

NUMERICAL ASSESSMENT OF TUNNEL ADVANCEMENT IN  
PROBLEMATIC SOIL CONDITIONS: A CASE STUDY

A THESIS SUBMITTED TO  
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES  
OF  
MIDDLE EAST TECHNICAL UNIVERSITY



BY  
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IN PARTIAL FULFILLMENT OF THE REQUIREMENTS  
FOR  
THE DEGREE OF MASTER OF SCIENCE  
IN  
CIVIL ENGINEERING

JANUARY 2021



Approval of the thesis:

**NUMERICAL ASSESSMENT OF TUNNEL ADVANCEMENT IN  
PROBLEMATIC SOIL CONDITIONS: A CASE STUDY**

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## **ABSTRACT**

### **NUMERICAL ASSESSMENT OF TUNNEL ADVANCEMENT IN PROBLEMATIC SOIL CONDITIONS: A CASE STUDY**

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January 2021, 134 pages

Modeling tunnel advancement in complex ground conditions requires significant engineering judgment and is an integral part of the design and construction of tunnels. Insufficient data and improper design procedures may lead to dramatic consequences. Realistic estimations of the ground behavior, displacements, and support pressures are possible with accurate numerical models. In this study, the advancement of Tunnel A93 is investigated by 1-D semi-empirical assessments and two-dimensional finite element analyses. This thesis aims to introduce an assessment of the tunnel advancement with TBM and NATM through adverse ground conditions based on the actual convergence measurements of Tunnel A93.

The tunnel A93 is located in Turkey, which is to be served as a railway tunnel for a high-speed railway line. Due to the challenging soil conditions, many problems, including several stoppages due to shield jamming and the partial collapse of the tunnel, have occurred during the excavation.

In order to assess the response of the tunnel and develop better support systems, field monitoring data have been compiled. Field performance and numerical analysis results are comparatively discussed, along with the effectiveness of mitigative design

solutions. Additionally, lining deformations and their interaction with excavation sequences are assessed.

Keywords: Tunnel, TBM, NATM, Collapse



## ÖZ

### **SORUNLU ZEMİN KOŞULLARINDA TÜNEL AÇIMININ SAYISAL DEĞERLENDİRİLMESİ: BİR VAKA ÖRNEĞİ**

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Ocak 2021, 134 sayfa

Zorlu zemin koşullarında tünel açımının modellenmesi, ciddi bir mühendislik muhakemesi gerektirir ve tünel tasarımı ve inşasının kayda değer bir parçasıdır. Yetersiz veriler ve hatalı tasarım yöntemleri istenmeyen sonuçlara yol açabilir. Zemin davranışı, tünel içi deplasmanlar ve tahkimatlar üzerinde oluşan basınçların gerçekçi tahminleri, doğru sayısal modellerle mümkündür. Bu çalışmada, A93 Tüneli'nin açımı, 1 boyutlu yarı ampirik değerlendirmeler ve iki boyutlu sonlu eleman analizleri ile incelenmiştir. Bu tezin amacı, Tünel A93'ün sahadan alınan konverjans ölçümlerine dayanarak olumsuz zemin koşullarında TBM ve NATM yöntemleriye açımının değerlendirilmesini sunmaktır.

Bir yüksek hızlı demiryolu hattında hizmet verecek olan A93 demiryolu tüneli, Türkiye'de yer almaktadır. Tünel açımı sırasında, son derece zorlu zemin koşullarından dolayı, şilt sıkışmasından dolayı tünel açımının duraksaması ve tünelin kısmen çökmesi de dahil olmak üzere birçok sorunla karşılaşmıştır.

Tünel açımının etkisini değerlendirmek ve uygun destek sistemleri geliştirmek için saha izleme verileri derlenmiştir. Saha performansı ve sayısal analiz sonuçları, tasarımsal çözümlerin etkinliği ile birlikte karşılaştırmalı olarak ele alınmıştır.

Ayrıca, tnel kaplamasında gzlenen deplasmanlar ve oluřan deplasmanların kazı ařamalarıyla etkileřimi deęerlendirilmiřtir.

Anahtar Kelimeler: Tnel, TDM, YATM, kme





To my family

## ACKNOWLEDGMENTS

I would like to express my deepest gratitude to my supervisor Prof. Dr. Kemal Önder Çetin, for his support, expertise, encouragement, and advice throughout the thesis journey. This thesis would not be completed without his precious contributions.

I am grateful to the members of the Examining Committee for their valuable comments and guidance.

Sincere thanks to Francesco Saverio Agresti and Paolo Occidente, and my colleagues in SWS Engineering SpA, Fugro Sial Geosciences Consulting and Engineering Co. Ltd., and Prota Engineering for their valuable suggestions.

I am thankful to Cihan Erdönmez and Gökhan Marangoz for their valuable contribution.

I owe special thanks to my friends, especially Uluhan Sezer, Ersin Turan, and Melis Kalkan, for their endless support and patience.

I would also like to express my gratitude to my mother, Güner Güneş, my father Sakin Güneş, and my dear sister, my mentor Hande Güneş. It has been a joy to have their support throughout my life.

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## LIST OF ABBREVIATIONS

### ABBREVIATIONS

|       |                                                     |
|-------|-----------------------------------------------------|
| AFTES | French Tunnelling and Underground Space Association |
| CCM   | Convergence-Confinement Method                      |
| FE    | Finite Element                                      |
| FEM   | Finite Element Method                               |
| GSI   | Geological Strength Index                           |
| GWL   | Groundwater level                                   |
| NATM  | New Austrian Tunnelling Method                      |
| TBM   | Tunnel Boring Machine                               |
| UCS   | Uniaxial Compressive Strength                       |





## **CHAPTER 1**

### **INTRODUCTION**

#### **1.1 Overview**

Tunnels are engineering subsurface structures, which are constructed by means of accomplishing safe, efficient, and serviceable transportation. The demands of modern transportation networks, which provide the continuity of the flow of passengers and goods, have become a significant phenomenon in civilizations. To design modern tunnels, vast knowledge, not just on-site geological conditions are needed, but also all the different methods of design, different support systems, rock mass, and soil behaviors, and numerous other parts of the tunnel design which have become a standard.

The basic principles are to provide well-timed, adequate, and economical solutions for different ground characteristics. Tunnels are constructed in numerous different ground environments, including soil and rock. A wide range of variations in the groundwater conditions, in-situ states of stress, and geological structures may be encountered during the tunnel advancement. Alternative construction techniques, such as conventional excavation, excavation with the drill-and-blast method, and mechanized tunneling techniques, are used to construct tunnels (O'Rourke, 1984).

Due to topographical, social, or environmental considerations, many transportation projects involve numerous tunnel structures constructed in difficult ground conditions. Thus, tunneling in adverse ground conditions is one of the most challenging phenomena in tunnel engineering. Therefore, tunnel construction, in terms of engineering, is a phenomenon that requires multidisciplinary optimization since the tunnel structure is aimed to be a part of the ground.

The lack of complete soil data and rapidly changing ground conditions may affect the design process of the tunnels. When unfavorable conditions arise without warning, the cost, time schedule of the project may be affected significantly. Even more, inadequate or incompatible design may result in local failures or total collapses.

The empirical methods are not sufficient to calculate the support deformation and performance, stress redistribution around the tunnel, which are susceptible elements to the field observation (Basarir et al., 2005). In this sense, numerical methods are widely employed in the design of tunnels to overcome many uncertainties throughout the design of tunnels (Barla, 2016).

## **1.2 Scope and Objective of the Study**

Tunnel advancement in weak ground conditions is one of the most problematic issues in tunnel engineering. Excavation in adverse geological conditions may lead to instability at the face and tunnel walls, jamming of the boring machine (in the case of shield tunneling), and even collapses. In addition, tunnel advancement in problematic soil conditions requires meticulous attention in geotechnical site investigations, construction methods, and monitoring.

In this study, a 13.4m diameter high-speed railway tunnel excavated in a highly squeezing ground was investigated. The tunnel construction has begun according to the principles of TBM. During the railway tunnel excavation, due to excessive ground pressures, unsymmetrical deformations are generated in the tunnel lining, thus resulting in a total collapse of the cavity. Consequently, the tunnel construction method is altered to NATM. The performance of both support systems and the observed deformations due to the tunnel excavation is assessed by means of numerical analyses. The numerical analyses are conducted based on the data acquired from site investigation results and the field deformation data. The scope of this thesis covers the evaluation of the tunnel advancement by following different

construction methods of the high-speed railway tunnel, before and after the occurrence of the partial collapse incident. The analyses performed with back analyzed strength parameters before and after the collapse are compared, and the verification of the results is achieved on the basis of field monitoring data.

### **1.3 Thesis Outline**

This thesis study includes six chapters. It starts with the introduction, which presents the scope and objective of the study, in Chapter 1. In Chapter 2, the theoretical background of tunnel construction methods and the application of the tunnel excavation to the numerical models are described. In Chapter 3, the features of the A93 Tunnel are summarized. The results of the geotechnical investigations, the different construction methods employed before and after the collapse, and the monitoring system are described in this chapter. Chapter 4 indicates the studies performed and the implemented methodology throughout the numerical analyses. In Chapter 5, the results of the analyses and the discussion of the obtained results are presented. In Chapter 6, the thesis study is summarized, and brief information about further studies are presented.



## **CHAPTER 2**

### **LITERATURE REVIEW**

A comprehensive literature review on tunnel construction methods and numerical modeling and the representation of the tunnel excavation are summarized in this chapter.

#### **2.1 Tunnel Construction Methods**

The tunneling comprises excavation, the removal of the excavated material (mucking), and supporting the formed cavity during and after the excavation. All of the mentioned procedures aim to achieve serviceability during the life-cycle of the structure. There are several proposed methods to perform the advancement in tunnels. In this section, two well-known tunneling methods, the New Austrian Tunneling Method (NATM) and mechanized continuous tunneling by a tunnel boring machine (TBM), are briefly described, and the advantages and disadvantages of each method are compared.

##### **2.1.1 New Austrian Tunneling Method (NATM)**

Rabcewicz, Müller, and Pacher developed NATM in 1957 through 1964 (Rabcewicz, 1964). The main principle of the method is to consider the surrounding ground as the main element for bearing the loads; in other words, the ground is used as a construction material. Adequate deformations in the ground are allowed to mobilize the strength of the rock or soil possesses. As a result, while the ground

resists most of the stresses exerted, the support elements, such as shotcrete and rock bolts, introduce a cavity confining and surface stabilizing function.

The New Austrian Tunneling Method is a type of cyclic tunneling method, which is defined as the execution of the tunneling in batches in a repetitive manner. Depending on the properties of the surrounding ground, the excavation progresses with blasting or digging machinery apart from any full-face tunnel boring machine. The excavated material is transported out of the tunnel, and the support measures are installed.

The general principles of the NATM construction are identified by several authors; Brown (1981), Sauer and Gold (1989), Scholey and Ingle (1989), Wallis (1995), and Bowers (1997) which are summarized as follows:

- The inherent strength of the surrounding ground shall be preserved and mobilized by minimizing the deformations and prevent the loosening of the soil or rock mass.
- The mobilization of the strength is achieved by the application of primary support. Several primary support measures can be identified in NATM, such as shotcrete, steel sets, and rock bolts. The application of the shotcrete has almost become a universally analogous element as the primary support in NATM. The excessive deformations are controlled with such elements to avoid excessive deformations, which will result in a loss of strength.
- The tunneling method mainly aims to learn from experience and continuously update the support system using the deformation and stress measurements taken around the tunnel domain. Such measurements shall be acquired from sophisticated instrumentation, and they shall provide an insight into the tunnel stability and optimization of the support system during the tunnel advancement.
- The NATM philosophy dictates the usage of flexible supports that can be adapted to various ground conditions. The supporting elements sprayed concrete, wire meshes, steel ribs, rock bolting, or anchoring are considered

the main load-bearing components in NATM tunneling. The feature of final cast in-situ concrete lining after the application of primary supports is sealing the tunnel and forming the required shape of the tunnel.

- The timing of the ring closure is recognized as a critical factor in bearing on the overall behavior of the lining and ground interaction. The appropriate time for the cast of the invert depends on the features of the soil or rock behavior. The invert closure is conducted as quickly as possible in the soft ground excavations. On the contrary, the invert is closed at some distance behind the core face to destress the host ground and allow controlled deformations.
- The design and execution of NATM tunnels shall be carried out co-operatively to resolve any problems encountered during the advancement. Therefore, the flexibility in the contractual agreements is an essential requirement in NATM constructions.
- The support measures are determined from the classification of the rock masses. The host ground is classified into several groups according to their ground properties, geological and hydrogeological features. Each class involves a specific amount of support with an indication of specific excavation steps.

The fundamentals of the tunnel construction with NATM have been described by Rabcewicz (1964) in terms of stress rearrangements pressures. The excavation of a tunnel causes a stress redistribution around the tunnel domain; thus, the rearranged pressures create a new equilibrium around the tunnel cavity. The equilibrium is reached with or without the application of supporting elements depending on the exceedance of the shear strength of the host ground. This is described as a mechanical and progressive process, which is investigated in three phases (Figure 2.1).

Table 2.1 Rock mass types in ÖNORM B 2203

| Main rock mass type       |                               |                                                                                                                                                                                     |
|---------------------------|-------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>A</b>                  | <b>Stable to overbreaking</b> | Stresses acting on rock mass do not cause major failures                                                                                                                            |
| <b>B</b>                  | <b>Friable</b>                | Disintegration due to structural weakness and/or lack of interlocking.                                                                                                              |
| <b>C</b>                  | <b>Squeezing</b>              | Strength of rock mass is exceeded to great depth; this type also includes rock bursts and swelling rock                                                                             |
| Rock mass types in detail |                               |                                                                                                                                                                                     |
| Type                      | Rock mass behavior            | Demands on excavation and support for conventional tunnel driving                                                                                                                   |
| <b>A1</b>                 | <b>Stable</b>                 | Minor deformations that decline rapidly, no spalling.                                                                                                                               |
| <b>A2</b>                 | <b>Overbreaking</b>           | Minor deformations that decline rapidly; some spalling at the crown due to discontinuities.                                                                                         |
| <b>B1</b>                 | <b>Friable</b>                | Minor deformations that decline rapidly; structural weakness and blasting operations lead to loosening and the separation of blocks in the crown and upper wall.                    |
| <b>B2</b>                 | <b>Very friable</b>           | Deformations decline rapidly; poor structural strength, little interlocking, high mobility of rock mass and blasting operations lead to rapid and deep loosening where unsupported. |
| <b>B3</b>                 | <b>Rolling</b>                | Excavation even in small cross sections leads to inflow of rock material; lack of cohesion and interlocking are responsible for insufficient stability.                             |
| <b>C1</b>                 | <b>Rock bursting</b>          | Sudden release of energy leads to explosive rock failure.                                                                                                                           |
| <b>C2</b>                 | <b>Squeezing</b>              | Pronounced deformations that take long to decline; development of failure zones and plastic zones in plastic, cohesive rock mass.                                                   |
| <b>C3</b>                 | <b>Heavily squeezing</b>      | Large deformations, rapid at the beginning, taking long to decline; development of deep reaching failure zones and plastic zones.                                                   |
| <b>C4</b>                 | <b>Flowing</b>                | Very low cohesion, low friction, soft and plastic consistency of rock mass; material will flow into the tunnel even through very small unsupported areas.                           |
| <b>C5</b>                 | <b>Swelling</b>               | Rock mass with mineral content that increases in volume by absorbing water, e.g. swelling clay- minerals, salts, anhydrite.                                                         |

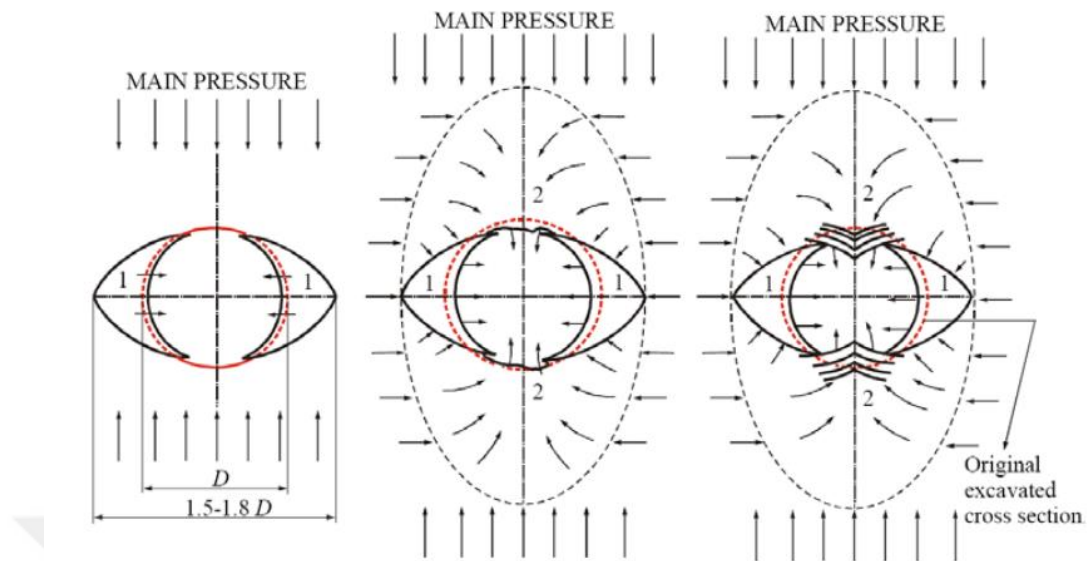


Figure 2.1 Mechanical process and sequence of failure around a cavity by stress rearrangement pressure (after Rabcewicz (1964), modified by Karakus & Fowell (2004))

At the first phase, wedge-shaped blocks are formed by shearing along the Mohr surfaces and move into the vertical direction of the maximum pressure into the cavity. This movement toward the tunnel domain and succeeding increase in the span causes convergence of the crown and the invert. Finally, at the last phase, since the movements increase and under sustained pressure, the surrounding rock may buckle and bulge outward the domain, into the cavity, which causes squeezing conditions.

### 2.1.2 Mechanized Continuous Tunneling

The tunnel construction methods contain digging a tube-like passage through the ground. According to the French Association of Tunnels and Underground Space (AFTES), the mechanized tunneling techniques (contrary to the conventional tunneling techniques) include the tunneling techniques where the excavation is performed mechanically utilizing teeth, picks, or discs. The mechanized tunneling involves various types of machines, from the backhoe diggers to the complicated

equipment such as shield TBMs, which carry out the excavation and provide support measures.

The definition TBM refers to a machine for excavating tunnels in rock with a circular full-cut cutter head, usually equipped with disc cutters. The rock is cut using the excavation tools by the rotation of the cutter head and the pressure exerted by the blades on the core face. In contrast to the methods of drilling and blasting, in which it is possible to consider the design measures for the resulting interaction of tunnel and rock, either by subdividing the excavated section or by a rapid fitting of the support elements to the geological conditions, this is not possible driving with TBMs (Maidl et al. 2008).

With the continual development of technologies, many TBMs are available to bore tunnels of different diameters through ground conditions ranging from hard rock to soft soil and grounds in between. Based on ground and groundwater conditions (i.e., hard rock, soft soils, mixed and changing grounds), tunnel face, and wall stability, TBMs can be broadly classified into seven types, as indicated in Figure 2.2. Extensive reviews of the historical development and opportunities of using TBMs in hard rock and soft soil are provided by Stack (1982) and Maidl et al. (2012, 2013).

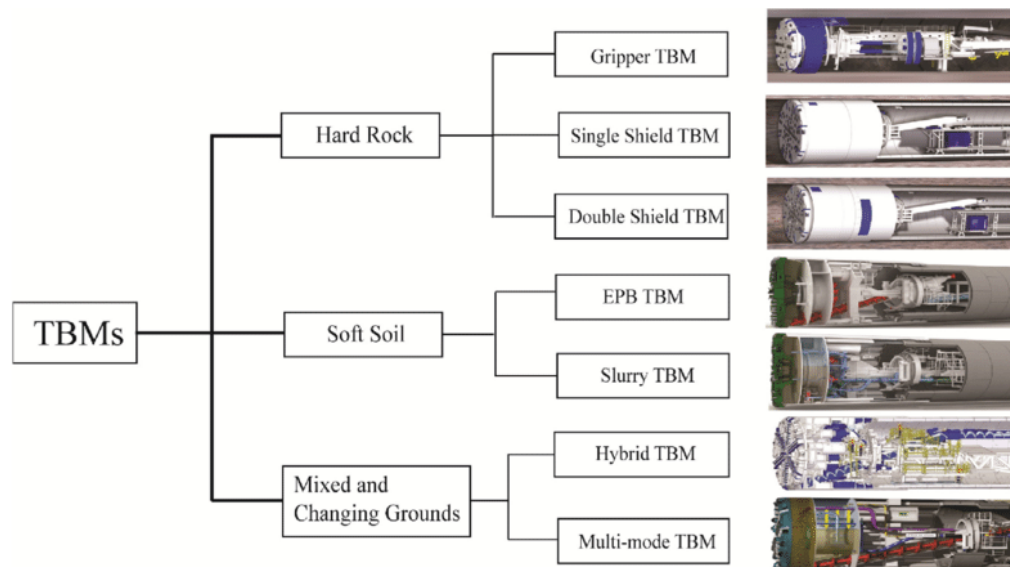


Figure 2.2 Classification of TBMs (Zheng et al., 2016)

### 2.1.3 Comparison of Conventional Tunneling and TBM Drives

Both conventional and TBM tunneling methods have significant advantages and disadvantages. The application of the methods shall be assessed with a comprehensive feasibility study for each tunnel project. The main differences between conventional tunneling and TBM tunneling in construction and operational stages are listed in Table 2.2 (Haack, 1996).

Table 2.2 Comparison of conventional tunneling and TBM tunneling

| <i>Phase</i>       | <i>Assessment Criteria</i>                                | <i>Conventional Tunneling</i> | <i>TBM Tunneling</i> |
|--------------------|-----------------------------------------------------------|-------------------------------|----------------------|
| Construction Phase | Supporting agent in face zone                             | Variable                      | Safer                |
|                    | Lining thickness                                          | Variable                      | Constant             |
|                    | Safety of crew                                            | Lower                         | Higher               |
|                    | Working and health protection                             | Lower                         | Higher               |
|                    | Degree of mechanization                                   | Limited                       | High                 |
|                    | Degree of standardization                                 | Conditional                   | High                 |
|                    | Danger of break                                           | Higher                        | Lower                |
|                    | Construction time – short tunnel                          | Shorter                       | Longer               |
|                    | Construction time – long tunnel                           | Longer                        | Shorter              |
|                    | Construction costs – short tunnel                         | Lower                         | Higher               |
|                    | Construction costs – long tunnel                          | Higher                        | Lower                |
| Operational Phase  | Tunnel cross-section                                      | Variable                      | Constant             |
|                    | Cross-section form                                        | As desired                    | Generally circular   |
|                    | Degree of the utilization of drive-related cross-sections | Generally higher              | Generally lower      |

Although the listed comparison in the table above indicates that TBM tunneling provides a relatively safe and standardized construction, conventional tunneling

offers a more flexible approach in the case of encountering adverse ground conditions. It shall also be noted that conventional tunneling presents efficient results in relatively shorter tunnels both economically and operationally.

According to Maidl et al. (2012), the essential disadvantages of TBM tunneling are summarized as follows:

- Better geological investigations and information are necessary than for drilling and blasting
- Extended lead time for the design, production, and assembly of the machine,
- Long familiarisation time,
- Elaborate and expensive site facilities (a separating plant may be required); tenders may only be competitive for longer tunnels,
- Performance risk in the changeable ground,
- The cross-section of the tunnel typically has to be round with little possibility of variation,
- The high cost of altering the excavated geometry and fixation of the mechanical failure of the equipment,
- The lining typically has to be specially designed to resist the thrust forces,
- Adaptation to varying rock types and high water inflow is only possible to a limited extent.

## **2.2 Modeling Tunnel Excavation**

### **2.2.1 The Gap Method**

The Gap Method was introduced by Rowe et al. (1983). A predefined void is presented into the FE mesh, which typifies the expected ground loss due to the excavation. Thus, the out-of-plane and in-plane ground losses are combined with additional losses to consider the misalignment of the shield, the quality of labor, and

the volume change due to the remolding of soil. Therefore, different tunnel construction methods can be accounted for by varying the dimensions of the gap.

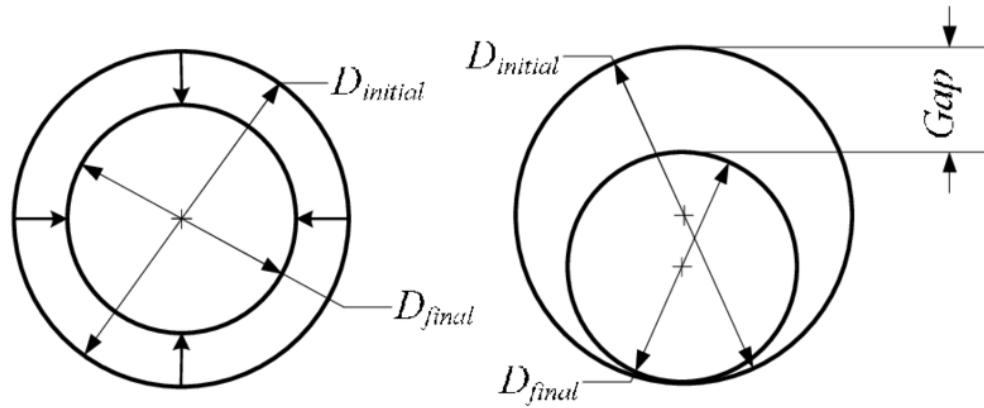


Figure 2.3 The Gap Method introduced by Rowe et al. (1983) (Koungelis, 2007)

The application procedure of the Gap Method starts with the placement of the gap around the final tunnel position ( $D_{final}$ ). Therefore, the soil boundary prior to excavation is located. The invert of the tunnel is rested on the underlying soil prescribing the gap parameter, which is the vertical distance between the crown of the tunnel and the initial position before tunneling.

The analysis proceeds with the removal of the boundary tractions located at the perimeter of the opening. The nodal displacements that take place at the perimeter of the opening are monitored. The soil-lining interaction is activated when the displacement of a node shows that the predefined gap has been closed and the soil is in contact with the lining. According to Rowe et al. (1978), the soil and the lining are considered as separate bodies, related only by nodal forces.

### **2.2.2 The Progressive Softening Method**

The Progressive Softening was introduced for the modeling of NATM tunneling by Swoboda (1979). The soil stiffness within the heading is progressively reduced in front of the face prior to the excavation by multiplying the soil stiffness by a reduction factor  $\beta$ . The softening effects are made clear when the fictitious forces are applied to the boundary of the tunnel. The lining is installed before the modeled excavation is complete.

If the tunnel is constructed with sequential excavation, the procedure is applied to each excavation area sequentially. The same methodology can also be applied to the analyses with side drifts.

### **2.2.3 The Volume Loss Control Method**

The Volume Loss Control Method is similar to the convergence-confinement method (Chapter 2.2.4), but instead of presenting the proportion of relaxation prior to the installation of the lining, the volume loss that will result on completion of the excavation is prescribed (Figure 2.4). Therefore, the method is suitable, especially for predictive analyses of tunnel excavation in soil types, where the expected volume loss can be conservatively determined. The Volume Loss Control Method is convenient to utilize in the back analysis of excavations for which the measurements of volume loss have been made.

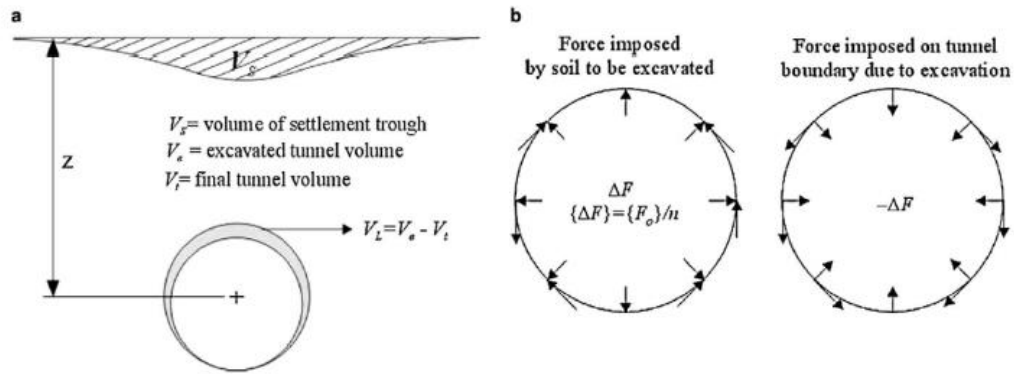


Figure 2.4 (a) Volume loss method; (b) modeling the excavation of solid elements (Potts et al., 2001).

