elemanlar analiz sonuçları birbiri ile benzer değerler taşırken sahada yapılan ölçümlerde deformasyonların daha fazla olduğu görülmüştür. Bunun sebebi olarak saha kazısında gösterilen hassasiyet ile ilişkili olduğu düşünülmüştür. Mevcut şartlarda var olan yöntemler ile gerçekleşmesi beklenen yüzey oturmaları, kabul edilebilir sınırların üstünde olduğu görülmüş ve bu tez kapsamında bu deformasyonların azaltılmasına yönelik ilave önlemler ve iyileştirmeler üzerinde çalışma yapılmıştır. Yapılan çalışmalarda yüzde 36-43 mertebelerinde mevcut duruma göre deformasyonların azaldığı görülmüştür.

Anahtar kelimeler: NATM (Yeni Avusturya Tünel Açma Metodu), yüzey oturması, yüzey çökmesi, bina deformasyonu, tünel, yeraltı metro.

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LIST OF SYMBOLS

C	:	Distance between top of tunnel and ground surface
D	:	Tunnel diameter
S_{max}	:	Maximum tunnel surface settlement value
S	:	Tunnel surface settlement value
x	:	Lateral distance from the tunnel centerline
I	:	Transverse distance from the centerline of the tunnel to the point of inflection
V_S	:	Surface volume loss per unit length due to tunnel excavation
Ø\`	:	Effective soil friction angle
V_L	:	Volume loss ratio
H	:	The height between the surface and the center of the tunnel
K*	:	Soil coefficient determining tunnel settlement
Z_0	:	The distances from the ground surface to respectively the tunnel axis
ΔV	:	Extra volume of excavated soil
A_t	:	Volume of the finished void of tunnel
N	:	Soil stability ratio
σ_S	:	Total Surcharge pressure
σ_T	:	TMB face pressure
γ_n	:	Unit weight of soil
E_s	:	Elastic modulus
a	:	Spacing between tunnels
d	:	Spacing between pipes
T	:	Temperature
σ_v	:	Pre-excavation existing load value at tunnel
L_1	:	Free Span Length
L_2	:	Full support excavation distance
P_L	:	Net limit pressure
E_m	:	Menard Elastic Modulus
γ_d	:	Dry unit weight of soil
C_u	:	Undrained shear strength
K	:	Permeability of soil

G	:	Shear modulus
I	:	Moment of inertia
σ_y	:	Yield stress
σ_t	:	Tensile strength
σ_1	:	Maximum principal stress
σ_3	:	Minimum principal stress
ν	:	Poisson ratio

CHAPTER ONE
INTRODUCTION

1.1 Motivation of the Thesis

Tunnels are underground structures that have been used for various purposes since ancient times in human history, either with closed or open ends, and are constructed in different types of soils. Because of rapid urbanization, tunnels have become increasingly prevalent within urban areas. Prior to tunnel construction, it is necessary to conduct the required research to ensure the safe and economical excavation of the tunnel. Geotechnical investigations are carried out to anticipate the impact on the tunnel after excavation and to define soil conditions.

Tunnel projects are more challenging and riskier in terms of design and construction compared to other projects due to the variability of geological conditions, groundwater conditions, and seismicity along the route. To mitigate or eliminate these risks, geotechnical, geological, and geophysical investigations must be conducted meticulously. With advancing technology, these investigations can be conducted more accurately, and tunnel excavation methods are being developed based on various soil types. One of these techniques is the New Austrian Tunneling Method (NATM), which is one of the most preferred methods today.

The New Austrian Tunneling Method (NATM) is a tunnel construction method that represents a flexible approach tailored to underground conditions and geological structures. NATM, short for "New Austrian Tunneling Method," was initially developed by Austrian engineers in the late 1960s and has since been used in numerous tunnel construction projects worldwide.

Tunnel construction typically begins with tunnel boring machines or the blasting method. In NATM, support structures are not immediately added to the tunnel's surroundings. This is done based on geological conditions and the progress of the tunnel. As the tunnel advances, if necessary, appropriate supports are added to the tunnel's surroundings. These supports can often include steel frames, shotcrete walls, or injected concrete. The supports help prevent the tunnel from collapsing or deforming.

The NATM technique is a system that operates as deformation occurs. However, since the levels of deformation that will occur can also cause deformations in structures directly above the tunnel, it is of great importance to anticipate and take necessary measures for the expected settlements before tunnel excavation. Therefore, within the scope of this thesis, the deformations caused by the excavation of a large-diameter shallow tunnel have been studied, and corrective methods have been developed by evaluating the magnitudes of these deformations.

1.2 Statement of the Problem

Surface deformations resulting from the excavation of a shallow, large-diameter tunnel using the NATM (New Austrian Tunneling Method) technique within the Izmir region were investigated. Initially, the magnitudes of these deformations were estimated using existing empirical approaches in the literature. Subsequently, analyses were conducted using the finite element method to explore the underlying causes of these deformations. The data obtained through the finite element method were compared to both the predictions made by empirical formulas and the site observations, enabling the identification of the factors contributing to the development of deformations. In this context, remedial methods were developed, and the results were evaluated.

1.3 Scope of the Thesis

The first chapter of the study begins by outlining the inception of the idea behind preparing the thesis and explaining the purpose of the thesis. In the second chapter, the phenomena leading to deformations encountered in urban underground tunnels excavated using the NATM technique are evaluated, and based on these observations, a review of the literature is conducted. The selection of soil parameters and geological structure for the area where the tunnel is excavated is explained in the third chapter. Numerical modeling is presented in the fourth chapter. The employed soil-structure model, the developed methodology, and the approach for generating results are explained. Empirical approach results for predicting surface deformations, the evaluation of field data, and the results obtained using the finite element method are discussed in the fifth chapter. Finally, in the sixth chapter, observations and recommendations for future research are provided.

CHAPTER TWO

LITERATURE REVIEW

2.1 History of Tunneling

For centuries, civilizations have constructed various structures such as roads, bridges, and tunnels to improve their lives. Tunneling is perhaps the earliest application of civil engineering by humanity. It is known that the ancient Egyptians dug tunnels for water supply in the construction of tombs. The Egyptians also engaged in mining to extract copper ore, which required the excavation of deep tunnels. Today, tunnels are primarily used for road, rail, and pedestrian transportation. There are numerous reasons why tunnel construction is preferred. Some of these reasons can be listed as follows:

- Tunnel construction may be more advantageous in situations where widening roads or opening new roads is not feasible or where surface expropriation costs are prohibitively expensive.
- Tunnel construction is often more cost-effective in projects planned for areas with high construction and excavation costs, such as ridges, hills, and rivers.
- Tunnel construction can be preferred due to significant reduction in route distance, which contributes to the regional economy.

2.2 Features of the Subway Tunneling Method

Subway tunneling methods can vary depending on many factors such as the processes to be carried out in its construction, the measures to be taken, the importance of the region where the tunnel will pass, the completion time of the tunnel and the budget of the project. However, the most important point here is to determine whether the chosen method is suitable.

The points to be considered in the methods to be chosen are that they are economical, practical in practice, easily revised against sudden situations, and most importantly, they contain low risk.

2.3 Factors Affecting Tunnel Cost

In order to be able to evaluate the tunnel cost, it is firstly related to what purpose and through which region the tunnel will pass. These measures will also directly affect the tunnel costs, as the measures to be taken will greatly affect whether the area where the tunnel will pass is urban or outside.

According to the region where the tunnel will pass, factors such as the tunnel excavation support system, the diameter of the tunnel, the depth of the tunnel from the surface, the length of the tunnel and the geological and hydrological features of the region are the main issues that will directly affect the tunnel cost.

2.4 Tunneling Methods

The general definition of tunnel is the passages created by digging underground. These tunnels should be built in the most reliable way due to their underground structure. But what's the best way to drill a tunnel? There are quite a few methods, these four methods are mostly used. The defining methods are the Tunnel Boring Machine Method (TBM), the Cut and Cover approach, Norwegian Method of Tunneling (NTM) and the New Austrian Tunneling Method (NATM). Each of them has some strengths and weakness.

2.4.1 Tunnel Boring Machine Method (TBM)

Tunnel boring machine is used to digs various soil or rock layers by circular cross sections. TBMs have great advantages because of less deformations on the surface and time saving. Therewithal, cost of lining decreases significantly. TBMs have substantial advantages as well as disadvantages such as expensive to construct and difficulty in transportation.

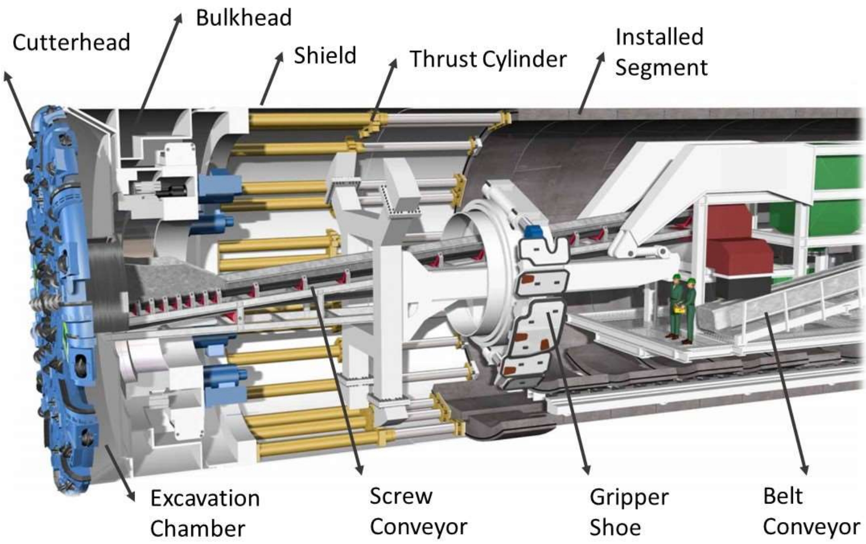


Figure 2.1 TBM longitudinal section view (http://www.railsystem.net/tunnel-boring-machine-tbm)

2.4.2 Cut and Cover Approach

Cut- and-cover is known the oldest method of tunneling. The general principle of this method is that the structure is built inside the excavated ground and this structure is covered with the ground. There are two basic forms of cut-and-cover tunneling that are bottom-up method and top-down method.

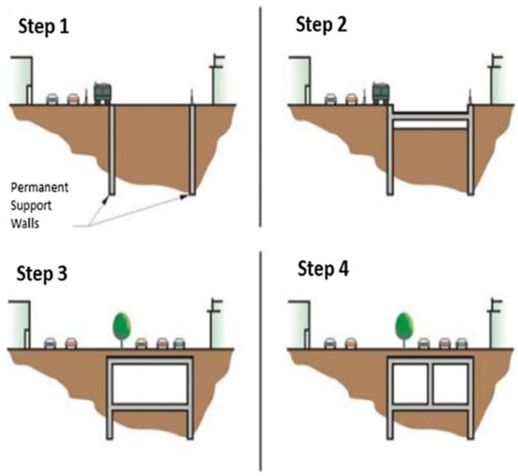


Figure 2.2 Stage construction top-down method (Golshani, & Rezaeibadashiani, 2020)

2.4.3 The New Austrian Method (NATM)

In recent years, one of the most frequently used tunneling methods is the New Austrian Tunneling Method (NATM), which is based on the principle of self-support in the rock environment where the tunnel is excavated. This means that the stress and deformations that will occur after excavation are directed within the rock through the implemented reinforcements. The patent for this method was obtained by A. Brunner in 1958 (Austrian patent number 197851) (Ergin, 1992).

However, its global introduction was carried out by Austrian tunnel experts Pacher, Müller, and Rabcewicz. Initially applied to tunnels excavated in rock, NATM has become a commonly used method for both soft and stable cohesive soils in modern times. In shallow depths and weak soil conditions, steel umbrella tubes or umbrella arch reinforcements are applied during tunnel advancement as primary reinforcement elements (Muraki, 1997). As shown in Figure 2.15, the effect of umbrella arch application on stress distribution on the tunnel surface is illustrated. With this method, before excavation of the tunnel face, a reinforcement arch is created along the tunnel axis in the roof, and then excavation progresses beneath this arch. Traditional methods like shotcrete, steel mesh, rock bolts, steel pipes, and ground anchor applications around the tunnel create a pseudo shell element (Aksoy, & Onargan, 2010).

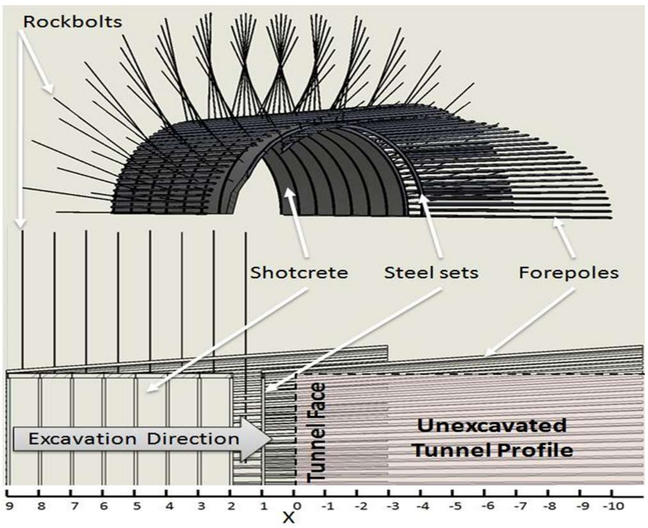


Figure 2.3 Temporary structural support system used in a NATM (Oke, Vlachopoulos, & Diederichs, 2012, May)

The conducted thesis study focuses on the analysis of surface deformations for the excavation of a large-diameter shallow tunnel using the NATM technique in a weak rock environment. Therefore, the identification of the reasons behind the deformations occurring during tunnel excavation is of paramount importance.

2.5 Causes of Surface Deformations in Tunnel Excavations

During the excavation of a tunnel at a specific depth beneath the surface, the natural structure of the ground or rock formation in which the tunnel is opened becomes disturbed, resulting in displacements due to volume loss caused by tunnel excavation.

These volume losses lead to changes in the equilibrium state of stresses in the soil surrounding the tunnel. These altered stresses, depending on soil parameters, induce deformations both around the tunnel perimeter and on the surface. While these deformations can extend to the surface for shallow depths, they are limited to a certain boundary in deeper tunnels, exerting less influence on surface structures. To define this effect, the concepts of deep tunnel and shallow tunnel have emerged.

If we denote the distance between the tunnel ceiling and the ground surface as C, and the tunnel diameter or width as D, when $C/D > 2.5$, the influence on the surface area is minimized. On the other hand, when $C/D < 2.5$, the tunnel is considered shallow, and it is anticipated that the deformations occurring during tunnel excavation will adversely affect surface settlement.

Particularly when tunnel excavations occur over a residential area, the resulting deformations become significantly more crucial. Structures situated on the surface or underground are highly sensitive to these settlements, and maintaining these settlements within acceptable levels is of paramount importance (Golpasand, Nikudel, & Uromeihy, 2016).

In the conducted studies, it has been observed that in order to maintain tunnel stability and prevent deformations, a thorough understanding of soil behavior during tunnel excavation is essential. To gain a better understanding of these behaviors, attempts have been made to compare site observations with results obtained through finite element methods. Numerous studies have been carried out in the literature regarding the fundamental factors that influence surface settlements. Some values considered important are summarized below.

2.6 Parameters Controlling Surface Settlements Due to Tunnel Excavations

Lecca and New (2007) have stated that the surface settlements occurring during tunnel excavation are dependent not only on the engineering characteristics of the ground, such as geological and hydrogeological properties, but also on tunnel geometry, excavation depth, excavation method, process management during excavation, and the quality of work.

• Volume losses occurring during tunnel excavation
• Control of groundwater levels
• Methods employed in tunnel excavation (umbrella arch-ground nails and tunnel lining stability)
• Tunnel face excavation stages (full face-stepwise excavation...)
• Improvement of parameters related to the ground above the tunnel (grouting-freezing

2.6.1 Prediction of Surface Deformations Using Empirical Methods

In relation to the prediction of surface settlements caused by tunnel excavations, various researchers like Cording and Hansmire (1975), Atkinson and Potts (1977), Attewel and Woodman (1982), O'Reilly and New (1982) have adopted different approaches. Particularly, the Gaussian Curve first described by Peck (1969) has been a guiding study for a more accurate understanding of these settlements.

In these studies, empirical formulas have been developed using values such as tunnel diameter, tunnel depth, volume losses, and engineering parameters of the soil. To comprehend the dominant parameters governing these formulas, the methods of Peck (1969) and O'Reilly and New (1982) are explained below.

2.6.1.1 Peck Formula Theory

In 1969, Peck proposed that the ground settlement conforms to the law of normal distribution by summarizing the measured data of ground settlement caused by a large number of tunnel construction at that time.

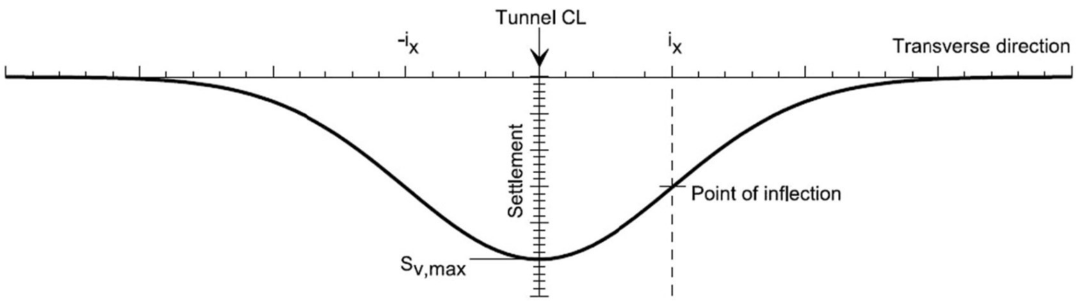


Figure 2.4 Transverse aspect of ground settlement due to tunneling (Golpasand, Nikudel, & Uromeihy, 2016)

$$S(x)=S_{(max)}\exp\left(-\frac{x^2}{2i^2}\right); \tag{2.1}$$

$$S_{(max)}=\frac{Vs}{i\sqrt{2\pi}}; \tag{2.2}$$

$$i=\frac{Z}{tan\left(45°-\frac{\emptyset}{2}\right)\sqrt{2\pi}} \tag{2.3}$$

The Gaussian, or normal distribution curve, has two significant variables related to the maximum settlement S_{max} that can occur on the surface and the inflection point (bend point) i. The definition of the inflection point is provided by various researchers' equations as shown in Table 2.1 for their studies.

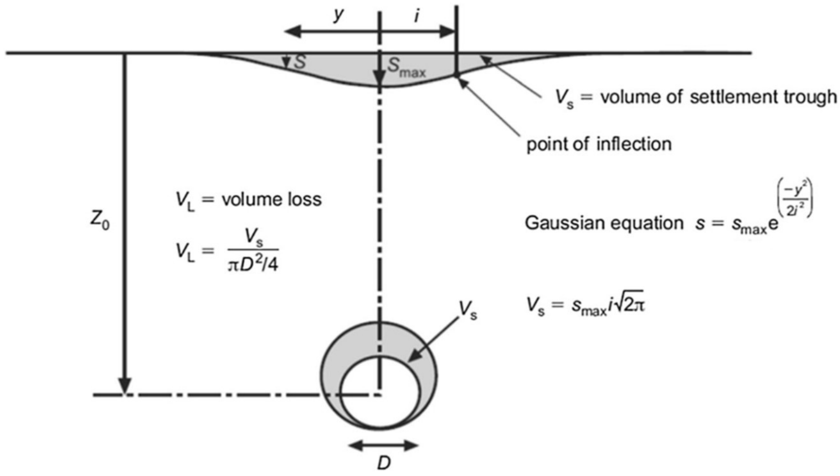


Figure 2.5 Volume loss and surface settlement (Rezaei, & Ahmadi, 2020)

Table 2.1 Definition of the inflection point is provided by various researchers.

Researchers	**Inflection Points (i)**	**Research Method**
Peck (1969)	$\frac{2i}{D}=\left(\frac{H}{D}\right)^{(0.80\sim1.0)}$	Site Observations
Attewel and Farmer (1974)	$\frac{2i}{D}=\left(\frac{H}{D}\right)^{(1.0)}$	Site Observations
Attewel (1977)	$i = 0.5H$	Only Clayey Soil
Atkinson and Potts (1977)	$i = 0.25(1.5C + D)$ $i = 0.25(C + D)$	Site Observations and Model Experiments
Clough and Schmidt (1981)	$i = 0.5H^{0.8}D^{0.2}$	-
O'Reilly and New (1982)	$(Cohhesive\ Soil)$ $i = 0.5H + 1.1\ m$ $(Cohessionless\ Soils)$ $i = 0.28H - 0.12m$	Site Observations
Mair vd. (1993)	$i = 0.5H$	Site Observations and Centrifugal Models
Arıoğlu (1992)	$i = 0.4H + 0.6$ $(Cohessive\ Soils, shielded\ machine)$	
Arıoğlu (1992)	$i = 0.9R\left(\frac{H}{2D}\right)^{(0.88)}$ $(All\ type\ soils, Shielded\ machine))$	
Arıoğlu (1992)	$i = 0.368H + 2.84$ $(All\ type\ soils)$	

In the studies conducted by O'Reilly and New, they indicated that the variable i has a stronger correlation with tunnel depth rather than tunnel diameter, and they straightforwardly formulated a simple equation for determining the inflection point value.

$$i = K^* \times H. \tag{2.4}$$

Where, K* represents the soil constant value. As K* varies according to different soil types, Rankin (1988) suggested taking K* = 0.5 for clayey soils and K* = 0.25 for cohesionless soils. Additionally, Mair et al. (1993) proposed the equation stated below for the K* constant. Through observations during tunnel construction on London clay and centrifuge tests, they obtained a reasonably feasible range of data for the K* constant as seen in Figure-2.6 (Taylor, 1995).

$$K^* = 0.325 + \frac{0.175}{(1 - z/z_0)} \tag{2.5}$$

Where, z_o and z are the distances from the ground surface to respectively the tunnel axis.