With the  $\lambda$  method, the support pressure applied outward on the tunnel boundary is progressively reduced. Alternatively, the value of  $\{F_0\}$ , the equivalent nodal forces which represent the pressure exerted on the representative tunnel boundary to be excavated, can be calculated. The nodal forces are linearly distributed to the number of increments “n” over which the excavation is to take place. The equal force vector  $\{-\Delta F\}$  is then applied at the tunnel excavation boundary for each of the n increments of excavation (Figure 2.4b). The volume loss induced by each increment of boundary loading can be detected, and the tunnel lining constructed on the increment where the desired volume loss is achieved. After the construction of the lining,  $\{-\Delta F\}$  is remained at the excavation perimeter for the remainder of the “n” increments, thus introducing initial stresses into the lining. Further volume losses may occur during the last execution, depending on the stiffness of the lining. In this case, the installation of the lining can be performed at an increment, which has a relatively smaller volume loss than the desired result. Therefore, after the full excavation, the expected volume loss is achieved.

## 2.2.4 Convergence-Confinement Method

The convergence-confinement method or so-called  $\lambda$ -Method is a part of the rational approach and utilizes an analytical type calculation. The basis of the method is the analysis of the stress and strain state developing in the “hot” ground around a tunnel. The simplicity of the method provides a widespread utilization to model the tunnel excavation for decades (Oreste, 2009).

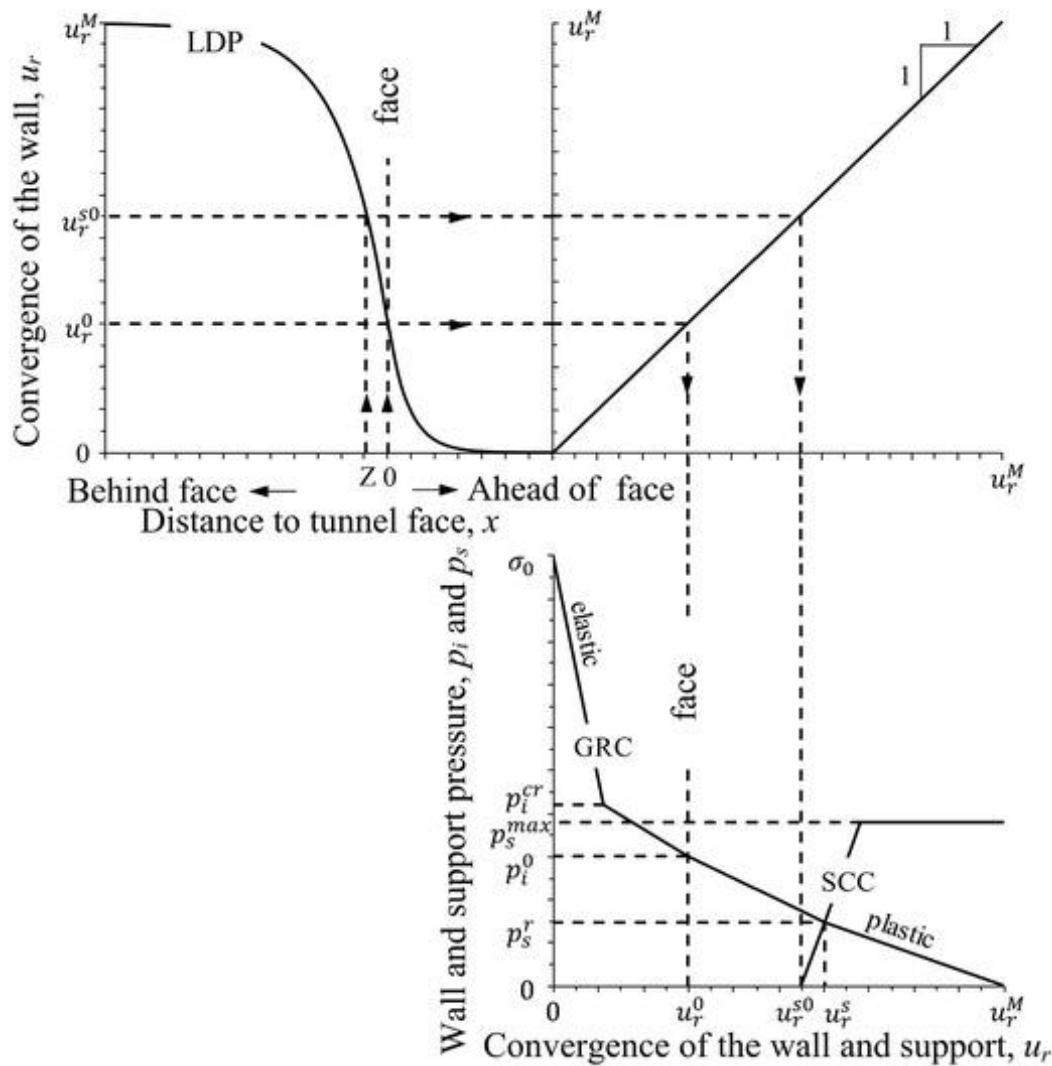


Figure 2.5 Schematic representation of the convergence-confinement method (Zhang et al., 2020)

The convergence-confinement method offers a dimensionless coefficient  $\lambda$  representing the relaxation (reduction) of the stresses in the tunnel walls at each excavation stress. This parameter is regarded as a constant number in previous studies. The parameter  $\lambda$  has an incisive effect on the generated stresses and deformations around the tunnel. The effects of various factors, such as strength parameters of the host ground and the depth of the tunnel, are not fully taken into account (Mousivand, 2018).

The convergence-confinement method was first established by studying a simple axisymmetric case of a deep tunnel with a hollow circular section in an isotropic elastic medium, where the initial stresses are hydrostatic. It makes it possible to deal with the three-dimensional problem of the interaction between the solid mass and the support by an equivalent plane strain problem (Panet and Guenot, 1982).

The field of displacements created by the excavation was studied by numerical models. At a distance  $x$  from the face, the radial displacement of the walls of the excavation is given by the expression:

$$u_R(x) = \lambda(x) u_R(\infty) \quad (2.1)$$

The variation of the deconfinement factor  $\lambda$ , as a function of  $x$ , behind the core face, is given by the following expression:

$$\lambda(x) = 0.28 + 0.72 \left[ 1 - \frac{0.84R}{x + 0.84R} \right] \quad (2.2)$$

Where,  $R$ : the equivalent radius of the tunnel.

The deconfinement factor,  $\lambda$ , is considered equal to 0.28 for  $x = 0$  (at the core face) and converges asymptotically towards 1 when  $x$  becomes infinite.

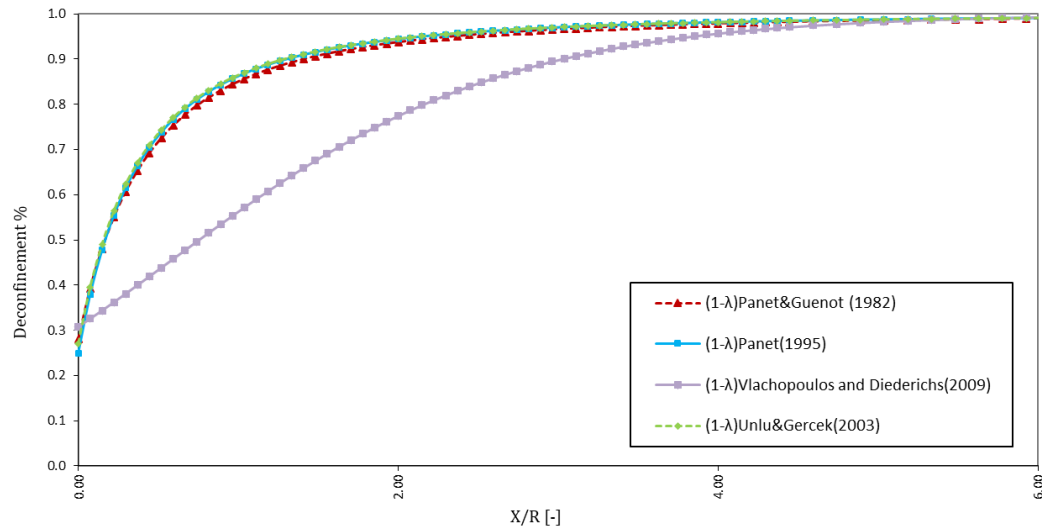


Figure 2.6 Radial displacement path of a point located on the surface of a circular tunnel

Among the several research results, the radial displacements are proportioned with the final elastic plane strain value. The comparison of a number of approaches is visualized in Figure 2.6.

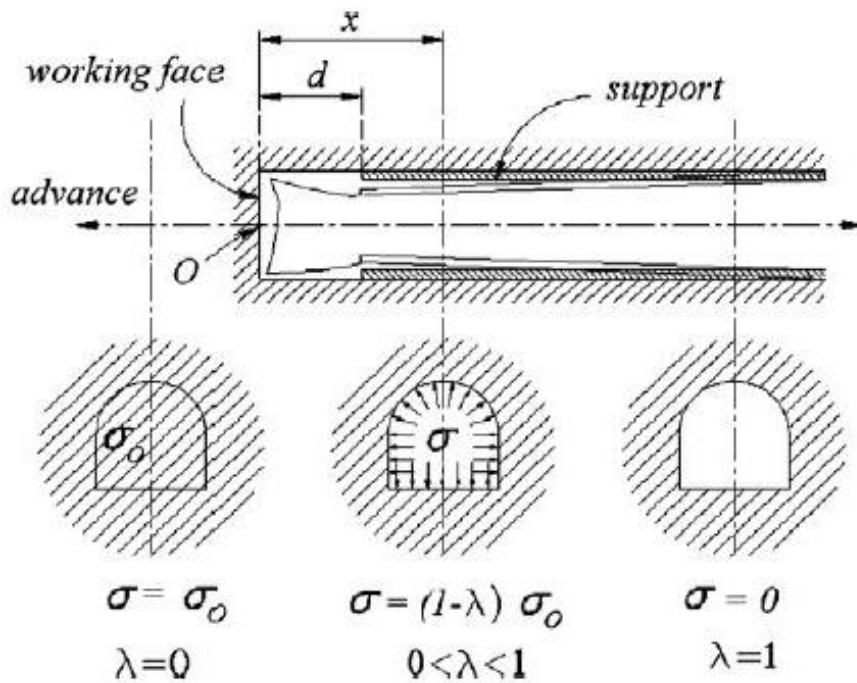


Figure 2.7 Confinement loss in a tunnel (González-Nicieza et al., 2008)

Instead of three-dimensional advancement case, the CCM approaches the problem as being one of the two-dimensional plane deformations that study the ground/support interaction (AFTES, 2002). The simulation of the fictitious stresses,  $\sigma$ , exerted to the walls of the excavation, are given by:

$$\sigma = (1 - \lambda) \sigma_0 \quad (2.3)$$

Where  $\sigma_0$  is the natural stress of the host ground and  $\lambda$  is the deconfinement factor. This parameter varies from 0 to 1, simulating the effect produced by the excavation (Figure 2.7).

### 2.3 Squeezing Ground Conditions

Squeezing of the host ground is the large time-dependent deformations that occur around the tunnel geometry. The time-dependent deformations are produced by the disturbance of the primitive stress field due to the excavation of the tunnel. The rock mass around the tunnel boundaries is strained under the influence of induced stresses. The deformations may terminate during construction or continue over a long period of time (Barla, 1995).

In the landmark paper on tunneling by Karl Terzaghi (1946), the prerequisite of the squeezing ground conditions is defined as the high percentage of microscopic or sub-microscopic particles of micaceous minerals or of clay minerals with low swelling capacity. Following the definitions of Terzaghi, several approaches are identified by various authors to assess the potential of squeezing problems in tunneling.

Hoek and Marinos (2000) indicated that the relationship of tunnel strain with the ratio of uniaxial compressive strength of rock mass to the in-situ stress state is indicative of predicting the squeezing problems of tunnels.

Based on the observations and case histories of several tunnels located in Venezuela, India, and Taiwan, Hoek (2000) proposed a curve to be used to estimate the degree of squeezing (Figure 2.8).

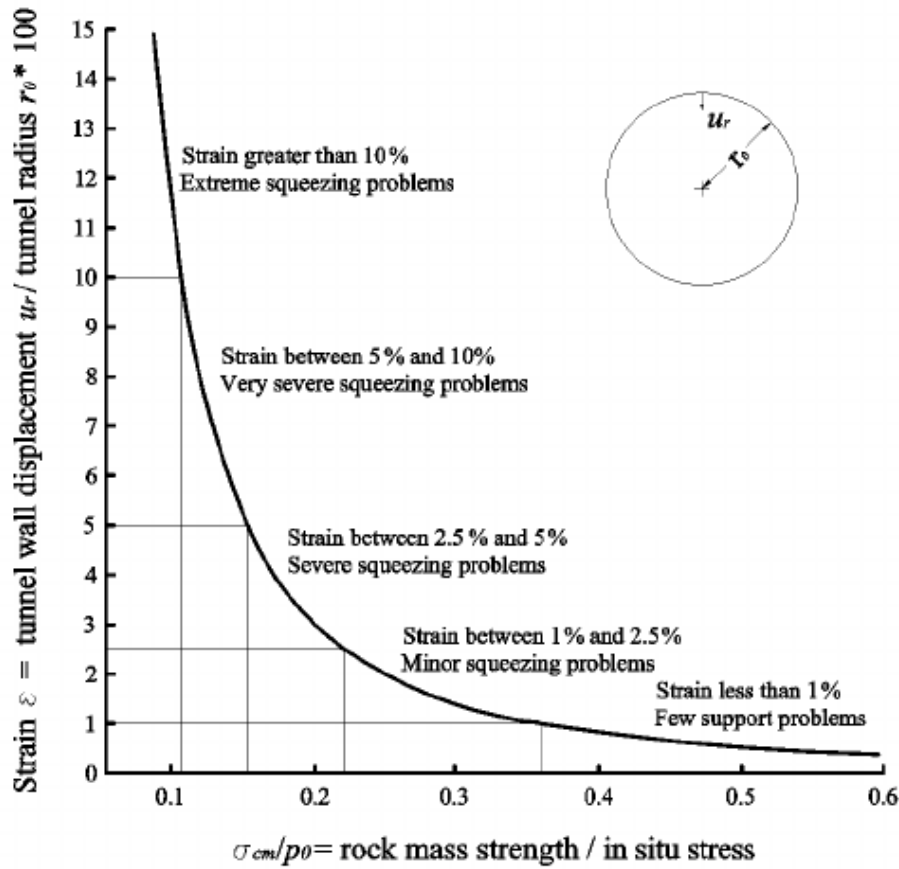
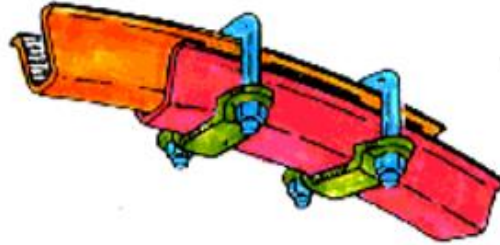


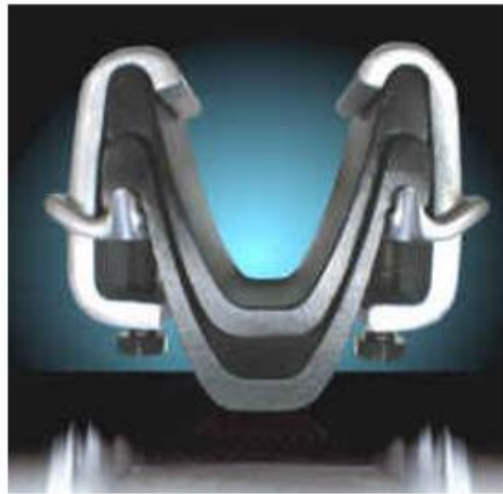
Figure 2.8 Classification of the squeezing behavior (Hoek, 2000)

The Yacambú-Quibor tunnel has a peculiar background in tunnel engineering in predicting the squeezing ground conditions. The tunnel water transmission tunnel was designed as a 5.5 m span 25 km tunnel being excavated through the Andes near the city of Barquisimeto in Venezuela. The maximum overburden observed on the tunnel is 1270 m, and the considerable proportion of the host ground through which it is advanced is graphitic phyllite. The construction of the tunnel started in 1975 and broke through in July 2008. In 1979 a TBM was trapped due to the squeezing ground conditions during a stoppage in the drive. The tunnel boring machine could not be mobilized further, and the squeezing ground occupied all the cavities in the machine structure gradually. The remains of this TBM were removed several years later. The complete closure of the tunnel occurred in several locations. The tunnel stability was retained by the installation of steel profiles with sliding joints (Figure 2.9) with a

specific amount of convergence to be locked. These steel sets were fully embedded in shotcrete apart from the gaps having a width of 1 m over the sliding joints.



(a)



(b)

Figure 2.9 (a) assembly of a sliding joint in a steel set; (b) cross-section detail (Barla, 2001)

After the tunnel advancement of 5 to 10 m, the sliding joints had closed, and the primary lining started to carry the load applied from the surrounding ground. The gaps were filled with shotcrete to complete the primary lining. After the application of the technique, the convergence measurements indicated that the sections were retained the stability, and the performance of these support measures was satisfactory (Sánchez Fernández and Terán Benítez, 1994).



## **CHAPTER 3**

### **CASE STUDY: A93 TUNNEL**

#### **3.1 Introduction**

Tunnel A93 is one of the tunnels under construction to serve in a high-speed railway line, with a significance to link one of the main junctions in the transportation system of Turkey. It is a single tube 6100 meters long tunnel with an approximate area of 141 m<sup>2</sup>. The tunnel construction has begun according to the principles of TBM. However, due to excessive deformations, a partial collapse has occurred after the tunnel advancement of approximately 1020 m; thus, the tunnel advancement was suspended. After the incident of collapse at KM: 217+280, in which the TBM was trapped under the host ground, the tunnel excavation has proceeded with an alteration in the construction technique. Consequently, the tunnel advancement has been chosen to progress complying with the NATM fundamentals. In the scope of this study, solely the collapse area is taken into account.

In this chapter, the geological and geotechnical characteristics of the host ground are assessed utilizing the available borehole data and laboratory test results. Thereafter, the construction methods of the tunnel are presented in detail. Following the discussion on the construction methods, the monitoring system of the tunnel is referred.

#### **3.2 Geological and Geotechnical Aspects**

According to the field report, the observations in field and borehole samples show that the metapelites are the most common unit on the tunnel route. They consist of gray-colored phyllites and intercalated gray-colored metasandstones, and metasiltsstones. Although rare, meta conglomerate levels are observed in this

intercalation. The phyllites are dark gray, gray-colored, dense foliated, brittle, and fragile, with locally crushed zones and broken zones, showing very weak to weak rock characteristics. The borehole photos indicate that the phyllites show a declining degradation profile from the surface to greater depths. Generally, in areas close to the surface and in areas where the effect of groundwater is more intense (under stream beds and valley floors), moderate to high degradation is observed, while the degree of degradation towards lower elevations gradually decreases. In general, the dominant foliation direction is towards the north, but local differences are observed due to frequent folding and faulting. Foliation angles show a wide distribution between horizontal and vertical, while the dominant slope amount is in the range of 45-60°. Foliation surfaces are wavy and smooth; polished surfaces are seen in places. Since the phyllites are easily crushed in areas close to folds and faults, they appear in dispersed forms. Such shearing and crushed cuts acquire a blackish color; therefore, they resemble graphitic schist. The most problematic unit along the alignment of the tunnel is observed as extremely weathered, tectonized, and highly jointed graphite schists.

Collapses occurred inside of the tunnel cavity during TBM advancement in the tunnel. The deformation caused by the collapse reached from the tunnel level to the surface, and three sinkholes of different sizes were formed on the surface. This situation shows that the rock mass is severely degraded and deformed in the section where the last point reached by the TBM.

### **3.2.1 Site Investigations**

Within the scope of the exploration program, seven boreholes were performed in the section between the project KM: 216+260 and KM:217+700. The depths of the boreholes were not identical. Each borehole has an individual depth, varying from 62.6 to 165.0m. The total depth of the boreholes in that area is about 955.0m. The plan view of the locations of boreholes is provided in Figure 3.1, and the profile view of them is provided in Figure 3.2.



Table 3.1 List of Boreholes in the Collapse Area

| <i>No</i> | <i>Borehole</i> | <i>Chainage</i> | <i>Elevation (m)</i> | <i>Depth (m)</i> | <i>Date</i> |
|-----------|-----------------|-----------------|----------------------|------------------|-------------|
| 1         | SK-216+840      | 216+840         | 552.3                | 70.0             | 31.03.2010  |
| 2         | FSK-1           | 217+060.35      | 625.0                | 165.0            | 17.11.2016  |
| 3         | KSK-5           | 217+183         | 655.7                | 62.6             | 17.02.2013  |
| 4         | KSK-7           | 217+315         | 655.7                | 165.3            | 24.02.2013  |
| 5         | KSK-9           | 217+227         | 665.0                | 165.0            | 14.03.2013  |
| 6         | SK-217+340      | 217+340         | 658.5                | 165.0            | 03.04.2010  |
| 7         | SK-217+700      | 217+700         | 696.7                | 195.0            | 30.05.2010  |

The groundwater levels were measured in some of the boreholes. The groundwater levels obtained from the measurements demonstrate that the groundwater levels vary from 8 m to 95 m above the top elevation of the tunnel geometry. At the area of interest, the groundwater level was measured approximately 80 m above the crown of the tunnel.

According to the site investigation report, the project alignment is located within Triassic aged semi-metamorphic rocks. In general, rock units consisting of phyllites, graphite schists, and chlorite schists are impermeable. Therefore, it was expected that no significant amount of water would leak into the tunnel. However, considering that the alignment is located in a highly tectonic area, the rocks are sheared by various faults. The fracture systems and the presence of a little amount of flow are expected along these fracture zones. During the tunnel advancement, water flow in the form of leakage or a small amount of flow could be expected along joints and fracture zones.

Table 3.2 Groundwater levels according to measurements

| <i>No</i> | <i>Borehole</i> | <i>Chainage</i> | <i>Elevation<br/>(m)</i> | <i>Depth<br/>(m)</i> | <i>GWL Elevation<br/>(m)</i> |
|-----------|-----------------|-----------------|--------------------------|----------------------|------------------------------|
| 1         | SK-216+840      | 216+840         | 552.3                    | 70.0                 | 497.3                        |
| 2         | FSK-1           | 217+060.35      | 625.0                    | 165.0                | 545.0                        |
| 3         | SK-217+340      | 217+340         | 658.5                    | 165.0                | 595.5                        |

### 3.2.2 Properties of the Soil and Rock Mass

Laboratory tests were performed on the samples obtained from the conducted borehole applications, based on the methods specified in ISRM (2007). In the area of interest, a total of 34 uniaxial compressive strength tests, 5 point load tests, and 41 unit weight tests were performed. The parameters of unit weight (Figure 3.3), uniaxial compressive strength (Figure 3.4), modulus of elasticity (Figure 3.5), and Poisson's ratio (Figure 3.6) of the rocks were determined with these experiments.

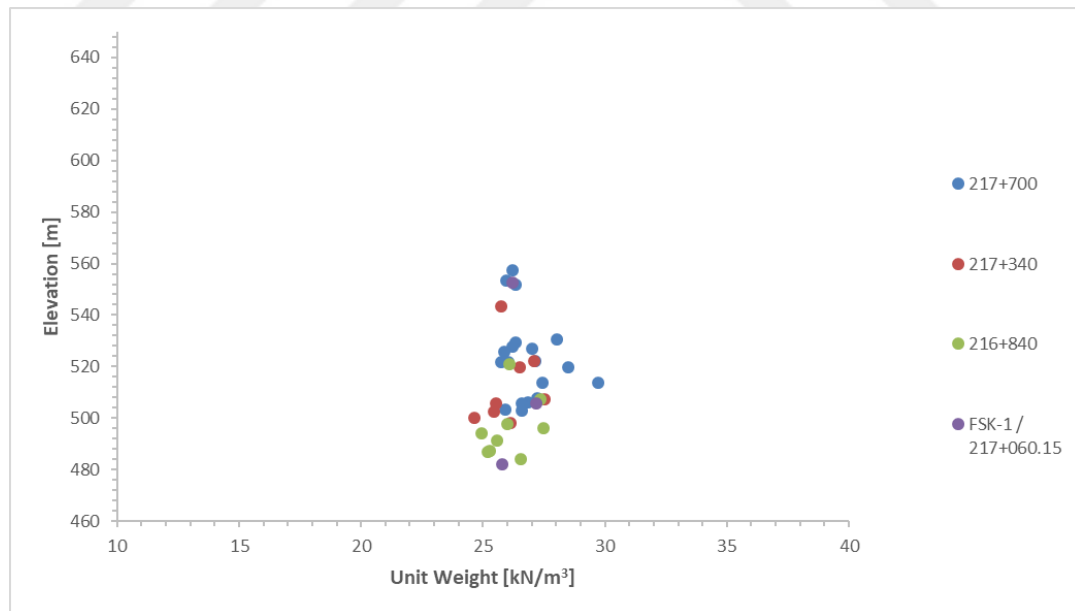


Figure 3.3 Variation of unit weight values with elevation

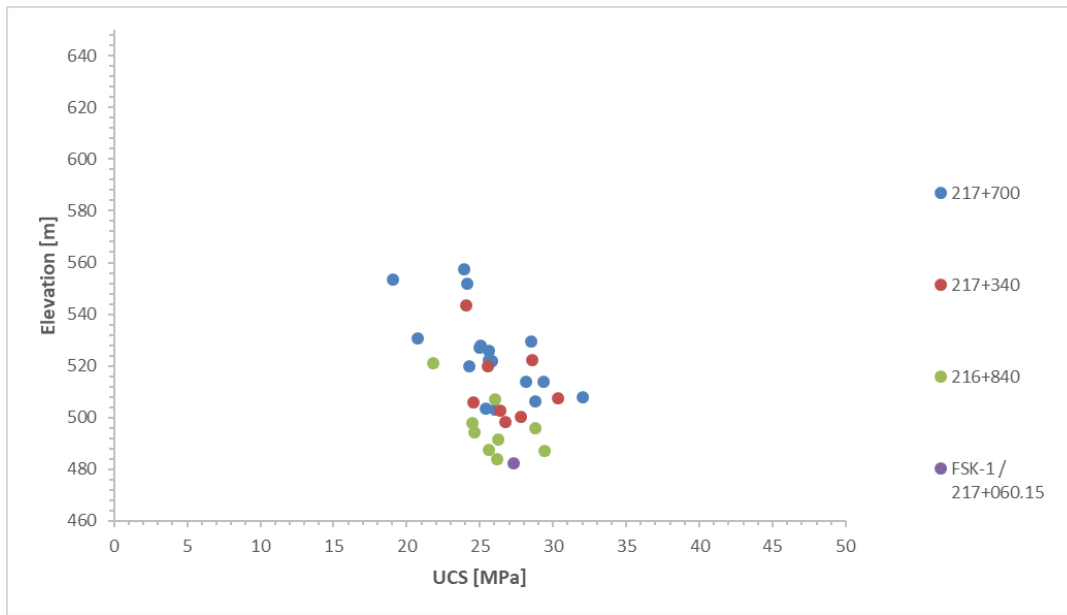


Figure 3.4 Variation of Uniaxial Compressive Strength values with elevation

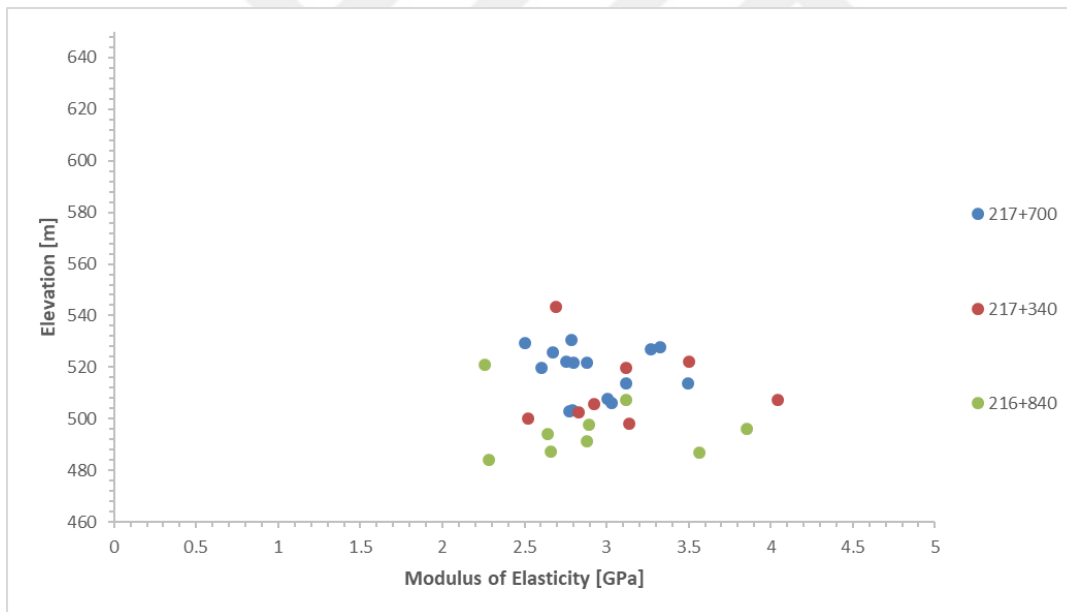


Figure 3.5 Variation of Modulus of Elasticity values with elevation

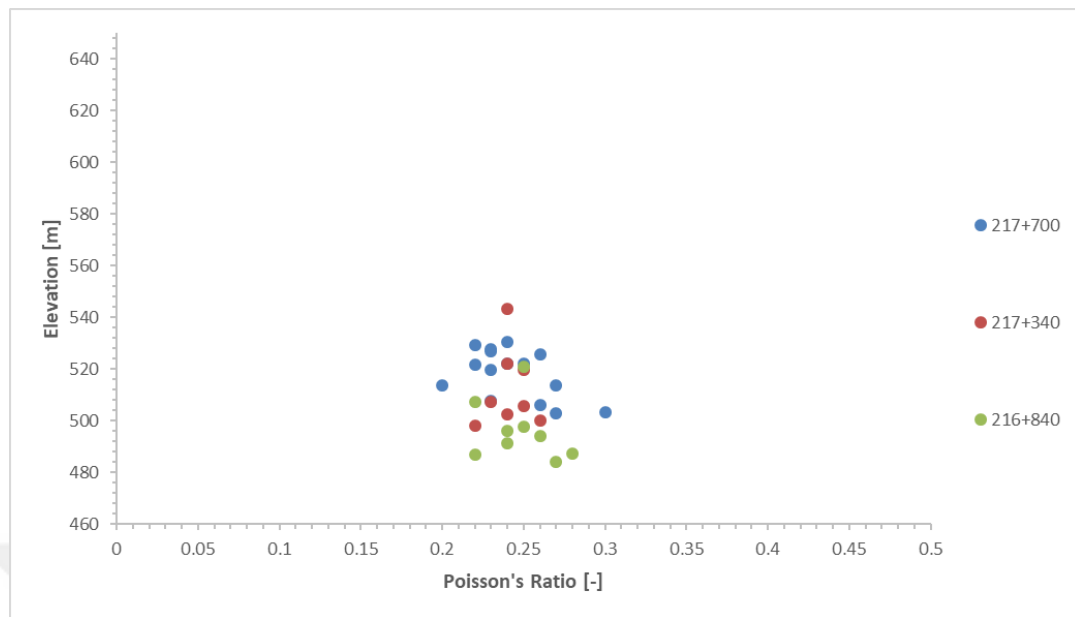
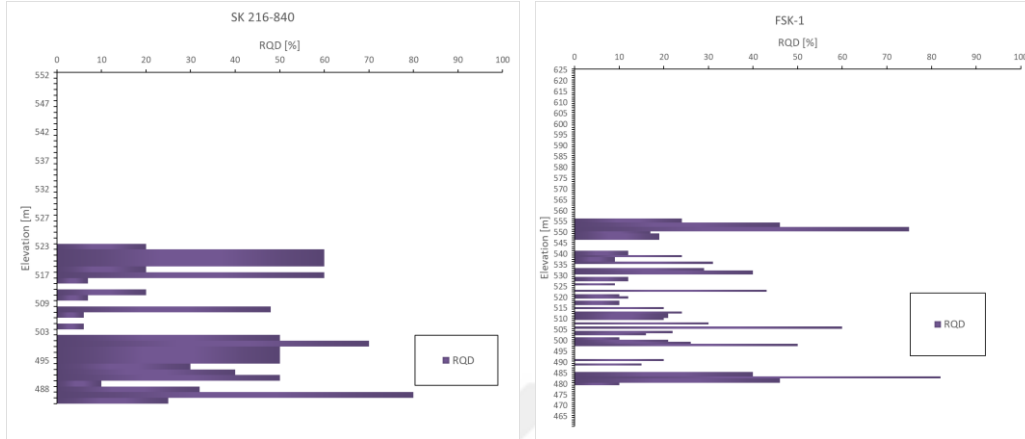


Figure 3.6 Variation of Poisson's Ratio values with elevation

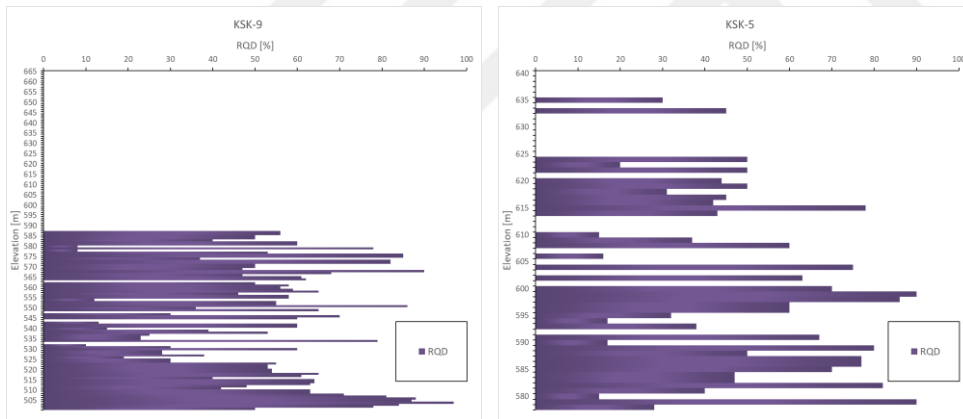
Based on the laboratory test results on the samples obtained from the boreholes, in the area of interest, the geotechnical properties of the soil associate a lack of increase with elevation. This peculiar phenomenon may be a consequence of the fault zones situated at the domain. The disturbance to the soil or rock beneath the ground caused by the fault motions could be an effective agent to constitute a relatively stable trend in the variation of strength parameters with depth.

The unit weight value of rocks is related to their composition. In practice, rocks with high unit volume weights generally have low porosity, low water absorption, and high specific gravity values. When the average unit weights obtained from the studied rocks are classified according to the work of Stille & Palmström (2015), they are defined as rocks with medium unit volume weight.

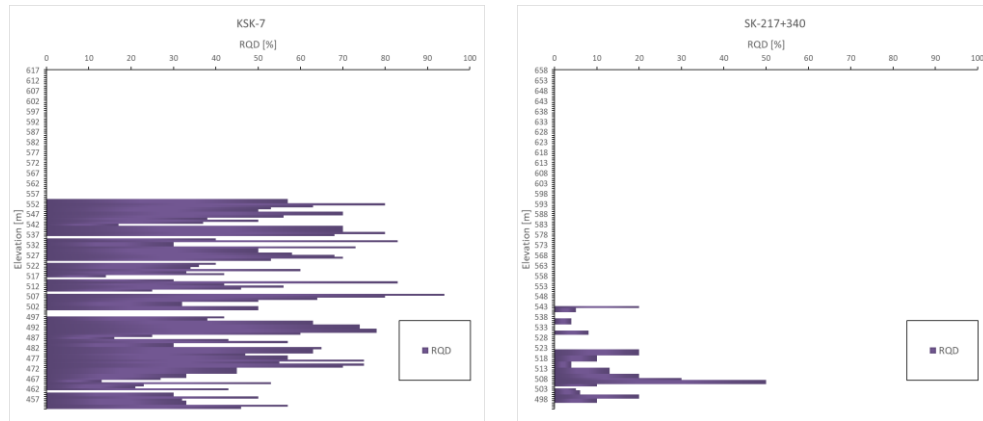
(a)



(b)



(c)



(d)

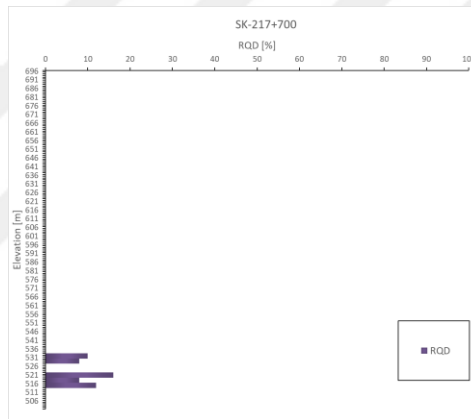


Figure 3.7 Distribution of RQD values (a) SK-216+840 and FSK-1, (b) KSK-9 and KSK-5, (c) KSK-7 and SK-217+340, (d) SK-217+700

Determining the mechanical properties of rocks is a significant step in assessing the strength and deformability behavior. According to the data obtained in this study, the compressive strength values were obtained in the range of 19.07-32 MPa in the average for metapelites, which were observed in the specified collapse area. According to Deere and Miller (1966), the investigated rocks fall in the “very low-low” strength rock class. When these rocks are classified considering their Modulus of Elasticity values, according to Deere and Miller (1966), they are classified as rocks with a “very low” Modulus of Elasticity. Geological characteristics of the

region and weathering are considered to be the most influential factors leading to low engineering strength values.

The geotechnical parameters of the metapelite unit, determined from the laboratory test results, are summarized in Table 3.3. These parameters are employed in the analyses, by which before the collapse performance is assessed.

Table 3.3 Geotechnical Parameters Based on Laboratory Test Results

| <i>Depth [m]</i> | <i>Parameter</i>                    | <i>Lowest Value</i> | <i>Highest Value</i> | <i>Mean <math>\pm</math> Standard Deviation</i> |
|------------------|-------------------------------------|---------------------|----------------------|-------------------------------------------------|
| 460-620          | Unit Weight<br>[kN/m <sup>3</sup> ] | 24.09               | 29.72                | 26.34 $\pm$ 1.12                                |
| 520-620          | UCS [MPa]                           | 19.07               | 29.40                | 24.06 $\pm$ 3.84                                |
| 460-520          |                                     | 21.83               | 32.00                | 26.63 $\pm$ 2.76                                |
| 460-620          | Modulus of<br>Elasticity [GPa]      | 2.25                | 4.04                 | 2.93 $\pm$ 0.45                                 |
| 460-620          | Poisson's Ratio [-]                 | 0.20                | 0.30                 | 0.24 $\pm$ 0.02                                 |

Another critical parameter is Geological Strength Index (GSI). According to the ground investigation report, the GSI value is calculated by the studies conducted on the borehole logs and outcrops encountered in the field. Therefore, the modified GSI system proposed by Sönmez and Ulusay (2002) (Figure 3.8) was utilized.

Structural properties were calculated by considering the number of volumetric joints in the calculation of the GSI value. Accordingly, the roughness rating ( $R_r$ ) was taken as 0 due to the slippery surface of the discontinuities. The weathering rating ( $R_w$ ) was taken as 3 due to moderate deteriorations. Lastly, the infilling rating ( $R_f$ ) was taken as 6 due to the lack of filling. The total Surface Condition Rating (SCR) score was calculated as 9 by taking the summation of these three scores.



given in Figure 3.8. Although the GSI value was calculated approximately as 28, the design value of GSI was taken as 25 to be on the safe side.

|             | Rock type  | Class             | Group                                         | Texture                              |                                                    |                                    |                                       |
|-------------|------------|-------------------|-----------------------------------------------|--------------------------------------|----------------------------------------------------|------------------------------------|---------------------------------------|
|             |            |                   |                                               | Coarse                               | Medium                                             | Fine                               | Very fine                             |
| SEDIMENTARY |            | Clastic           |                                               | Conglomerates<br>*                   | Sandstones<br>17 ± 4                               | Siltstones<br>7 ± 2                | Claystones<br>4 ± 2                   |
|             |            |                   |                                               | Breccias<br>*                        |                                                    | Greywackes<br>(18 ± 3)             | Shales<br>(6 ± 2)<br>Marls<br>(7 ± 2) |
|             |            | Non-Clastic       | Carbonates                                    | Crystalline<br>Limestone<br>(12 ± 3) | Sparitic<br>Limestones<br>( 10 ± 2)                | Micritic<br>Limestones<br>(9 ± 2 ) | Dolomites<br>(9 ± 3)                  |
|             |            |                   | Evaporites                                    |                                      | Gypsum<br>8 ± 2                                    | Anhydrite<br>12 ± 2                |                                       |
|             |            |                   | Organic                                       |                                      |                                                    |                                    | Chalk<br>7 ± 2                        |
| METAMORPHIC |            | Non Foliated      |                                               | Marble<br>9 ± 3                      | Hornfels<br>(19 ± 4 )<br>Metasandstone<br>(19 ± 3) | Quartzites<br>20 ± 3               |                                       |
|             |            | Slightly foliated |                                               | Migmatite<br>(29 ± 3)                | Amphibolites<br>26 ± 6                             | Gneiss<br>28 ± 5                   |                                       |
|             |            | Foliated**        |                                               |                                      | Schists<br>12 ± 3                                  | Phyllites<br>(7 ± 3)               | Slates<br>7 ± 4                       |
| IGNEOUS     | Plutonic   | Light             | Granite<br>32 ± 3<br>Granodiorite<br>(29 ± 3) |                                      | Diorite<br>25 ± 5                                  |                                    |                                       |
|             |            | Dark              | Gabbro<br>27 ± 3<br>Norite<br>20 ± 5          | Dolerite<br>(16 ± 5)                 |                                                    |                                    |                                       |
|             | Hypabyssal |                   | Porphyries<br>(20 ± 5)                        |                                      | Diabase<br>(15 ± 5)                                | Peridotite<br>(25 ± 5)             |                                       |
|             | Volcanic   | Lava              | Rhyolite<br>(25 ± 5)<br>Andesite<br>25 ± 5    |                                      | Dacite<br>(25 ± 3)<br>Basalt<br>(25 ± 5)           |                                    |                                       |
|             |            | Pyroclastic       | Agglomerate<br>(19 ± 3)                       | Breccia<br>(19 ± 5)                  | Tuff<br>(13 ± 5)                                   |                                    |                                       |

Figure 3.9 Intact rock strength constant ( $m_i$ ) values for different rock types (Marinos and Hoek, 2000)

The last parameter to successfully satisfy the requirements for the application of the Hoek-Brown material model is the material-dependent constant  $m_i$ . This constant is

related to the frictional characteristics of the rock material. The material constant  $m_i$  can be specified by the application of triaxial tests. However, the value of  $m_i$  is estimated practically from a table of typical values. As previously mentioned, the rock unit is defined as extremely weathered, tectonized, and highly jointed graphite schists. Hoek and Marinos (2000) suggested that the  $m_i$  value of schists can be classified as 12 (Figure 3.9). However, it is underlined that this value is for intact rock specimens tested normal to bedding or foliation. In case of a failure along a weakness plane, the value of  $m_i$  would diverge from the suggested rating. According to the investigation report, the material constant  $m_i$  was used as 12.

To determine the rock mass strength parameters of the surrounding ground considering the Hoek-Brown Failure Criterion RocLab software is used. RocLab is a software by RocScience Inc., which is used to correlate Hoek-Brown and Mohr-Coulomb strength parameters. Therefore, the rock mass strength parameters are specified for both of the failure criteria. The summary table of the rock mass strength parameters is provided in Table 3.4.

Table 3.4 Geotechnical Parameters Employed in FE Analyses

| <i>Depth</i> | $\gamma$             | UCS   | E     | $\nu$ | GSI | $m_i$ | c     | $\phi$ |
|--------------|----------------------|-------|-------|-------|-----|-------|-------|--------|
| [m]          | [kN/m <sup>3</sup> ] | [MPa] | [GPa] | [-]   | [-] | [-]   | [kPa] | [°]    |
| 0-100        | 26.3                 | 24.0  | 2.9   | 0.24  | 25  | 12    | 277.0 | 36.7   |
| 100-250      | 26.3                 | 26.0  | 2.9   | 0.24  | 25  | 12    | 386.0 | 33.8   |

### 3.3 Construction of A93 Tunnel

The construction of the tunnel started in 2011. The TBM excavation of the A93 Tunnel started from KM:216+500. Approximately 1020 m of the tunnel was excavated by a TBM. However, during the tunnel advancement, excessive deformations were observed at the tunnel linings, starting from KM: 217+300. Due to the increasing trend of the deformations observed at the precast segments, the tunnel advancement was halted at KM: 217+525. Following the delay in the

excavation process, these excessive deformations caused a failure in the segments inside of the tunnels. Even more, on the ground surface, sinkholes were observed, located at the surface projection of partial collapse (located at approximately KM: 216+900 and KM: 217+010, Figure 3.1).

Consequently, continuous reinforcement attempts were carried out, and TBM made approximately 250 m of tunnel advancement. However, the machine was failed to move further due to the highly squeezing ground conditions. As a result, the construction works were ceased, and the construction method was altered. Therefore, in the area of collapse occurrence, the tunnel advancement was carried out by the sequential excavation method.

### **3.3.1 Construction with TBM**

Tunnel construction has started according to the TBM principles. A single shield, which is usually chosen to excavate hard rocks where geological discontinuities are frequently observed (Maidl et al. 2008), 13.77m diameter TBM with the ability to make a full circular excavation was mobilized for the project. The general layout and geometrical definition of the segments are provided in Figure 3.10.

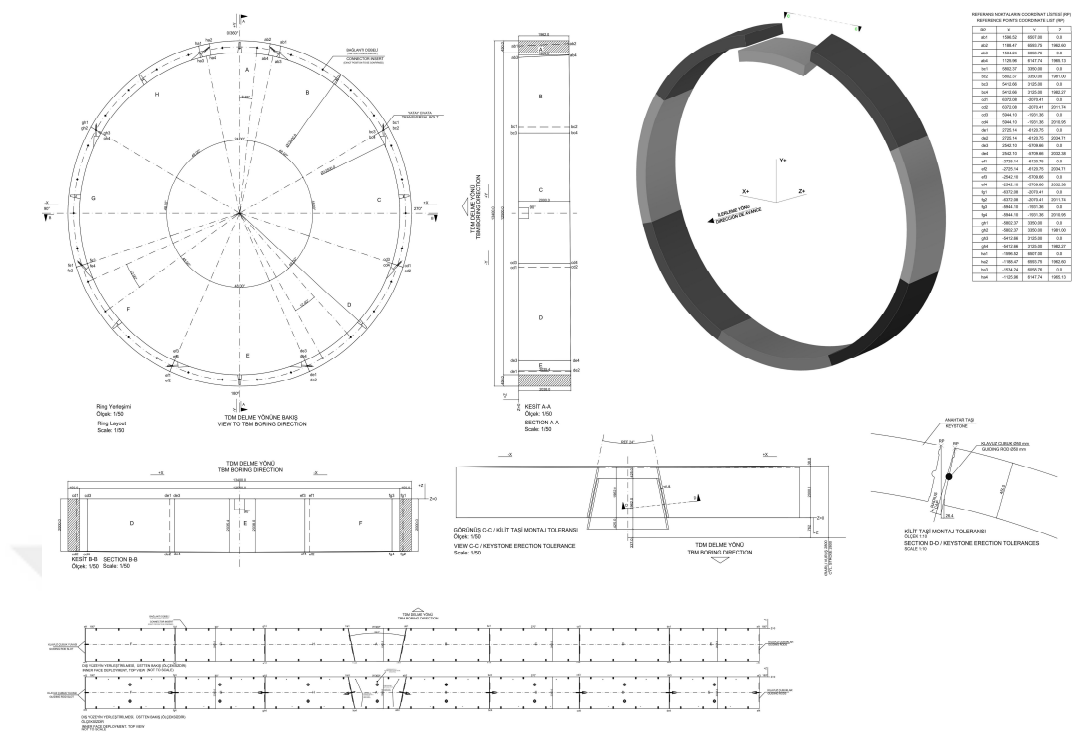


Figure 3.10 Geometrical Definition of Segments (Tunnelconsult)

A total of 8 segments, including a keystone (7+1), were utilized as the tunnel's segmental lining. The TBM segments were produced complying with the universal ring principles. The general properties of the linings are provided in Table 3.5.

Table 3.5 General Properties of the TBM Lining Segments

| Type of Segments                     | Universal Ring |
|--------------------------------------|----------------|
| Length                               | 2000mm         |
| Thickness                            | 450 mm         |
| External Diameter                    | 13,400 mm      |
| Internal Diameter                    | 12,500 mm      |
| Total Number of Segments in One Ring | 7+1            |
| Weight of a Segment                  | 6100kg         |
| Concrete Class                       | C40/50         |
| Steel Reinforcement Class            | S420           |
| Minimum Concrete Cover               | 30 mm          |

### 3.3.2 Construction with NATM

After the collapse in the TBM excavated sections, the tunnel construction method was changed from TBM to NATM. Complying with the NATM construction philosophy, the sequential excavation technique has been implemented for the construction of the A93 Tunnel. The tunnel cross-section was excavated in three phases: top-heading excavation, lower bench excavation, and the invert excavation.

In the collapsed area, where the NATM tunnel passes through, the geological units consist of the dark gray colored, densely foliated, locally crushed zoned, medium weathered, very weak to weak phyllites - graphite schists.

Due to the excessive deformations observed during the tunnel advancement, the TBM was stuck at KM: 217+280. At this section of the project route, failure and relaxation zones have occurred around the tunnel excavation. Therefore, the rock conditions in this section have been considered that they completely lost their original strength properties. Thus, in the design stage, the ground is assumed as flowing ground conditions. For this reason, conforming with the NATM Rock Mass Classes (Geoconsult, 1993 and ÖNORM B 2203, 1994), C4 Flowing Rock Class was chosen as the excavation class in this section (Figure 3.11).

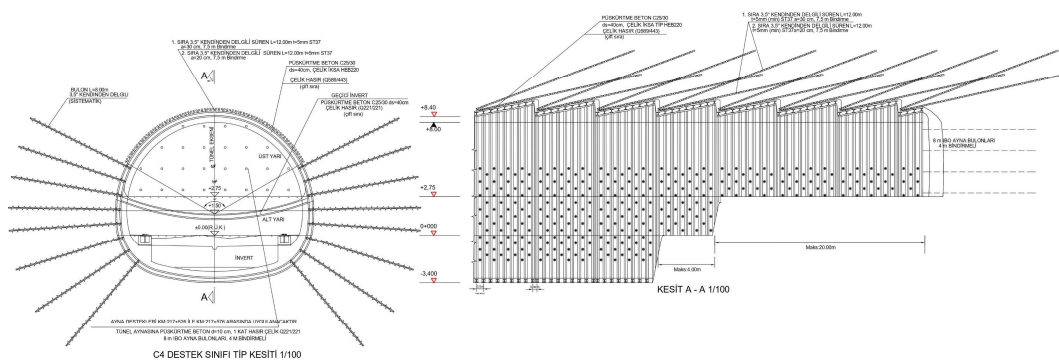


Figure 3.11 The Details of the C4 Support System (Fugro Sial)

The properties of the various support elements for the primary lining are given as follows:

- 40cm thick shotcrete (C25/30),
- HEB 220 steel ribs, having an interval of 50cm
- Double-row wire meshes Q589/443,
- Systematic 8m long  $\phi 32$ mm diameter self-drilling rock bolts (bore diameter: 3.5inches) with a grid of 0.50m (longitudinal direction) x 1.00m (transversal direction) starting from the elevation of +4.74m to -1.79m (Taking the top of rail elevation as 0.00m),
- Forepoling constitutes two rows of self-drilling pipes:
  - 1<sup>st</sup> row of forepoling: 5mm thick, 12.0m long, installed with 30cm interval, 7.5m overlapped, with a boring diameter of 3.5inches, aligned with 21°,
  - 2<sup>nd</sup> row of forepoling: 5mm thick, 12.0m long, installed with 20cm interval, 7.5m overlapped, with a boring diameter of 3.5inches, aligned with 13°,
- 8m long  $\phi 32$ mm diameter self-drilling rock bolts with 4m overlap, applied at the core face (Only applied at the top heading, a total number of 26 rock bolts are applied to a cross-section of 80.65 m<sup>2</sup>),
- 10cm thick shotcrete (C25/30) application to the core face reinforced with a single row of wire mesh,
- 40cm thick temporary invert, formed by shotcrete (C25/30) and double-row wire meshes (Q221/221).

After the primary lining application, the application of the supporting elements is completed with the installation of the final lining. The final lining was designed to have a 40cm thickness at the crown, and a variable thickness, from 40cm to 70cm, at the invert with a concrete class of C30/37. At the crown, double-row  $\phi 28/20$ cm steel reinforcements and double-row Q589/443 wire meshes are used to reinforce the concrete lining. Double rowed Q589/443 wire meshes were used as the reinforcement of the final lining at the invert.

### 3.4 Monitoring

The philosophy of NATM construction dictates a comprehensive monitoring program to detect the behavior of the surrounding rock mass as the tunnel advancement proceeds (Bieniawski, 1989). The monitoring of the deformations encountered in the primary support elements enables engineers to make sufficiently fast and economic revisions to the support system throughout the project cycle. An extensive monitoring program urges a continuous, systematic, and coherent measurement system.

The monitoring of the A93 Tunnel in the TBM method was performed in taking measurements in three axes related to the unit coordinate system. Additionally, the settlement of the TBM segments was measured.

However, in NATM, the monitoring in the A93 Tunnel is conducted by the use of optical reflectors installed on the tunnel lining in several sections (Figure 3.12).

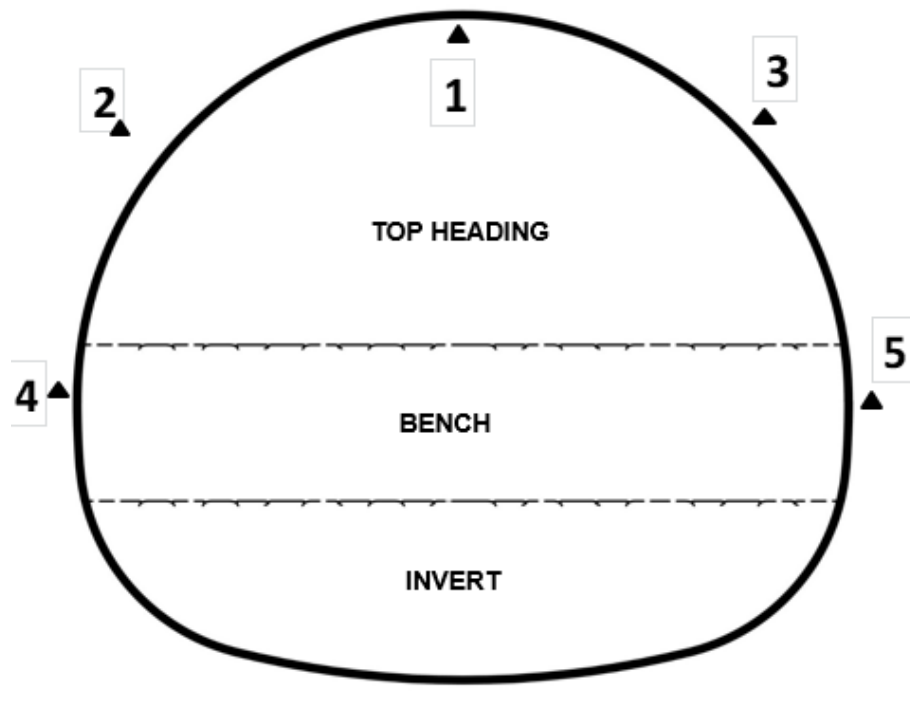


Figure 3.12 Optical measurement points

Measurements at these five provided a basis to evaluate the rock mass behavior observed in the field. This value does not ring an emergency warning at first glance. These five sections were selected from the area of interest, which is located between KM:217+280 and KM:217+530. The monitoring data of the chosen sections are provided in Figure 3.13 through Figure 3.17. The first section was chosen from the location that is behind the collapsed area, in which the excavation was completed by TBM. The following two sections indicate the monitoring data obtained before the incident of collapse, in which the excavation was conducted by a TBM. The last two of the total of five sections are devoted to the monitoring data of the sections obtained from the altered, sequentially excavated tunnel advancement method. The last two sections were selected from the available points closest to the points at which the second and third sections were located.