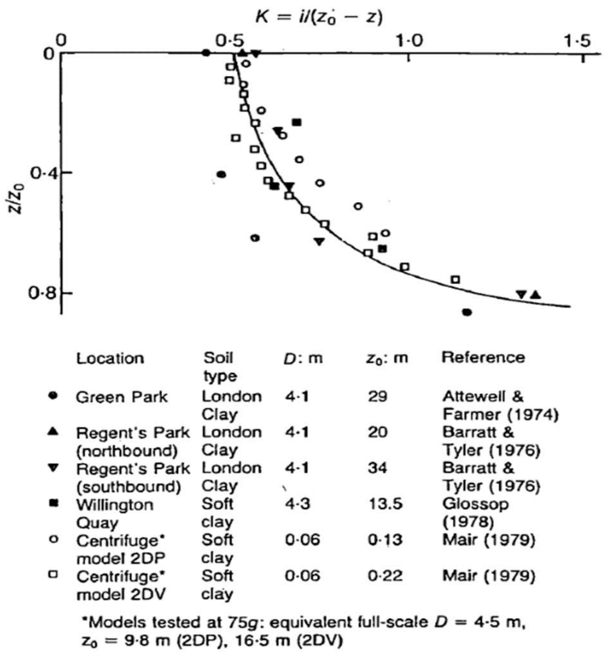


Figure 2.6 Variation of K with depth for subsurface settlement profiles above tunnels in clays (Mair et al, 1993)

The equation derived by O'Reilly and New (1982) for the maximum anticipated settlement on the surface is given below.

$$S_{(max)} = 0.313 V_L \left(\frac{D^2}{i}\right) \tag{2.6}$$

Where V_L is the volume loss, D is the diameter of the tunnel, i is the settlement trough width coefficient, i.e., the abscissa of the inflection point of subsidence curve, S_{max} is land subsidence at the center line of the tunnel.

Both Peck (1969) and O'Reilly and New (1982), as well as numerous researchers who have derived equations using empirical methods, have demonstrated that the maximum surface settlements are primarily influenced by the volume loss parameter. To define volume loss, it can be characterized as the additional excavated volume (ΔV) resulting from creating a larger void than the permanent tunnel excavation void, divided by the permanent tunnel void ratio (At). Volume loss is often referred to as ground loss and contributes to surface settlements due to displacement toward the tunnel (Golpasand, Nikudel, & Uromeihy, 2016).

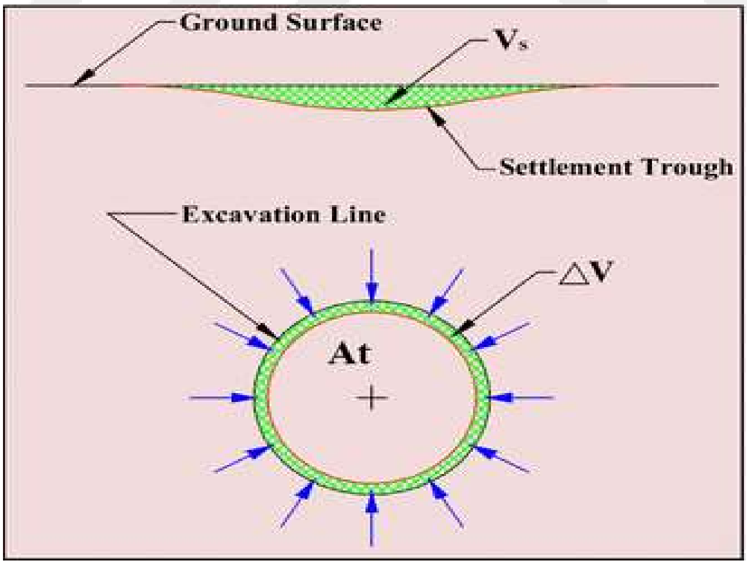


Figure 2.7 Settlement of the ground due to volume loss (Golpasand, Nikudel, & Uromeihy, 2016)

In many studies related to volume loss, specific intervals have been obtained for predicting surface settlements depending on the excavation method. Arıoğlu (1992) derived an empirical formula for determining the volume loss ratio and highlighted a significant relationship with the soil stability ratio (N). Additionally, Schmidt (1968) proposed the following equation for predicting the maximum surface settlement.

$$S_{(max)} = 0.0125K\left(\frac{R^2}{i}\right) \quad (2.7)$$

$$K = 0.87e^{0.26N} \quad (2.8)$$

$$N = \frac{\gamma_n H + \sigma_S - \sigma_T}{c_u} \quad (2.9)$$

Where

σ_S:Total surcharge pressure

σ_T: TMB face pressure

γ_n: Unit weight of soil

Table 2.2 Relationship between volume loss (VL), construction practice, and ground conditions (Ahmet, & Iskander, 2011)

Case	V_L (%)
Good practice in firm ground; tight control of face pressure within closed face machine in slowly raveling or squeezing ground	0.5
Usual practice with closed face machine in slowly raveling or squeezing ground	1.0
Poor practice with closed face in raveling ground	2.0
Poor practice with closed face machine in poor (fast raveling) ground	3.0
Poor practice with little face control in running ground	4.0 or more

In addition to the empirical formulas for predicting maximum surface settlement obtained by some researchers considering the volume loss ratio, Herzog (1965) proposed the following equations by associating them with independent parameters of the ground, tunnel diameter and depth. However, in a study conducted for a tunnel opened in Istanbul, the equations proposed by Herzog (1965) were compared with values obtained through numerical methods and real surface measurements, revealing significant discrepancies. This was attributed to the fact that this equation did not account for the characteristics of umbrella arches created in tunnels opened using the NATM technique, leading to its modification. The modified equation's results, obtained using the modified equation, were found to support the results obtained from numerical and actual surface measurements (Hasanpour, Chakeri, Ozcelik, & Denek, 2012).

For single tunnel:

$$S_{(max)} = 0.785(\gamma_n Z_0 + \sigma_s)\left(\frac{D^2}{iE}\right) \tag{2.10}$$

For twin tunnel:

$$S_{(max)} = 4.71(\gamma_n Z_0 + \sigma_s)\left(\frac{D^2}{(3i+a)E}\right) \tag{2.11}$$

Where, E is elasticity modulus of the soil (ton/m²), a is spacing between tunnels (m), Z_0 the distances from the ground surface to respectively the tunnel axis (m), γ_n is the natural unit weight of soil (ton/m³), σ_s is the total surcharge load (ton/m²) and d is spacing between pipes (m).

The formula proposed by Herzog (1965) is recommended to be used for shallow tunnels where the primary support is completed using the NATM technique. Additionally, these equations do not consider volume loss and long-term settlements.

Summarizing the studies conducted for predicting maximum surface settlements obtained through site observations and experiments, it can be observed that the surface settlements are influenced by factors such as tunnel diameter, tunnel depth, soil structure, and the volume losses incurred due to tunnel excavation methods. However, it should be noted that the empirical formulas outlined above do not account for seepage and consolidation effects in predicting surface settlements. Especially for tunnel excavations within urban areas, both tunnel diameter and tunnel depth, as well as excavation methods, should be reviewed to determine a preliminary settlement value. The impact of these settlements on surface structures should be assessed, and if necessary, optimizations should be carried out.

2.6.2 Groundwater Level Control

One of the most common challenges in tunneling is encountering water in the ground. When water enters tunnel excavations conducted below the groundwater level, it is drained and removed from the tunnel to create a safe excavation environment. However, this drainage process, depending on the amount of incoming water, can lead to a lowering of the groundwater level. The decrease in groundwater level causes an increase in the weight of soil particles floating in the water. This leads to additional loads on the lower layers of soil, resulting in an increase in effective stress. Due to these additional loads, the soil layers start to settle again, causing settlements.

Furthermore, water that constantly flows into the tunnel carries soil particles with it. This can lead to soil loss above the tunnel, accelerating surface settlements. To control groundwater levels, various methods such as contact injection, consolidation injection, or permeation grouting with both chemical and cement injections are applied within the soil. In this regard, Aksoy, (2008) conducted a chemical injection study from a shaft opened for tunnel excavation to halt excessive water ingress causing a drop in the existing groundwater level. This application successfully stopped excessive water ingress and restored the groundwater level to its original state. As a result, this approach prevented settlements caused by the drop in groundwater level on the structures above the tunnel.

As a result of the lowering of the groundwater level, the weight of soil particles floating in the water increases. This causes additional loads to be applied to the lower layers of soil. Due to these additional loads, the soil layers start to settle again.

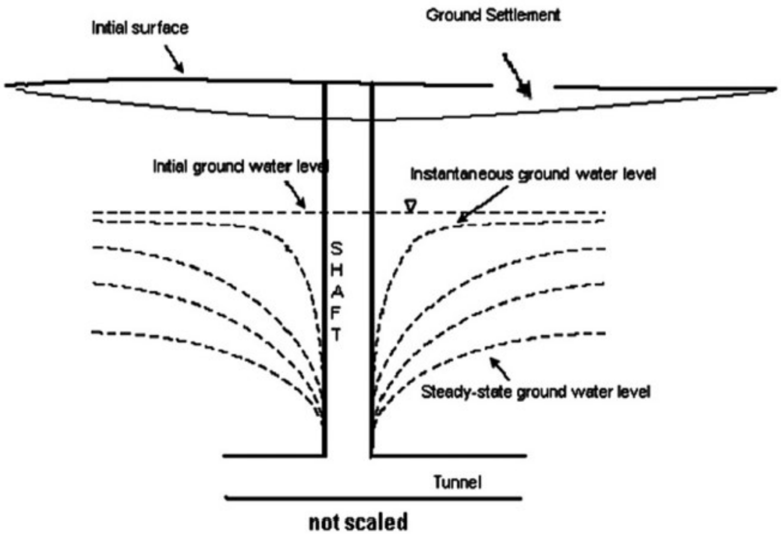


Figure 2.8 Groundwater flow behavior and ground settlement. (Aksoy, 2008)

In another study, through numerical analysis, the expected maximum surface settlement value was found to be between 1.5 to 2.0 mm. However, site measurements revealed that this value reached levels of around 25 mm. Upon evaluation, it was determined that in a tunnel where 3.5 l/s of water was measured to be coming, the factor contributing to the increased surface settlements was excessive water ingress. To mitigate this, a chemical injection was applied within the tunnel to reduce the water ingress. Following the chemical injection, the water influx from the tunnel decreased to levels of 0-0.5 l/s, and the time-dependent settlement values exhibited a decreasing trend after the chemical injection (Kucuk, Genis, Onargan, Aksoy, Guney, & Altındağ, 2009).

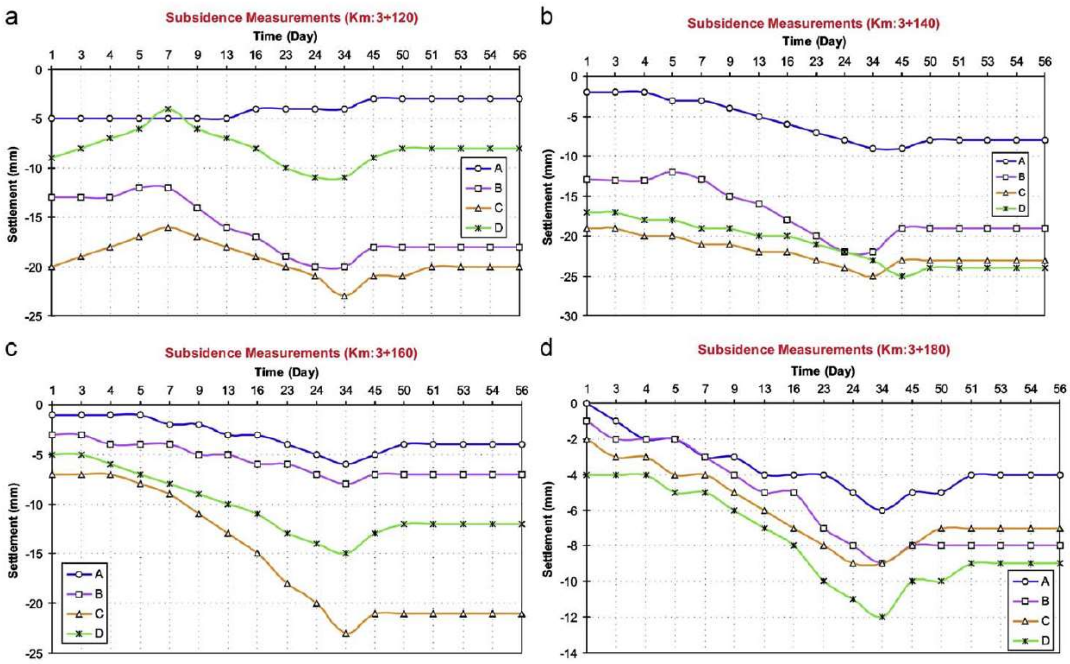


Figure 2.9 Time depended on ground settlement measurement before and after chemical injection (Kucuk, et al., 2009)

In areas where groundwater is prevalent, the methods chosen for groundwater level control must be selected with great caution. Interventions within the soil can alter the flow direction and speed of groundwater due to reasons like changing the water flow direction or slowing it down at certain points. These changes can disrupt the existing water balance. The disturbed groundwater pressure equilibrium within the soil can add to subsidence on the tunnel or the surface structures.

Kucuk, (2011) evaluated a subsidence that occurred in the tunnel lining and attributed it to injection columns that altered the flow direction of groundwater. This study highlights the importance of thoroughly evaluating the hydrological characteristics of groundwater. The efforts to prevent water from entering the tunnel can potentially lead to worse outcomes for the tunnel. Therefore, the hydrological nature of groundwater should be carefully assessed, and the chosen method should be decided accordingly.

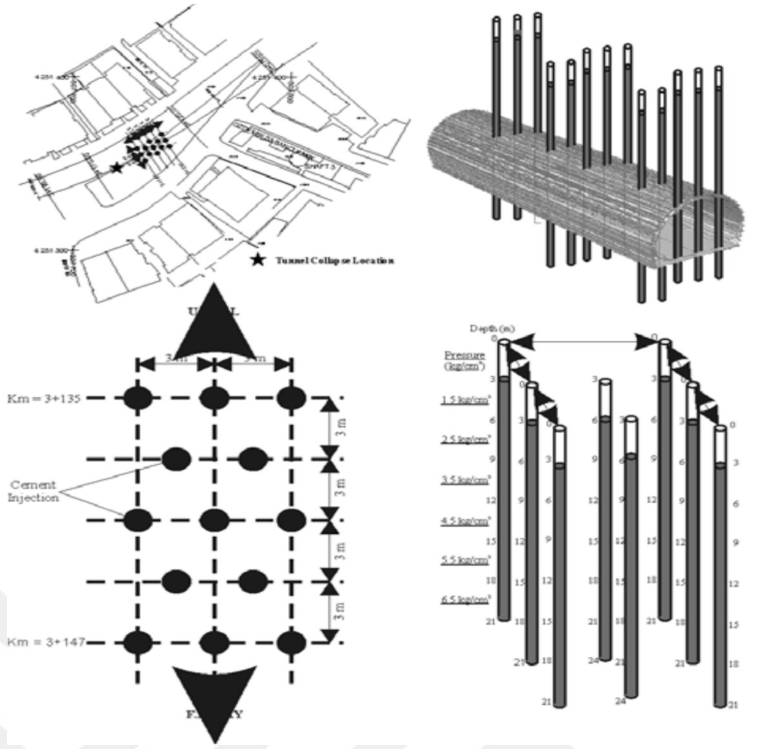


Figure 2.10 Cement injection details around Ozdemir Sabanci location (Kucuk, 2011)

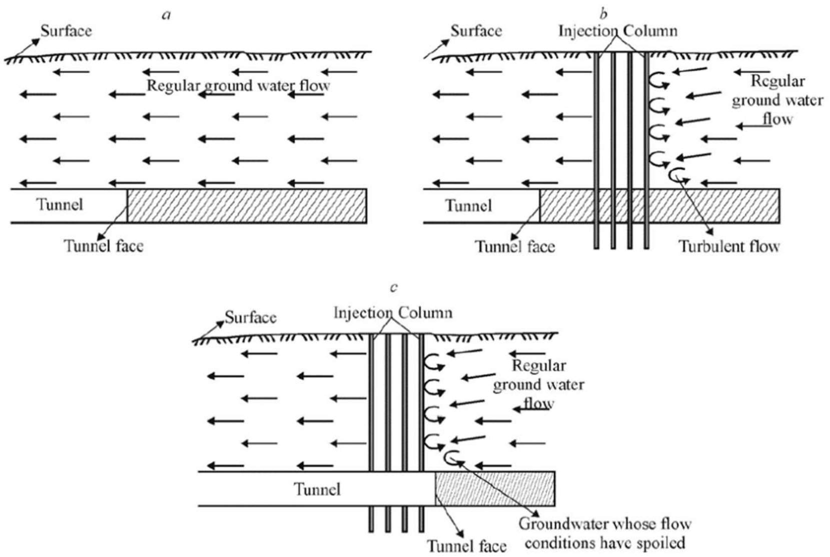


Figure 2.11 Bibliography of ground water problem (Kucuk, 2011)

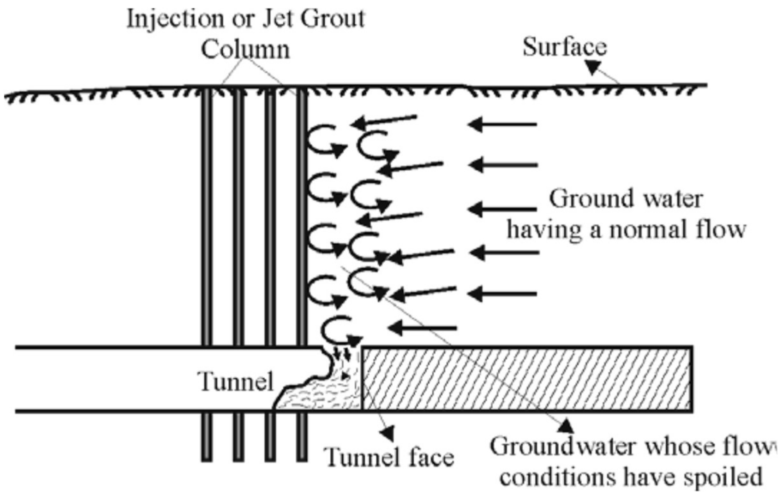


Figure 2.12 Mechanism of shallow tunnel collapse induced by ground water Bibliography of ground water problem (Kucuk, 2011)

Indeed, as observed in the study conducted by (Wei, & Zhu, 2021), reductions in groundwater levels have been shown to increase surface settlements.

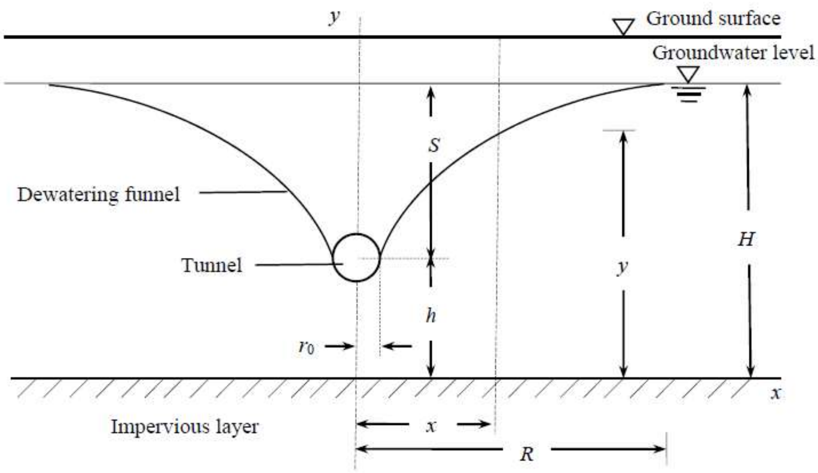


Figure 2.13 Profile of the dewatering funnel a tunnel (Wei, & Zhu, 2021)

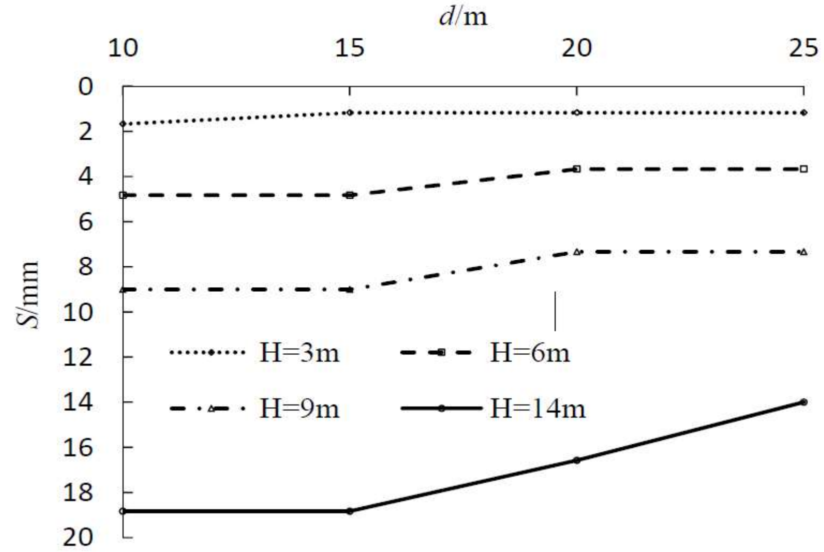


Figure 2.14 Influence of different drawdown on surface subsidence (Wei, & Zhu, 2021)

2.6.3 Methods Applied in Tunnel Excavation (Umbrella Arch and Soil Nail Applications)

During tunneling operations, sensitivity analysis and particularly the evaluation of displacements that can affect the surface, especially in shallow depths, are crucial. Over time, various tunneling methods have been developed internationally to enhance sensitivity and address surface displacement concerns. Some well-known methods include the German Tunneling Method, American and Italian Tunneling Methods, New Austrian Tunneling Method (NATM), British Tunneling Method, Tunnel Boring Machine (TBM) Tunneling Method, and tunneling by blasting. Each method has its own advantages and disadvantages.