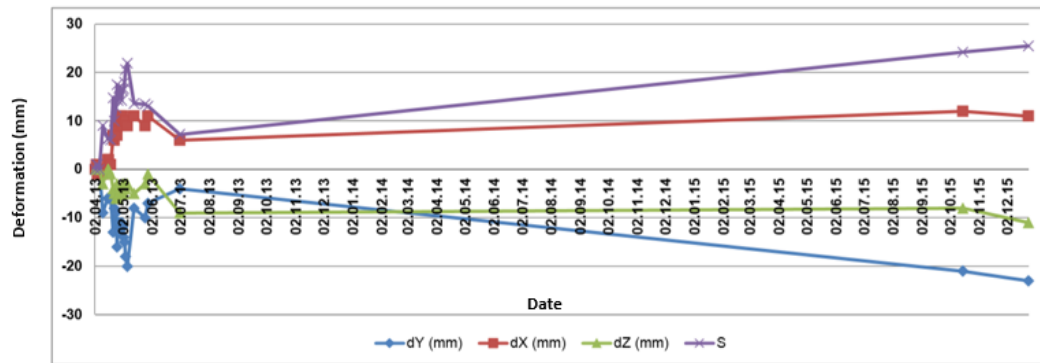


Figure 3.13 Monitoring data of KM: 217+265 with TBM excavation

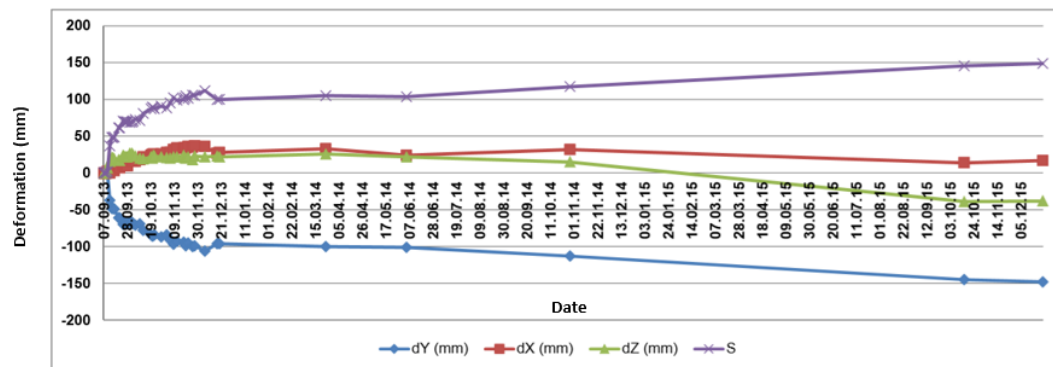


Figure 3.14 Monitoring data of KM: 217+378 with TBM excavation

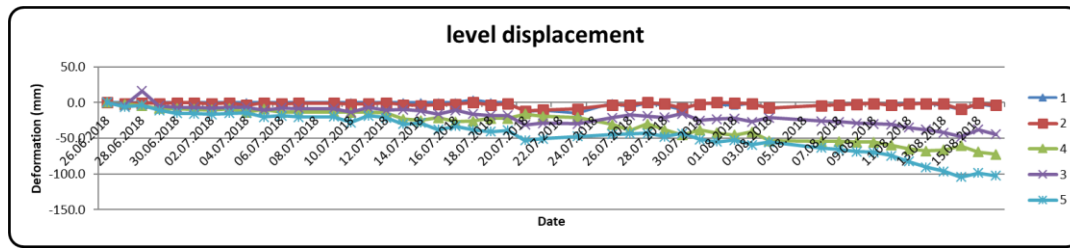


Figure 3.15 Monitoring data of KM: 217+373 with NATM excavation

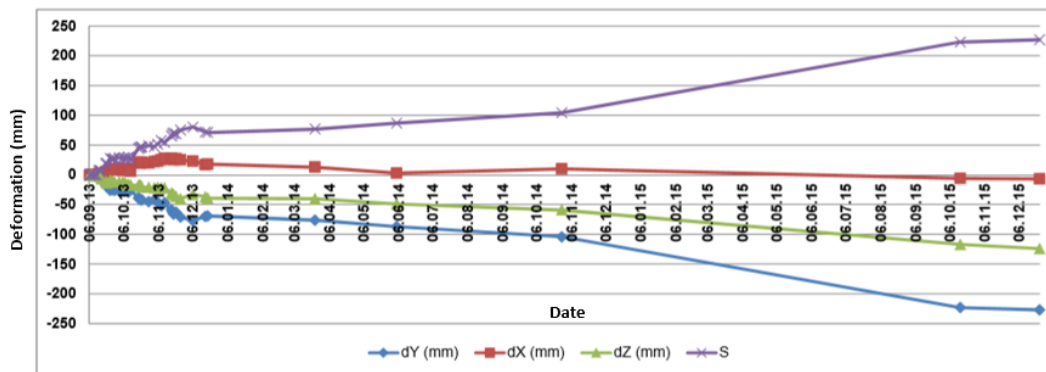


Figure 3.16 Monitoring data of KM: 217+395 with TBM excavation

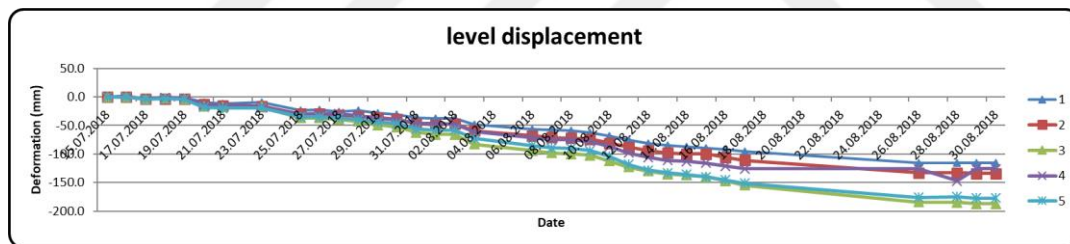


Figure 3.17 Monitoring data of KM: 217+385 with NATM excavation

In Figure 3.13, the settlement and deformation measurements indicate that the observed maximum deformation at the section is around 25mm. This value does not ring an emergency warning at first glance. The monitoring data presented in Figure 3.14 indicate that the maximum displacement at the segments reaches approximately 150mm in this section, and in Figure 3.16, it is interpreted that the displacements at the segments extend to approximately 231mm. The monitoring data obtained in Figure 3.15 and Figure 3.17 indicate that the displacements observed in the lining tend to decrease with the application of the alteration in the tunnel advancement method.

The monitoring data is another component that further investigations are based on in the scope of this study. The inputs and outputs of the numerical analyses presented in the next chapter are utterly related to the tunnel monitoring data evaluations.





## **CHAPTER 4**

### **METHODOLOGY OF THE NUMERICAL ANALYSES OF THE A93 TUNNEL**

#### **4.1 Introduction**

Although conventional design tools are comfortable and relatively quick to use and provide decent predictions, they provide uncoupled answers, in which the stresses are usually determined by elastic solutions and the initial displacements by empirical methods. In contrast, the real behavior of a tunnel, in general, is reflected in pore pressure changes, lining deformations, plastic deformations. The finite element technique provides a framework handling such complex scenarios such as modeling the excavation sequences, a complex ground condition in a more realistic manner, and reflecting the soil behavior, considering the effect of adjacent structures and even the cases with multiple tunnels. Therefore, finite element analyses provide a strong basis for implementing numerical methods in design (David and Zdravkovic, 2001).

This study focuses on an extensive assessment of the tunnel construction techniques, behavior of tunnel supports, and the ground reactions by comparing the observed field deformation data and interpretation of these data by utilizing software tools. First, a parametric study was performed to evaluate the conformance of the site investigation results with the actual response of the ground to the tunnel excavation and the implemented support system. To visualize the ground-support interactions, RocSupport software, by RocScience Inc., was used. Referring to the preliminary analyses conducted using RocSupport, 2D finite element analyses were performed using the geotechnical finite element software Plaxis 2D by Bentley Systems Inc.

The performed analyses are investigated in two individual chapters. The first chapter is devoted to the analyses performed for the case of the tunnel advancement executed with the principles of TBM construction techniques, while the second chapter

consists of the tunnel construction complying with the NATM techniques after the occurrence of the collapse.

The analysis conducted for the first case is referred to as Analysis 1. In Analysis 1, the expected behavior of the host ground and the support reactions in advancement with TBM are investigated considering the data obtained in the design stage. Subsequently, the accuracy of the expected behavior is assessed by combining the monitoring data acquired from the field.

The analyses performed to seek the compatibility of the design parameters and the real behavior observed during the construction are named Analysis 2, Analysis 3-Alternative A, and Analysis 4-Alternative A. Analysis 2 presents the case of tunnel response to TBM excavation in KM:217+265, in which the total deformation after the excavation is reached around 25mm. The strength parameters were attained by utilizing the field data. Analysis 3-Alternative A and Analysis 4-Alternative A are devoted to assessing the tunnel performance in TBM excavation in KM:217+378 and KM:217+395, respectively. The analyses are based on the deformation increments along the route, in which the expected total deformations in KM:217+378 and KM: 217+395 are 150mm and 231mm, respectively. These three analyses were performed by interpreting the monitoring data obtained during and after the construction of the tunnel. In these analyses, three individual cases presenting particular deformation trends were used.

The partial collapse incident led to a pause in the excavation progress and a change in the tunnel construction technique. Consequently, the altered construction method and the implemented solutions were analyzed in Analysis 3-Alternative B and Analysis 4-Alternative B. Analysis 3-Alternative B is devoted to assessing the tunnel advancement in NATM after the partial collapse in KM: 217+373. Based on the monitoring data, the expected maximum deformation in KM:217+373 with the NATM technique is 102mm. In Analysis 3-Alternative B, the strength parameters were obtained from the previously studied case, Analysis 3-Alternative A, in which the analyzed section is closely located to KM:217+373. The other analysis, Analysis

4-Alternative B, investigating the altered construction method, presents the assessment of tunnel advancement after the partial collapse in KM: 217+385. The expected maximum deformation in KM:217+385 with the NATM technique is 186mm, based on the monitoring data. In Analysis 4-Alternative B, the strength parameters were obtained from the previously studied case, Analysis 4-Alternative A, in which the analyzed section is closely located to KM:217+395. These two analyses address the tunnel performance with the altered construction method. The verification of the proposed technique and analysis results are demonstrated with the observed field data.

## **4.2 Modeling of Tunnel Advancement with TBM**

The construction of the A93 Tunnel was started in 2011 with a TBM designed to excavate the 6100m long tunnel route. The properties of the machine are given in Table 3.5. Initially, the assessment of the support design was carried out by using RocSupport software. In the design stage, the geotechnical parameters were determined from the data acquired from the existing boreholes and laboratory tests performed on the samples prepared from the related boreholes. A total of four analyses are performed in this section, namely, Analysis 1, Analysis 2, Analysis 3-Alternative A, and Analysis 4-Alternative A. In the next chapters, the modeling procedure of these analyses is explained in detail.

### **4.2.1 Ground-Support Reactions**

As a first step, the visualization of the ground-support reaction curve was executed by utilizing RocSupport software. The Duncan Fama solution, among the several methods that the software presents, is based on the Mohr-Coulomb Failure Criterion. This approach assumes that the surrounding jointed rock mass behaves as elastic-perfectly plastic material, in which the zero plastic volume change occurrence is assumed in a failure involving slip along intersecting discontinuities (Duncan Fama,

1993). Therefore, in this study, since the obtained rock mass parameters are compatible with the Mohr-Coulomb Failure Criterion, the Duncan Fama solution method was chosen.

The maximum overburden in the collapse area was determined as 160m. Therefore, the tunnel and soil parameters that are used in the creation of the ground-support reaction curve are tabulated for each case (Table 4.1- Table 4.4).

Table 4.1 Parameters Employed in Analysis 1

| <i>Parameter</i>                                 | <i>Results</i>                  |
|--------------------------------------------------|---------------------------------|
| In-situ Stress [MPa]                             | $p_0 = 160 \times 0.026 = 4.21$ |
| Modulus of Elasticity [MPa]                      | $E = 2900$                      |
| Poisson's Ratio [-]                              | $\nu = 0.24$                    |
| Uniaxial Compressive Strength of Rock Mass [MPa] | $\sigma_{cm} = 0.311$           |
| Friction Angle [°]                               | $\phi = 33.8$                   |
| Tunnel Radius [m]                                | $r_0 = 6.7$                     |

Table 4.2 Parameters Employed in Analysis 2

| <i>Parameter</i>                                 | <i>Results</i>                  |
|--------------------------------------------------|---------------------------------|
| In-situ Stress [MPa]                             | $p_0 = 160 \times 0.026 = 4.21$ |
| Modulus of Elasticity [MPa]                      | $E = 1700$                      |
| Poisson's Ratio [-]                              | $\nu = 0.24$                    |
| Uniaxial Compressive Strength of Rock Mass [MPa] | $\sigma_{cm} = 0.207$           |
| Friction Angle [°]                               | $\phi = 30.2$                   |
| Tunnel Radius [m]                                | $r_0 = 6.7$                     |

Table 4.3 Parameters Employed in Analysis 3-Alternative A

| <i>Parameter</i>                                 | <i>Results</i>                  |
|--------------------------------------------------|---------------------------------|
| In-situ Stress [MPa]                             | $p_0 = 160 \times 0.026 = 4.21$ |
| Modulus of Elasticity [MPa]                      | $E = 130$                       |
| Poisson's Ratio [-]                              | $\nu = 0.24$                    |
| Uniaxial Compressive Strength of Rock Mass [MPa] | $\sigma_{cm} = 0.012$           |
| Friction Angle [°]                               | $\phi = 12.3$                   |
| Tunnel Radius [m]                                | $r_0 = 6.7$                     |

Table 4.4 Parameters Employed in Analysis 4-Alternative A

| <i>Parameter</i>                                 | <i>Results</i>                  |
|--------------------------------------------------|---------------------------------|
| In-situ Stress [MPa]                             | $p_0 = 160 \times 0.026 = 4.21$ |
| Modulus of Elasticity [MPa]                      | $E = 118$                       |
| Poisson's Ratio [-]                              | $\nu = 0.24$                    |
| Uniaxial Compressive Strength of Rock Mass [MPa] | $\sigma_{cm} = 0.009$           |
| Friction Angle [°]                               | $\phi = 9.94$                   |
| Tunnel Radius [m]                                | $r_0 = 6.7$                     |

The support system consists of the precast segments installed after the tunnel excavation with TBM. The maximum support pressure [Eq. (4.1)] and the stiffness of the segments [Eq. (4.2)] are calculated by the formulae given below:

$$P_{cmax} = \frac{1}{2} \sigma_{cc} \left[ 1 - \frac{(r_o - t_c)^2}{(r_o)^2} \right] \quad (4.1)$$

Where,  $P_{cmax}$  = maximum support pressure of segments (MPa),

$\sigma_{cc}$  = compressive strength of the concrete (MPa),

$r_o$  = radius of the tunnel (m),

$t_c$  = thickness of the lining (m).

$$k_c = \frac{E_c (r_o^2 - (r_o - t_c)^2)}{2(1 - \nu_c^2)(r_o - t_c)r_o^2} \quad (4.2)$$

Where,  $k_c$  = stiffness of segments (MPa/m)

$E_c$  = Young's Modulus of the concrete (MPa),

$\nu_c$  = Poisson's ratio of the concrete.

Therefore, the displacement capacity of the precast segments is calculated by assuming the segments exhibit linear elastic behavior. The results of the maximum support pressure, stiffness, and displacement capacity of segments are provided in Table 4.5 :

Table 4.5 Support Properties of Precast Segments

| <i>Property</i>                          | <i>Input Parameter</i> |
|------------------------------------------|------------------------|
| Maximum Available Support Pressure [MPa] | P = 2.59               |
| Stiffness [MPa/m]                        | k = 381                |
| Displacement Capacity [%]                | P/k = 0.68             |

The detailed results of the above mentioned ground-support reactions are provided in Chapter 5.

#### 4.2.2 Finite Element Analyses

In the FE analyses, plane strain models with 15-noded triangular elements are used. Plaxis 2D automatically generates the fixities at the boundaries. The top boundaries of the models are not fixed to simulate the actual ground behavior. The bottom

boundary is restrained in both directions for the displacement. The horizontal displacements are restrained in the vertical left and right boundaries of the models, while the vertical displacements were not restrained.

The sizes of the model geometry were created wide enough to prevent the undesirable effects of boundary conditions. Therefore, the model geometry was created as 220m wide and 220m high.

The models' coarseness factor was selected as 0.25 around 30m of the vicinity of the tunnel axis, and the mesh creation was conducted by fine elements. The coarseness factor was gradually decreased by 0.5 to 1 as the distance of the models' clusters to the tunnel geometry increase.

The water table is created through the field data obtained from boreholes. The water table was created as a continuous horizontal line 80 m above the top of the segmental linings.

According to the observations from the field data, two different units of materials were used. The material at the top of the model geometry is named Unit 1, and the material below it was named Unit 2. Based on the variation of RQD and UCS values with depth, Unit 1 was considered to be relatively more weathered comparing with Unit 2. Based on the results of monitoring data, the tunnel lining was subjected to large-scale displacements; therefore, it can be inferred that there is a significant possibility of the existence of locally disturbed zones around the tunnel boundary. However, in the analyses, for the sake of simplicity and to reflect the influence of disturbed zones thoroughly, two uniform materials were considered in the FE analyses.

Unit 1 has a thickness of 100m, and Unit 2 was modeled as 120m thick. The equivalent mesh discretization for Analysis 1, Analysis 2, Analysis 3-Alternative A, and Analysis 4-Alternative A is provided in Figure 4.1.

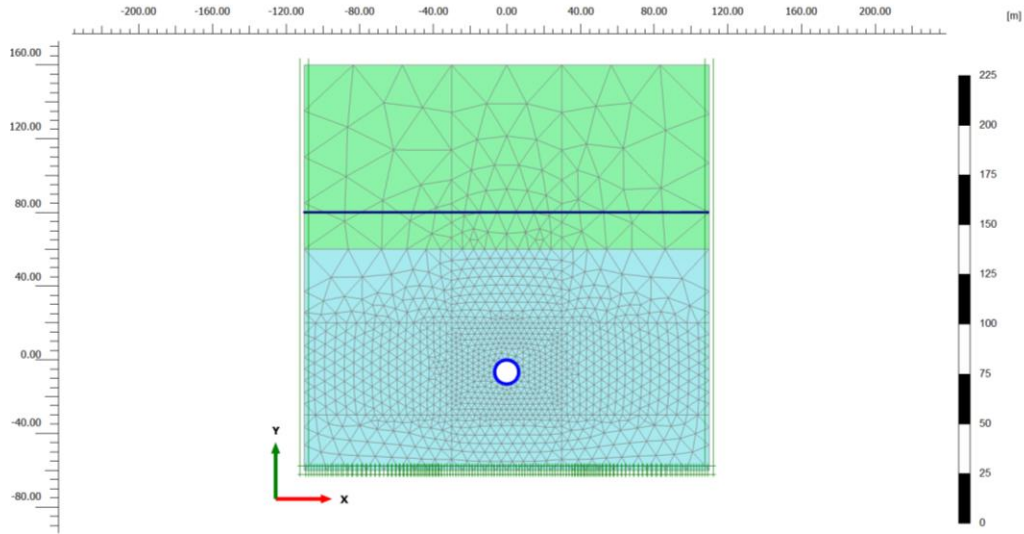


Figure 4.1 Generic mesh discretization of the 2D models with TBM

In the FE analyses, the Mohr-Coulomb material model was used. As stated in Hoek and Brown (1997), the mathematical relationship between the Hoek-Brown criterion and Mohr-Coulomb criterion was expressed in terms of the major and minor principal stresses by relating normal and shear stresses. For this purpose, the rock parameters were converted to the Mohr-Coulomb strength parameters by the well-known program RocLab, by RocScience. In each analysis, different geotechnical parameters are assigned to the FE models. The geotechnical parameters of Analysis 2, Analysis 3-Alternative A and Analysis 4-Alternative A are determined from the back analyses performed by taking the field monitoring data as the reference. The strength parameters of  $c'$ ,  $\phi'$ , and the Young's Modulus are chosen as the variable parameters of the back analyses. The  $K_0$  value is determined from the Eq. (4.3) (Sheorey, 1994). The employed geotechnical parameters in each analysis are tabulated in Table 4.6.

$$K_0 = 0.25 + 7E_h \left( 0.001 + \frac{1}{z} \right) \quad (4.3)$$

Table 4.6 Geotechnical Parameters Employed in Analyses with TBM

| <i>Analysis</i> | <i>Unit</i> | $\gamma_u$<br>[kN/m <sup>3</sup> ] | $\gamma_s$<br>[kN/m <sup>3</sup> ] | $c'$<br>[kPa] | $\phi'$<br>[°] | $\psi$<br>[°] | $\nu$<br>[-] | E [kPa] | $K_0$<br>[-] |
|-----------------|-------------|------------------------------------|------------------------------------|---------------|----------------|---------------|--------------|---------|--------------|
| Analysis 1      | Unit 1      | 26.3                               | 26.3                               | 277.0         | 37.0           | 0             | 0.24         | 2.90E6  | 0.398        |
|                 | Unit 2      | 26.3                               | 26.3                               | 386.0         | 34.0           | 0             | 0.24         | 2.90E6  | 0.440        |
| Analysis 2      | Unit 1      | 26.3                               | 26.3                               | 225.0         | 33.0           | 0             | 0.24         | 1.70E6  | 0.455        |
|                 | Unit 2      | 26.3                               | 26.3                               | 314.0         | 30.0           | 0             | 0.24         | 1.70E6  | 0.500        |
| Analysis 3-     | Unit 1      | 26.3                               | 26.3                               | 54.0          | 14.0           | 0             | 0.24         | 130000  | 0.758        |
| Alternative A   | Unit 2      | 26.3                               | 26.3                               | 74.0          | 12.0           | 0             | 0.24         | 130000  | 0.792        |
| Analysis 4-     | Unit 1      | 26.3                               | 26.3                               | 42.0          | 11.0           | 0             | 0.24         | 118000  | 0.809        |
| Alternative A   | Unit 2      | 26.3                               | 26.3                               | 58.0          | 10.0           | 0             | 0.24         | 118000  | 0.826        |

The tunnel lining consists of 200cm long, 45 cm thick precast segments. The precast segments are modeled as plate elements that satisfy the circularity of the tunnel geometry. As the calculations are performed in a plane-strain FE model, the length of the segments was modeled as 1.0m. The necessary modifications are conducted during the calculation procedure of the outputs. Elastic material properties were chosen when modeling the plate elements that simulate the tunnel lining. Since it is not possible to model coupled double rings in 2D FE programs, Muir-Wood's formula [Eq. (4.4)] was used to simulate the reduced area-wise moments while assigning the properties of the plate.

$$I = I_s + I_n \left( \frac{4}{m} \right) \quad (4.4)$$

Where; I: reduced area-wise moment

$I_s$ : the area-wise moment of the force transmission zone

$I_n$ : the area-wise moment of the complete section

m: number of segments (excluding the key segment)

The segmental ring usually is calculated as a coupled double ring. In a FE code, only a single ring can be calculated, as coupling of two rings is not possible or very time consuming for most of all calculation programs. A single ring with articulations and a single rigid ring is calculated, as well as a rigid ring with a reduced area-wise moment according to Muir-Wood's formula. The properties of the plate are provided in Table 4.7.

Table 4.7 Input Parameters of the Plate Element of TBM

| <i>Parameter</i>  | <i>Unit</i>         | <i>Symbol</i> | <i>Value</i> |
|-------------------|---------------------|---------------|--------------|
| Material Type     | Elastic             | -             | -            |
| Axial Stiffness   | kN/m                | EA            | 15642818     |
| Bending Stiffness | kNm <sup>2</sup> /m | EI            | 527945       |
| Weight            | kN/m/m              | w             | 11.25        |
| Poisson's Ratio   | -                   | $\nu$         | 0.2          |

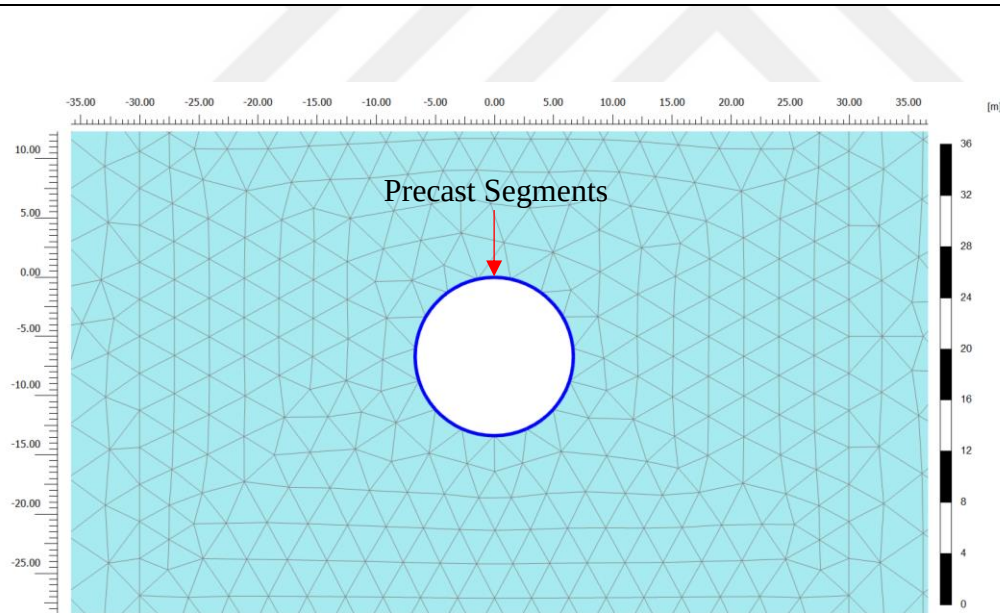


Figure 4.2 Generic presentation of support elements of TBM tunnel in FE models

The tunnel advancement in the analyses with TBM was performed in three executive phases with the staged construction option. The initial phase consists of the generation of the initial stress state throughout the model with the  $K_0$  procedure. After that, the internal cluster of the tunnel geometry is deactivated to simulate the

full face excavation performed by the TBM. The deconfinement ratio of 28% was applied to the excavated cluster. Ultimately, at the last phase, the tunnel linings are installed around the tunnel periphery, and the internal pressure was utterly relaxed. The results and the related discussions are presented in Chapter 5.

### **4.3 Modeling of Tunnel Advancement with NATM**

After observing excessive deformations during the excavation of Tunnel A93 with TBM, the tunnel advancement was stopped. The tunnel advancement has been decided to be carried out with a sequential excavation method. Thereafter, the design project of the tunnel construction was started over based on NATM philosophy.

According to the new project, the C4 Class of Support System is used in the construction of the tunnel in the collapse area. As the support system of the tunnel was changed entirely, a re-evaluation of the designed support system was to be done. Therefore, as previously done, the support system and the behavior of the ground were assessed first by plotting the ground-support reaction curves. Thereafter, the finite element models are constructed in the light of the obtained data.

In this chapter, three cases are investigated, namely, Analysis 3-Alternative B, and Analysis 4-Alternative B. These cases consist of the re-evaluation of Analysis 3-Alternativa A and Analysis 4-Alternative A, with the newly proposed support system.