When deciding on a tunneling method, several factors need to be considered together, such as the geological composition (rock or soil), tunnel diameter, tunnel depth, existing structures along the tunnel route, and socio-economic factors. A comprehensive understanding of these factors is essential for selecting the appropriate tunneling method.

For our thesis, which focuses on reducing surface deformations in tunneling using the NATM (New Austrian Tunneling Method), the emphasis has been placed on the NATM method. More detailed explanations about the mentioned methods can be found in the study by Satıcı and Topal, (2015).

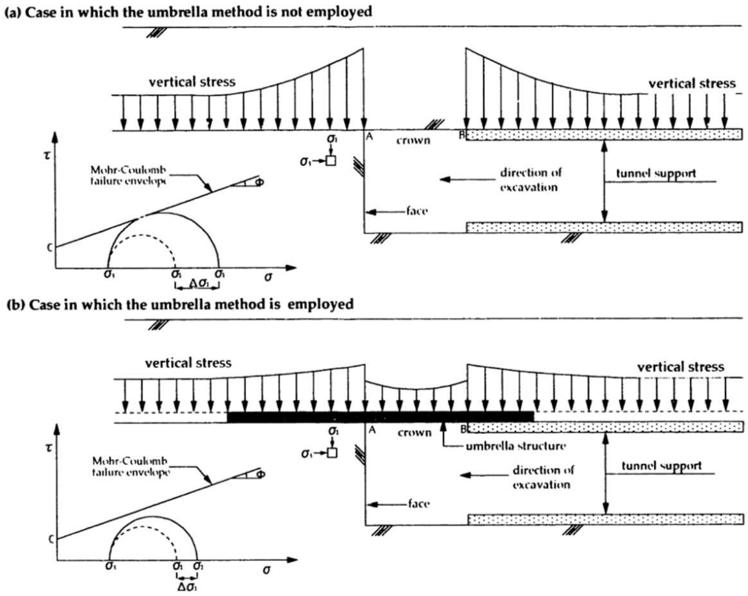


Figure 2.15 Vertical stress distribution alon a longitudinal line at tunnel crown and stress state ahead of the face (Muraki,1997) (a) Umbrella method is not employed, (b) Umbrella method is employed

Surface deformations caused by tunnels constructed in urban areas can lead to significant damage to the existing structures in the vicinity. In tunneling using the NATM (New Austrian Tunneling Method), which aims to reduce these deformations, the use of steel pipes and ground anchors in the upper region of the tunnel has been observed to play a crucial role in minimizing surface settlements. Several studies have indicated that this preference for utilizing steel pipes and ground anchors above the tunnel significantly contributes to reducing surface settlements. In a study comparing the use and non-use of steel pipes above the tunnel, it was observed that surface settlements could be reduced by up to 65% when steel pipes were implemented, as given in Table 2.3 (Hasanpour, et al. 2012).

Table 2.3 Surface settlement values (Hasanpour, et al., 2012)

Smax FLAC3D with pipe-roofing	Smax FLAC3D without pipe-roofing
23.6 mm	67 mm

Similarly, in a study comparing the usage and non-usage of steel pipes and ground anchors above the tunnel, a reduction of surface settlements by 69% (from 23 mm to 6.8 mm) was observed, as depicted in Figure 2.16 (Aksoy, & Onargan, 2010).

In another study conducted in the Yenikapı-Taksim metro tunnel, particularly in clayey soil formations, surface settlements were observed to decrease by 75% when steel pipes and umbrella arch were used (Ocak, 2008).

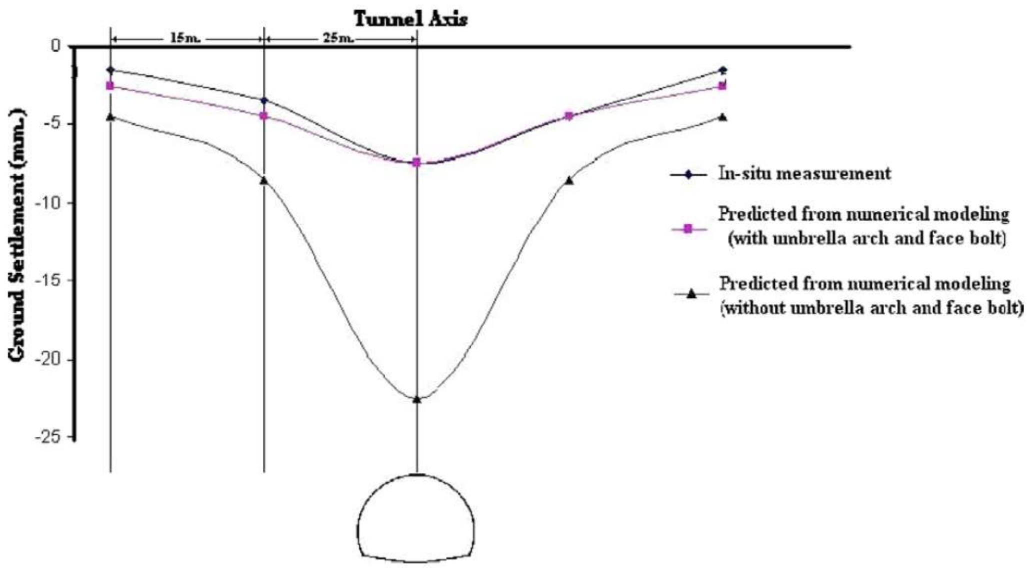


Figure 2.16 Ground settlement values obtained from in situ measurements and numerical models (Aksoy, & Onargan, 2010)

In another study, it was observed that the application of steel pipes had a positive effect on surface settlements (Yeo, Lee, Tan, Hasegawa, Suzukı, & Shınjı, 2009).

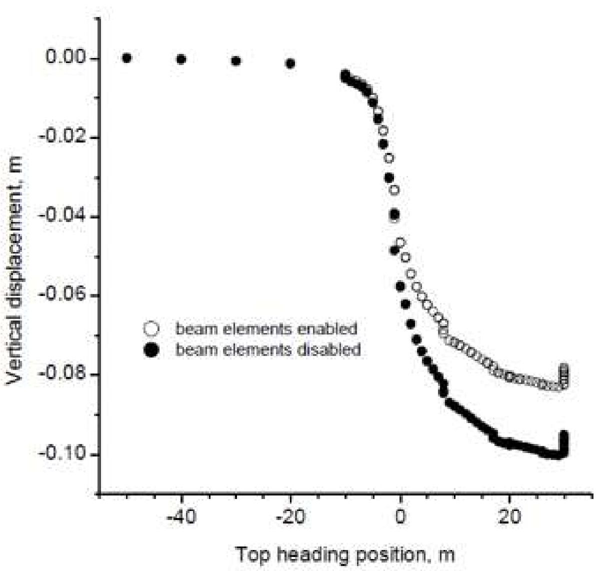


Figure 2.17 Vertical displacement progresses in top heading (Yeo, et al.,2009)

Another crucial aspect in tunnel excavations is ensuring the proper establishment of tunnel lining stability. Possible movements in the tunnel lining can lead to both tunnel collapses and significant surface settlements. The effect of the soil nails applied to the tunnel lining is similar to the concept of reinforced concrete, where the applied soil nails generate lateral stresses through friction. This effect is better understood on the Mohr's circle diagram shown in Figure 2.18. Findings from a study involving numerical analyses and laboratory experiments on the reinforcement of tunnel linings revealed that the reinforcement elements used in the tunnel lining not only enhanced the lining stability but also reduced surface settlements. The usage rates, quantities, and lengths of the reinforcement elements in the tunnel lining were found to decrease the displacements occurring in the tunnel lining. Yoo, C., & Shin, H. K. (2003). Reductions in deformations within the tunnel lining will directly result in decreased surface settlements.

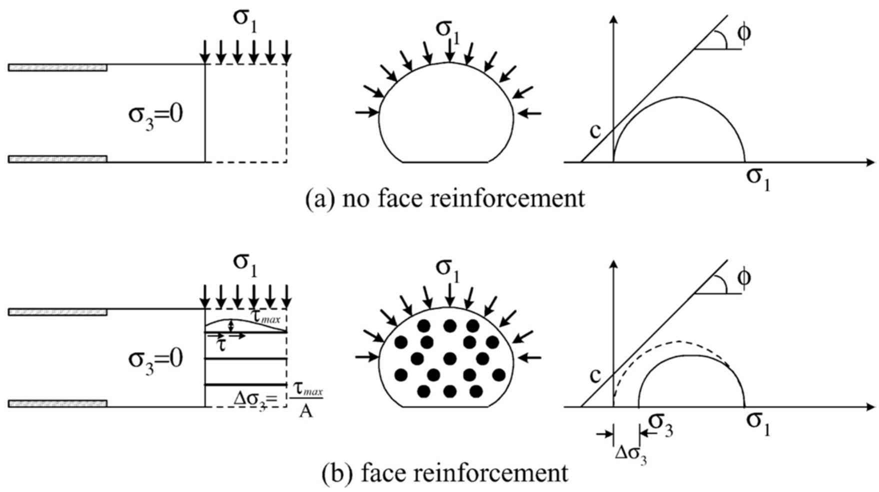


Figure 2.18 Theoretical representation of face reinforcement technique (Scholsser, 1997)

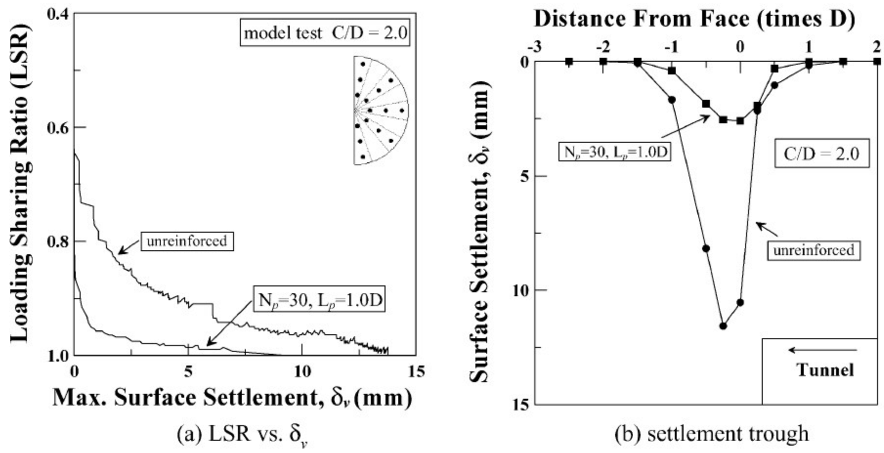


Figure 2.19 Effect of face reinforcement on surface settlement (model test): (a) LSR-dv relationship; (b) longitudinal settlement trough (Yoo, & Shin, 2003)

2.6.4 Improvement of Parameters Related to the Ground Above the Tunnel

2.6.4.1 Ground Improvement Through Injection

During tunnel excavation, one of the measures taken to ensure the safety and protection of surrounding structures is the implementation of injection processes beneath buildings and above tunnel areas. If we evaluate these separately

Subgrade injection beneath buildings involves the injection of materials such as polyurethane, a mixture of cement, water, and sand, or epoxy resins into the ground. This process aims to compact the soil beneath and prevent potential deformations in the building's foundation. By performing this injection process beforehand, it preemptively mitigates deformations caused by tunnel excavation and is frequently utilized for this purpose.

Grout application on the tunnel surface leads to a decrease in surface settlement values directly or indirectly due to the following conditions:

- The injection process applied on the tunnel surface fills voids that may occur in the upper part of the tunnel, enhancing the stability of the soil or rock formations. This situation reduces the deformations that may occur on the surface.

- The injection process applied on the tunnel surface fills voids within the ground, resulting in a more impermeable soil in the upper area of the tunnel. This helps reduce water flow into the tunnel, leading to a decrease in water drawdown from within the tunnel. This reduction in water drawdown prevents surface settlements caused by the lowering of the groundwater level, as explained in the upper sections.

In a study conducted to reduce the deformation on the existing tunnel due to excavation in the surrounding area, grout injection into the ground was performed. It was observed that the grout application reduced the horizontal displacements from 9.2 mm to 4.6 mm (Zheng, Pan, Cheng, Bai, Du, Diao, & Ng, 2020).

2.6.4.2 Soil Improvement with Freezing Method

In tunnel constructions facing challenging ground conditions, the method of freezing the ground can be employed. This method is based on the principle of solidifying the ground by freezing the water within it. In summary, the freezing process is achieved through a cooling system and pipes utilized during tunnel construction. These pipes are inserted into the ground and circulated with a cooling fluid. By freezing the water in the ground, the cooling fluid causes the ground to harden. Once the tunnel construction is completed, the cooling system and pipes are removed. The water in the ground naturally melts, returning to its normal state. While the ground freezing method for tunneling above is costly, it remains an effective means to ensure the safety of surrounding structures during tunnel construction. However, prior to implementing this method, a detailed ground examination is necessary, and suitable cooling equipment must be selected. Due to its specialized expertise requirement and cost-intensive nature, this method is less frequently preferred in practical applications.

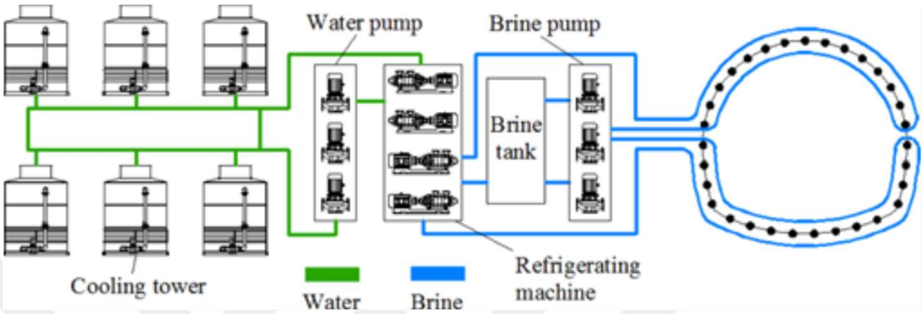


Figure 2.20 Freezing system (Yan, Wu, Zhang, Ma, & Li, 2019)

The ground freezing method is sometimes used not only to control surface settlements but also to allow the construction of interconnected or very closely spaced underground structures. In the city of Kobe, Japan, for instance, the ground freezing method was employed to resolve the issue of potential interference between newly planned tunnels and existing underground structures. This method was successfully applied, and the desired outcome was achieved. In this study, the following equations were utilized to predict the stress.

Deformation Modulus

$$E = 13.75\,(-T)^{1.18}\ \text{(frozen clay)} \tag{2.13}$$

$$E = 27.5\,(-T)^{1.18}\ \text{(frozen sand)} \tag{2.14}$$

Yield Stress

$$\sigma_y = 0.531\,(-T)^{0.404}\ \text{(MPa) (frozen clay)} \tag{2.15}$$

$$\sigma_y = 1.06(-T)^{0.404}\text{(MPa) (frozen sand)} \tag{2.16}$$

where T is the temperature.

2.2.5 Excavation Stage and Importance of Sequential Excavation

The New Austrian Tunneling Method (NATM), which enables excavation in various types of ground, is a frequently favored technique. The distinct engineering parameters of each type of ground have led to different approaches within the same excavation method. Both site observations and academic studies have revealed a significant finding in the NATM technique, involving multiple excavation stages that control surface settlements. A gradual excavation profile is illustrated in Figure 2.21, where the excavation stages and the impact of the unsupported portion's (free span) length on surface settlements have been assessed and presented in tabular form.

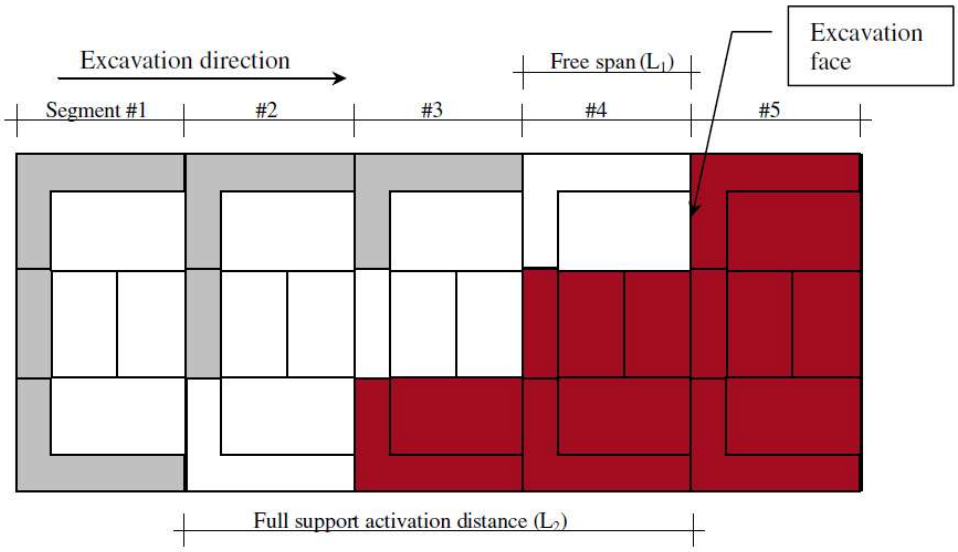


Figure 2.21 Schematic representation of 3-D excavation (De Farias, Junior, & De Assis, 2004)

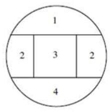
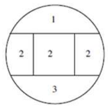
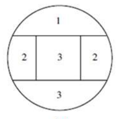
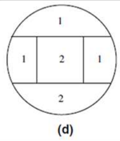
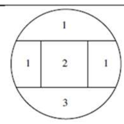
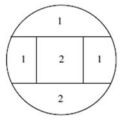
	CONDITIONS	PARTIAL FACE SCHEMA
CASE-1	L1=0.25D L2=0.25D FULL FACE	
CASE-2	L1=0.50D L2=0.50D FULL FACE	
CASE-3	L1=0.25D L2=1.00D PARTIAL-FACE (a)	 (a)
CASE-4	L1=0.25D L2=0.75D PARTIAL- FACE (b)	 (b)
CASE-5	L1=0.25D L2=0.75D PARTIAL-FACE (c)	 (c)
CASE-6	L1=0.25D L2=0.50D PARTIAL-FACE (d)	 (d)
CASE-7	L1=0.25D L2=0.75D PARTIAL-FACE (e)	 (e)
CASE-8	L1=0.50D L2=0.25D PARTIAL-FACE (d)	 (d)

Figure 2.22 Condition and cross section of cases

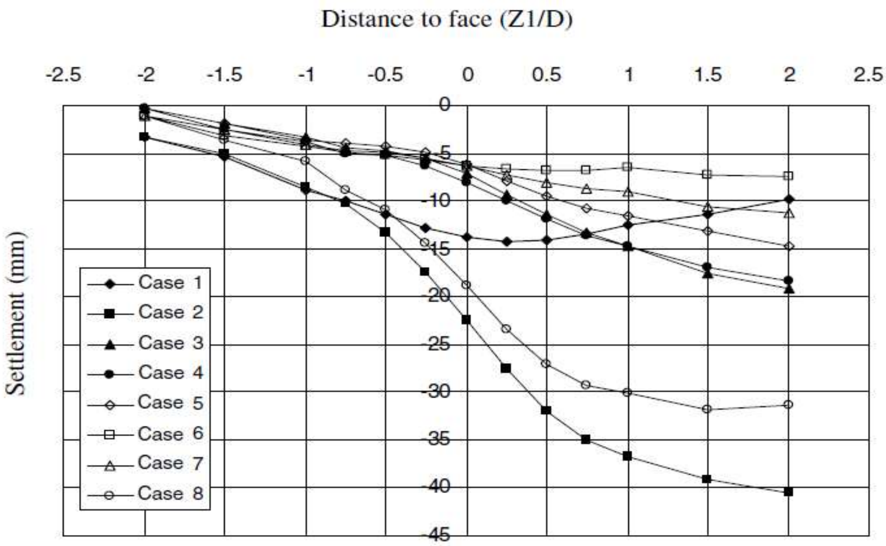


Figure 2.23 Settlement of differences cases. De Farias, et al.,2004)

Note that: Negative value of Z_1=D indicates that the excavation face did not reach the point section yet.

Upon examining the above tables and graphs, it is evident that an increase in the L_1, or free span, distance has a more dominant influence on surface settlements compared to other conditions. Another significant factor is not incrementing the L_1 distance gradually.

In another study, different tunnel excavation approaches with varying excavation steps, as shown in Figure 2.24, were compared for tunnel type-2 using Sequential excavation model (SEM), Two-stage excavation model (TSM), and Full-face excavation model (FFM). Through the conducted analysis, it was observed that, in the case of type-2 excavation stages, the sequential excavation method, as implemented in the field, yielded values closer to real site measurements. In tunnels excavated in soft ground conditions, performing fewer steps in the internal support system in TSM and FFM models led to delayed closure of the tunnel support ring and greater surface deformation (Karakus, & Fowell, 2003).

In the introduced models for type-2 excavation steps, they are described as follows.