After the collapse incident, the vertical effective stress around the tunnel is assumed to decrease remarkably. The assumption of the amount of decrease in the in-situ stresses is made by utilizing the ground-support reaction curves of the analyzed cases. The re-evaluated cases, in which the collapse incident was demonstrated with the conducted analyses, with the new support system were studied considering a decrease in the vertical effective stresses. As a consequence, in these cases, the strength parameters of the soil were altered according to the in situ stresses by using RocLab software.

### 4.3.1 Ground-Support Reactions

As a first step, as previously explained, RocSupport software was used to visualize the ground-support reaction curve. Again, in these cases, the Duncan Fama solution was chosen to determine the ground-support reaction curves as the Mohr-Colomb Failure Criterion is the most suitable method to represent the flowing ground after the collapse. Therefore, the tunnel and soil parameters complying with Duncan Fama solution are provided as follows (Table 4.8 - Table 4.9):

Table 4.8 Parameters Employed in Analysis 3-Alternative B

| <i>Parameter</i>                                 | <i>Results</i>                   |
|--------------------------------------------------|----------------------------------|
| In-situ Stress [MPa]                             | $p_0 = 160 \times 0.0075 = 1.20$ |
| Modulus of Elasticity [MPa]                      | $E = 130$                        |
| Poisson's Ratio [-]                              | $\nu = 0.24$                     |
| Uniaxial Compressive Strength of Rock Mass [MPa] | $\sigma_{cm} = 0.012$            |
| Friction Angle [ $^{\circ}$ ]                    | $\phi = 17.3$                    |
| Tunnel Radius [m]                                | $r_0 = 6.7$                      |

Table 4.9 Parameters Employed in Analysis 4-Alternative B

| <i>Parameter</i>                                 | <i>Results</i>                   |
|--------------------------------------------------|----------------------------------|
| In-situ Stress [MPa]                             | $p_0 = 160 \times 0.0094 = 1.50$ |
| Modulus of Elasticity [MPa]                      | $E = 118$                        |
| Poisson's Ratio [-]                              | $\nu = 0.24$                     |
| Uniaxial Compressive Strength of Rock Mass [MPa] | $\sigma_{cm} = 0.009$            |
| Friction Angle [ $^{\circ}$ ]                    | $\phi = 13.4$                    |
| Tunnel Radius [m]                                | $r_0 = 6.7$                      |

This time the proposed support system contains more than one element. The support system contains shotcrete application, steel rib installation, and the radially applied rock bolts. The maximum support pressure [Eq. (4.1)] and the stiffness [Eq. (4.2)] of the shotcrete are calculated by the formulas given in Chapter 4.2.1. The support properties of steel ribs and rock bolts are calculated by the formulas given below [Eq. (4.5) - (4.8)]:

$$P_{ssmax} = \frac{A_s \sigma_{ys}}{s_l r_o} \quad (4.5)$$

Where,  $P_{ssmax}$  = maximum support pressure of steel sets (MPa),

$A_s$  = cross-sectional area of the section (MPa),

$\sigma_{cc}$  = yield strength of the steel (MPa),

$s_l$  = steel set spacing along the tunnel axis (m),

$r_o$  = radius of the tunnel (m).

$$K_{ss} = \frac{E_s A_s}{s_l r_o^2} \quad (4.6)$$

Where,  $K_{ss}$  = stiffness of segments (MPa/m)

$E_{ss}$  = Young's Modulus of the steel (MPa),

$A_s$  = cross-sectional area of the section (MPa),

$s_l$  = steel set spacing along the tunnel axis (m),

$r_o$  = radius of the tunnel (m).

$$P_{sbmax} = \frac{T_{bf}}{s_l s_c} \quad (4.7)$$

Where,  $P_{sbmax}$  = maximum support pressure provided by a rock bolt pattern (MPa),

$T_{bf}$  = ultimate load capacity obtained from a pull-out test (MN),

$s_l$  = longitudinal bolt spacing (m),

$s_c$  = circumferential bolt spacing (m).

$$K_{sb} = \frac{E_s \pi d_b^2}{4 l s_l s_c} \quad (4.8)$$

Where,  $K_{sb}$  = stiffness of rock bolts (MPa/m)

$E_s$  = Young's Modulus of the bolt (MPa),

$d_b$  = diameter of rock bolt (m),

$l$  = free length of the bolt (MPa),

$s_l$  = longitudinal bolt spacing (m),

$s_c$  = circumferential bolt spacing (m).

The support system's linear displacement capacity is calculated by proportioning the maximum available support pressure of the support system to the total stiffness of the support system. The results of the total available maximum support pressure, total stiffness, and total displacement capacity of the system are provided in Table 4.10:

Table 4.10 Support Properties of the Support System

| <i>Type of Support</i> | <i>Property</i>                          | <i>Input<br/>Parameter</i> |
|------------------------|------------------------------------------|----------------------------|
| Shotcrete              | Maximum Available Support Pressure [MPa] | P = 1.14                   |
|                        | Stiffness [MPa/m]                        | k = 570.7                  |
|                        | Displacement Capacity [%]                | P/k = 0.2                  |
| Steel Ribs             | Maximum Available Support Pressure [MPa] | P = 1.45                   |
|                        | Stiffness [MPa/m]                        | k = 255                    |
|                        | Displacement Capacity [%]                | P/k = 0.56                 |
| Rock Bolts             | Maximum Available Support Pressure [MPa] | P = 0.5                    |
|                        | Stiffness [MPa/m]                        | k = 1319                   |
|                        | Displacement Capacity [%]                | P/k = 0.04                 |
| Total                  | Maximum Available Support Pressure [MPa] | P = 3.09                   |
|                        | Stiffness [MPa/m]                        | k = 2144.7                 |
|                        | Displacement Capacity [%]                | P/k = 0.14                 |

#### 4.3.2 Finite Element Analyses

In the FE analyses, including the sequential excavation method, plane strain models with 15-noded triangular elements are used. The boundary conditions are chosen as the same as the restrictions defined in Chapter 4.2.2.

The model geometry is selected wide and high enough to assure that the boundary conditions do not affect the results. The selection of the model geometry complies with the suggestions indicated in several papers mentioned in the literature studies. Therefore, the model geometry was created as 220m wide and 220m high.

The models' coarseness factor was selected as 0.5 around 30m of the vicinity of the tunnel axis, and the mesh creation was performed by fine elements. The coarseness factor was gradually decreased by 0.5 to 1 as the distance of the models' clusters to

the tunnel geometry increase. The conditions of the water table were assumed to remain unchanged as there were not any measurements taken after the occurrence of collapse.

The arrangement (i.e., the thickness, sequence, and the denomination of the units) of the units that are used in the FE models also remained unchanged. The equivalent mesh discretization for Analysis 3-Alternative B, and Analysis 4-Alternative B is provided in Figure 5.2.

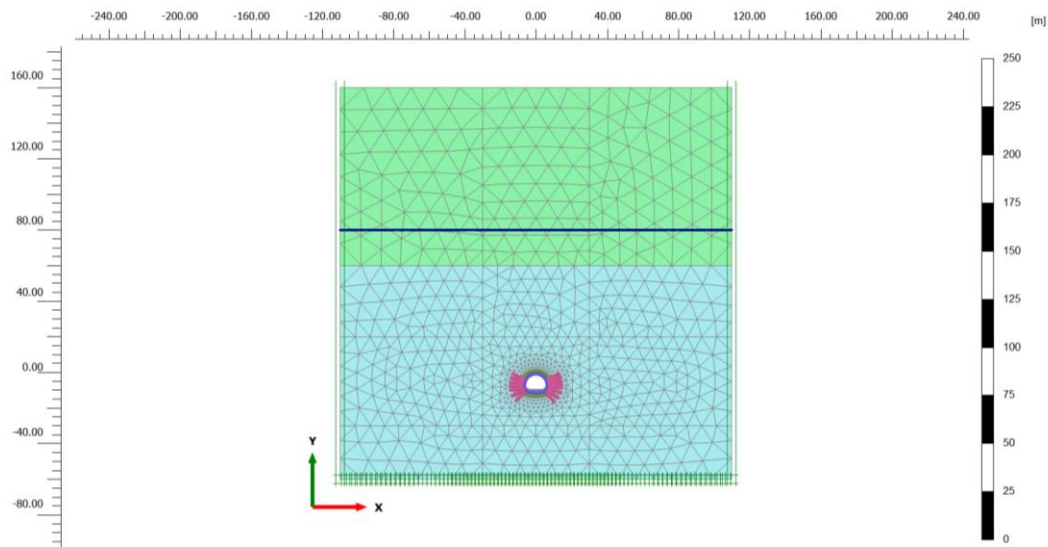


Figure 4.3 Generic mesh discretization of the 2D models with NATM

In the FE analyses, the Mohr-Coulomb material model was used. As stated in Hoek and Brown (1997), the mathematical relationship between the Hoek-Brown criterion and Mohr-Coulomb criterion was expressed in terms of the major and minor principal stresses by relating normal and shear stresses. For this purpose, the rock parameters were converted to the Mohr-Coulomb strength parameters by the well-known program RocLab, by RocScience. In each analysis, different geotechnical parameters are assigned to the FE models. The geotechnical parameters of Analysis 3-Alternative B, and Analysis 4-Alternative B are determined from the back-analyses mentioned in Chapter 4.2.2. The unit weight of the host ground was decreased to simulate the behavior of the soil after the collapse incident. Therefore, after making necessary modifications to the strength parameters with the effect of

the reduced unit weights, the geotechnical parameters used in the analyses are summarized in Table 4.11. The  $K_0$  value is determined from the Eq. (4.3) (Sheorey, 1994).

Table 4.11 Geotechnical Parameters Employed in Analyses with NATM

| <i>Analysis</i> | <i>Unit</i> | $\gamma_u$<br>[kN/m <sup>3</sup> ] | $\gamma_s$<br>[kN/m <sup>3</sup> ] | $c'$<br>[kPa] | $\phi'$<br>[°] | $\psi$<br>[°] | $\nu$<br>[-] | $E$ [kPa] | $K_0$<br>[-] |
|-----------------|-------------|------------------------------------|------------------------------------|---------------|----------------|---------------|--------------|-----------|--------------|
| Analysis 3-     | Unit 1      | 7.50                               | 7.50                               | 24.0          | 19.0           | 0             | 0.24         | 130000    | 0.674        |
| Alternative B   | Unit 2      | 7.50                               | 7.50                               | 33.0          | 17.0           | 0             | 0.24         | 130000    | 0.707        |
| Analysis 4-     | Unit 1      | 9.375                              | 9.375                              | 22.0          | 14.0           | 0             | 0.24         | 118000    | 0.758        |
| Alternative B   | Unit 2      | 9.375                              | 9.375                              | 30.0          | 13.0           | 0             | 0.24         | 118000    | 0.775        |

The primary support system comprises of 40 cm thick shotcrete (C25/30) lining, steel ribs composed of HEB220 steel profiles installed with 0.5m intervals along the tunnel advancement,  $\phi=32$ mm diameter rock bolts with a mesh of 0.5 (longitudinal direction) x 1.0m (transversal direction). The final lining has a thickness of 40cm at the crown and a variable thickness of 40 to 70 cm at the invert. Shotcrete lining and the steel sets are assumed to function as a hybrid lining. Therefore, both components are modeled as one plate element of which reflects the hybrid element.

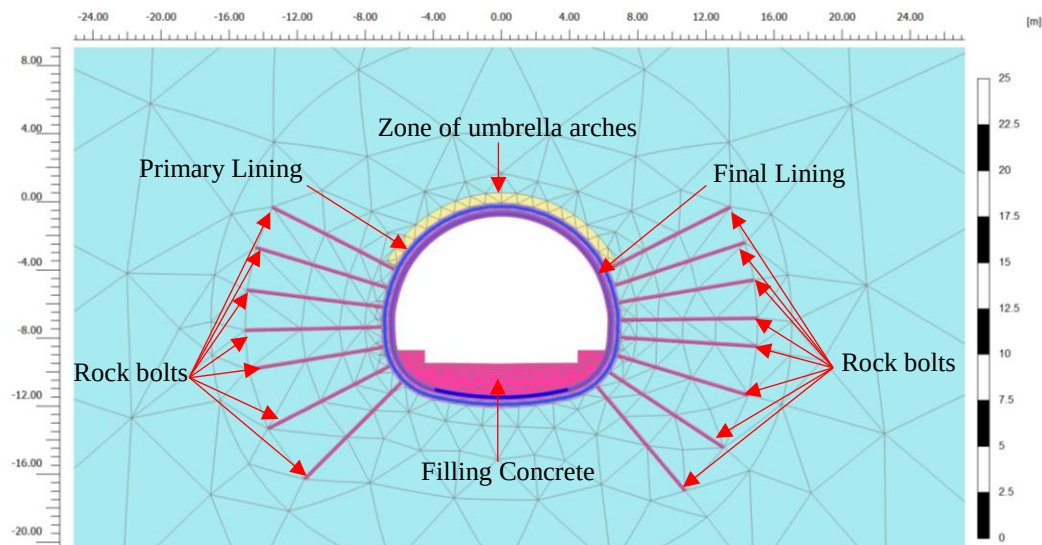


Figure 4.4 Generic presentation of support elements of NATM tunnel in FE models

The properties of the hybrid shotcrete and steel rib elements deviate by the maturation of the shotcrete by the advancement of the tunnel. In the staged construction, the excavation sequences are simulated gradually. Therefore, since the tunnel advancement progresses, the maturation rates of the shotcrete alter accordingly. As a practical approach, the shotcrete reaches its two days of maturation rate as the tunnel advances for 2m. Similarly, at the tunnel advancement rate of 4m, the shotcrete reaches its four-day maturation rate. The shotcrete is assumed to reach its 28 days of maturation rate no less than 12m of tunnel advancement.

The primary and final lining boundaries are defined as linear elastic soils, and the plates are embedded into the middle of the presumably circular geometry to obtain relatively smooth readings on the plates. Therefore, the plates' parameters (EA, EI, and weight) are reduced by 1000, not to affect the results. The outputs are adjusted as they were multiplied by 1000 when the calculation results are presented. Thus, the properties of the plate elements are assigned, considering these assumptions.

The rock bolts are defined as elastic embedded beam rows installed radially along the tunnel boundary. The properties of the plates and the rock bolts that are used in the analyses with NATM tunnels are provided in Table 4.12.

Table 4.12 Input Parameters of the Support Elements of NATM

| <i>Element</i>                                            | <i>Parameter</i>     | <i>Unit</i>            | Symbol               | Value   |
|-----------------------------------------------------------|----------------------|------------------------|----------------------|---------|
| Embedded Beam<br>Row:<br>Rock Bolts                       | Material Type        | Elastic                | -                    | -       |
|                                                           | Young's Modulus      | kN/m <sup>2</sup>      | E                    | 210.0E6 |
|                                                           | Unit Weight          | kN/m <sup>3</sup>      | $\gamma$             | 76.982  |
|                                                           | Pile Type            | Predefined             | -                    | -       |
|                                                           | Predefined Pile Type | Massive Circular Plate | -                    | -       |
|                                                           | Diameter             | m                      | D                    | 0.032   |
|                                                           | Out-of-plane Spacing | m                      | L <sub>spacing</sub> | 0.5     |
| Plate:<br>Shotcrete + Steel Ribs<br>(Maturation: 2 days)  | Material Type        | Elastic                | -                    | -       |
|                                                           | Axial Stiffness      | kN/m                   | EA                   | 9895.04 |
|                                                           | Bending Stiffness    | kNm <sup>2</sup> /m    | EI                   | 114.93  |
|                                                           | Weight               | kN/m/m                 | w                    | 0.01    |
|                                                           | Poisson's Ratio      | -                      | $\nu$                | 0.2     |
| Plate:<br>Shotcrete + Steel Ribs<br>(Maturation: 4 days)  | Material Type        | Elastic                | -                    | -       |
|                                                           | Axial Stiffness      | kN/m                   | EA                   | 12557.9 |
|                                                           | Bending Stiffness    | kNm <sup>2</sup> /m    | EI                   | 150.44  |
|                                                           | Weight               | kN/m/m                 | w                    | 0.01    |
|                                                           | Poisson's Ratio      | -                      | $\nu$                | 0.2     |
| Plate:<br>Shotcrete + Steel Ribs<br>(Maturation: 28 days) | Material Type        | Elastic                | -                    | -       |
|                                                           | Axial Stiffness      | kN/m                   | EA                   | 14475.3 |
|                                                           | Bending Stiffness    | kNm <sup>2</sup> /m    | EI                   | 176.0   |
|                                                           | Weight               | kN/m/m                 | w                    | 0.01    |
|                                                           | Poisson's Ratio      | -                      | $\nu$                | 0.2     |

Table 4.12 (continued) Input Parameters of the Support Elements of NATM

|                                       |                   |                     |       |         |
|---------------------------------------|-------------------|---------------------|-------|---------|
| Plate:                                | Material Type     | Elastic             | -     | -       |
| Final Lining<br>(Maturation: 28 days) | Axial Stiffness   | kN/m                | EA    | 11882.8 |
|                                       | Bending Stiffness | kNm <sup>2</sup> /m | EI    | 158.0   |
|                                       | Weight            | kN/m/m              | w     | 0.01    |
|                                       | Poisson's Ratio   | -                   | $\nu$ | 0.2     |

As stated by Hoek (2003), there are no available general design rules in the design of forepoles in 2D modeling. Therefore, in the absence of such context, a crude equivalent modeling approach is used in the modeling of forepoles. This approach assumes that the estimation of weighted averages of each component can be used to determine the strength of a 'consolidation zone' of the reinforced ground. The strength of each element, including the steel, grout, and the host ground, is multiplied by the cross-sectional area of each element, and the resulting product is divided by the total area of all elements in the strengthened zone. In this study, the double layer of forepoles installed above the excavation boundary is modeled as 0.6m deep, 1m wide beams.

The host ground at the tunnel boundary roof is strengthened before starting the excavation of the top heading. The 1<sup>st</sup> row of forepoles is comprised of 12.0m long, 88.9 mm diameter steel pipes of 5 mm thickness, overlapping length of 7.5m, aligned with 21° relative to the tunnel axis, 30 cm of longitudinal spacing. The 2<sup>nd</sup> row of forepoles is comprised of 12.0m long, 88.9 mm diameter steel pipes of 5 mm thickness, overlapping length of 7.5m, aligned with 13° relative to the tunnel axis, 20 cm of longitudinal spacing. Therefore, the resulting equivalent strength parameters of the consolidation zone are provided in Table 4.13.

Table 4.13 Equivalent strength parameters of the consolidation zone above the tunnel crown

| <i>Analysis</i>              | <i>Element</i> | <i>Area [m<sup>2</sup>]</i> | <i>Strength [MPa]</i> | <i>Product [MN]</i> |
|------------------------------|----------------|-----------------------------|-----------------------|---------------------|
| Analysis 3-<br>Alternative B | Host Ground    | 0.589                       | 0.012                 | 0.007               |
|                              | Grouting       | 0.005                       | 26.629                | 0.130               |
|                              | Steel Pipes    | 0.006                       | 370.0                 | 2.268               |
|                              | Sum            | 0.6                         | -                     | 2.405               |
| Analysis 4-<br>Alternative B | Host Ground    | 0.589                       | 0.012                 | 0.005               |
|                              | Grouting       | 0.005                       | 26.629                | 0.130               |
|                              | Steel Pipes    | 0.006                       | 370.0                 | 2.268               |
|                              | Sum            | 0.6                         | -                     | 2.403               |

The sequential excavation of the tunnel was executed in nine consecutive phases with the staged construction option. Each phase with the applied appropriate relaxation factors and the assumed maturation rates of the shotcrete are summarized in Table 4.14. The results and the related discussions are presented in Chapter 5.

Table 4.14 Relaxation ratios employed in FE models

| Phase         | Description                                                                                                                                                                   | Top Heading     |            | Bench           |            | Invert          |            | Maturation of Shotcrete |         |         |
|---------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------|------------|-----------------|------------|-----------------|------------|-------------------------|---------|---------|
|               |                                                                                                                                                                               | $x_{front}$ [m] | Relaxation | $x_{front}$ [m] | Relaxation | $x_{front}$ [m] | Relaxation | Top Heading             | Bench   | Invert  |
| Initial Phase | KO-Procedure                                                                                                                                                                  | -∞              | 0%         | -∞              | 0%         | -∞              | 0%         | -                       | -       | -       |
| Phase 1       | Installation of forepolings                                                                                                                                                   | -∞              | 0%         | -∞              | 0%         | -∞              | 0%         | -                       | -       | -       |
| Phase 2       | Excavation of top heading                                                                                                                                                     | 0               | 28%        | -∞              | 0%         | -∞              | 0%         | -                       | -       | -       |
| Phase 3       | 2.0m of advancement in top heading + application of shotcrete, steel ribs, and rock bolts                                                                                     | 2               | 67%        | -∞              | 0%         | -∞              | 0%         | 2 days                  | -       | -       |
| Phase 4       | 4.0m of advancement in top heading + maturation of shotcrete                                                                                                                  | 4               | 81%        | -∞              | 0%         | -∞              | 0%         | 4 days                  | -       | -       |
| Phase 5       | 12.0 m of advancement in top heading + 4.0m advancement at the bench + maturation of shotcrete in top heading + application of shotcrete, steel ribs, and rock bolts in bench | 12              | 100%       | 4               | 77%        | -∞              | 0%         | 28 days                 | 4 days  | -       |
| Phase 6       | 12.0 m of advancement in bench + maturation of shotcrete in bench                                                                                                             | 20              | 100%       | 12              | 100%       | -∞              | 0%         | 28 days                 | 28 days | -       |
| Phase 7       | 4.0m of advancement in invert + application of shotcrete, steel ribs, and rock bolts in invert                                                                                | ≈ 10 D          | 100%       | 20              | 100%       | 4               | 75%        | 28 days                 | 28 days | 4 days  |
| Phase 8       | 12.0m of advancement in invert + maturation of shotcrete in invert + application of the final lining of the invert                                                            | ≈ 10 D          | 100%       | ≈ 10 D          | 100%       | 12              | 100%       | 28 days                 | 28 days | 28 days |
| Phase 9       | Application of the final lining in crown                                                                                                                                      | ≈ 10 D          | 100%       | ≈ 10 D          | 100%       | ≈ 10 D          | 100%       | 28 days                 | 28 days | 28 days |

## **CHAPTER 5**

### **RESULTS AND DISCUSSION**

This section is devoted to the presentation of the results of the numerical models conducted for the cases described in the previous chapter. An extensive presentation and discussion of the results, including the parametric studies to determine the real behavior of the host ground due to excavation, the correlation between the monitoring data and the results of the numerical models, deformation characteristics, resultant plastic zone around the tunnel boundary, and support reactions are displayed in this section.

The summary table, composed of the radius of the plastic zones, total displacements, maximum axial forces, shear forces, and bending moments at the support systems, is provided in Table 5.1.

Table 5.1 Summary of Analysis Results

| Analysis                  | Description                                                                                                                                                                                             | Total Displacements at the Support (In Monitoring Data) [m] | Total Displacements at the Support (In Preliminary Analyses) [m] | Difference with Monitoring Data [%] | Total Displacements (In FE Analyses) [m] | Difference with Monitoring Data [%] | Radius of Plastic Zone (In Preliminary Analyses) [m] | Radius of Plastic Zone (In FE Analyses) [m] | Difference [%] |
|---------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------|------------------------------------------------------------------|-------------------------------------|------------------------------------------|-------------------------------------|------------------------------------------------------|---------------------------------------------|----------------|
| Analysis 1                | Design parameters were based on site investigation studies                                                                                                                                              | 0.01-0.30 (Crown)                                           | 0.019 (Crown)                                                    | 61-93                               | 0.0164 (Crown)                           | 67-94                               | 10.27                                                | 10.33                                       | 0.58           |
| Analysis 2                | This case represents the stable section in TBM excavation, design parameters were back-calculated based on monitoring data.                                                                             | 0.025 (Crown)                                               | 0.026 (Crown)                                                    | 3.55                                | 0.0252 (Crown)                           | 0.83                                | 9.71                                                 | 10.64                                       | 8.74           |
| Analysis 3, Alternative A | The displacements exhibit an increasing trend, and the partial collapse was initiated. The design parameters were back-calculated based on monitoring data.                                             | 0.150 (Crown)                                               | 0.162 (Crown)                                                    | 7.46                                | 0.159 (Crown)                            | 5.78                                | 10.44                                                | 9.72                                        | 6.90           |
| Analysis 3, Alternative B | The altered tunnel construction method of NATM after the partial collapse demonstrated in Analysis 3-Alternative A. The geotechnical parameters specified in Analysis 3-Alternative A were implemented. | 0.102 (Side Walls)                                          | 0.101 (Side Walls)                                               | 0.72                                | 0.098 (Side Walls)                       | 3.82                                | 12.64                                                | 10.95                                       | 13.37          |
| Analysis 4, Alternative A | The geotechnical parameters were obtained based on monitoring data. The observed deformations reached their peak, and the partial collapse occurred.                                                    | 0.231 (Crown)                                               | 0.256 (Crown)                                                    | 9.71                                | 0.251 (Crown)                            | 8.08                                | 13.44                                                | 11.71                                       | 12.87          |
| Analysis 4, Alternative B | The altered tunnel construction method of NATM after the partial collapse demonstrated in Analysis 4-Alternative A. The geotechnical parameters specified in Analysis 4-Alternative A were implemented. | 0.186 (Side Walls)                                          | 0.172 (Side Walls)                                               | 7.44                                | 0.168 (Side Walls)                       | 9.57                                | 15.58                                                | 13.43                                       | 13.80          |

## 5.1 Analysis 1

Analysis 1 is devoted to the investigation of the expected behavior in advancement with TBM considering the site investigations and laboratory test results acquired in the design stage. The base parameters obtained from the available data before the initiation of the advancement are applied, and the expected results are evaluated accordingly.

As seen in Figure 5.1 and Figure 5.2, the radius of the plastic zone is calculated as 10.27 m, the maximum displacement expected at the tunnel walls are determined as 19.39mm. In addition, the factor of safety is calculated as 4.85. Hence, according to the preliminary investigation results, the support system interacts adequately with the ground conditions specified according to the available site investigation results.

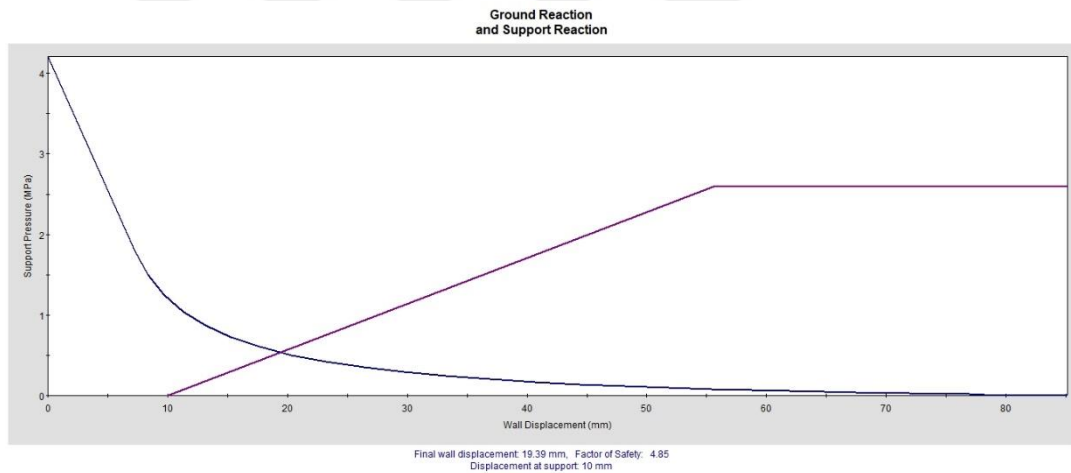


Figure 5.1 Ground-Support Reaction Curve of Analysis 1

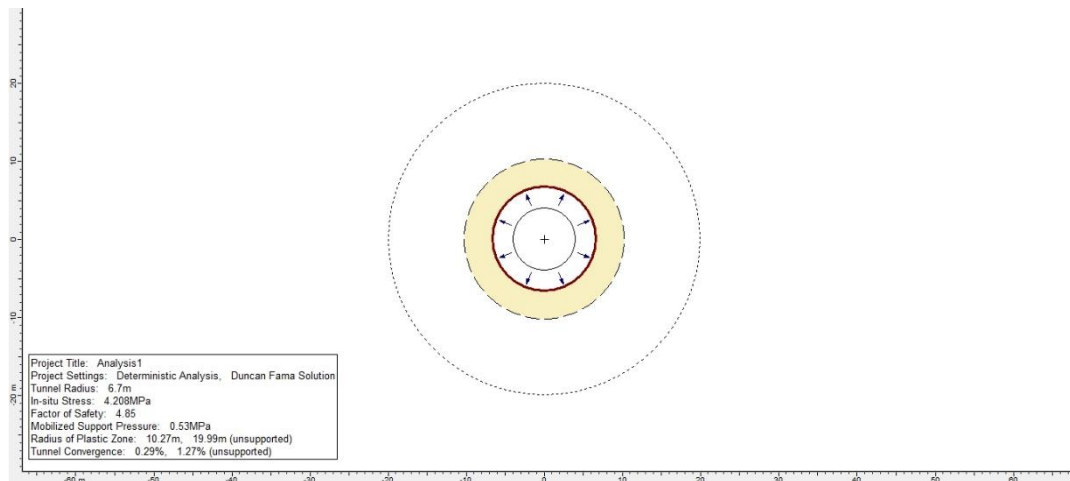


Figure 5.2 Tunnel Section View of Analysis 1

Following the analyses performed on RocSupport, 2D finite element analyses are performed for further evaluation of the deformations, stress distributions, and support reactions. The radius of the plastic zone, expected deformations at the segments, and compatibility of the support reactions are compared in the analyses conducted in Plaxis 2D.

The finite element analyses of Analysis 1 are performed according to the procedures described in Chapter 4.2.

The resulting plastic zone, principal stress contours around the tunnel boundary, deformations at the precast segments and the host ground, axial forces, shear forces, and bending moment are presented in Figures 5.3 through 5.10 from the results obtained from the last phase of the construction stages.

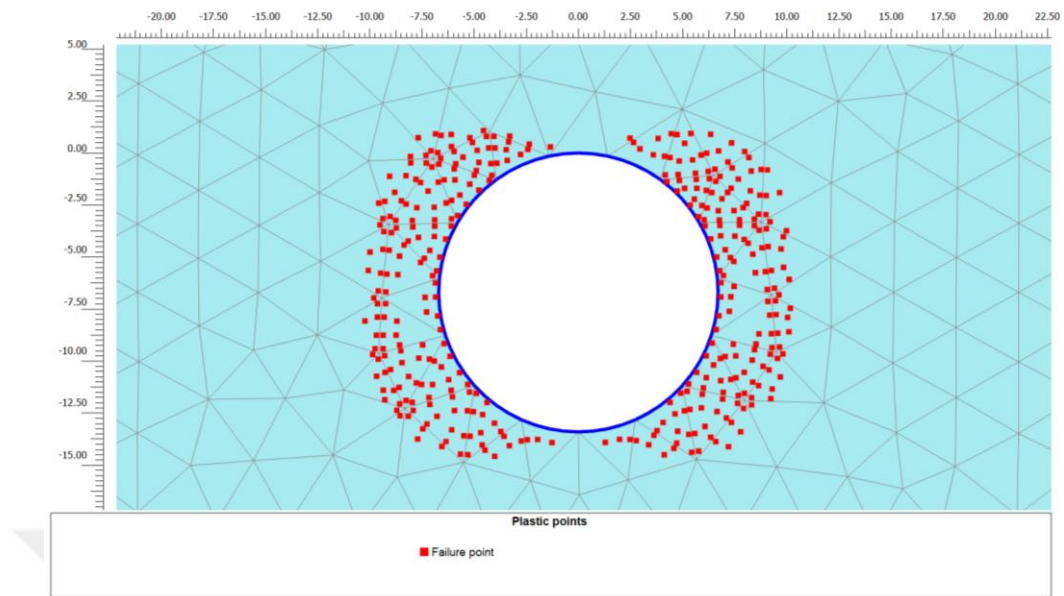


Figure 5.3 Plastic Zone Around the Tunnel in Analysis 1

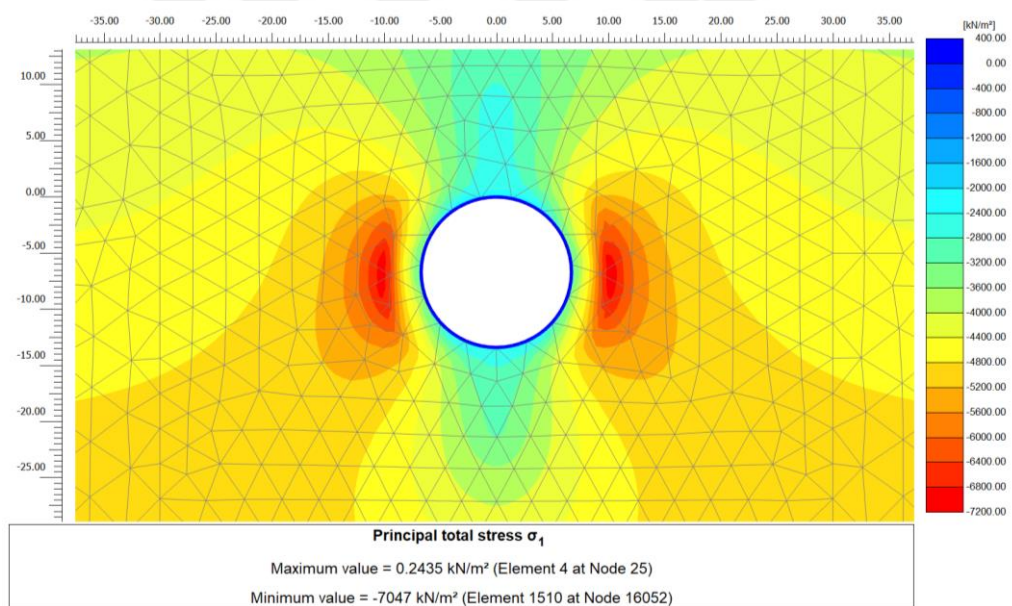


Figure 5.4 Major Principal Stress Contours Around the Tunnel in Analysis 1

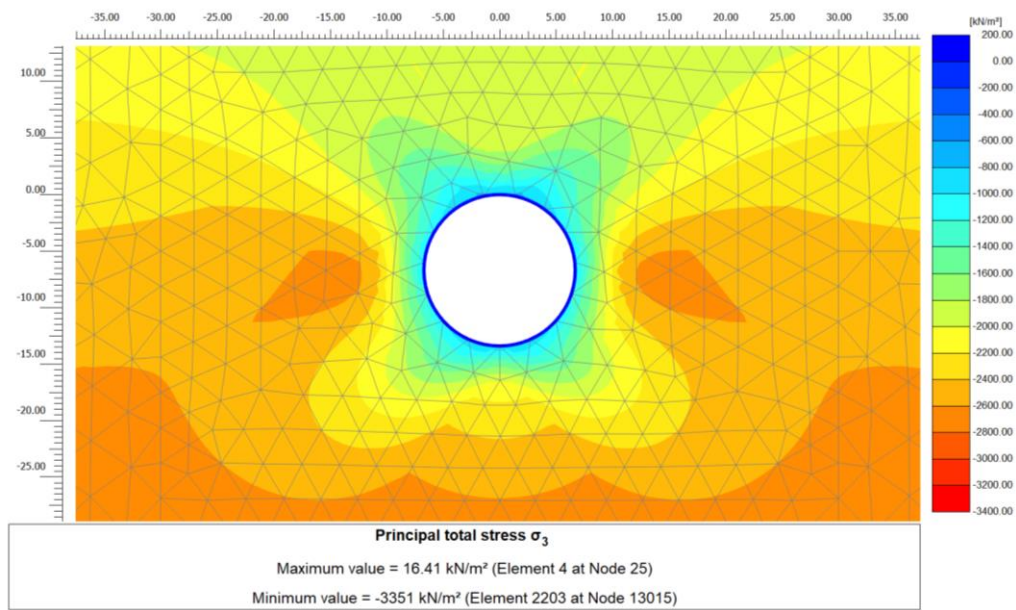


Figure 5.5 Minor Principal Stress Contours Around the Tunnel in Analysis 1

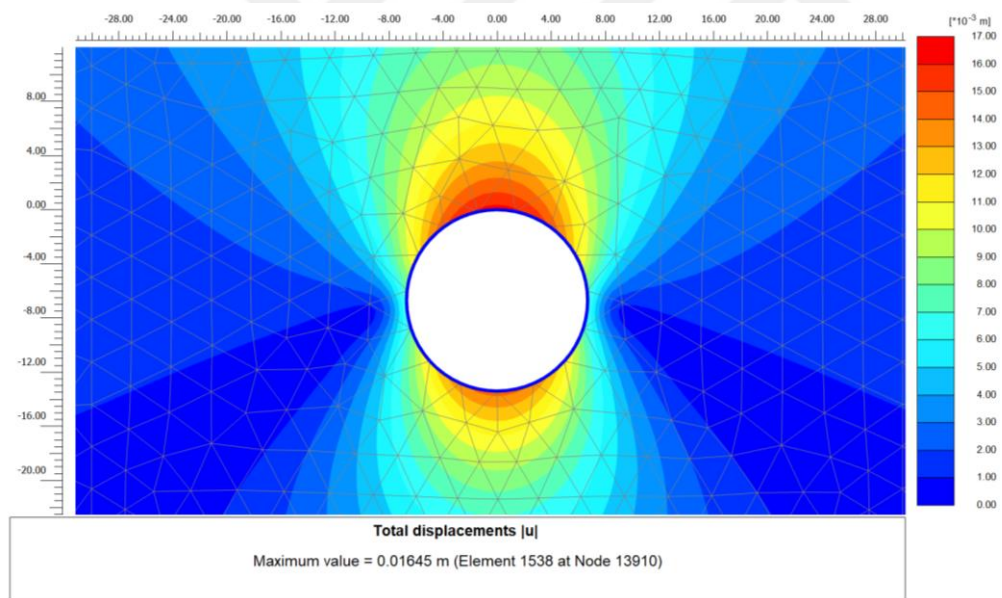


Figure 5.6 Total Displacements Around Tunnel Boundary in Analysis 1

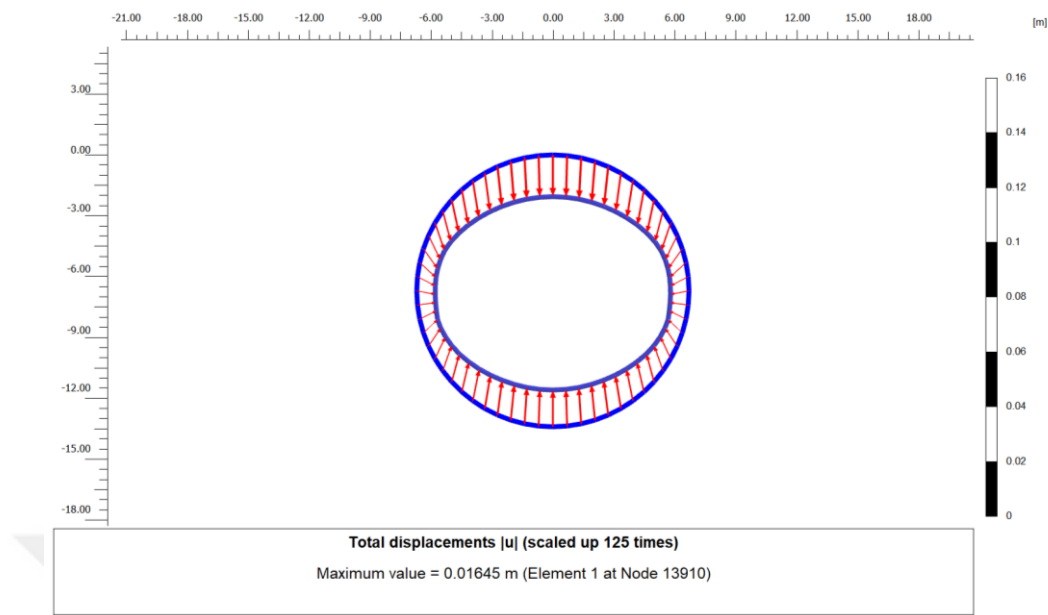


Figure 5.7 Total Displacements at the Precast Segments in Analysis 1

The results of the preliminary analyses conducted by using RocSupport and the results obtained from 2D FE analyses show coherent outcomes. In FE analysis, the radius of the plastic zone is determined as 10.33m, which deviates from the preliminary studies only with a 0.5% margin. The resulting displacements obtained from the FE analysis throughout the precast segments, however, deviate from the preliminary assessment with a margin of a %15.1. This margin can be caused by the presence of the water table, the applied relaxation factors, and consideration of double coupled rings, and the reduction of stiffness of the plate elements in FE analyses as in the analyses performed in RocSupport; the latter mentioned factors are not taken into account.

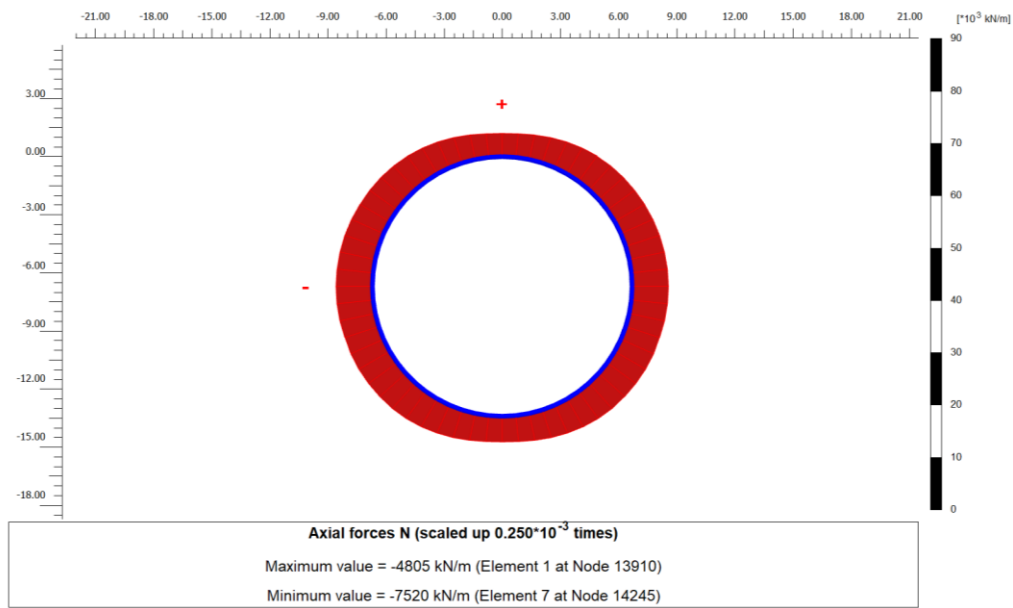


Figure 5.8 Axial Forces at the Precast Segments in Analysis 1

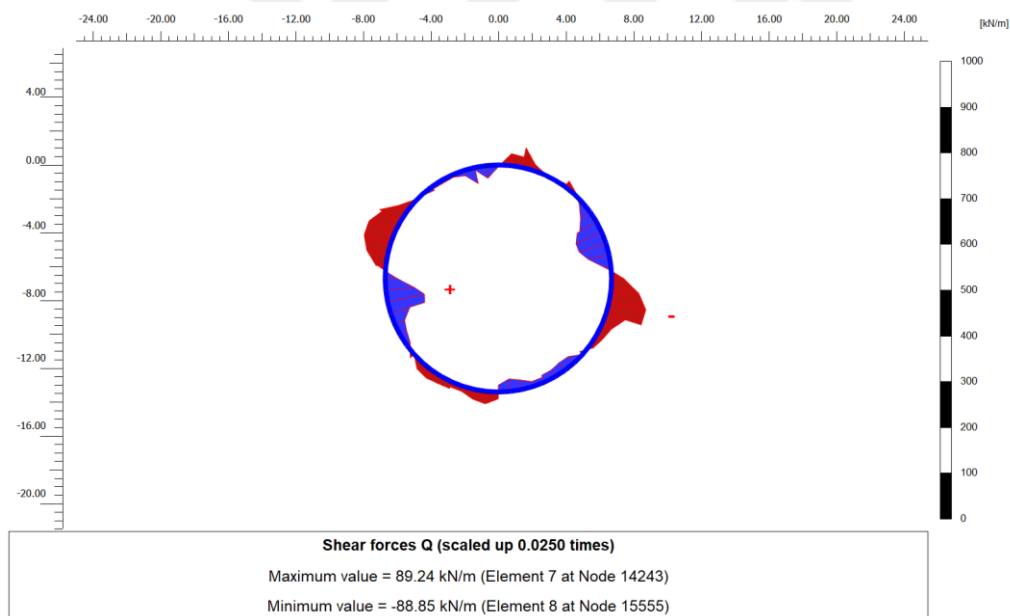


Figure 5.9 Shear Forces at the Precast Segments in Analysis 1

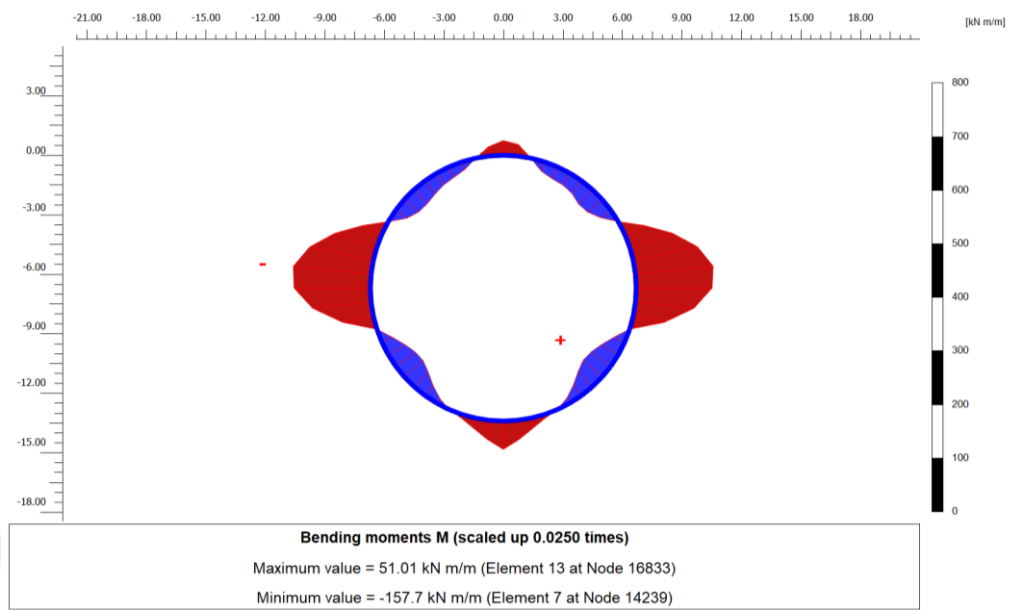


Figure 5.10 Bending Moments at the Precast Segments in Analysis 1

According to the resulting axial force, shear force, and bending moment diagrams, the static verification of the sections was performed, and the interaction diagram is provided in Figure 5.11.

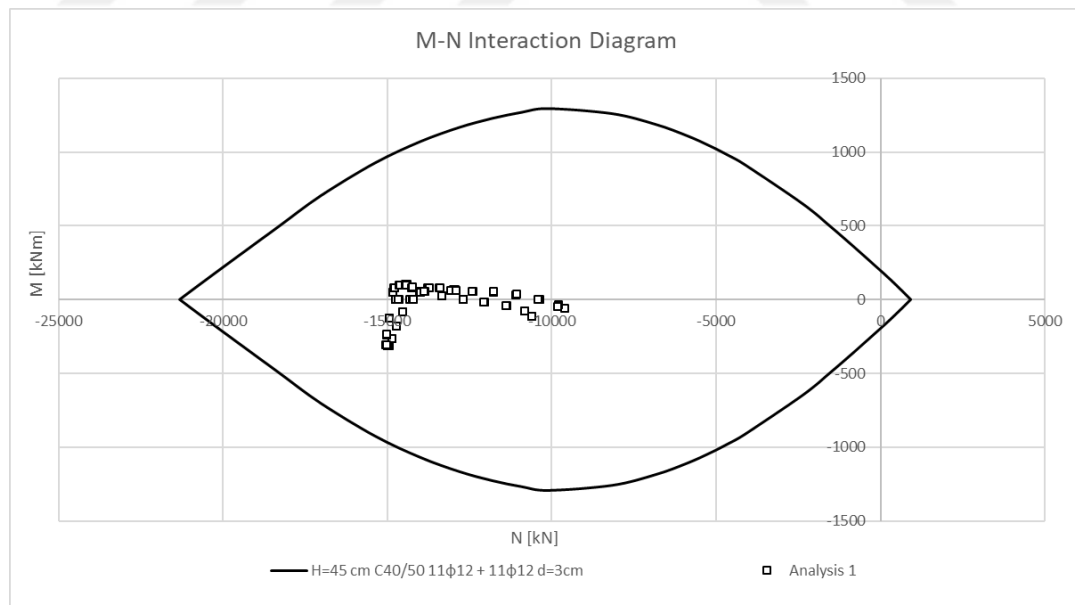


Figure 5.11 Interaction Diagram of Analysis 1

Even though the analyzed section satisfies the structural requirements, it does not reflect the monitoring results' outcomes. The results obtained from Analysis 1 do not comply with the monitoring data and the incident of collapse. Therefore, the following analyses are conducted to comprehend the real behavior observed at the site.

## 5.2 Analysis 2

Analysis 2 presents the case of tunnel response to TBM excavation in KM:217+265. Analysis 2 is conducted according to geotechnical parameters determined from the back analysis. According to the monitoring data, the expected deformation in the precast segments is determined approximately 25mm in this section.

In Figure 5.12 and Figure 5.13, the ground-support reaction curve and the tunnel section views are provided. The maximum displacement at the walls is calculated as 25.92mm, and the radius of the plastic zone is calculated as 9.71 m. Additionally, the factor of safety is calculated as 2.86. Therefore, according to the preliminary estimation, the provided support system shows adequate safety in this section of the tunnel alignment.

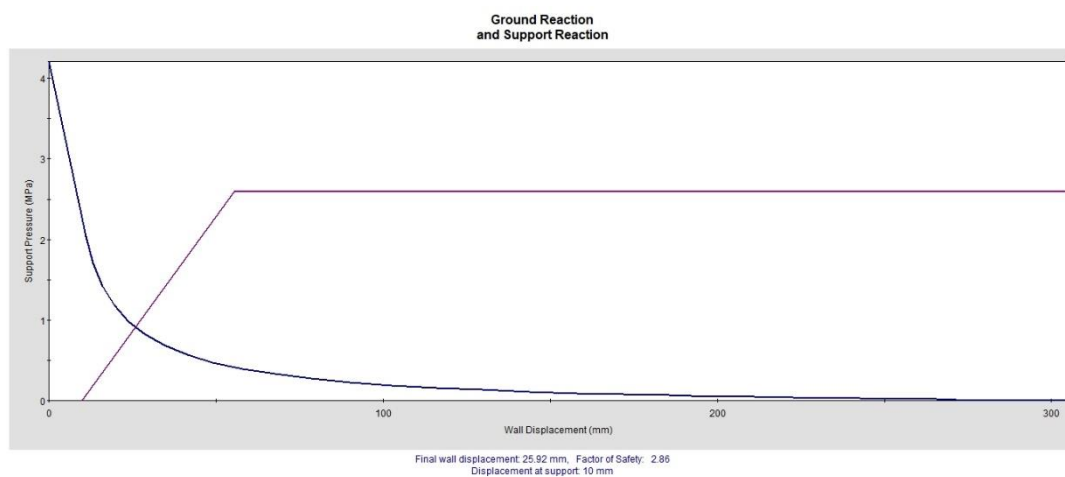


Figure 5.12 Ground-Support Reaction Curve of Analysis 2

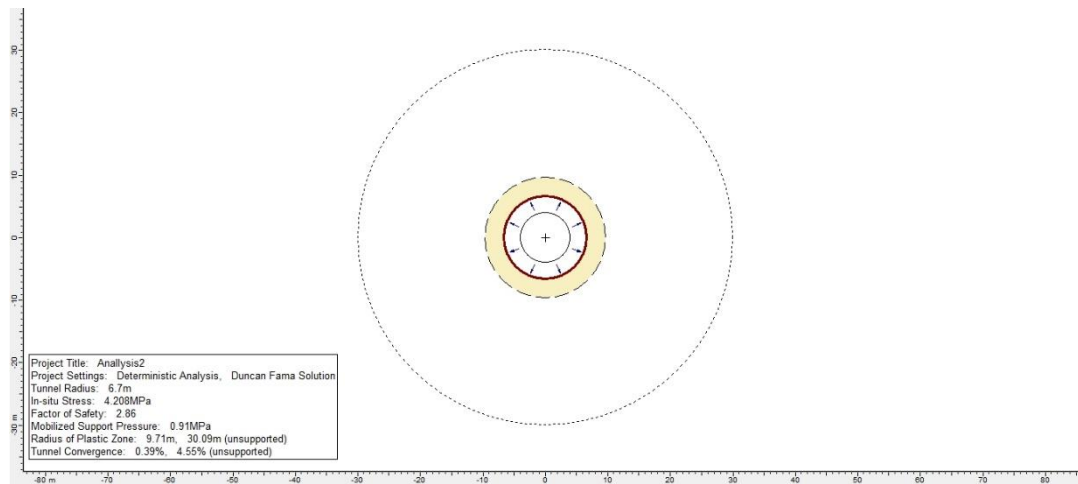


Figure 5.13 Tunnel Section View of Analysis 2

The radius of the plastic zone, principal stress contours around the tunnel boundary, expected deformations at the segments, and compatibility of the support reactions are compared in the analyses conducted in Plaxis 2D. The finite element analyses of Analysis 2 are performed according to the procedures described in Chapter 4.2. The results of the FE analyses are provided from Figure 5.14 to Figure 5.21.