- Sequential Excavation Model (SEM): Stages 1-2-3 and 4 are excavated sequentially according to a specific progression.
- Two-stage Excavation Model (TSM): Stages 1 and 2 are excavated simultaneously, followed by the simultaneous excavation of stages 3 and 4.
- Full-face Excavation Model (FFM): Stages 1-2-3 and 4 are excavated all at once.

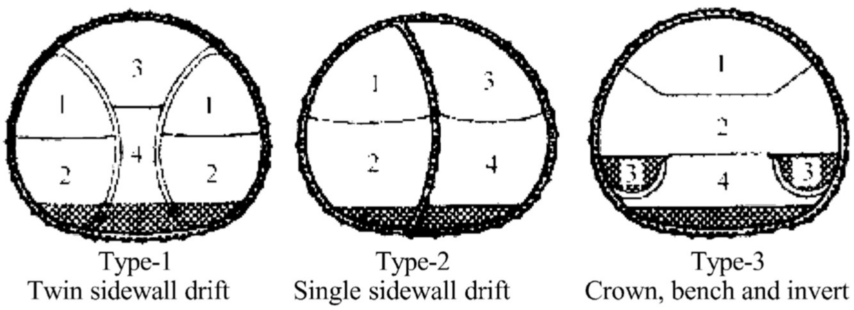


Figure 2.24 Heathrow Express Trial tunnels (Bowers, 1997)

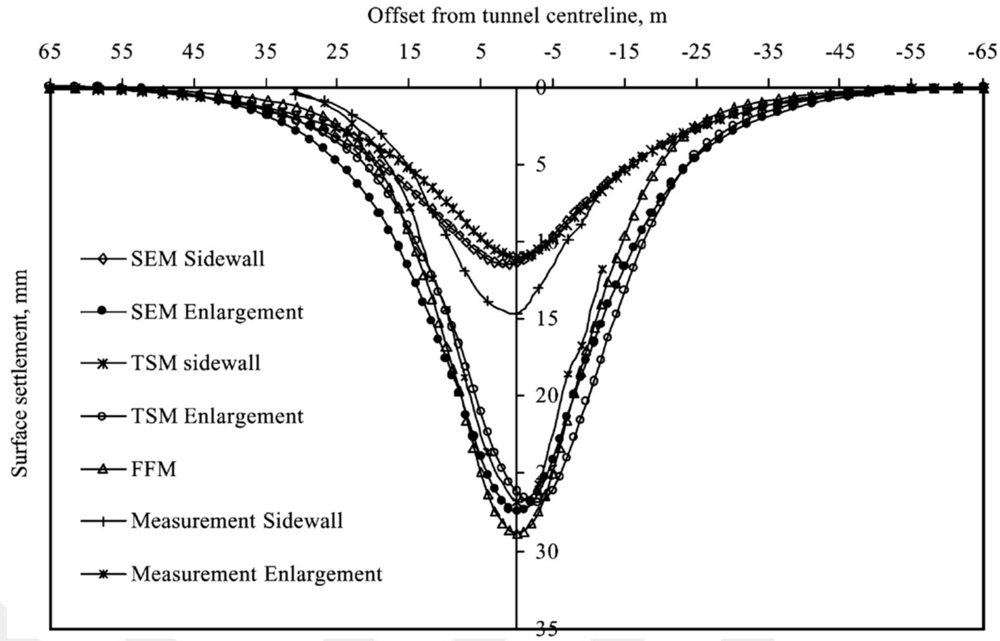


Figure 2.25 Prediction surface settlements of SEM, TSM and FFM models (Karakus, & Fowell, 2003)

As outlined in the mentioned academic studies, deformations on the surface due to tunnel excavations are not solely attributed to a single cause but are observed to result from multiple factors. Therefore, the soil conditions of the area where tunnel excavation will take place, as well as the characteristics of the chosen excavation method, need to be evaluated meticulously, and the most suitable method should be selected. Especially in urban areas, tunnel excavations pose risks for existing structures due to the potential surface deformations they might cause.

This thesis focuses on mitigating the surface deformations caused by a tunnel excavation taking place within a city. Despite efforts to reduce these deformations using some of the methods mentioned above, there remains a need for an approach that addresses the issue when settlements do not meet accepted values. The thesis seeks to develop such a method within its scope.

CHAPTER THREE

GEOTECHNICAL CHARACTERISTICS OF TUNNEL CONSTRUCTION SITE

Tunnel construction is a complex engineering endeavor that necessitates a crucial geotechnical investigation phase for successful execution. Here are some reasons why geotechnical investigation is crucial prior to tunnel construction:

- **Determination of Soil and Geological Conditions**: Determining the optimal route and designing for tunnel construction necessitates accurate assessment of soil and geological conditions. Geotechnical investigation is used to determine factors such as soil bearing capacity, settlement characteristics of geological layers, and soil stability. This information provides the foundational parameters for the safe construction of the tunnel.

- **Soil Study and Soil Mechanics Analysis:** Geotechnical investigation is crucial for determining the characteristics and behavior of the soil to be used in tunnel construction. Soil study encompasses factors such as soil composition, properties, compaction behavior, permeability, and durability. This information allows for the consideration of soil effects and soil strength in tunnel design. Additionally, soil mechanics analyses, including stability and deformation of the soil, are integral parts of geotechnical investigation, ensuring the safe and stable progress of tunnel construction.
- **Groundwater Control**: Monitoring the groundwater level during tunnel construction is of paramount importance. Geotechnical investigation is used to identify water sources in the tunnel area and establish groundwater level control. Groundwater level is a critical factor that needs to be considered for managing drainage and water pressure issues during tunnel construction. Geotechnical investigation assists in determining strategies for accurately monitoring the

groundwater level through analyses of water flow and filtration, ensuring effective groundwater control measures.

- **Soil Stability and Soil Improvement:** Soil stability is of great importance during tunnel construction. Geotechnical investigation is utilized to identify areas where soil needs to be stabilized or strengthened. If problematic areas of soil stability are not identified, collapses, slides, or other soil movements can be observed during tunnel construction.

The importance of geotechnical investigation for tunnel construction has been conveyed above. The parameters associated with the rock and soil to be used in tunnel design can be acquired through field tests or laboratory studies. Within the framework of this thesis, tunnel design was formulated based on the assumption of constructing a tunnel in the same region by extrapolating from two closely situated borehole logs utilized in another project. Subsequently, the results were assessed.

3.1 Tunnel Line Geology

Within the scope of this thesis, a large-diameter and shallow tunnel excavated using the NATM technique in the Izmir region has been investigated. The relevant tunnel is located 30 meters below the surface, with the groundwater level measured at 29 meters from the surface. The geological investigations pertaining to the region through which the tunnel passes have been determined based on the borehole log data collected in this area and accepted approaches in the literature. Upon a detailed examination of the borehole log, the relevant soil layers can be described as follows:

Within 1-1.5 meters below the borehole: Probable soil composed of clay, sand, gravel, and excavation debris, gravel-block rubble, and various rock fragments. This filling material continues up to 12 meters. The N_{30} values of the alluvial units that continue up to 12 meters are within the range of 30 to >50 blow counts. These units consist of dense to very dense predominantly clayey, silty sandy gravel layers (GP-GM).

Between 12 and 23 meters: Open greenish-gray layers dominated by meta sandstone, fractured, and fragmented flysch.

Between 23 and 45 meters: Brownish-gray layers, generally weathered, fractured, and occasionally gravel-block in nature, weak to very weak strength shale interbeds, observed. The characteristics of these layers are described in detail below.

3.1.1 Made Ground

The thickness of the filling material formed by the clay-sand-gravel units varies between 0.50 and 1.00 meters according to the drilling works. However, since the drilling technique gives one-dimensional information about the soil and the soil properties between drillings can only be determined by interpolation, it should be considered that the fill thickness may increase in places where the drilling points do not coincide.

In the borehole, a potential fill material consisting of clay, sand, gravel, and excavation debris, as well as gravel-block-sized rubble and various rock fragments, has been observed with an average thickness of 1.5 meters.

3.1.2 Alluvium

Yellowish brown-gray colored, less clayey silty sandy pebbles, blocks and gravels of sandstone origin; It is traced up to 12 m after the filling material such as clayey, yellowish brown pebbly clayey sand and block size materials. Grain sizing to unit transitions can be observed closely.

The N_{30} value of the alluvial layer ranges between 28 and 50 blow counts, with an average value of 30. This layer predominantly consists of dense to very dense, predominantly clayey, silty sandy gravel units (GP-GM).

3.1.3 Weathered and Partially Decomposed Flysch

This zone, which is observed as an alternation of yellowish brown, light greenish gray, fractured, fragmented, moderately weathered sandstone shale, is generally interpreted as the Alluvial Flysch transition. In general, the separation of the lower parts of the alluvial succession and Weathered flysch is quite difficult.

The weathered flysch layer has an SPT-N_{30} value ranging from 41 to 50, with an average N_{30} value of 46. Although it appears generally cohesionless, due to the presence of a small amount of silty clay material, the effective shear strength has been estimated at c’ 4 kPa.

3.1.4 Flysch

The flysch, which is generally dark in color, consists mostly of sandstone, shale and occasionally diabases. The unit is very cracked and fractured. Quartzites are generally found in the cracks. The flysch unit forms steep and sloping slopes in the region. The predominantly observed shale unit presents a foliated structure. Limestones in the flysch unit are in the position of solitaire and can be found in very large masses. The limestones are gray black in color and very thick or massive. Generally dark brown colored flysch is represented by various argillaceous flyschs, micaceous sandstones, arkose, conglomerate, crystalline or dolomitic limestones, reddish pink or grayish brown flysch limestones and radiolarites. Sometimes quartz, sometimes calcite, sometimes veins of both minerals cut these layers. Various clayey flyschs and sometimes very crystallized limestones in the flysch show that this formation has undergone severe folding and metamorphosis.

3.2 Field Tests

In-situ SPT tests were conducted during drilling to determine soil strength parameters. Since correlations based on SPT results are frequently used to obtain various soil parameters, SPT results are highly significant findings in obtaining a multitude of soil properties.

3.2.1 Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) is a widely used geotechnical testing method to evaluate the engineering properties of soils and determine design parameters. This test is considered one of the fundamental tests used to ensure safe and sound construction of structures, minimize risks related to soil, comprehend soil's engineering properties, and establish appropriate design parameters. The classification of SPT-N_{30} values proposed by Clayton (1993) is presented in the table below:

Table 3.1 The classification of SPT-N_{30} values proposed by Clayton (1993)

Soil Type	SPT		Soil Classification
Sand	$(N_1)_{60}$	0-3	Very Loose
		3-8	Loose
		8-25	Medium
		25-42	Dense
		42-58	Very Dense
Clay	N_{60}	0-4	Very Soft
		4-8	Soft
		8-15	Firm
		15-30	Stiff
		30-60	Very Stiff
		>60	Hard

3.2.2 Menard Pressuremeter Test

It is known that the deformation modulus for soils, apart from laboratory experiments, is generally determined using empirical (observational) approaches. Despite the widespread use of this critical geotechnical design parameter in calculations performed with two or three-dimensional finite elements, there is a necessity for this value to be determined under conditions closest to the natural soil environment.

Especially in environments where there are frequent transitions between soil and weathered rock masses, the inability to perform SPT field tests, the unavailability of undisturbed soil samples, or the difficulty in obtaining them highlights the need for the deformation modulus to be determined in situ through pressuremeter testing. The values obtained from pressuremeter tests conducted in the field are shown in the table below:

Field pressuremeter tests provide valuable insight into the deformation properties of the soil and contribute to more accurate geotechnical analyses and design considerations.

Table 3.2 Pressuremeter values in situ

PRESSUREMETER TEST RESULTS				
SOIL LAYER	**Net Limit Pressure**	**Menard Modulus**	**α (Briaud, 1992)**	**Deformation Modulus**
	P_L, MPa	**E_m, MPa**	**-**	**E, MPa**
Made Ground	0.3	4	0.67	6
Alluvium	3.2	18	0.67	27
Weathered and Partially Decomposed Flysch	2.7	29	0.67	43
Flysch	2.9	145	0.67	216

3.3 Laboratory Tests

During the drilling process, field tests were conducted on disturbed Standard Penetration Test (SPT) samples and preserved core samples. These tests included the determination of natural and dry unit weight, moisture content, sieve analyses, Atterberg limits, and direct shear box tests. For units displaying rock characteristics in the researched areas, unconfined compression tests and point load tests were conducted on core samples.

3.3.1 Sieve Analysis

Sieve analysis is a test method used to determine the particle size distribution of soil samples. This analysis enables the determination of the percentage distribution of different particle sizes present in the soil. In the sieve analysis of soil, the sample is initially placed in a suitable set of sieves and then the sample is shaken over the sieve stack. Using sieves with different particle sizes in the sieve set, the soil sample is classified based on particle size as it is shaken through a series of sieves. The results obtained from sieve analysis are used to classify the soil according to ASTM or other standards. The classification of soil into different particle sizes allows for the identification of the soil and the determination of appropriate design parameters.

The granulometric composition of the soil plays a vital role in recognizing important engineering properties such as compressibility, permeability, strength, and liquefaction potential, thus enabling accurate design and assessment. Sieve analysis conducted on samples collected from the site is presented in the table below:

Table 3. 3 Sieve analyses results

Depth(m)	Soil Class	Water Content (%)	Atterberg Limit Value	Wet Sieving Results	
3,00-3,45	GP-GM	5.5	NP	+#10=%68,1	-#200=%9,3
9-00-9,45	GP-GM	7.4	NP	+#10=%65,8	-#200=%11
10,50-10,95	GP-GM	6.3	NP	+#10=%72,3	-#200=%10,6

3.3.1 Borehole Log

In this study, the strength parameters of the soil have been obtained based on a sample borehole log. The relevant borehole log is shown in Appendix A

It can be observed that the relatively rock-like flysch soils generally exhibit a fracture frequency and weathering degree of W5-M5. The quality of the Rock Quality Designation (RQD), fracture frequency, and weathering degrees are used for rock classification and the corresponding Table 3.4 is shared below for reference:

This information aids in the classification of rock based on the quality, frequency of fractures, and degree of weathering, providing valuable insight into the engineering behavior of the rock-like flysch soils.

Table 3.4 Rock mass proposed classification

FRACTURE FREQUENCY (M)		RQD QUALITY (%)		ROCK WEATHERING RATE (W)	
M1<1	Massive	0-25	Very poor	W1	Fresh-unweathered
M2=1~3	Lightly fractured	25-50	Poor	W2	Slightly weathered
M3=3~10	Fractured	50-75	Fair	W3	Moderately weathered
M4=10~50	Highly fractured	75-90	Good	W4	Higly weathered
M5>50	Disintegrated	90-100	Excellent	W5	Completely weathered

3.4 Strength Properties of Soil Materials.

Based on the drilling, field, and laboratory test results, geological units to be encountered along the tunnel route have been identified. The methodology used to determine Geotechnical design parameters for these units is outlined below in sequence. In these parameter studies, where certain soil parameters haven't been directly determined through laboratory tests, correlations from the literature and results from field experiments have been utilized, along with some indirect laboratory test results. These correlations are provided below in order. Especially, soils are divided into two subgroups: cohesive and non-cohesive soils. The methodology is examined according to these two subgroups. The made ground layer is anticipated to be cohesive soil, while alluvium and weathered flysch layers are considered non-cohesive. The flysch layer exhibiting rock properties has been characterized and strength parameters have been obtained using rock classification.

3.4.1 Soil Unit Weights

Unit weight values for the soil materials were determined based on the relationship between soil density/compactness and natural unit weight, utilizing the table proposed by Carter & Bentley in 1991. The unit weight values for all layers are provided in Table 3.5.

Table 3.5 Bulk density of soil layers

SOIL LAYER THICKNESS	**SOIL TYPE**	**Natural Density**	**Dry Density**
		γ_n (kN/m³)	γ_d (kN/m³)
0.00~1.500	Made Ground	17	16
1.500~12.00	Alluvium	19	18
12.00~23.00	Weathered and Partially Decomposed Flysch	22	21
23.00~	Flysch	23	21

3.4.2 Undrained Shear Strength. (C_u)

In the conducted drilling investigations, the soil profile has been generally assumed to be non-cohesive, while cohesive characteristics have been attributed to the made ground soil. Based on the results of the Menard Pressuremeter test conducted on artificial fill layers, the following conclusions can be drawn:

Net limit pressure P_L is 3 kg/cm².
Menard Modulus E_m is 4 MPa.

The net limit pressure is defined as the maximum effective stress at which soil fails along a critical slip surface or failure plane. It is a measure of the shear strength of the soil and is often used in geotechnical analyses and stability calculations.

P_L (kPa)= 15.214 $N_{1(60)}$ + 89.276; $N_{1(60)}$ = 14 (3.1)

E_m (kPa) =165.88 $N_{1(60)}$ + 1.364,1 $N_{1(60)}$ = 16 (3.2)

An average $N_{1(60)}$ value of 15 has been obtained. Based on this value, an estimated N_{30} value of 29 has been calculated. Using Figure 3.1, an undrained shear strength of 165 kPa has been selected.

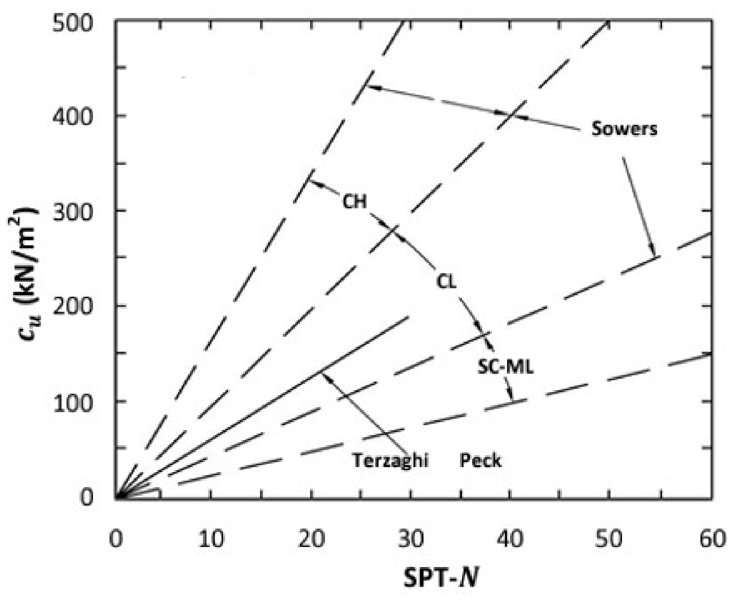


Figure 3.1 Relationship SPT-$\boldsymbol{N}$ – $\boldsymbol{cu}$ (Terzaghi, & Peck, 1967 and Sowers, 1979)

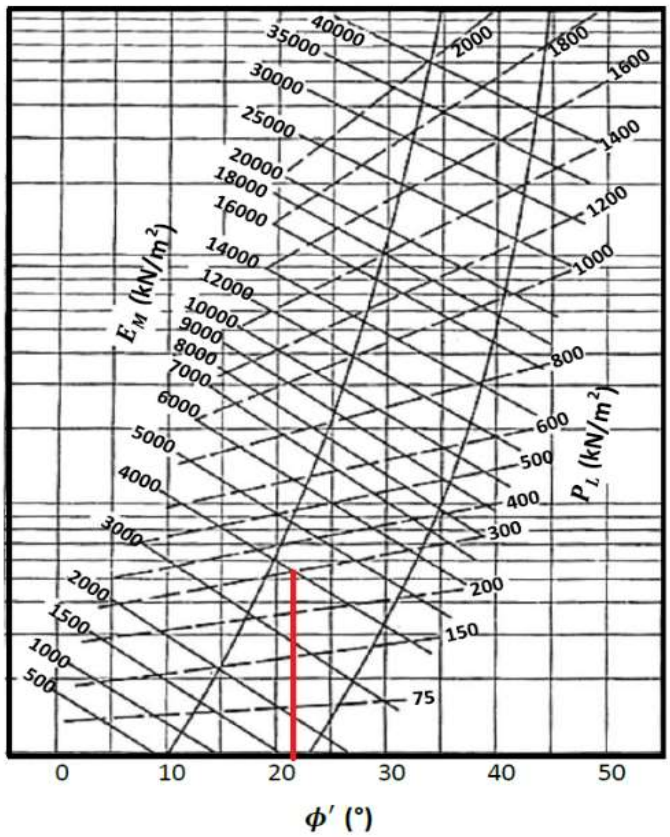
Additionally, the undrained shear strength parameter can be obtained from the following equation proposed by Baguelin (1978).

$$C_u = 0.67 \times (P_L) \times 0.75 \text{ (kPa)}; \quad C_u = 0.67 \times 300 \times 0.75 = 151 \text{ kPa} \qquad (3.3)$$

P_L: Net limit pressure obtained from the pressuremeter test (kPa). Mean C_u value has been taken as 158 kPa.

3.4.3 Effective Shear Strength Parameters (Ø′, c′)

Upon examining the overall soil profile, although a cohesionless structure is observed, the thin material layer containing made ground has been considered cohesive due to its density, and a cohesion value of 5 kPa has been assumed. The ultimate bearing capacity has been determined with a Ø′ value of 22°, utilizing the graph in Figure 3.2 as suggested by Calhoon in 1970, based on the Menard Module.



s

Figure 3.2 Relationship PL-Em - ϕ' (Calhoon, 1970)

When examining the borehole log, layers exhibiting other soil properties were assumed to be cohesionless, excluding the made ground soil layer. According to these assumptions, correlations and graphs recommended for Standard Penetration Test (SPT) values, which are particularly useful in determining various soil strength parameters in cohesionless soils, have been utilized. For cohesive soils, empirical formulas developed using SPT values by numerous researchers have been employed, and in Table 3.6, friction angles for alluvium and weathered and partially decomposed flysch layers are presented.