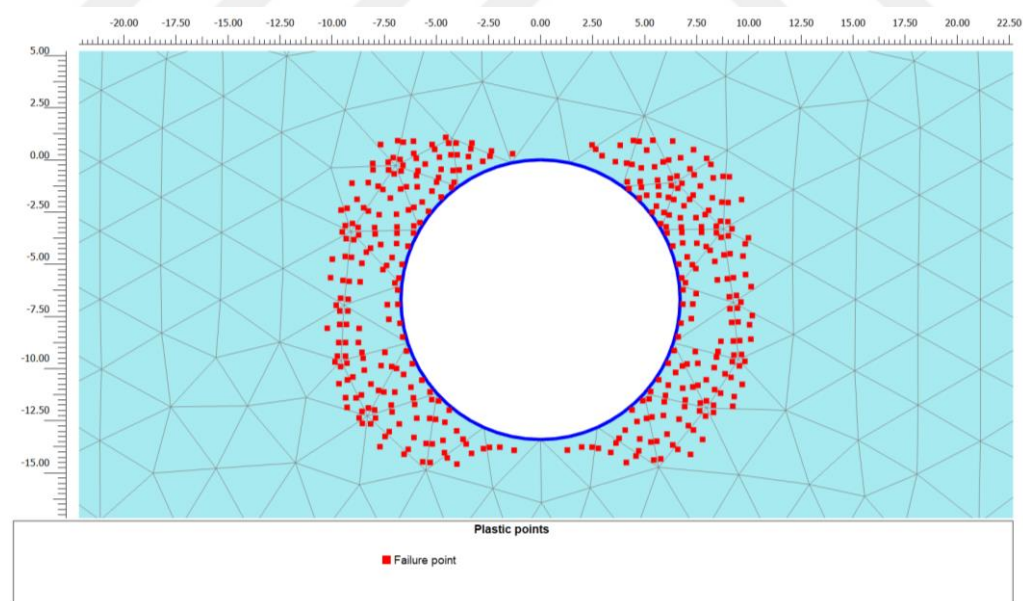


Figure 5.14 Plastic Zone Around the Tunnel in Analysis 2

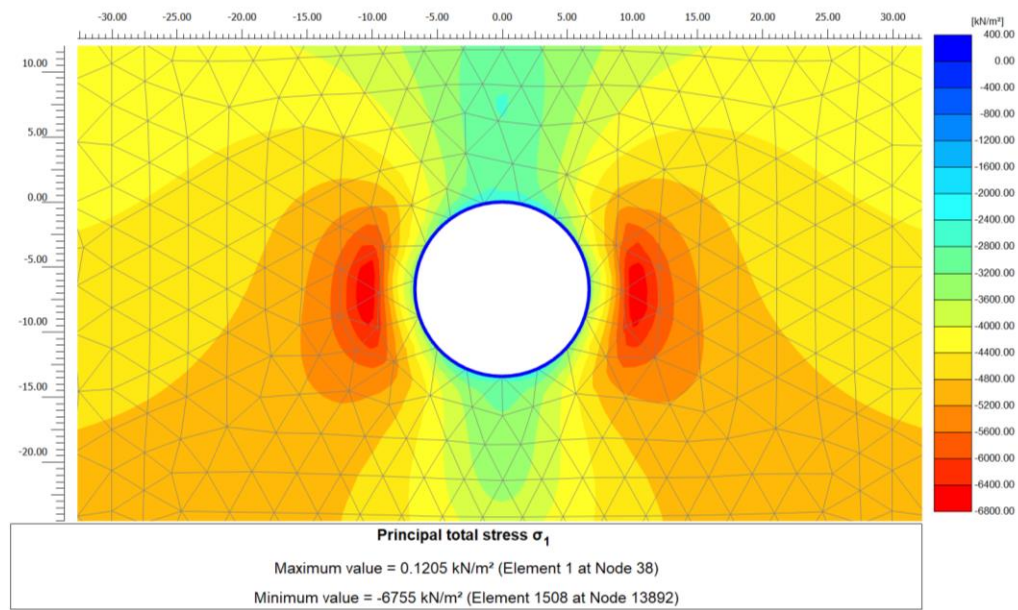


Figure 5.15 Major Principal Stress Contours Around the Tunnel in Analysis 2

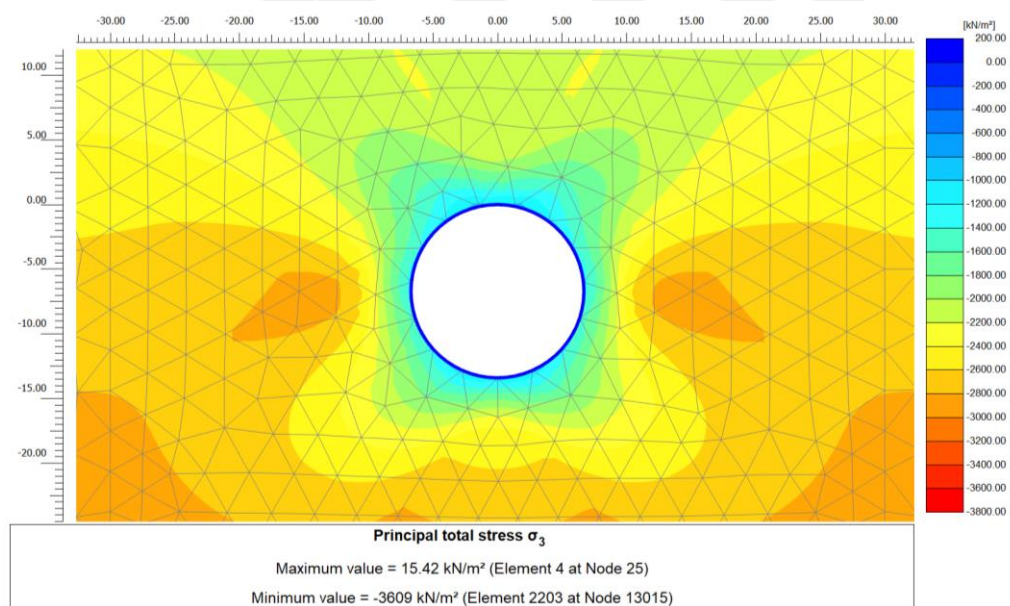


Figure 5.16 Minor Principal Stress Contours Around the Tunnel in Analysis 2

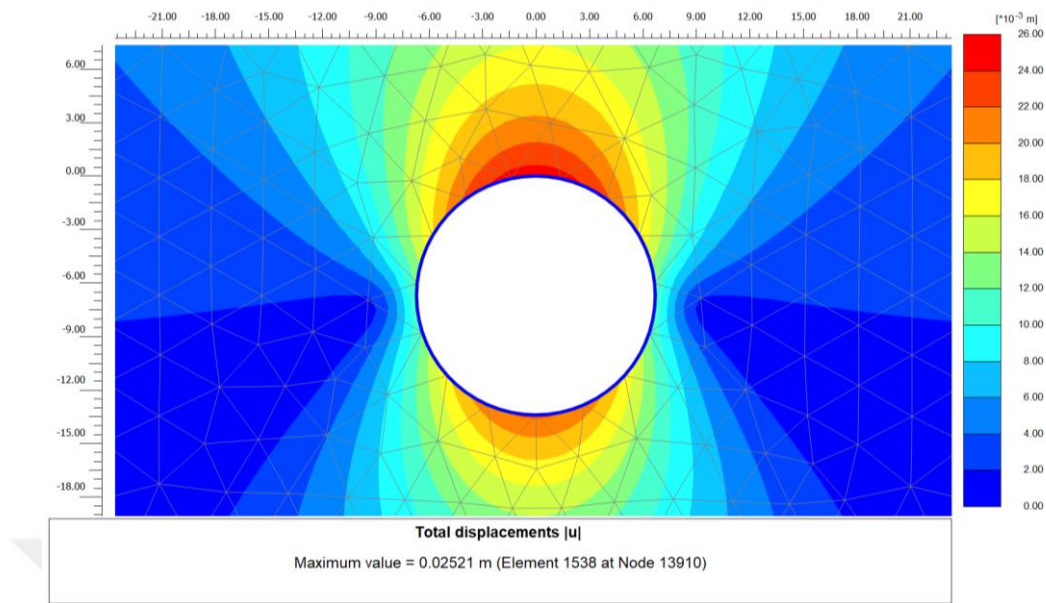


Figure 5.17 Total Displacements Around Tunnel Boundary in Analysis 2

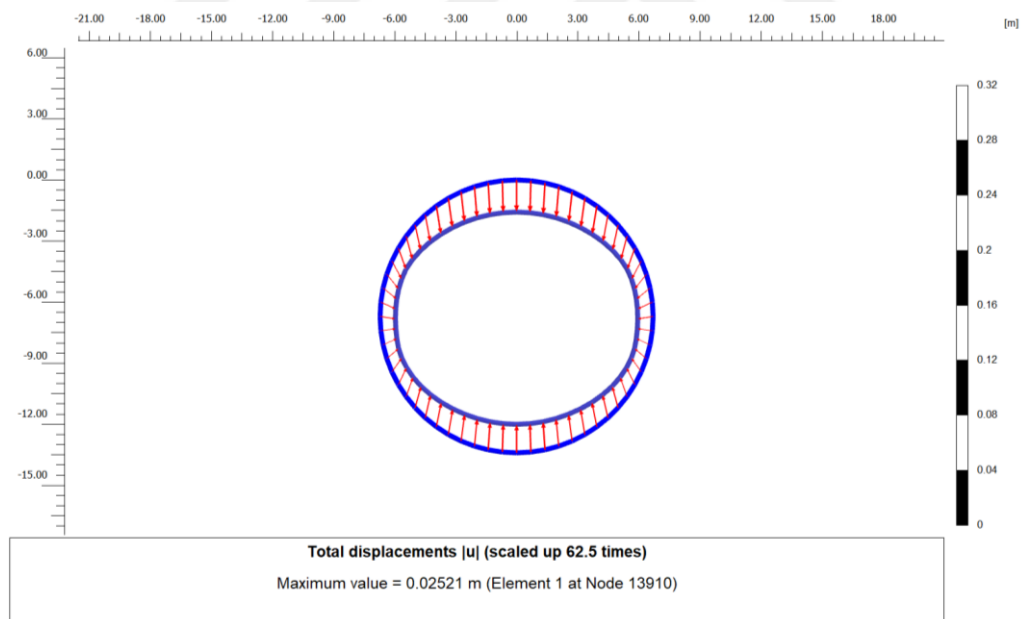


Figure 5.18 Total Displacements at the Precast Segments in Analysis 2

In FE analysis, the radius of the plastic zone is determined as 10.64m, which results in an 8.74% error with the preliminary assessment. Additionally, in the FE analysis, the displacements in the precast segments are calculated as 25.21mm, which

smoothly fits the preliminary observations with only 2.74% deviation. Therefore, the verification of the preliminary assessment and the FE results are satisfied.

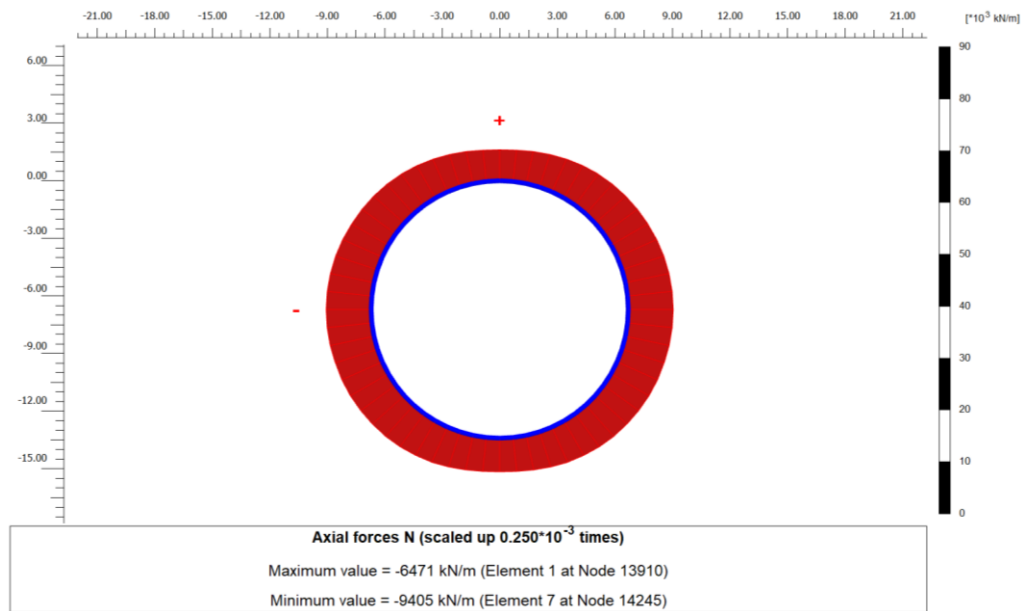


Figure 5.19 Axial Forces at the Precast Segments in Analysis 2

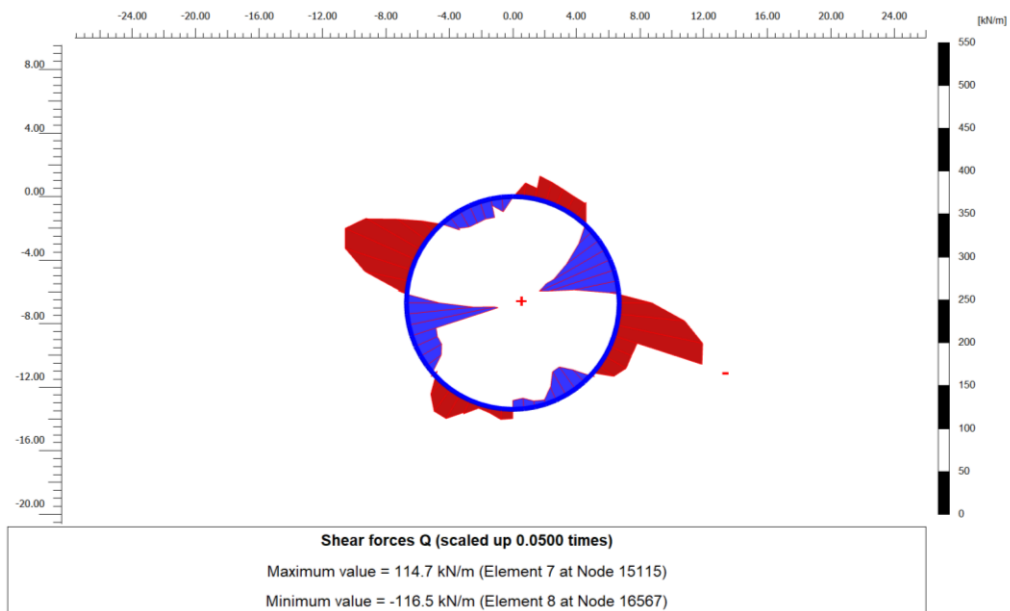


Figure 5.20 Shear Forces at the Precast Segments in Analysis 2

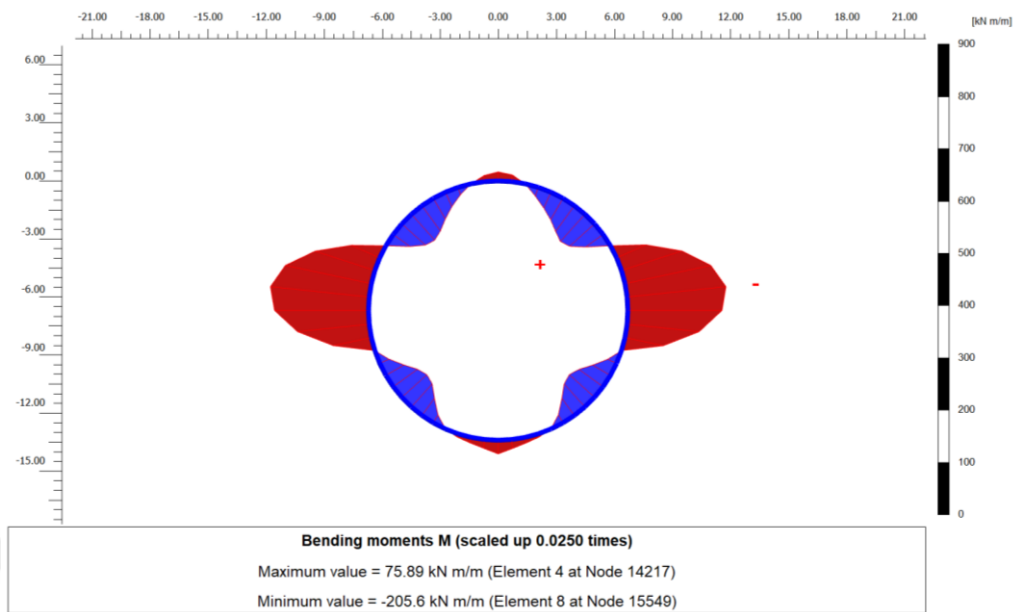


Figure 5.21 Bending Moments at the Precast Segments in Analysis 2

According to the resulting axial force, shear force, and bending moment diagrams, the static verification of the sections was performed, and the interaction diagram is provided in Figure 5.22.

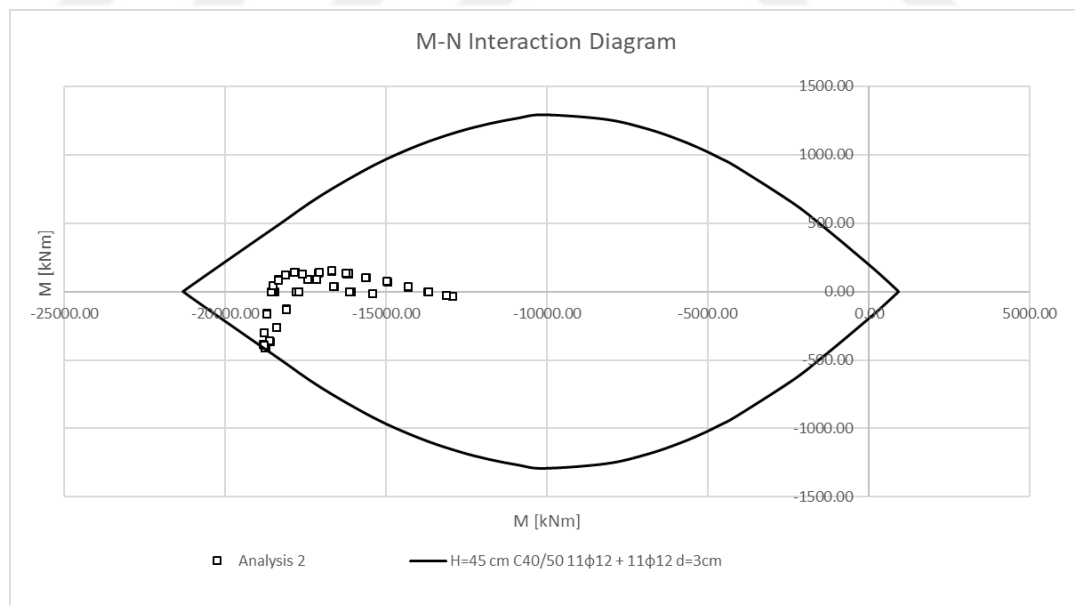


Figure 5.22 Interaction Diagram of Analysis 2

The outcomes of Analysis 2, as expected, are not tended to result in an occurrence of a collapse. The deflections at the tunnel support are verified with numerical models and result compatibly with the monitoring data. The static verifications also show that there is no evidence of a failure throughout the precast segments in this section.

### **5.3 Analysis 3-Alternative A**

Analysis 3-Alternative is devoted to assessing the tunnel performance in TBM excavation in KM:217+378. Analysis 3-Alternative A is conducted according to geotechnical parameters of the soil determined from the back analysis. According to the monitoring data, the expected deformation in the precast segments is determined approximately 150mm in this section. In this case, the displacements exhibit an increasing trend, and the partial collapse was initiated.

Figure 5.23 and Figure 5.24 indicate the ground-support reaction curve and the tunnel section view. The maximum displacement at the walls is calculated as 162.09mm, and the radius of the plastic zone is calculated as 10.44m. Additionally, the factor of safety is calculated as 0.98. As the terminology of the factor of safety suggests, the provided support measures are not able to carry the load applied to them. Therefore, according to the preliminary assessment, the provided support system is not safe to resist the loads applied to it.

As demonstrated in Figure 5.23, the support installation took place earlier than the optimal relaxation rate of the host ground. The capacity of the precast segments was not sufficient to counterbalance the pressure applied from the host ground at the time the *equilibrium*, in which point of junction of the ground reaction curve and the support reaction curve, was reached.

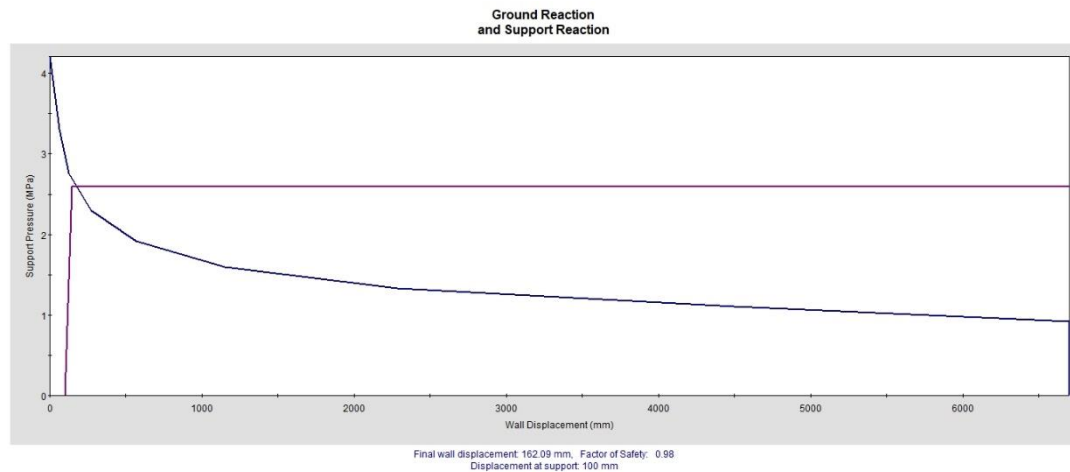


Figure 5.23 Ground-Support Reaction Curve of Analysis 3-Alternative A

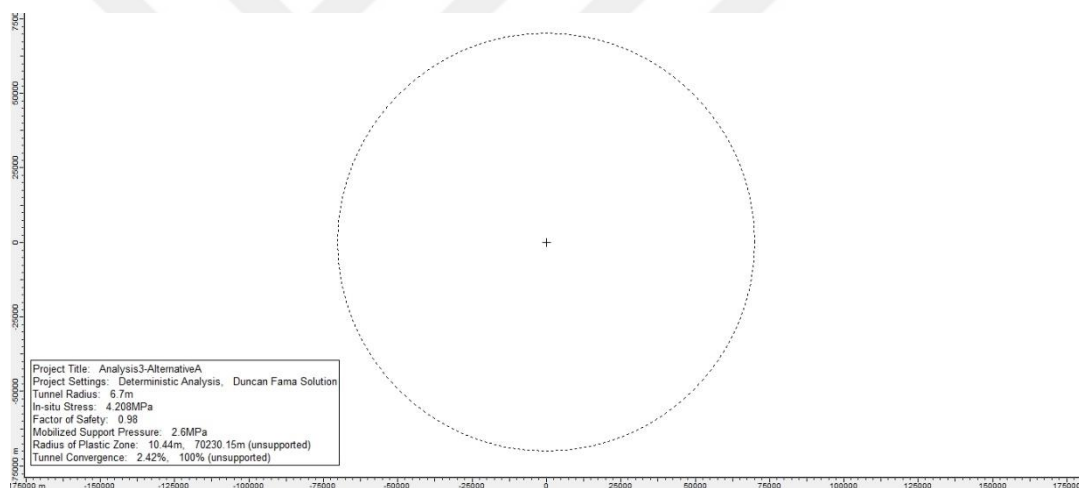


Figure 5.24 Tunnel Section View of Analysis 3-Alternative A

Following the analyses performed on RocSupport, 2D finite element analyses are performed for further evaluation of the deformations, stress distributions, and support reactions. The radius of the plastic zone, principal stress contours around the tunnel boundary, expected deformations at the segments, and compatibility of the support reactions are compared in the analyses conducted in Plaxis 2D. The finite element analyses of Analysis 3-Alternative A, are performed according to the procedures described in Chapter 4.2.

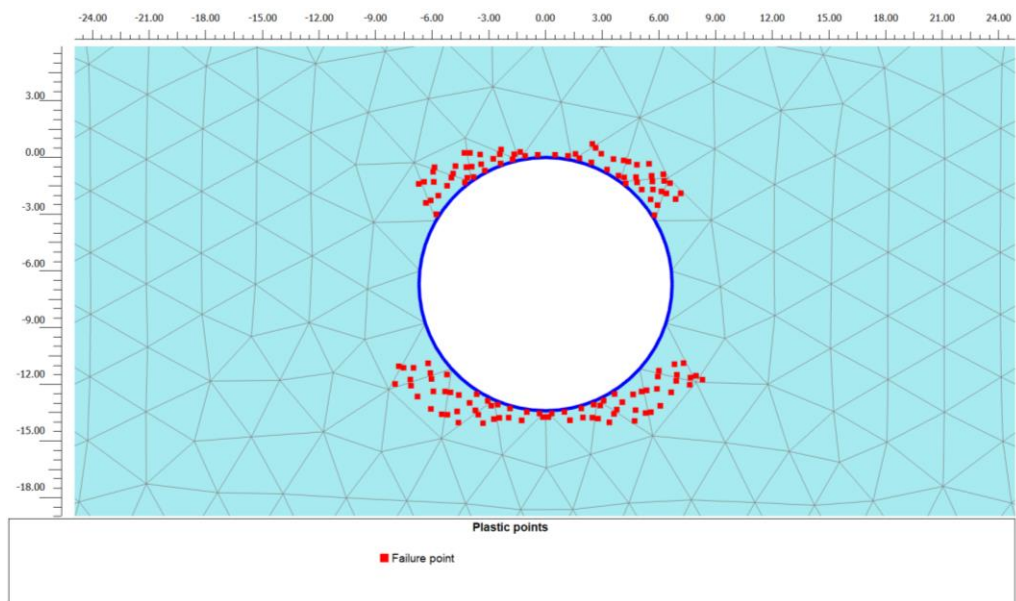


Figure 5.25 Resulting Plastic Zone Around the Tunnel in Analysis 3-Alternative A

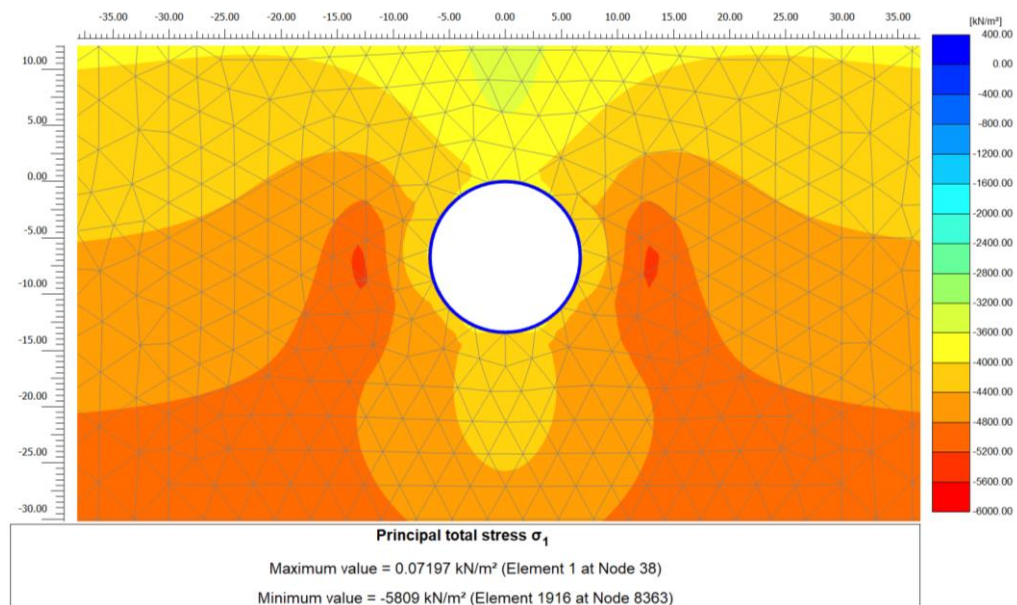


Figure 5.26 Major Principal Stress Contours Around the Tunnel in Analysis 3-Alternative A

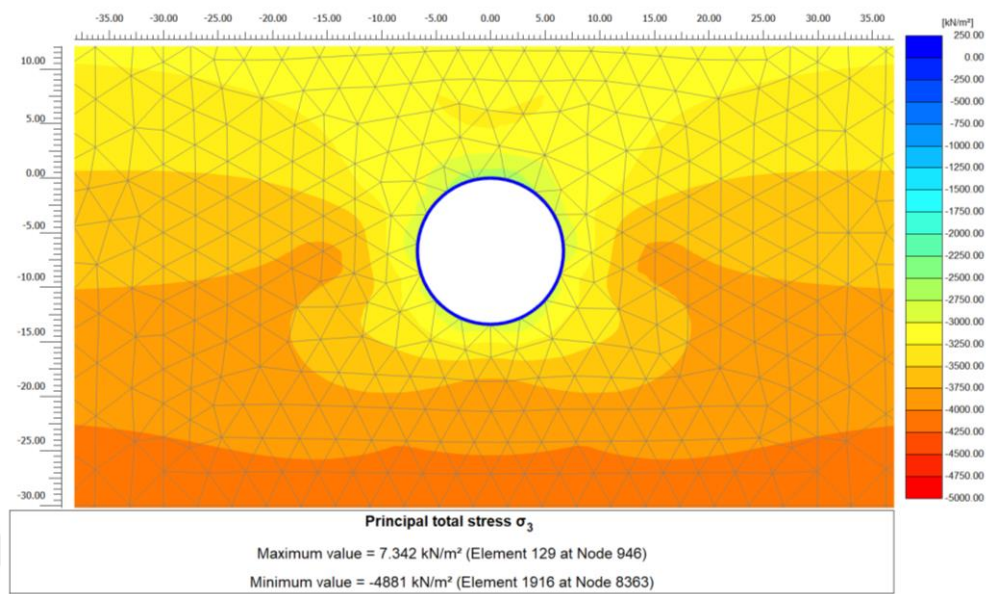


Figure 5.27 Minor Principal Stress Contours Around the Tunnel in Analysis 3-Alternative A