Table 3.6 Effective friction angle for cohesive soil layers

SOIL PROPERTIES & SPT VALUES					Kulhawy & Mayne, 1990		De Mello, 1971		Bowles, 1996	
SOIL LAYER	**SPT Depth(m)**	**Effective Stress (kPa)**	**SPT-N**	**SPT-N_{60}**	**Ø'**	**Ø'$_{mean}$**	**Ø'**	**Ø'$_{mean}$**	**Ø'**	**Ø'$_{mean}$**
Alluvium GP-GM	3	82.5	28	21	45	42	39	37	38	38
	4.5	111	27	20	43		37		37	
	6	139.5	32	23	43		37		38	
	7.5	168	36	26	43		35		38	
	10.5	225	31	23	39		36		37	
	12	253.5	42	31	41		38		38	
Weathered and Partially Decomposed Flysch GP-GM	24	487.5	41	30	35	35	36	36	37	37

3.4.4 Deformation Modulus (E_s)

The deformation modulus of the made ground layer was determined using the results of the pressuremeter test, and the obtained value is presented in Table-3.2.

In determining the deformation modulus for alluvium and weathered flysch layers, empirical formulas associated with SPT values, which are significantly important for cohesionless soils, and pressuremeter test results were considered together. The obtained results for each soil layer are presented in Table-3.7

Table 3.7 Deformation modulus of alluvium and weathered and partially decomposed flysch layers

DEFORMATION MODULUS		(McGregor, & Duncan,1998)	AASHTO, 2014	FHWA, 2002a	Pressuremeter Test	Mean
Soil Layer	Depth (m)	E_s, MPa	E_s, MPa	E_s, MPa	E_s, MPA	E_s, MPa
Alluvium GP-GM	3	18	21	27	27	21
	4.5	17	18	22		
	6	20	19	22		
	7.5	21	19	23		
	10.5	19	14	24		
	12	24	18	18		
Weathered and Partially Decomposed Flysch GP-GM	24	24	15	23	43	26

3.4.5 Permeability

As a result of the geotechnical investigations, falling head permeability tests were conducted for cohesive soil layers, and constant head permeability tests were conducted for cohesionless soil layers related to the soil strata, and the results are provided in Table-11. Additionally, a Lugeon test was performed for the flysch layer exhibiting rock properties. When referring to the table proposed by Quiñones- Rozo (2010), it can be observed that the alluvium and weathered flysch layers have relatively high permeability, whereas the flysch layer possesses low permeability.

Table 3.8 Permeability coefficient of layers

SOIL	Lugeon Range	Permeability Coefficient K, cm/sec
Alluvium	-	$2x10^{-3}$
Weathered and Partially Decomposed Flysch	-	$2x10^{-3}$
Flysch	2	$1x10^{-5}$~$6x10^{-5}$

Table 3.9 Range of Lugeon values and the corresponding rock condition Quiñones-Rozo (2010)

Lugeon Range	Classification	Hydraulic Conductivity Range (cm/sec)	Condition of Rock Mass Discontinuities	Reporting Precision (Lugeons)
<1	Very Low	$< 1 \times 10^{-5}$	Very tight	<1
1-5	Low	1×10^{-5} - 6×10^{-5}	Tight	± 0
5-15	Moderate	6×10^{-5} - 2×10^{-4}	Few partly open	± 1
15-50	Medium	2×10^{-4} - 6×10^{-4}	Some open	± 5
50-100	High	6×10^{-4} - 1×10^{-3}	Many open	± 10
>100	Very High	$> 1 \times 10^{-3}$	Open closely spaced or voids	>100

3.4.6 Geotechnical Design Parameters for Soils

The soil strength parameters to be utilized in the design for layers exhibiting soil properties have been summarized in the following table, based on various tests conducted both in the field and laboratory.

Table 3.10 Soil strength parameters

SOIL LAYER THICKNESS	SOIL TYPE	Natural Density	Dry Density	Undrained Shear Strength	Effective Cohession Strength	Effective Friction Angle	Poisson Ratio	Deformation Modulus
		γ_n (kN/m³)	γ_d (kN/m³)	C_u, kPa	c' (kPa)	Ø°	-	Es, MPa
0.00~1.500	Made Ground	17	16	158	4	22	0.25	6
1.500~12.00	Alluvium	19	18	-	4	39	0.25	21
12.00~23.00	Weathered and Partially Decomposed Flysch	22	21		-	36	0.30	26
23.00~	Flysch	23	21		62	29	0.30	176

3.5 Strength Properties and Classification of Rock Mass

3.5.1 Definition and Strength Properties of Rock Mass

Rock masses are not composed of continuous, homogeneous, and isotropic materials; instead, they are intersected by various discontinuities. Additionally, they encompass rock types that have undergone varying degrees of weathering. Hence, for modeling their behavior under loads and defining rock masses, it is essential to identify discontinuities and similar geological elements.

Intact rock material, also known as sound rock material, refers to the smallest rock element within which no fractures exist. While microcracks may sometimes be present in rock material, they are not considered as discontinuities or fractures. These rock fragments can range in length from a few millimeters to several meters. In other words, rock fragments of varying sizes that lie between natural discontinuities in rock masses such as joints, bedding planes, schistosity, faults, etc., and do not contain any fractures or weakness planes that could lead to a decrease in tensile strength of the material.

Discontinuities in rock masses that lack tensile strength or have very low tensile strength, encompassing geological weakness planes like bedding planes, joints, faults, shear zones, are referred to as structural discontinuities. The concept of a rock mass (rock mass concept) is the combination of the discontinuity network and the rock material, forming a mass or system together.

The strength of a rock material is the resistance it exhibits against the applied stress. When the stress to which the rock is subjected exceeds its strength, failure will occur in the rock. Some values required for the characterization of rock masses are obtained through field and laboratory studies. Several significant experiments are explained below.

3.5.1.1 Uniaxial Compressive Strength

The value at which a material (such as rock) yields under the influence of uniaxial compressive stress is known as Uniaxial Compressive Strength (UCS or σ_{ci}). It serves as a fundamental mechanical property of intact rock material and is also used as a key input parameter in rock mass classification systems. It is determined through uniaxial loading conditions in a laboratory, where a cylindrical core sample is subjected to compression until failure. For the cylindrical core sample obtained from drilling, in the uniaxial compression test conducted for the flysch layer exhibiting rock properties, a UCS value of 29.4 MPa was obtained.

3.5.1.2 Tensile Strength

The point load test is conducted to determine the point load strength index, which is used in the classification of rocks based on their strength. The point load strength index is employed to indirectly determine other strength parameters such as uniaxial compressive and tensile strengths. Based on the test results, the rock's "Point Load Strength Index" and the "Strength Anisotropy Index" are determined (Topal, 2000). Tensile strength σ_t is the stress value at which a rock fails under the influence of uniaxial tensile stress.

3.5.1.3 Shear Strength

The stress acting on the failure plane at the moment of failure in a rock material subjected to stress is known as the failure stress.

3.5.1.5 Point Load Test

The point load test is conducted by applying a load to a specific point on a rock sample, usually utilizing a diametric pair. This load focuses pressure onto the sample, exerting a compressive force. The results of the point load test are typically represented by a value known as the "point load index" (PLI). This value reflects the strength of the tested rock specimen. The PLI value can serve as a basis for engineering analyses, including structural calculations and stability assessments. For the cylindrical core sample obtained from drilling and exhibiting rock properties in the flysch layer, point load tests were conducted, yielding three different values. The average value obtained was 2.9 MPa.

The point load tests, and uniaxial compressive test results conducted on the core samples obtained from the drilling work are presented in table 3.11. Furthermore, Bieniawski (1975) proposed an approach regarding the strength of rocks based on the point load index and UCS values, and this approach is shown in Table 3.12.

Table 3.11 UCS and point load test results for flysch soil layer

Depth(m)	Point Load Is(50) (kg/cm²)	The Uniaxial Compression Strength (kg/cm²)
28.50-30.00		294.30
34.50-36.00	11.70	
37.50-39.00	45.20	
43.50-45.00	28.90	

Table 3.12 Strength classification for rock materials (Bieniawski,1975)

Strength classification for rock materials

Description	Uniaxial compressive strength (MPa)	Point-load index (MPa)
Very high strength	> 200	> 8
High strength	100–200	4–8
Medium strength	50–100	2–4
Low strength	25–50	1–2
Very low strength	< 25	< 1

3.5.2. Classification of Rock Mass

The mechanical properties of rocks play a crucial role in determining the necessary support and reinforcement measures for the safe and stable excavation of tunnels. Rock strength, deformation characteristics, and fracture behavior guide the selection of appropriate support methods and the design of reinforcement systems used to ensure tunnel stability. Additionally, the mechanical properties of the rock affect the speed, cost, and difficulty of tunnel excavation. Hard rocks with high strength are generally excavated more quickly and easily, whereas low-strength rock types and brittle rocks may require more effort and time. Therefore, accurately determining the mechanical properties of the rock is of paramount importance, both during the design phase and throughout the excavation process.

In cases where there is limited information about the rock mass, rock mass classification systems are utilized during feasibility and design stages of underground excavations to conduct preliminary assessments of support systems. Additionally, these systems can be used as input parameters in empirical failure criteria recommended for rock masses, such as the Hoek-Brown failure criterion. At an international level, there is no comprehensive standard published for determining the required rock parameters. Designs in rock units still rely on empirical evaluations of rock masses guided by experience, the use of rock mass structure and index properties, as well as correlations based on parameters and index properties such as joint spacing, roughness, and RQD (Rock Quality Designation).

In contemporary times, the determination of rock parameters commonly relies on the Hoek-Brown empirical failure criteria. This approach, recommended by Hoek, integrates the RMR (Rock Mass Rating) system and Q (Rock Mass Quality) classification, incorporating GSI (Geological Strength Index) which is essentially related to RMR and Q, to establish rock mass constants. Within the scope of this thesis, RMR and GSI classifications have been applied, and strength parameters of the rock unit have been obtained.

3.5.2.1 RMR Classification

The RMR classification system was first proposed by Bieniawski in 1973. The RMR score, with multiple inputs as shown in the table below, is a highly effective and widely accepted method for characterizing complex rock structures. Based on field studies and examination of borehole logs, the point load index Is50 has been observed in the range of 11.70 to 45.20 kg/cm², with an average value of 25.75 kg/cm². The uniaxial compressive strength σ_{ci} was measured as 294.30 kg/cm². Upon reviewing the core values, it's evident that the core recovery percentage is quite low, and an average RQD (%) of 6 has been considered. Upon further examination of the borehole log, it is noticeable that weathering and discontinuities are quite abundant, indicating a weak rock feature. With this information, the RMR value is calculated as follows.

Table 3.13 Calculation based on the RMR system

	Calculation Based on the RMR System	Value	Raiting
1	Unconfined compression strength UCS=18~41 mPa	14~18 MPa	4
2	RQD= %6~13	%6~13	3
3	Joint spacing 200 mm~600 mm	200m~600 mm	10
4	Joint conditions	Slickensidd surfaces	10
5	Groundwater condition	Dripping	4
B	Joint orientations	Fair	-5
		TOTAL RMR	**26**

Table 3.14 Determination of the RMR value

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	2% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensidd surfaces or Gouge 5 mm thick or Separaton 1 -5 mm Continuou	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)									
Strike and dip orientations			Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable		
Ratings		Tunnels & mines	0	-2	-5	-10	-12		
		Foundations	0	-2	-7	-15	-25		
		Slopes	0	-5	-25	-50			

3.5.2.3 GSI Rock Mass Classification

Rock classification involves the process of identifying and categorizing different types of rocks based on specific characteristics. This process plays a significant role in engineering projects, mining, construction, geological research, and other related fields. One such classification, GSI (Geological Strength Index), is a system that considers factors such as the structural and durability properties of a rock mass. These factors include rock type, cracks and joints within structures, permeability, degree of weathering, deformation potential, and other geological features. GSI combines these characteristics to classify the rock mass's level of strength and determines parameters used in engineering designs. Many researchers contribute to this classification system, offering improved tables based on their work. In this thesis, the study conducted by Sonmez, & Ulusay, (1999), and their modified graph, were utilized. Through the combined evaluation of borehole logs and test results, a GSI value of 20 was obtained.

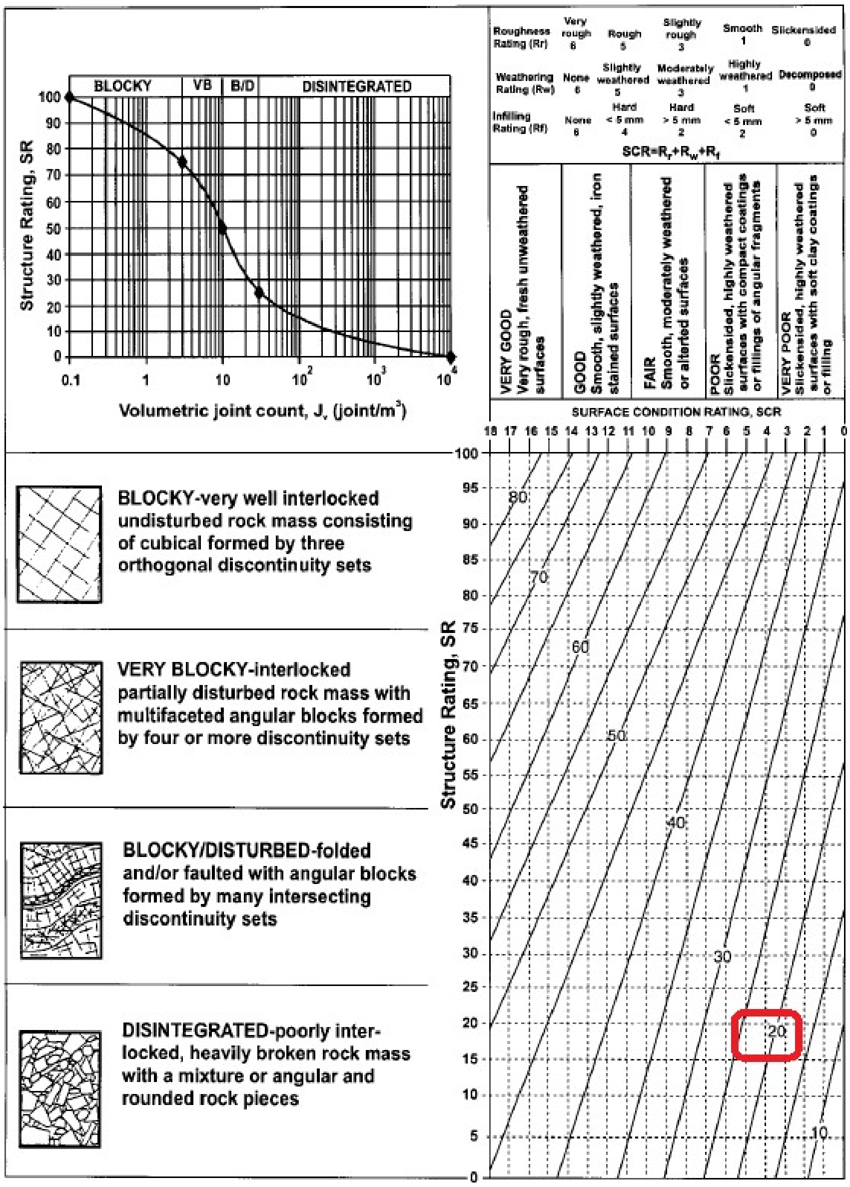


Figure 3.3 The modified GSI classification (Sonmez, & Ulusay, 1999)

3.5.3 Geotechnical Design Parameters for Rock

3.5.3.1 Deformation Modulus

Particularly in efforts aimed at predicting the compression, stress, and displacement responses of rock masses, the accurate determination of the elasticity modulus stands out as a primary concern. Due to the limited nature of data acquired from field studies, empirical methods will be utilized to compare these data, and the most suitable value for the elasticity modulus will be selected. Within the scope of this thesis, rock strength properties were obtained through empirical equations derived from GSI and RMR classifications. These obtained values were compared with those obtained in both field and laboratory settings to arrive at average rock strength properties.

Hoek and Brown (1980) and Hoek (1983) examined published information regarding the strength of intact rock and proposed empirical failure criterion equations for rocks. In the equations they suggested, it was observed that the Geological Strength Index (GSI) value plays a significant role in determining the engineering properties of rocks. Hoek and Brown determined through their studies that there is a curvilinear relationship between the maximum principal stress (σ_1) and the minimum principal stress (σ_3), or in other words, between normal stress and shear strength, and introduced the failure criterion for intact rocks as the 'Hoek and Brown Failure Criterion,' represented by a square root parabola. This contribution has been recognized in the technical literature (Hoek, Carranza, Torres, & Corkum, 2002).

Some of the formulas proposed by various researchers for determining the elasticity modulus using the RMR value are shown in Table 3.15. It has been observed in studies conducted by Kıncal, & Koca, (2019) and Hoek, & Diederichs (2005) that empirical formulas utilizing the RMR parameter tend to yield overestimated results and should be used cautiously in engineering calculations.

Table 3.15 Proposed by various researchers to estimate of the deformation modulus (EM) based on the RMR (Kıncal, & Koca,2019)

$E_m = 10^{(RMR - 10)/40}$ for RMR < 50 (GPa) (Serafim and Pereira 1983)
$E_m = 300 \times e^{(0.07 \times RMR)} \times 10-3$ (GPa) (Kim 1993)
$E_m = 0.0097 \times RMR^{3.54}$ (MPa) (Aydan and Kawamoto, 2000)
$E_m = 0.0736 \times e^{(0.0755 \times RMR)}$ (GPa) (Gökçeoglu et al. 2003)
$E_m = Ei \times e^{((RMR-100)/36)}$ (Gpa) (Galera et al. 2005)
$E_m = 0.3228 \times e^{(0.0485 \times RMR)}$ (GPa) Chun et al. 2006
$E_m = 10^{(RMR - 16)/50} \cong 10^{(0.019 \times RMR - 0.314)}$ (GPa) (Kang et al. 2013)

Hoek & Diederichs:

$$E_m = E_i\left(0.02 + \frac{1-\frac{D}{2}}{1+e^{\left(\frac{60+15D-GSI}{11}\right)}}\right) \tag{3.4}$$

Simplified Hoek & Diederichs:

$$E_m = 100000\left(\frac{1-\frac{D}{2}}{1+e^{\left(\frac{75+25D-GSI}{11}\right)}}\right) \tag{3.5}$$

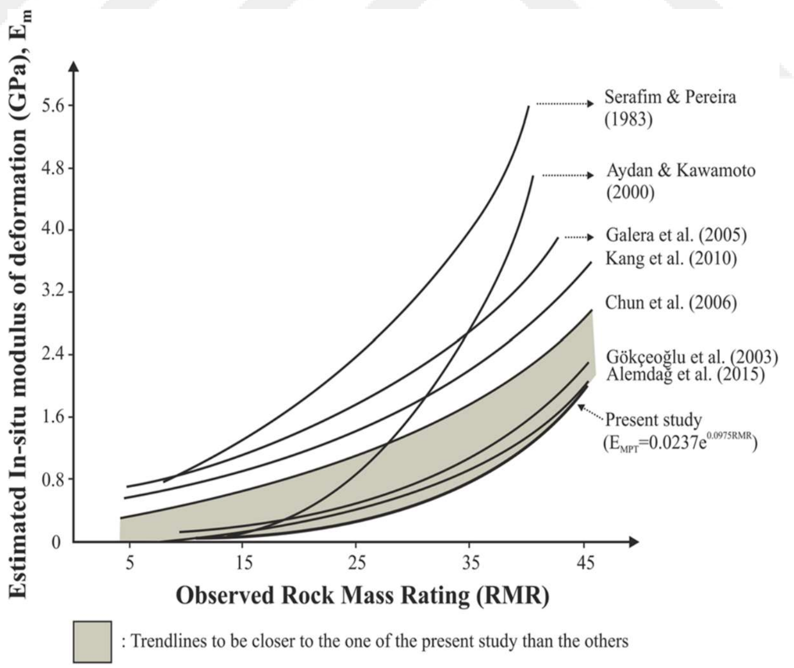


Figure 3.4 Estimated in situ modulus of deformation (E_M) values from the observed RMR values according to various researchers. (Kıncal, & Koca,2019)

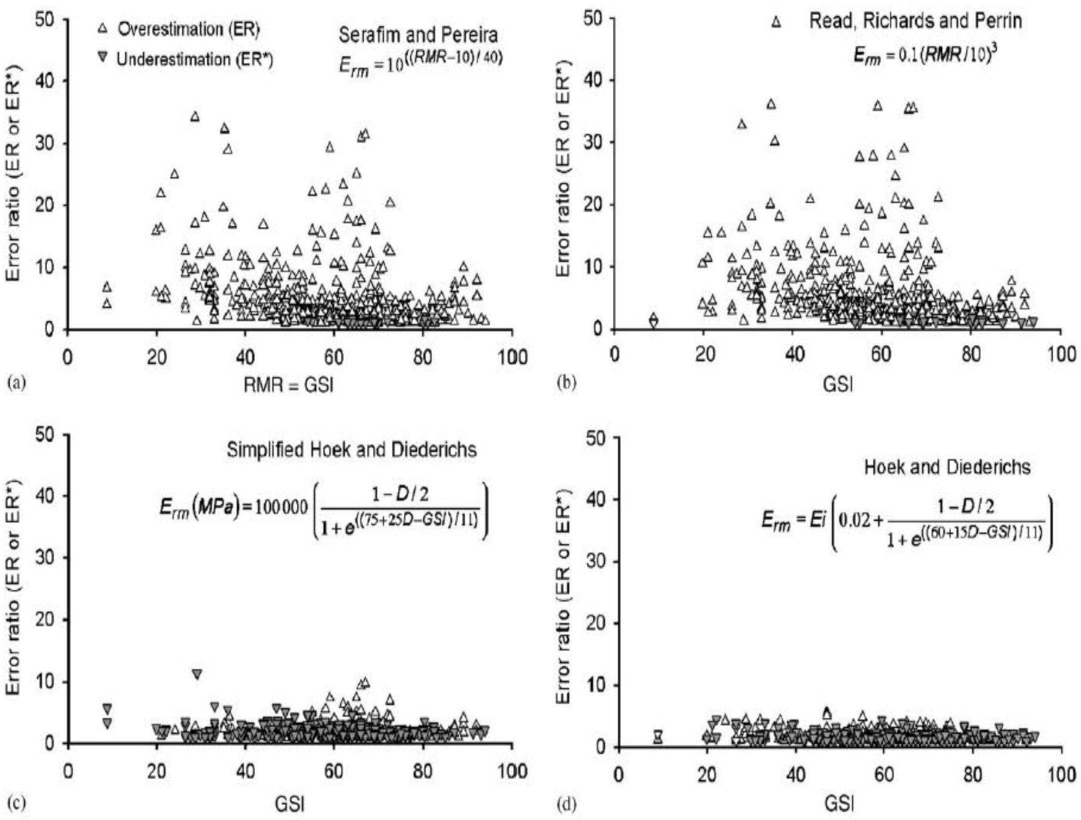


Figure 3.5 Comparison of prediction errors D=0.5 (Hoek and Diederichs, 2005)

In figure 3.5, this comparison is based upon an error ratio, ER, for overestimated values and ER* for underestimated values defined as

$$ER = \frac{\text{Calculated E}_{\text{rm}}}{Measured \text{ E}_{\text{rm}}} \tag{3.6}$$

$$ER^{*} = \frac{\text{Calculated E}_{\text{rm}}}{Measured \text{ E}_{\text{rm}}} \tag{3.7}$$

When determining the intact rock modulus value, some of the empirical formulas defined in the literature were utilized, and the results obtained are presented in the table below. Three result values were averaged, and for the flysch layer, the average intact rock modulus value was determined as 5.1 GPa.

Table 3.16 Determination of intact rock modulus

		INTACK ROCK MODULUS E_i (GPa)			
Point Load	**The Uniaxial Compression Strength**	**Ocak, İ. (2008)**	**Teymen, A.**	**Kıncal, C., & Koca, M. Y. (2019)**	**Mean**
$Is_{(50)}$,(kg/cm²)	UCS, (kg/cm²)	$E_i = 0.4.UCS^{0.854}$	$E_i = 1.403 I_{s(50)}^{1.219}$	$E_i = 157.4\sigma_{ci}$	E_i
29	294.3	7.2	5.1	4.6	5.1

3.5.3.2 Effective Shear Strength Parameters (Ø′, c′)

The determination of effective cohesion and internal friction angle is based on the Hoek-Brown Failure Criterion equations and the results of triaxial compression tests conducted on flysch soil. The strength parameters for the flysch units that can be selected depending on tunnel depth are obtained. The strength parameters for selecting the support units according to the Hoek-Brown failure criterion for tunnel depth, with GSI (Geological Strength Index) 20, σ_{ci} (uniaxial compressive strength of intact rock) 29.4 MPa, E_i (Young's modulus of intact rock) 5.1 GPa, H (tunnel depth) 30 m, and m_i (material constant) 4, are in Figure 3.6.

The effective shear strength angle Ø' was determined as 32^{o}, which was found to be consistent with the results of triaxial compression tests conducted on the flysch soil, as shown in Table 3.17.

Table 3.17 Triaxial test results of flysch layer

Triaxial Test		
SOIL	**c (kg/cm²)**	**Ø°**
Flysch	5.57	29

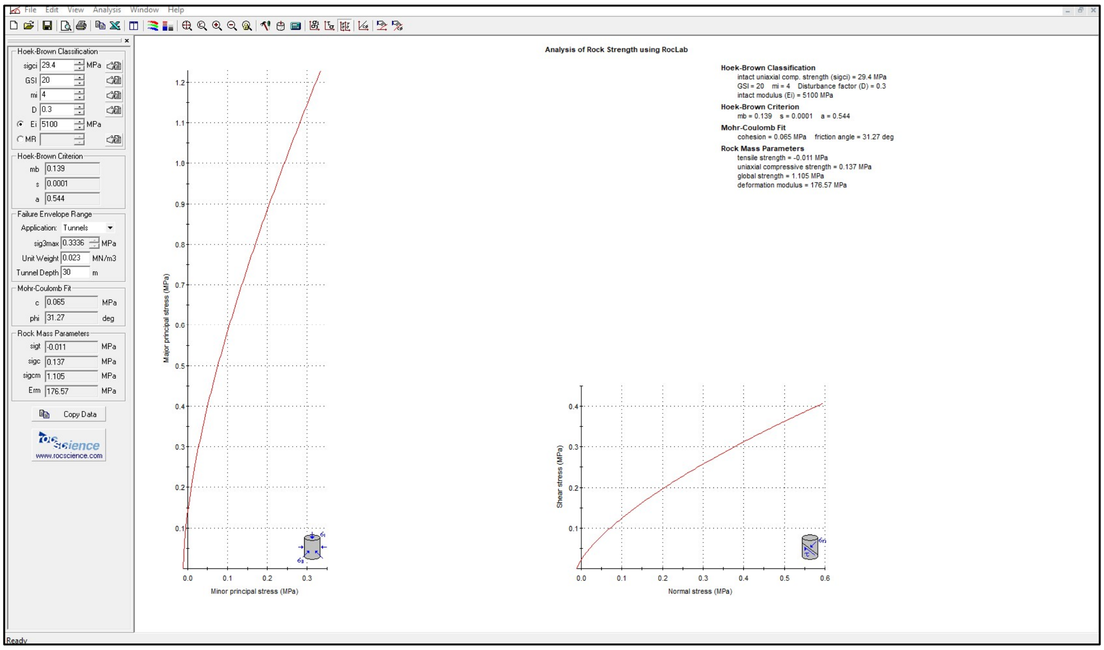


Figure 3.6 Elastic modulus and effective shear strength parameters of the flysch soil

Table 3.18 Strength properties of soil layers

SOIL LAYER THICKNESS	SOIL TYPE	Natural Density	Dry Density	Undrained Shear Strength	Effective Cohession Strength	Effective Friction Angle	Poisson Ratio	Deformation Modulus
		γ_n (kN/m³)	γ_d (kN/m³)	C_u, kPa	c' (kPa)	Ø°	ν	Es, MPa
0.00~1.500	Made Ground	17	16	158	4	22	0.25	6
1.500~12.00	Alluvium	19	18	-	4	39	0.25	21
12.00~23.00	Weathered and Partially Decomposed Flysch	22	21		-	36	0.30	26
23.00~	Flysch	23	21		62	29	0.30	176

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CHAPTER FOUR

2D NUMERICAL MODELING AND ANALYSIS OF THE TUNNEL

In this section, the excavation of a large-diameter tunnel exhibiting shallow tunnel characteristics using the NATM (New Austrian Tunneling Method) will be discussed in relation to the comparison of surface deformations through empirical formulas and numerical analyses. For numerical analyses, 2D models were established using the Plaxis 2D finite element software.

4.1 Two-Dimensional Numerical Analyses Tunnel Construction Model

A rock-soil model was created using the finite element method with Plaxis2D software. When forming the soil-rock, it is recommended to consider distances of 2D ~ 3D from the tunnel axis to the base and 4D ~ 5D from the tunnel axis to the vertical boundaries, based on the tunnel diameter (D) (Meissner, 1996; Möller, 2006). In this model, considering an overlying layer thickness of 30 m, vertical dimensions of 80 m and horizontal dimensions of 150 m were considered.

Table 4.1 Tunnel support parameters used in 2D numerical analyses.

Support Type	**E (MPa)**	**Poisson Ratio, n**	**Compressive Strength (MPa)**	**Tensile Strength (MPa)**	**Dimensions**
Rock bolt	200 000	0.3	420	420	Ø26/Ø32 mm, L=5 m & 9 m
Umbrella Pipe	200 000	0.3	20 cm spacing, 114 mm with 10 mm thickness, 9.0 m length, application angle is 10°		
Steel Ribs	200 000	0.3	355	355	IPN220
Shotcrete	22 000	0.2	30	2.35	35 cm
Wire Mesh	200 000	0.3	500	500	Q335/335

The composite elements formed by rock bolts and steel pipes, along with the grout material, have been modeled as embedded beams in the 2D numerical analysis model.

The excavation stages and structural support elements of the tunnel excavation support system are illustrated in Figure 4.1.

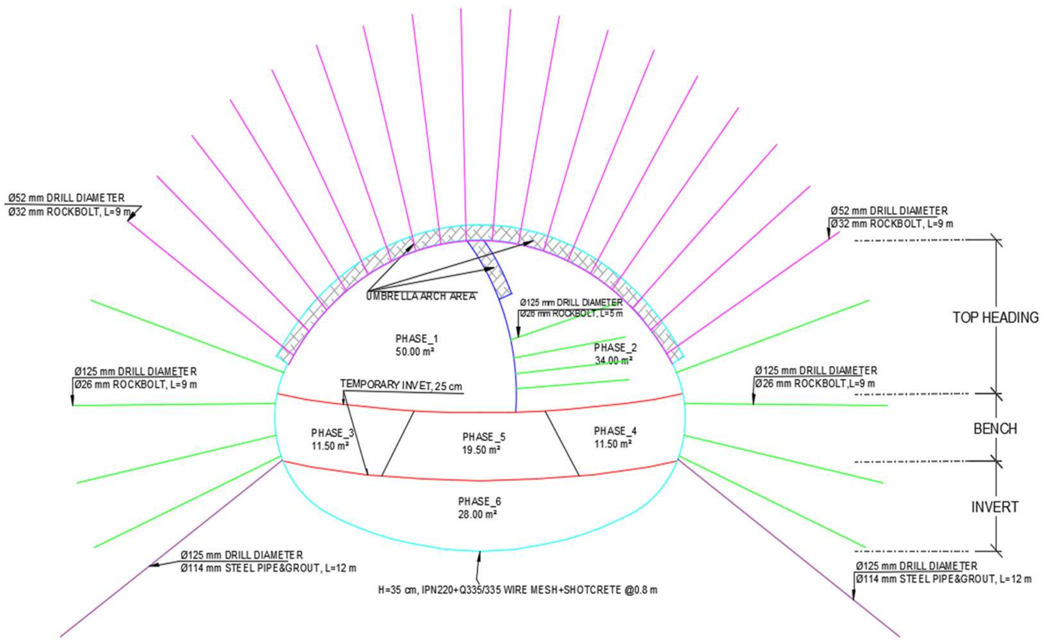

Figure 4.1 Forepoling modeling of the tunnel section

The analysis model created with Plaxis 2D finite element software is shown in Figure 4.2.

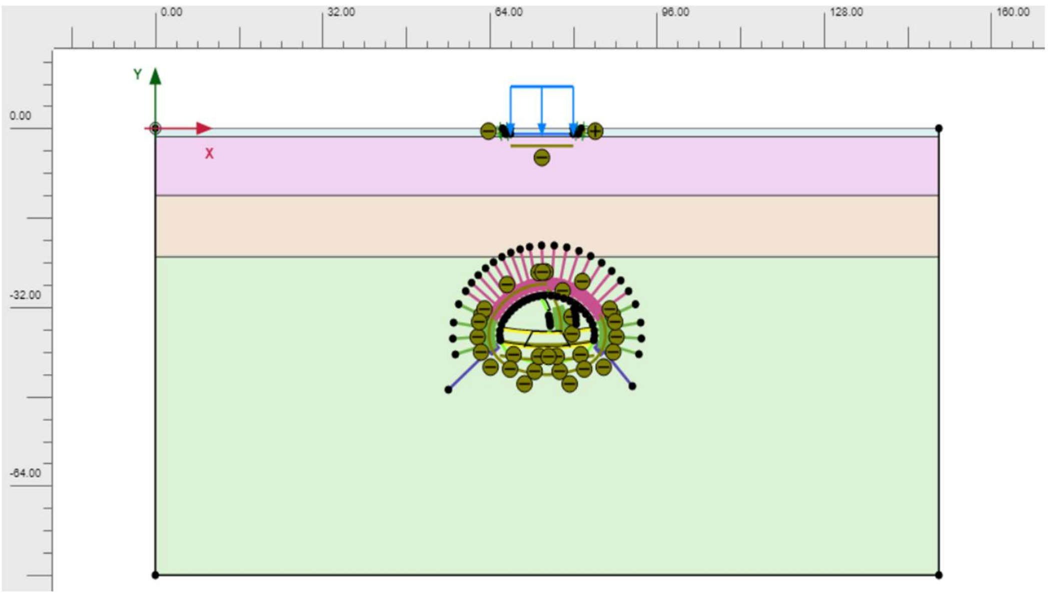

Figure 4.2 Boundary condition of the tunnel fort this study

One of the methods used in two-dimensional analyses for simulating the placement of support systems and the distance of the excavation section to the tunnel face is the "equivalent excavation forces." These "equivalent forces" are the forces that arise due to the lithostatic load applied to the original state of the tunnel excavation surroundings. Defining the release law for the tunnel core is required to determine the "face effect" in the calculations. This release law should be capable of simulating a decrease in the pressure on the support system with increasing deformations of the tunnel cavity. This concept is analytically expressed by the curve $1-\lambda = \sigma_r/\sigma_0$ (Panet). With the help of this curve, the transition from three-dimensional behavior in terms of radial forces to planar behavior is represented in terms of radial forces.

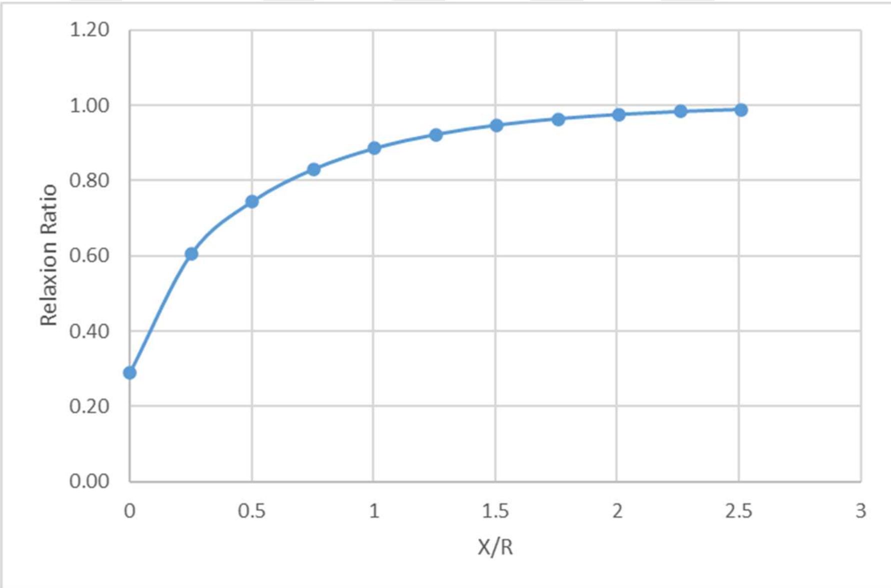

Figure 4.3 Stress reduction ratio

Within the scope of this thesis, there is a three-story building located over the tunnel in the analyzed area. The relevant building foundation and retaining walls have been modeled using plate elements. To simulate the building load, a uniform distributed load of 15 kPa has been assumed for each floor, resulting in a total building load of 45 kPa on the foundation. In 2D analyses, the umbrella zone has been modeled using the method known in the literature as the equivalent beam method. The strength parameters obtained for the equivalent beam zone are presented in the table below, and this zone is modeled using the Mohr-Coulomb soil model.

Table 4.2 Properties of the equivalent beam soil layer

	Section Properties of Component						**Area**	**Elastic Modulus**	**Product**
Support Type	Height (m)	Lentgh (m)	Outside Diamater (m)	Thickness (m)	Inside Diamater (m)	Pcs/per meter	m^2	MPa	-
Rock	0.60	1.00					0.58	176.00	102.72
Grout	0.60	1.00					0.03	30000.00	1040.97
Steel Pipe (Umbrella Arch)	-	-	0.114	0.010	0.09400	5	0.02	200000.00	3267.26
Sum							0.63		4410.95
Equivalent Deformation Modulus	4410.95								
kPa	4410947.90								

CHAPTER FIVE
RESULTS AND DISCUSSIONS

Vertical settlement values of the structure over the tunnel excavated using the NATM technique have been compared using the 2D finite element method, empirical approaches, and site measurements.

5.1 2D Finite Element Method

The tunnel excavation has been completed in six stages using Plaxis 2D software. The analysis results indicate that a vertical settlement of 7.55 cm in Figure 5.1 is anticipated at the surface in the current state of the building foundation located at the center of the tunnel.

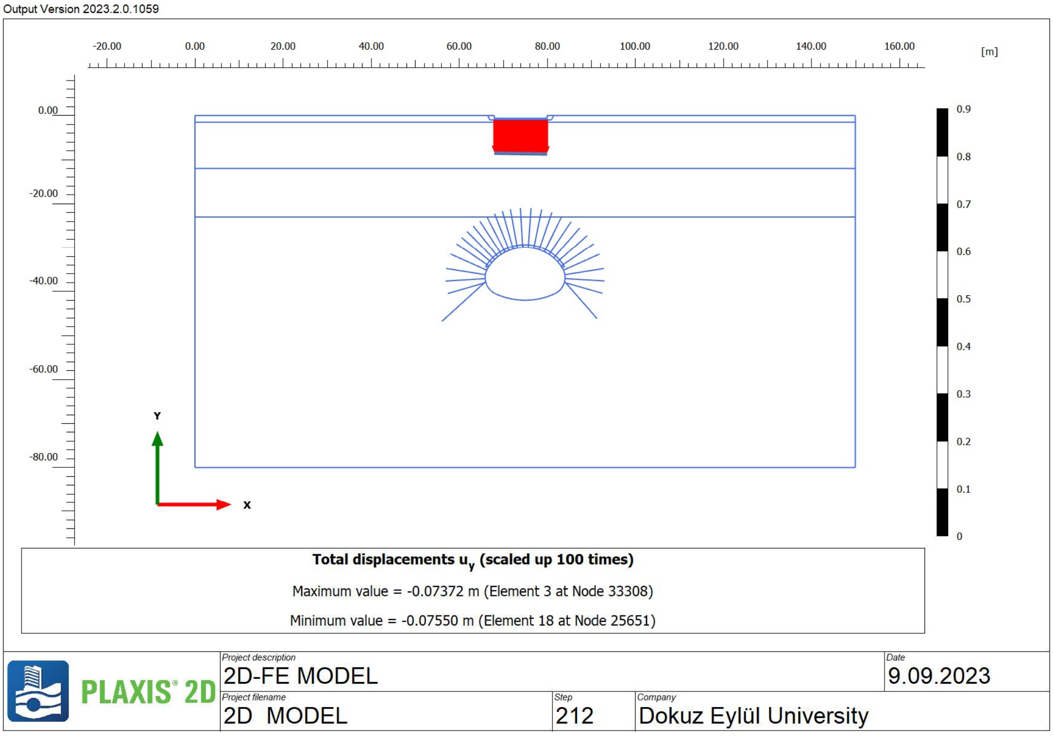

Figure 5.1 Vertical displacement of existing model

5.2 Empirical Methods

Empirical methods, 2D finite element Analyses, and site measurements have been compared in Figure 5.3 for surface settlement values. In this comparison, it is observed that the values obtained using the 2D finite element method are closer to the values obtained through empirical methods. However, it has been observed that the actual surface settlement values obtained from the site are higher, and this difference is attributed to factors such as the sensitivity exhibited during excavation and a decrease in the groundwater table level, as mentioned in Section 2. Additionally, in all three comparative cases, it has been observed that the expected settlements at the surface are high. Within the scope of this thesis, a method has been developed to reduce surface settlements.

Estimated settlement values obtained through empirical formulas proposed by various researchers are provided in Table 5.2, 5.3 and 5.4. In determining the inflection point, a critical input for estimating surface settlement values, numerous studies have been conducted. An average inflection point value of 15.85 meters, obtained by referencing some of these studies, is presented in Table 5.1.

Table 5.1 Inflection point calculations suggested by various researchers.

The Height Between the Surface and the Center of the Tunnel	The Equivalent Tunnel Diameter	The Distance Between the Top of the Tunnel and the Surface	Peck (1964)	Attewel and Farmer (1974)	Attewel (1977)	Atkinson and Potts (1977)[1]	Atkinson and Potts (1977)[2]	Clough & Schimdt (1981)	O'Reilly and New (1982)	Mair vd. (1993)	Arığolu (1992)	**MEAN**
H (m)	D (m)	C (m)	i (m)	i (m)	i (m)	i (m)	i (m)	i (m)	i (m)	i (m)	i (m)	**i (m)**
35	15	30	14.77	17.5	17.5	15	11.25	14.77	18.6	17.5	15.72	**15.85**

Table 5.2 Surface deformations calculated according to the empirical equation (Arıoğlu, Yüksel, & Arıoğlu,2002)

ARIOĞLU, B., YÜKSEL, A., & ARIOĞLU, E. (2002)		**UNIT**	**VALUE**
Ps	TRAFFIC&SURCHAGE LOAD	kN/m²	45
i	INFLECTION POINT	m	15.85
D	TUNNEL DIAMETER	m	15
Z_0	DISTANCE OF BETWWEN TUNNEL CENTER AND SURFACE	m	35
$\Upsilon_{avg.}$	EQUIVALENT UNIT WEIGHT OF SOIL LAYERS	kN/m³	20.708
$E_{avg.}$	EQUIVALENT ELASTIC MODULUS OF SOIL LAYERS	kN/m²	102406.3
υ	EQUIVALENT POISSON RATIO OF SOIL LAYERS	-	0.276
S_{max}	MAXIMUM SURFACE SETTLEMENT	mm	**85.5329**

Table 5.3 Surface deformations calculated according to the empirical equation Herzog (1985)

	HERZOG (1985)	**UNIT**	**VALUE**
P_s	TRAFFIC&SURCHAGE LOAD	ton/m²	4.5
i	INFLECTION POINT	m	15.85
D	TUNNEL DIAMETER	m	15
Z_0	DISTANCE OF BETWEEN TUNNEL CENTER AND SURFACE	m	35
$\Upsilon_{avg.}$	EQUIVALENT UNIT WEIGHT OF SOIL LAYERS	ton/m³	2.071
$E_{avg.}$	EQUIVALENT ELASTIC MODULUS OF SOIL LAYERS	ton/m²	10240.63
υ	EQUIVALENT POISSON RATIO OF SOIL LAYERS	-	0.276
S_{max}	MAXIMUM SURFACE SETTLEMENT	mm	83.78737

Table 5. 4 Surface deformations calculated according to the empirical equation Peck (1969)

	PECK(1969)	**UNIT**	**VALUE**
i	INFLECTION POINT	m	15.85
V_L	VOLUME LOSS	%	0.02
D	TUNNEL DIAMETER	m	15
Z_0	DISTANCE OF BETWWEN TUNNEL CENTER AND SURFACE	m	35
S_{max}	MAXIMUM SURFACE SETTLEMENT	mm	88.97

5.3 Site Measurements

As a result of daily measurements conducted parallel to the excavation, the vertical settlement amount measured in the area where the 6-stage full-section excavation was completed was found to be 165 mm at the tunnel center. The settlement amounts measured in the excavations in the site being greater than the values obtained through 2D finite element analysis and empirical methods can be attributed to various factors, as explained in Section 2, such as a decrease in the groundwater level accompanying the excavation and the possibility that the strength parameters of the soil profile where the tunnel is excavated may be lower than the values obtained from drill logs.

In the relevant tunnel excavation support system, economical and commonly used structural elements have been employed, and further improvements beyond this stage would require both expensive and more specific measures. Within the scope of this thesis, a method has been developed that differs from existing improvements, and the expected deformations on the structure have been examined using this developed method.

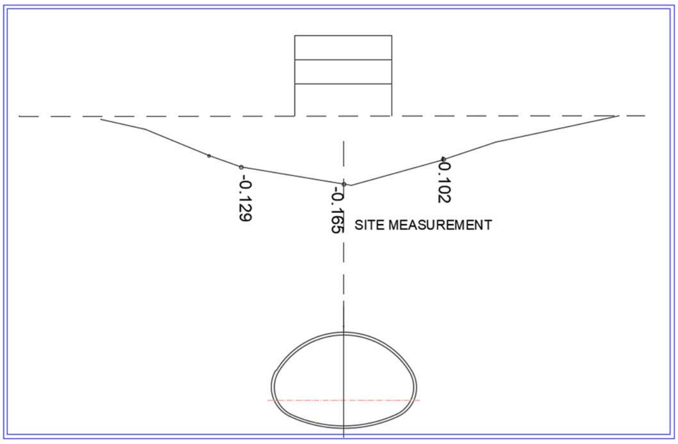

Figure 5.2 Vertical displacement measured in site

The values obtained through the 2D finite element method, empirical formulas, and site measurements as mentioned above have been compared and are presented in Figure 5.3.

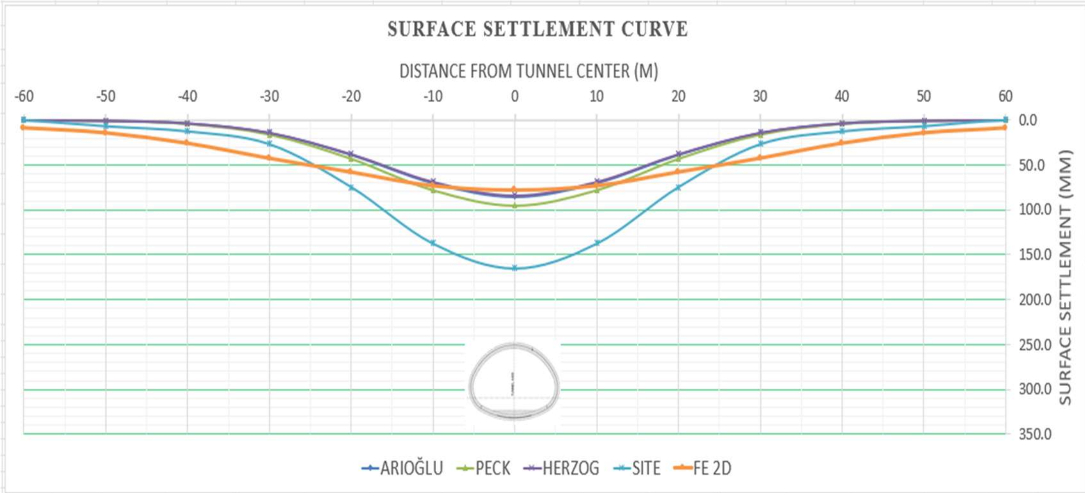

Figure 5.3 Comparative surface deformation curves

5.4 The Method Developed for Reducing Surface Settlements

In the developed method, instead of making improvements to the tunnel and foundation soil, which are often preferred but whose net effect cannot be fully determined to reduce surface settlements, the principle is to create a frame system by passing through regions where the displacements caused by tunnel excavation are zeroed or minimized. The aim is to minimize the impact of soil movement on the frame system, and after the interaction between the structure and the soil, transfer the loads generated to this frame system in order to limit displacements in the structures.

In summary, the developed method is a ground improvement or soil reinforcement approach that differs from the conventional techniques used in the current practice. Instead of making improvements to soil parameters as mentioned in the conventional techniques to control surface settlements, it focuses on a more controllable and targeted enhancement method. A key element of the developed method is the horizontal reinforcement elements, which are placed below the building foundation by horizontal drilling in a manner that is anchored outside the expected surface settlement trough's turning point, depending on the tunnel diameter and depth. These elements, in a sense, act as rigid beams. These horizontal beams will be fully anchored with piles acting as columns, thus forming a frame system beneath the foundation. By utilizing this frame system, the structure will be minimally affected by deformations occurring in the soil.

5.4.1 Description of the Developed Models

Before commencing tunnel excavation with structural elements in place, vertical structural elements, namely piles, are initially constructed at a sufficient distance from the tunnel inflection point distance. Subsequently, as shown in the visual below, an excavation is created outside the building to allow for the installation of the horizontal beam element that will pass beneath the building foundation. The relevant system is established to ensure that it does not cause settlements in the building foundation. Steel pipes of the required diameter and length will be installed to create the necessary horizontal beam element. Following this, within the steel pipes, a composite system, characterized by high resistance to bending and formed by concrete and steel profiles, or a system taking advantage of the inability to compress liquids, will be created by filling the pipes with the desired liquid at the desired pressure.

Once the installation of the horizontal beam elements is completed, a frame system will be established in full anchorage with the vertical elements, namely piles. Depending on the requirements, the first method involves creating a system with only pile vertical elements and beam horizontal elements, as depicted in Figure 5.5. In the second method, in addition to the first method, vertical structural curtain elements will be added. These will be provided with the necessary number and length of anchoring cables, applying the required pre-tension load. The goal of the second method is to limit the vertical displacements expected to occur with tunnel excavation by creating pressure and reverse moments in the horizontal beam elements. It has been observed that an improvement has been achieved in vertical displacements at the level of approximately 36-43%.

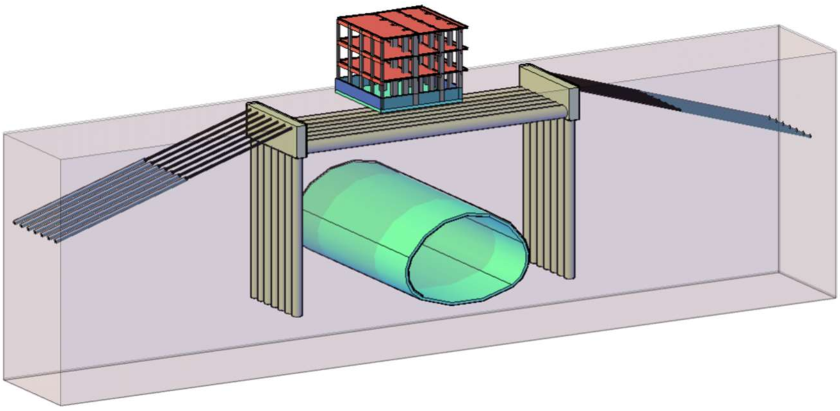

Figure 5.4 Model-1 3D view

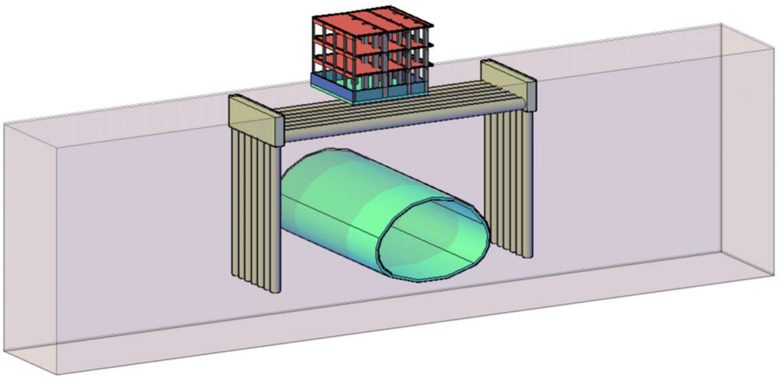

Figure 5.5 Model-2 3D view

5.4.2 Finite Element Analyses of the Developed Model

The diameter, spacing, length, and strength parameters of the horizontal beam elements and vertical pile elements used in the developed method are shown in Table 5.5. Information regarding the tendons and grout zones for the anchored system, which is the second alternative in the developed method, is presented in Table 5.6.

Table 5.5 Properties of structural elements in the developed models

PROPERTIES	Diameter	L_{space}	Length	Area	Inertia	Elastic Modulus	EA	EI
Unit	m	m	m	m^2	m^4	kPa	kN	kNm²
Circular Beam	1.8	2.5	30	2.54	0.52	3.70E+07	3.77E+07	7.63E+06
Pile	1.2	2.5	55	1.13	0.10	3.70E+07	4.18E+07	3.77E+06

Table 5.6 Properties of anchor system in the developed models

	Diameter	Length	L_{space}	Number of Tendon	Tendon Specified Strength	Elastic Modulus	Soil	Section Area	Friction Resistance (FHWA)	Safety Factor	T_{design}	EA
	m	m	m	pcs	kN	kPa	-	m²	kPa	-	kN	kN/m
Grout Body	0.2	12	1.2	-		20000	Cohessionless (Sitly sands)	0.0314	290	2	1093	
Anchor	0.018	30	1.2	4	380	2E+08	Cohessionless (Sitly sands)	0.0003	-	1.6	950	127235

5.4.2.1 Analysis Results of the Developed Model

When examining the analysis results for Model-1 and Model-2, it is observed that the anchored system is more effective compared to the non-anchored system. The preferred methods should be evaluated based on site-specific conditions, project budget, and time considerations to choose the most optimal option. Within the scope of this thesis, various alternatives such as pile length, horizontal beam length, pile and horizontal beam diameter, anchor angles, and root zone length have been evaluated, and the most optimal results have been demonstrated in this study. Depending on preferences, adjustments can be made to these variables to implement different choices. When the analysis results are evaluated, in the anchored system, a maximum vertical settlement of 43.2 mm is expected in the building foundation, while in the non-anchored system, it is expected to be 48.4 mm. Currently, this value has been measured at 75.5 mm.

Table 5.7 Comparison of the total vertical displacement values

TOTAL VERTICAL DISPLACEMENT (mm)			IMPROVEMENT (%)	
Existing Model	Model-1	Model-2	Model-1	Model-2
75.5	43.2	48.4	43%	36%

Table 5.8 Structural element in model 1 & 2

PROPERTIES	Diameter	L_{space}	Length	Number of Tendon	Angle of Anchor	Pre-Tension	Model
Unit	m	m	m		°	kN	
Circular Beam	1.8	2.5	30				Model-1&2
Pile	1.2	2.5	55				Model-1&2
Grout Body	0.2	1.2	12	-	15		Model-1
Anchor	0.018	1.2	30	4	15	650	Model-1

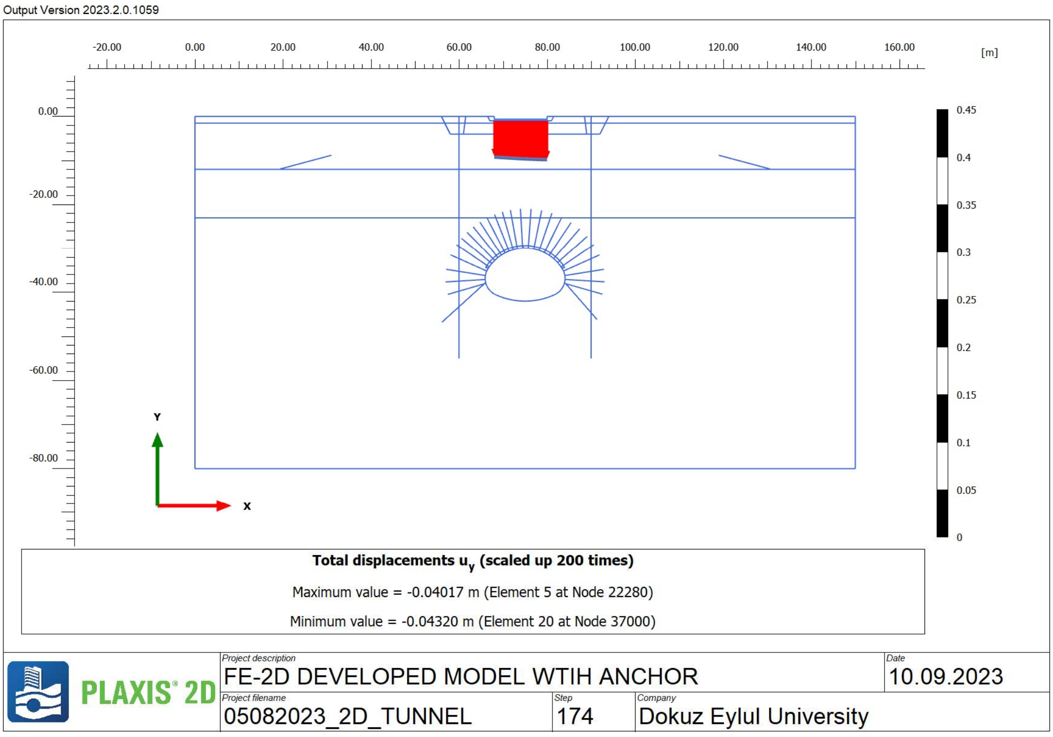

Figure 5.6 Vertical displacement of the developed model 1

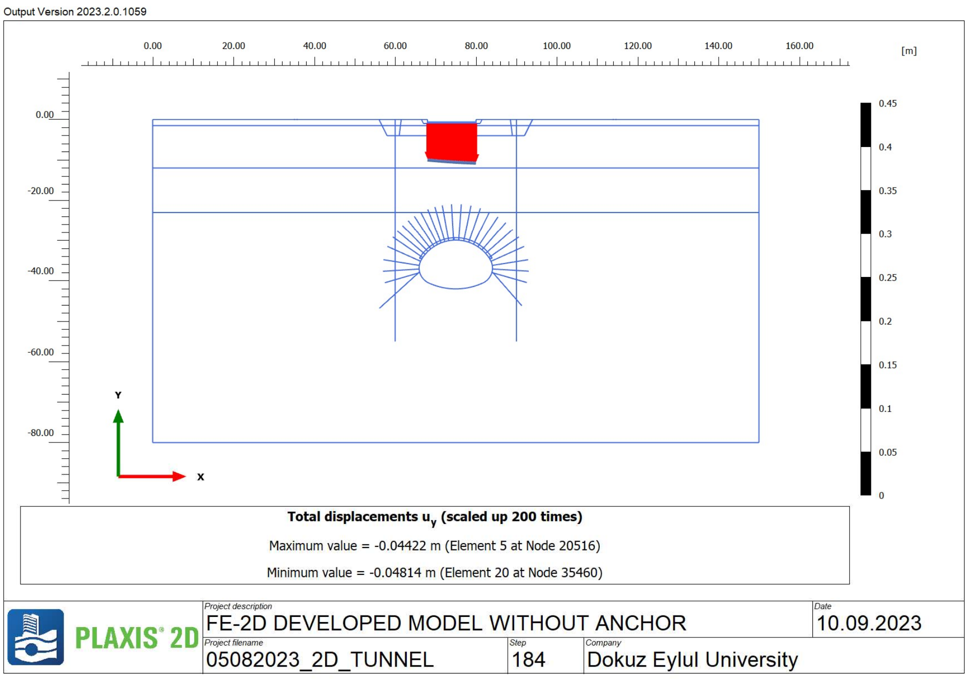

Figure 5.7 Vertical displacement of the developed model 2

5.5 Discussion

The tunnel design analysis results provided above have been discussed and interpreted as follows.

1. It has been observed that the accurate determination of the soil profile in which the tunnel excavation is performed is one of the most crucial findings for the correct prediction of surface deformations. In the site borehole logs, the elastic modulus of the relevant soil profile was determined as 176 MPa, and a surface settlement of 75.5 mm was anticipated. By changing only, the elastic modulus to 85 MPa, a surface settlement of 122 mm was measured, and when it was altered to 330 MPa, a settlement of 50 mm was recorded.

2. It has been recognized that the accurate prediction of groundwater levels and the control of this level during tunnel excavation are important inputs for accurately estimating surface settlements. The decrease in the groundwater level during excavation increases effective stresses, thereby accelerating settlements in structures above the tunnel.

3. It has been observed that there is no significant difference in surface settlements between choosing the tunnel primary lining system as very strong or sufficiently strong. In fact, changing the profiles selected as IPN220 in the current situation to IPN450 has resulted in a 5% improvement in surface settlements.

4. It has been observed that the definition of the temporary invert as a plate and the interface definition are crucial for accurately predicting surface deformations. In the same analysis, there was approximately a 27% difference between defining the temporary invert with an interface coefficient of 0.8 and not defining it, assuming it to be fully rigid. Surface settlements decrease as it transitions towards full rigidity. However, for smaller coefficients, surface settlements increase. This phenomenon can be explained by the transfer of stresses generated by excavation from structural elements to the ground. The

more effective the structure-soil interaction, the more stable the system behaves.

5. As evident in empirical formulas, the percentage of tunnel volume loss is one of the most critical parameters for predicting tunnel settlements. Quality implementation during excavation reduces tunnel volume loss, thereby decreasing surface deformations. In the empirical formula proposed by Peck (1969), if the existing tunnel excavation is carried out with a 2% volume loss, a settlement of 89 mm is anticipated. However, when the volume loss reaches around 3%, this value increases to approximately 133 mm.

6. In the developed methods, increasing the bending stiffness of the horizontal beam element, which is subjected to bending, reduces the vertical settlements in the surface structure.

7. In the developed methods, it has been observed that there is an inverse relationship between the length of the vertical structural elements, which are piles, and surface settlements up to a certain point. Beyond a certain point, extending the pile length further is not effective. It has been found necessary to extend the pile length at least to the tunnel's lower excavation level. Shorter piles remained within the tunnel excavation zone and did not achieve the desired effect.

8. In the method where the developed anchored system is applied, increasing the anchor prestressing force increases the reverse moment on the horizontal beam, thereby reducing deformations at the surface.

9. In the developed methods, it has been observed that the junction point of the horizontal beam and vertical piles should be fully fixed to ensure stability.

10. In the developed method, it has been envisaged that vertical structural elements should remain as much behind the tunnel inflection point as possible. However, attempting to stay outside the inflection point in tunnels of this size will

increase the horizontal beam length, making it more susceptible to bending, which will reduce its resistance to surface settlements.

CHAPTER SIX
CONCLUSIONS AND RECOMMENDATIONS

This thesis study has been pursued to evaluate surface settlements of an existing tunnel upon completion of excavation, make comparisons among different approaches, and develop a methodology that can be applied in the field prior to tunnel construction so that surface settlements can be reduced.

Considering the site construction conditions, it has been observed that the results achieved through two-dimensional analyses during the design phase may result in even greater settlements for large diameter and shallow depth tunnels. Therefore, it is considered essential not to rely solely on the results of a single method.

Findings that contribute to surface settlements have been identified, and with the help of these findings, information has been provided about the parameters to be considered in tunnel route selection.

It has been observed that the groundwater level remaining above the tunnel negatively affects surface settlements if water intake to the tunnel took place.

The parameter largely controlling surface settlements comes from the soil profile surrounding the tunnel being excavated as well. Unless necessary, it is recommended to route the tunnel through solid rock in such cases. In addition to this, there is a serious need for new techniques to open tunnels under critical structures. Keeping this in mind, an attempt was made to come up with such a methodology, which was never been applied before. The details of this technique and analyses results are given in Chapter Five and Appendices of this thesis. The technique was officially accepted by Dokuz Eylül University to be supported for a possible patent as part of the thesis with an application number 2023008301. The requests, specifications and summary of the patent application are presented in Appendix B, C, D and E. One may note, however, that there is a need for 3D analyses and calibration of the soil parameters shall be made to match 3D analyses results with field measurements.

Development of a laboratory-based study for enhancing the horizontal beam element's rigidity in a more cost-effective manner by using pressurized fluids within steel pipes and incorporating their non-compressibility feature into the system. One may note that doing this is quite essential since use of such a structural member would be specific to the developed methodology.

Creating prototypes of the proposed methodology with various structural members and doing small scale model tests in the laboratory environment would be quite enlightening to foresee full scale behavior of the system.

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APPENDICES

Appendix-A Borehole log

SAMPLE		STANDARTD PENETRATION							ROCK MASS PROPERTIES			
		BLOW COUNT			N_{30}							
SAMPLE TYPE & NUMBER	Depth (m)	15	30	45	SPT-N_{30}	SOIL GROUP SYMBOL	GROUND WATER LEVEL	SOIL CHARACTERIZATION	RQD (%)	CORE (%)	ROCK WEATHERING RATE (W)	FRACTURE FREQUENCY (M)
	0.00-1.50							0.00~1.50 m FILL				
SPT-1	1.50-1.95	8	9	9	18	GP-GM		1.50~12.00 m Yellowish brown-gray colored, less clayey silty sandy pebbles, blocks and gravels of sandstone origin; It is traced up to 12 m after the filling material such as clayey, yellowish brown pebbly clayey sand and block size materials.				
SPT-2	3.00-3.45	11	12	16	28							
SPT-3	4.50-4.95	8	12	15	27							
SPT-4	6.00-6.45	8	15	17	32							
SPT-5	7.50-7.95	14	18	18	36							
SPT-6	9.00-9.45	17	28	36	>50							
SPT-7	10.50-10.95	12	14	17	31							
SPT-8	12.00-12.45	21	18	24	42							
SPT-9	13.50-13.95	50/3			>50			12.00~23.00 m This zone, which is observed as an alternation of yellowish brown, light greenish gray, fractured, fragmented, moderately weathered sandstone shale, is generally interpreted as the Alluvial Flysch transition.	0	35	W2	M3
CORE	13.50-15.00								6	28	W2	M3
CORE	15.00-16.50								0	6	W5	M5
CORE	16.50-18.00								0	6	W5	M5
CORE	18.00-19.50								0	29	W5	M5
SPT-10	19.50-19.95	50/4			>50							
CORE	21.00-22.50								0	15	W5	M5
CORE	22.50-24.00							23.00-45.00 m Generally brownish-gray in color, fragmented, commonly weathered, with occasional gravel-block features, flysch.The flysch, which is generally dark in color, consists mostly of sandstone, shale and occasionally diabases. The unit is very cracked and fractured. Quartzites are generally found in the cracks.	0	15	W5	M5
SPT-11	24.00-24.45	14	18	23	41							
CORE	25.50-27.00								0	30	W5	M5
SPT-12	27.00-27.45	16	19	20	39				0	18	W5	M5
CORE	27.45-28.95						-30.0 m		13	41	W4	M5
CORE	28.95-30.45								0	10	W5	M5
CORE	30.45-31.95								0	8	W5	M5
SPT-13	31.95-32.40	17	23	25	48							
CORE	32.40-33.90								0	20	W5	M5
SPT-14	33.90-34.35	12	18	16	34							
CORE	34.35-35.85								0	13	W5	M5
CORE	35.85-37.35								0	27	W5	M5
CORE	37.35-39.85								0	22	W5	M5
SPT--15	39.85-40.30	50/10			>50							
CORE	40.30-41.80								0	28	W5	M5
CORE	41.80-43.30								11	24	W4	M5
SPT-16	43.30-43.75	17	26	23	49							

Appendix-B The patent specificaitons

A SUPPORT SYSTEM FOR REDUCING DEFORMATIONS IN STRUCTURES AROUND TUNNEL ROUTES

Technological Sector

The invention pertains to a new support system aimed at reducing deformations in structures along the alignment of tunnels excavated for various purposes in any type of terrain.

State of the Art

In today's practice, the occurrence of surface deformations on the ground above tunnels is a commonly encountered situation. This issue is particularly pronounced for tunnels constructed using the NATM technique. Especially in tunnels opened within urban areas, such as metro constructions, surface deformations can lead to serious problems in buildings. Existing techniques to mitigate these deformations often involve maintaining the strength of the tunnel construction system, making improvements targeting the foundations of structures from within the tunnel, reinforcing the tunnel lining, injecting materials aimed at the foundation or sublayers of the structure from the surface, and employing methods like ground freezing or soil heating.

During tunnel excavation, ground loss often leads to surface deformations and displacements, which are commonly encountered issues. To mitigate these deformations, various methods are generally employed, including efforts to reinforce the tunnel construction system, improvements targeting the foundations of structures from within the tunnel, enhancing tunnel lining stability, or injecting materials aimed at the foundation or sublayers of the structure from the surface. More costly methods such as ground freezing or heating are rarely used. The effectiveness of the preferred improvements is often not clearly determined. Therefore, excavation progress is regularly monitored through instrumental measurements, and additional measures are taken as deemed necessary.

When examining existing techniques, there always remains a question mark regarding the kind of improvement injection and similar methods bring about in the ground. Additionally, simultaneous injection during tunnel excavation causes the ground to become denser, which can increase surface displacements. In the case of injections initiated before tunnel excavation, the applied injection may not fully penetrate beneath the target structure, and because injection is applied at an oblique angle to the ground, the core of the structure remains unprotected. If the injection pressure is kept high, the structure can even suffer damage due to swelling before the tunnel is excavated, whereas using low pressure may result in the injected mixture not infiltrating the ground as desired.

The existing techniques such as ground heating and ground freezing require specialized technology and have a more specific application phase, making them expensive methods that are generally not preferred unless in very specialized and essential situations.

Another technique, temporarily lowering the groundwater level, directly leads to an increase in effective stress within the ground, potentially causing settlements at the surface, especially in areas with existing structures. It is essential to exercise caution when considering this method in regions with existing buildings. Continuous pumping will be required to maintain the groundwater level at a certain depth, resulting in energy consumption throughout the tunnel excavation process. Additionally, these pumping operations can create environmental impacts in the area, potentially leading to legal challenges.

In the Chinese patent document with number CN104895121A encountered during the literature review, a system designed beneath the structure foundation is described. In the mentioned document, pits are excavated around the structure foundation, and these pits are connected with a reinforcing layer. Unlike the method in the present invention, this approach does not provide sufficient support because it does not rely on a frame system based on creating moments and displacements in the opposite direction of vertical settlement on the beam element beneath the structure before commencing tunnel excavation, which consists of fully anchored connections for loads.

The Chinese patent document with number CN102704957 (A) encountered during the literature review discusses a method like the top-down excavation method for underground construction, aiming to preserve the existing topography on the surface without disruption for the structures above. It is not related to the present invention and does not contribute to mitigating deformations in surrounding structures.

The Japanese patent document with number JPH1129949A encountered during the literature review discusses a system placed over the tunnel aimed at preventing settlement in tunnel grounds. However, this system poses a risk to the tunnel itself as it is installed immediately above it and does not provide any benefit in terms of protecting the structure foundations. As a result, there is a need for a new support system that surpasses the prior art, addressing its disadvantages.

Brief Description of the Invention

The invention is a support system that surpasses the prior art, addresses its disadvantages, and additionally offers extra advantages.

The objective of the invention is to introduce a support system for urban underground tunnel constructions where critical structures (historical-military buildings, critical infrastructure lines such as natural gas, wastewater, drinking water, electricity, etc.) are present, aiming to reduce the impact of displacements caused by tunnel excavation on these relevant structures.

In the invention, rather than making improvements in the tunnel ground, the principle is to create a frame system by passing through areas where displacements caused by tunnel excavation are either zero or minimized, aiming to minimize the impact of ground movement on this frame system. The goal is to transfer the loads generated after the interaction between the structure and the ground to this frame system, thereby limiting the displacements in the structures.

The invention, unlike the methods used in the prior art, is a ground improvement or reinforcement method that aims to control settlements on the surface through a more controllable and directly targeted approach, rather than making improvements in the soil parameters mentioned in the prior art. An important aspect of the invention is the horizontal strengthening elements, which are expected to be placed beneath the structure foundation through horizontal drilling, beyond the turning point of the anticipated surface settlement trough, depending on the tunnel diameter and depth. These elements act as rigid beams in a sense.

These horizontal beams will be fully anchored with piles, effectively forming a frame system beneath the foundation. The structure will be supported by this frame system, minimizing its susceptibility to ground deformations.

Explanation of Figures

The invention, the figures attached hereto will be referred to in order to provide a clearer understanding of the features of the invention. However, the purpose is not to limit the invention with these specific arrangements. On the contrary, it is intended to encompass all alternatives, modifications, and equivalents that may fall within the scope defined by the claims attached hereto. The details shown are presented solely for the purpose of illustrating preferred embodiments of the present invention and should be understood to serve both the shaping of the methods and the provision of the most useful and easily understandable description of the rules and conceptual features of the invention. In these drawings.

Figure-1 Front view showing the subject of the invention system

Figure-2 Top View showing the subject of the invention system

The figures that will aid in understanding this invention are included as indicated in the attached diagram and are numbered and labeled with their names below.

Explanation of References

10. Tunnel

11. Pile

12. Circular sheath

13. Horizontal beam

14. Pre-tension tendon

A. Structure
B. Substructure zone

C. Excavation soil layer
D. Undisturbed soil

Explanation of the Invention:

In this detailed explanation, the subject of the invention, the support system, is only explained for better understanding with examples that do not create any restrictive effects.

In Figure 1, the front view of the support system under consideration is provided, and Figure 2 shows its top view. Accordingly, the support system is positioned in the substructure (B) of the building foundation beneath the tunnel (10) excavation ground (C). The term 'structure' (A) encompasses any type of construction. Located along the tunnel (10) route. The substructure (B) is the area beneath the foundations of the structure (A). The excavation ground (C) describes the ground that has experienced settlement due to the tunnel (10) excavation. While all the systems before the invention offer solutions to improve the excavation ground (C) mentioned here, the invention uses the load applied by the structures (A) to redirect it towards the undisturbed soil (D). The undisturbed soil (D) remains unaffected by any settlement or displacement after the tunnel (10) excavation. Therefore, it prevents deformations in the structures (A).

In the invention, a series of horizontal beams (13) and end-anchored bored piles (11) are used to redirect the loads of the structures (A). The number of horizontal beams (13) varies depending on the tunnel (10) length or the number of structures (A). Accordingly, in the invention, within the substructure (B) of the building foundation, at least one horizontal beam (13) longer than the width of the tunnel (10) is positioned in a way that does not include the excavation ground (C). At both ends of the horizontal beams (13), there is one bored pile (11) each. The horizontal beams (13) transfer the loads from the structures (A) to the aforementioned bored piles (11). Since the bored piles (13) are located on the undisturbed soil (D), the effect of settlement in the excavation ground (C) on the structures (A) is reduced.

In the invention, as an alternative, bored piles (11) or horizontal beams (13) can be transformed into an anchored system by being tensioned with pre-stressing cables (14). The pre-stressing cable (14) forms a rigid connection with the bored piles (11) or horizontal beams (13), providing both full anchorage conditions for the relevant area, limiting displacement, and reducing the displacements that would occur with tunnel excavation by creating reverse moments on the horizontal beam (13) element.

In the invention, either pressurized liquid or a reinforced concrete + steel composite element can be used as the horizontal beam (13). The advantage of using a reinforced concrete + steel composite element lies in its ability to easily achieve the required strength parameters by using concrete and steel, depending on the level of displacements related to the diameter and depth of the tunnel (10), and its economic feasibility. On the other hand, the fundamental principle of providing rigidity with pressurized liquid is the ability to control the pressure externally at any given time, monitor pressure changes that occur during tunnel excavation, and take advantage of the non-compressibility property of liquids. Either one of these two methods, or both simultaneously, can be used within the system.

Additionally, in the invention, circular casings (12) are used between the horizontal beams (13) to control the controlled placement of the horizontal beam (13) and to prevent settlements that the excavation during horizontal drilling may cause on the surface. Furthermore, the circular casing (12) directs the pressure that will be generated inside when the horizontal beam (13) is created with pressurized liquid.

In the liquid pressure-based horizontal beam (13) method of the invention, the void within the horizontal beam (13) is pressurized with a fluid. The pressurization process is facilitated by a liquid propulsion tank and a pump. Here, the fluid is directed into the horizontal beam (13) through watertight connections and the circular casing (12).

Appendix C The patent requests

REQUESTS

1. The invention is a new support system designed to reduce the deformation caused by tunnels excavated for any purpose in any field in structures (A) located along their routes. Its feature is;

- It comprises at least one horizontal beam (13) that is longer than the width of the tunnel (10) and is placed beneath the structure's foundation area (B) without extending into the excavation area (C).

- It is characterized by having at least one bored pile (11) at each end of the mentioned horizontal beam (13) where the load from the horizontal beam (13) is transferred, all located in the area of undisturbed ground (D) unaffected by the excavation.

2. Request 1 corresponds to a support system that is characterized by containing a reinforced concrete + steel composite horizontal beam (13) for the purpose of easily achieving the required strength parameters using concrete and steel depending on the magnitude of the displacements associated with the diameter and depth of the excavated tunnel (10). This system is designed to be economically feasible.

3. Request 1 corresponds to a support system that is characterized by containing a horizontal beam (13) filled with pressurized fluid for the purpose of enabling real-time external control of pressure, monitoring pressure changes occurring during tunnel excavation, and benefiting from the non-compressibility property of fluids.

4. A support system that complies with any of the above requests, characterized by including a cylindrical casing (12) between the horizontal beams (13) to withstand the settlements that may occur on the surface during horizontal drilling, ensuring controlled placement of the horizontal beam (13).

5. A support system that complies with any of the above requests, characterized by including a cylindrical casing (12) that directs the pressure generated inside when the horizontal beam (13) is formed with pressurized fluid.

6. A support system that complies with any of the above requests, characterized by the length of the horizontal beam (13) being greater than the width of the tunnel (10).

7. A support system that complies with any of the above requests, characterized by the inclusion of at least one pre-stressing cable (14) capable of making a rigid connection with bored piles (11) or horizontal beams (13), providing support for the full anchorage conditions of the relevant area, displacement limitation, and creating reverse moments on the horizontal beam (13) element with pre-stressing load, thereby reducing the displacements that will occur with the tunnel excavation.

Appendix D The patent summary

A SUPPORT SYSTEM FOR REDUCING DEFORMATIONS IN STRUCTURES AROUND TUNNEL ROUTES

The invention relates to a new support system aimed at reducing the deformations caused by tunnels excavated for any purpose in any area along their routes.

Appendix E The patent figures

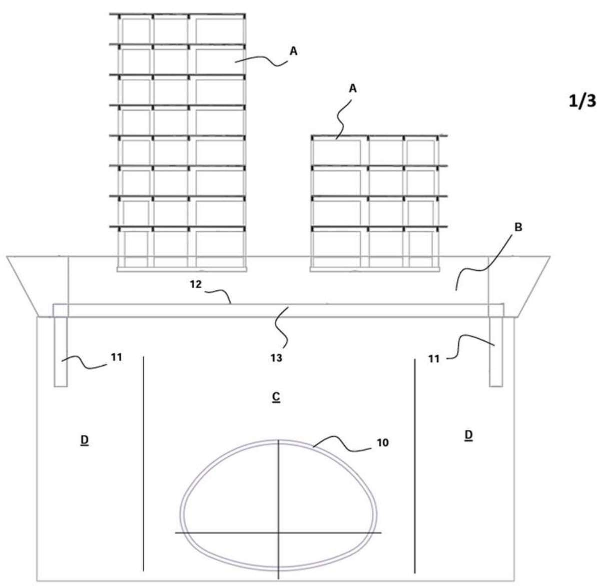


Figure – 1

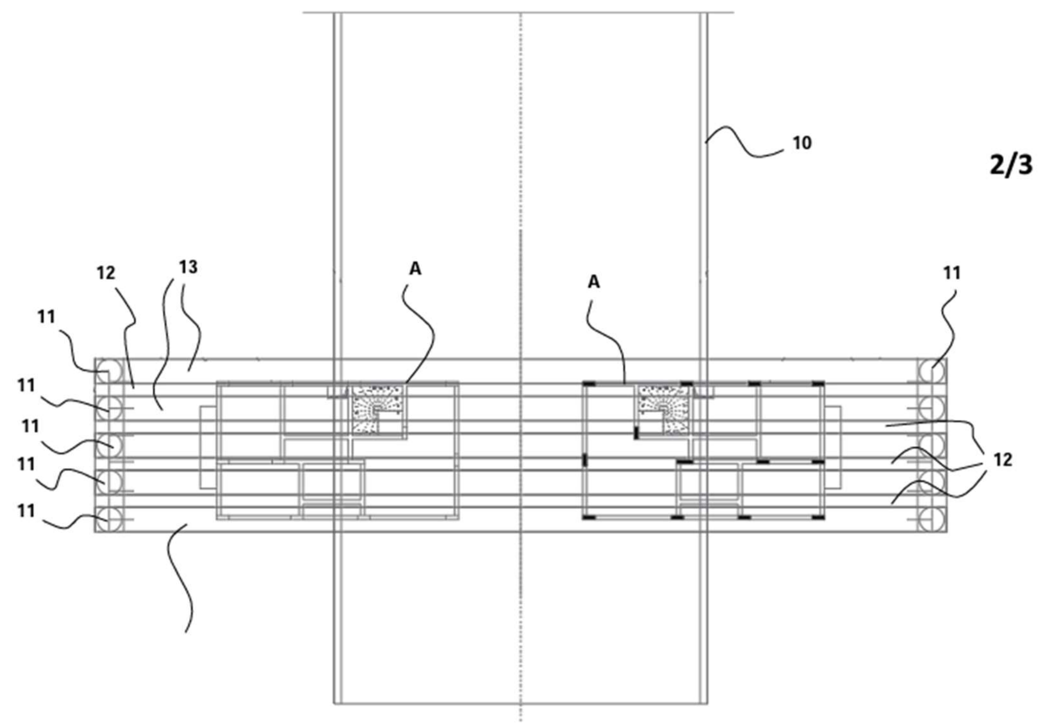


Figure-2

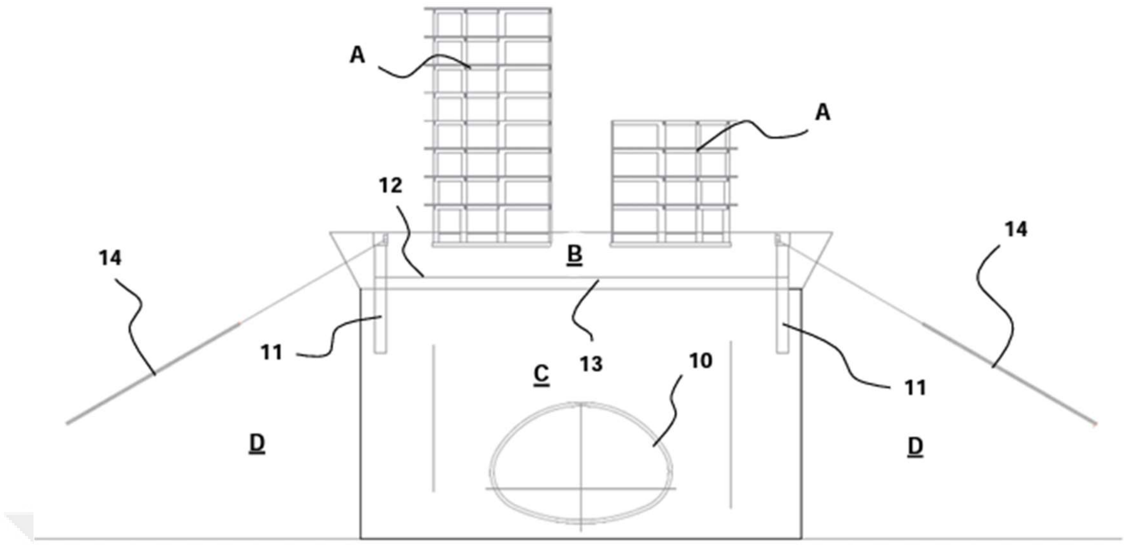
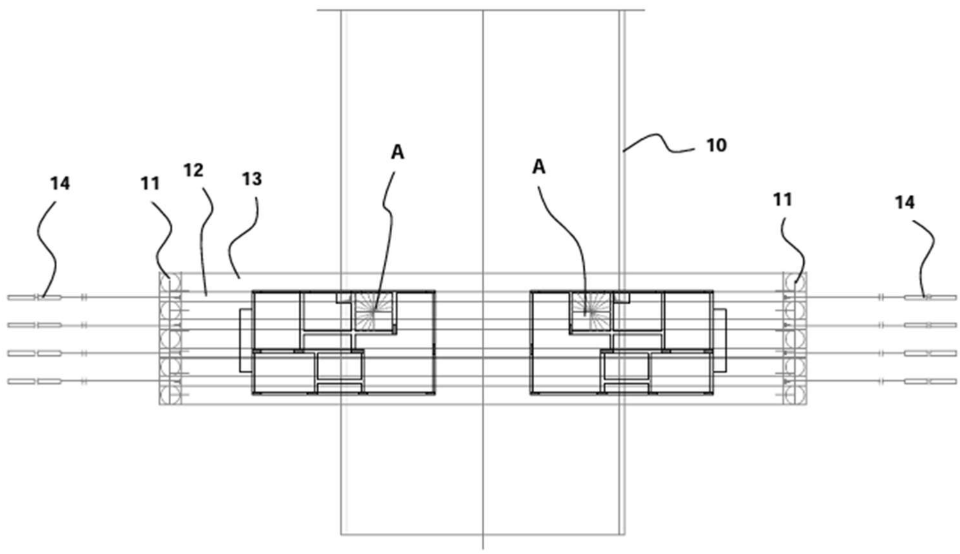


Figure – 3



3/3

Figure– 4