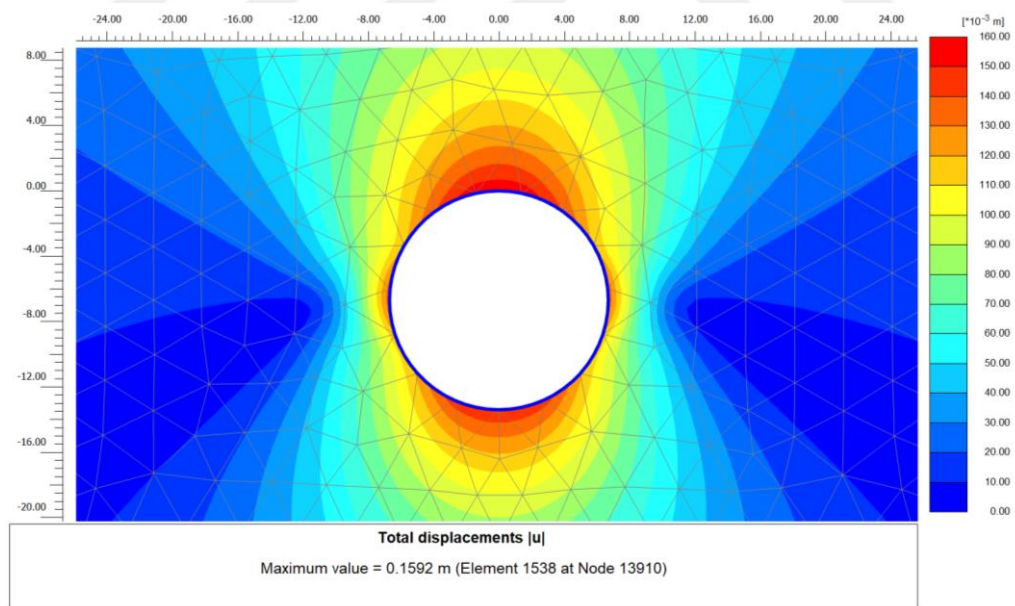


Figure 5.28 Resulting Total Displacements Around Tunnel Boundary in Analysis 3-Alternative A

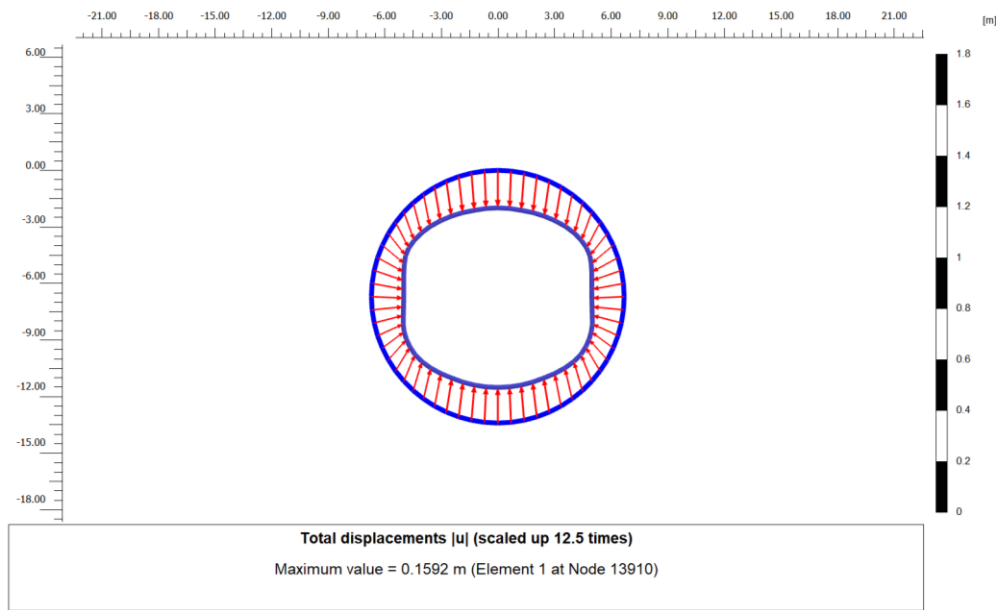


Figure 5.29 Total Displacements at the Precast Segments in Analysis 3-Alternative A

In FE analysis, the radius of the plastic zone is determined as 9.72m, which results in a 6.90% error with the preliminary assessment. Additionally, in the FE analysis, the displacements in the precast segments are calculated as 15.92mm, which gives a highly consistent result with the preliminary observations with only 1.78% deviation. Therefore, the verification of the preliminary assessment and the FE results are satisfied. In the preliminary evaluations, the support system was determined to perform inadequately to resist failure due to the generated stresses on them. The further evaluation of the statical verification of the cross-section is conducted in FE analyses.

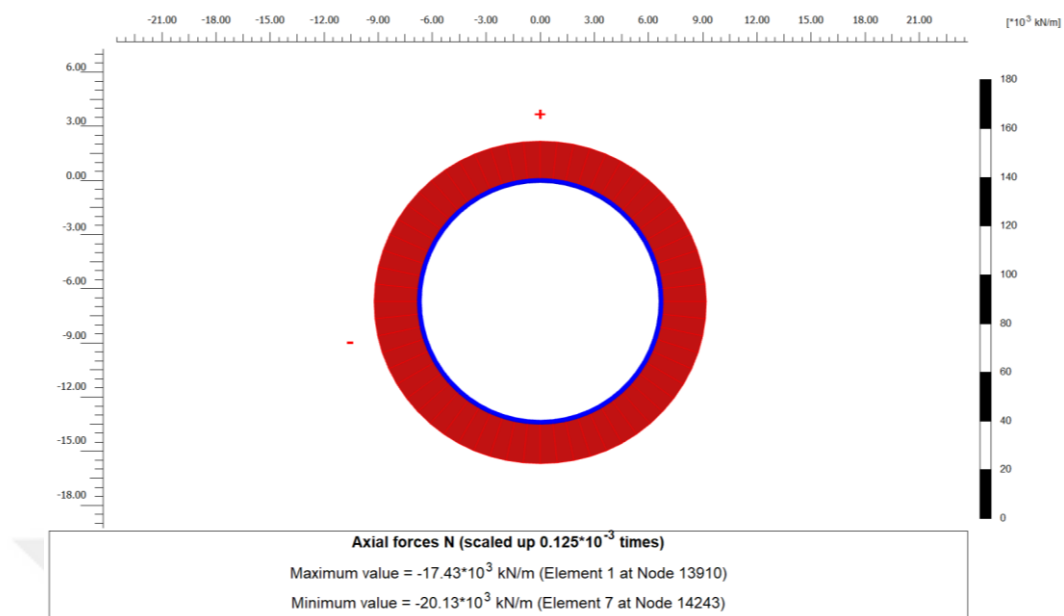


Figure 5.30 Axial Forces at the Precast Segments in Analysis 3-Alternative A

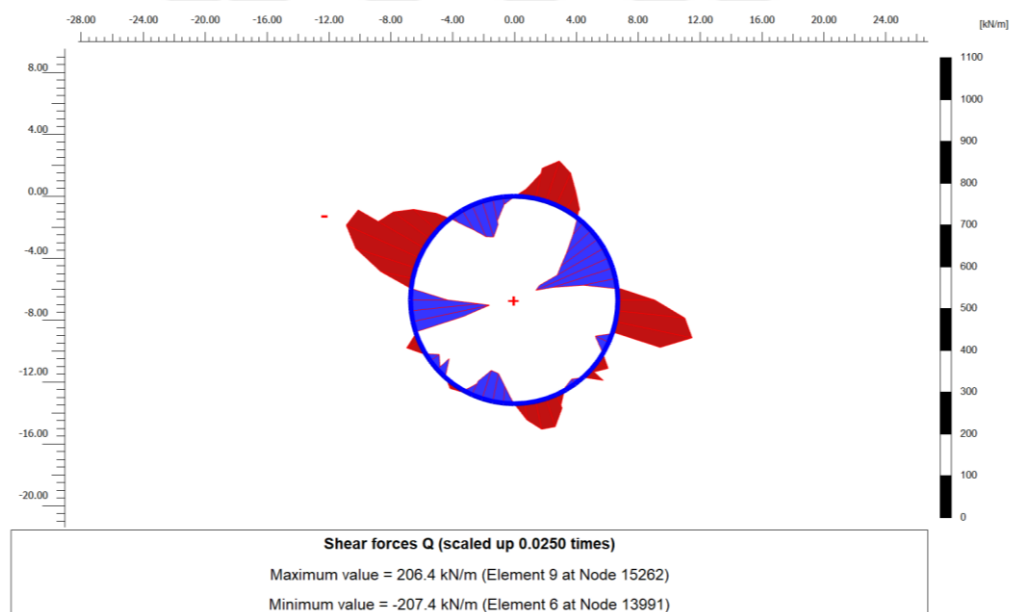


Figure 5.31 Shear Forces at the Precast Segments in Analysis 3-Alternative A

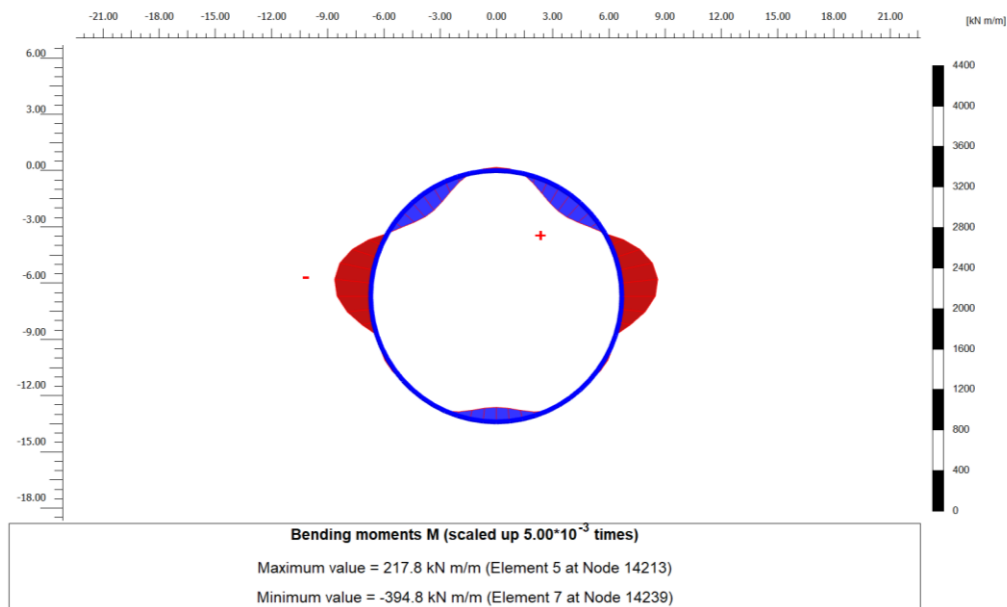


Figure 5.32 Bending Moments at the Precast Segments in Analysis 3-Alternative A

According to the resulting axial force, shear force, and bending moment diagrams, the static verification of the sections was performed, and the interaction diagrams are provided in Figure 5.33.

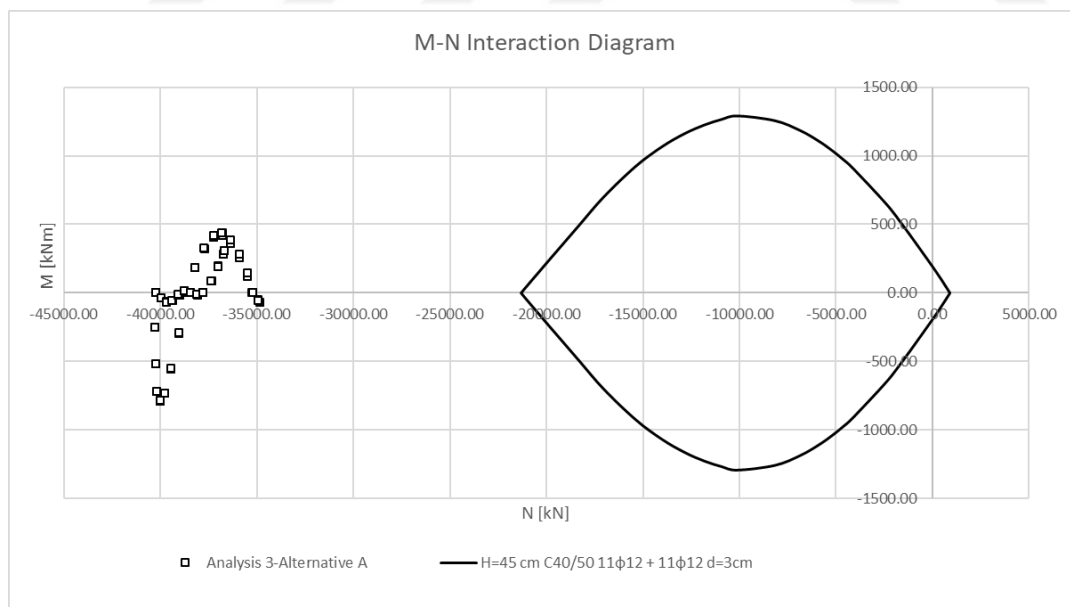


Figure 5.33 Interaction Diagram of Analysis 3-Alternative A

The deflections at the tunnel support are verified with numerical models and result compatibly with the monitoring data. However, the outcomes of Analysis 3-Alternative A, show that the tunnel's stability is not satisfied with the proposed support system, both in the preliminary analyses and in the static verification of the segments.

#### 5.4 Analysis 3-Alternative B

Analysis 3-Alternative B is devoted to assessing the tunnel advancement in NATM after the partial collapse in KM: 217+373. Based on the monitoring data, the expected maximum deformation in KM:217+373 with the NATM technique is 102mm. Analysis 3-Alternative B is composed of an altered tunnel construction after the partial collapse incident demonstrated in Analysis 3-Alternative A.

As seen in Figure 5.34 and Figure 5.35, the radius of the plastic zone is calculated as 12.64m, the maximum displacement expected at the tunnel walls are determined as 101.46mm. In addition, the factor of safety is calculated as 6.58. Hence, the support system interacts adequately with the specified ground conditions, according to the preliminary analyses.

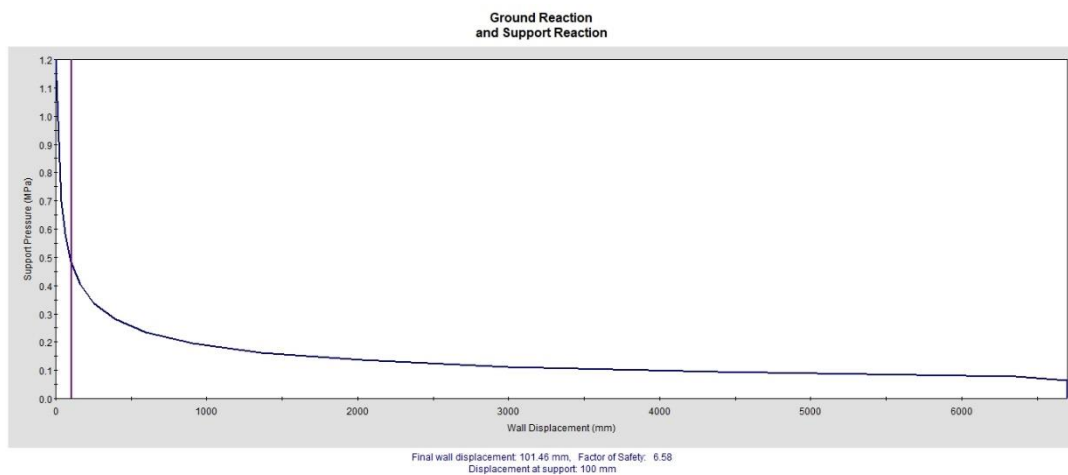


Figure 5.34 Ground-Support Reaction Curve of Analysis 3-Alternative B

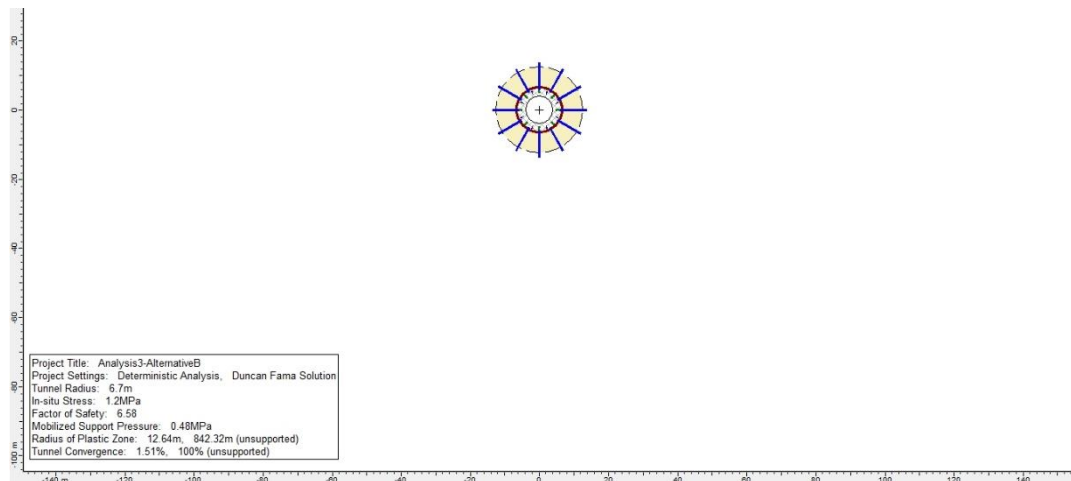


Figure 5.35 Tunnel Section View of Analysis 3-Alternative B

Following the analyses performed on RocSupport, 2D finite element analyses are performed for further evaluation of the deformations, stress distributions, and support reactions. The radius of the plastic zone, principal stress contours around the tunnel boundary, expected deformations at the segments, and compatibility of the support reactions are compared in the analyses conducted in Plaxis 2D. The finite element analysis of Analysis 3-Alternative B was performed according to the procedures described in Chapter 4.3.

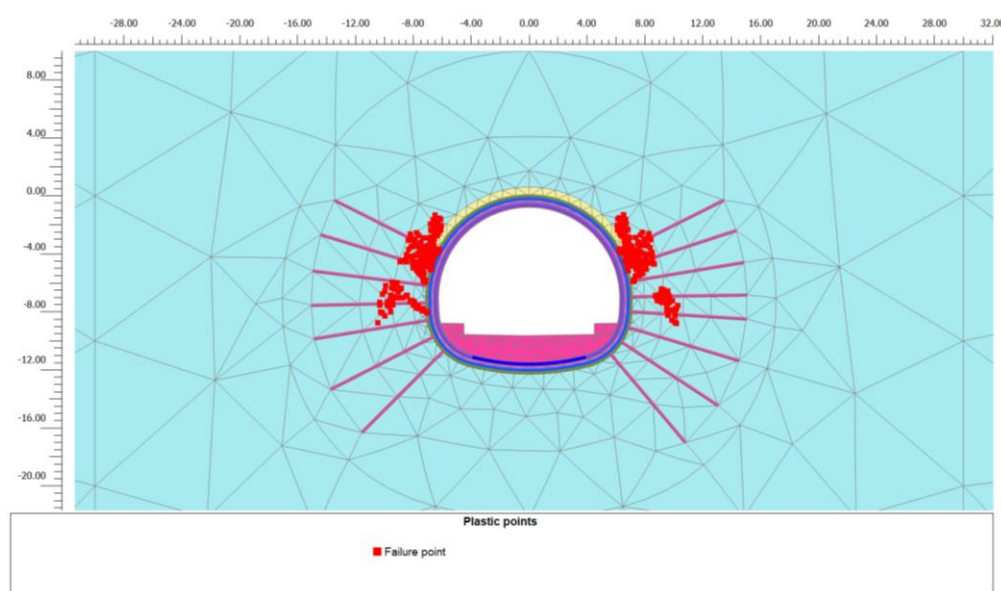


Figure 5.36 Plastic Zone Around the Tunnel in Analysis 3-Alternative B

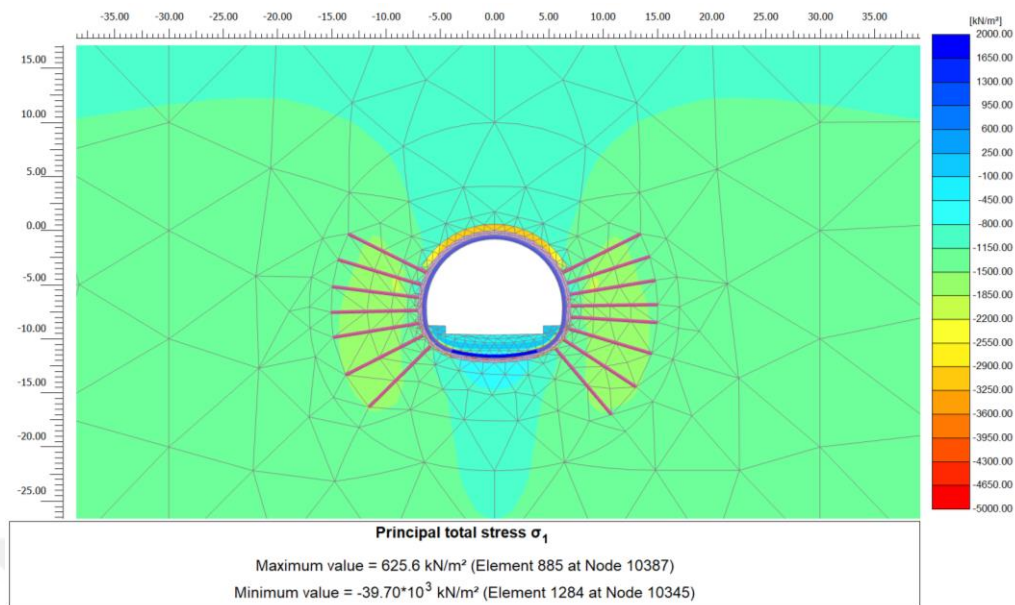


Figure 5.37 Major Principal Stress Contours Around the Tunnel in Analysis 3-Alternative B

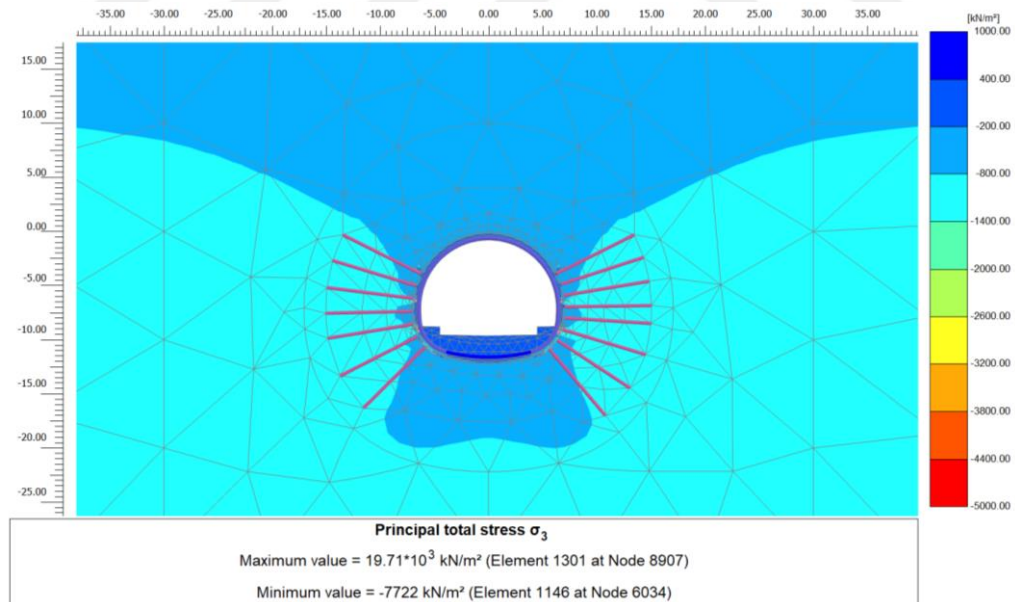


Figure 5.38 Minor Principal Stress Contours Around the Tunnel in Analysis 3-Alternative B

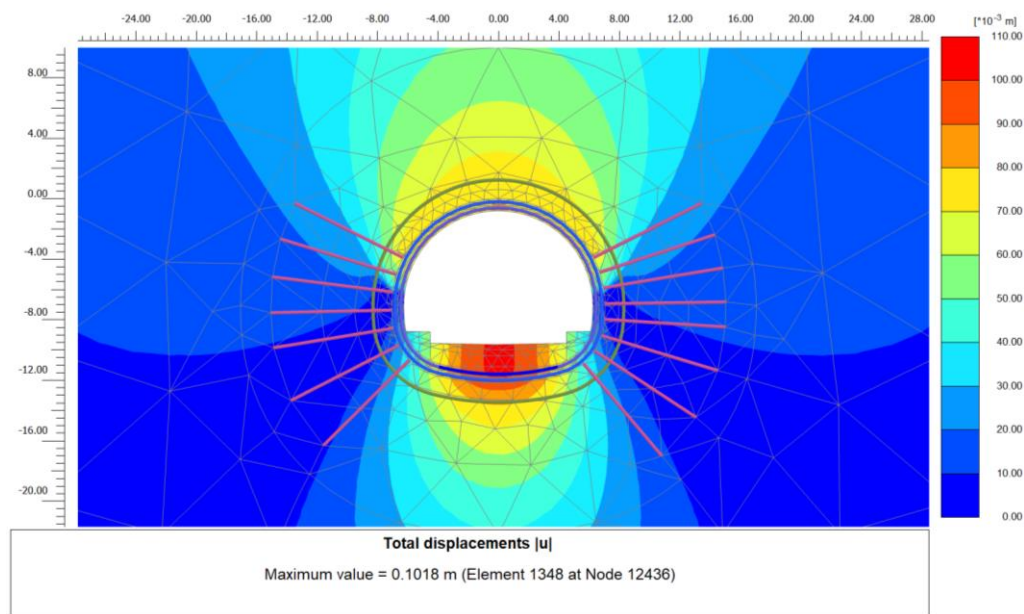


Figure 5.39 Total Displacements Around Tunnel Boundary in Analysis 3-Alternative B

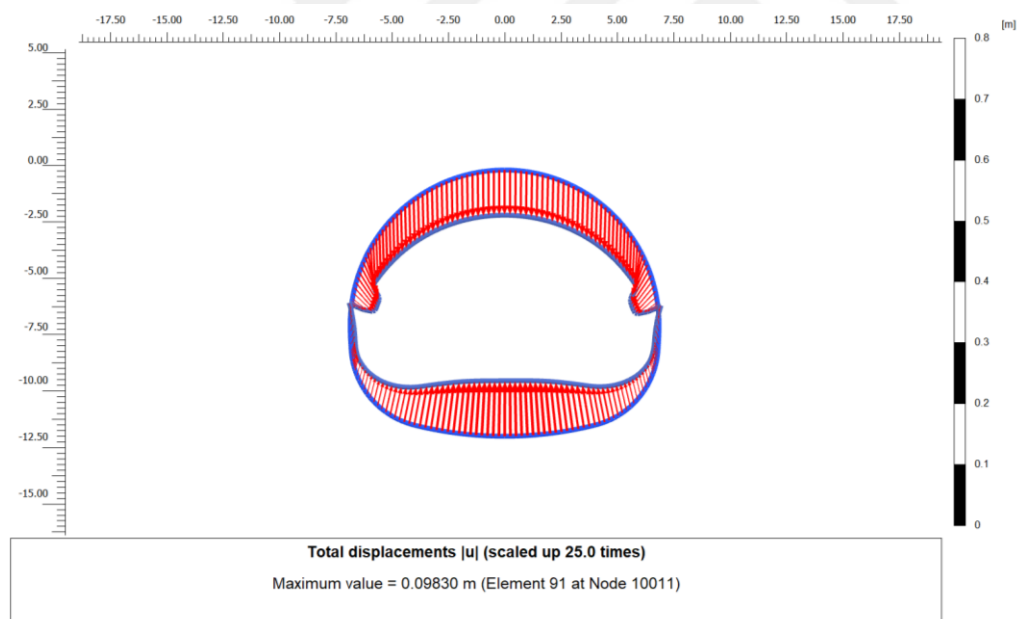


Figure 5.40 Total Displacements at the Primary Lining in Analysis 3-Alternative B

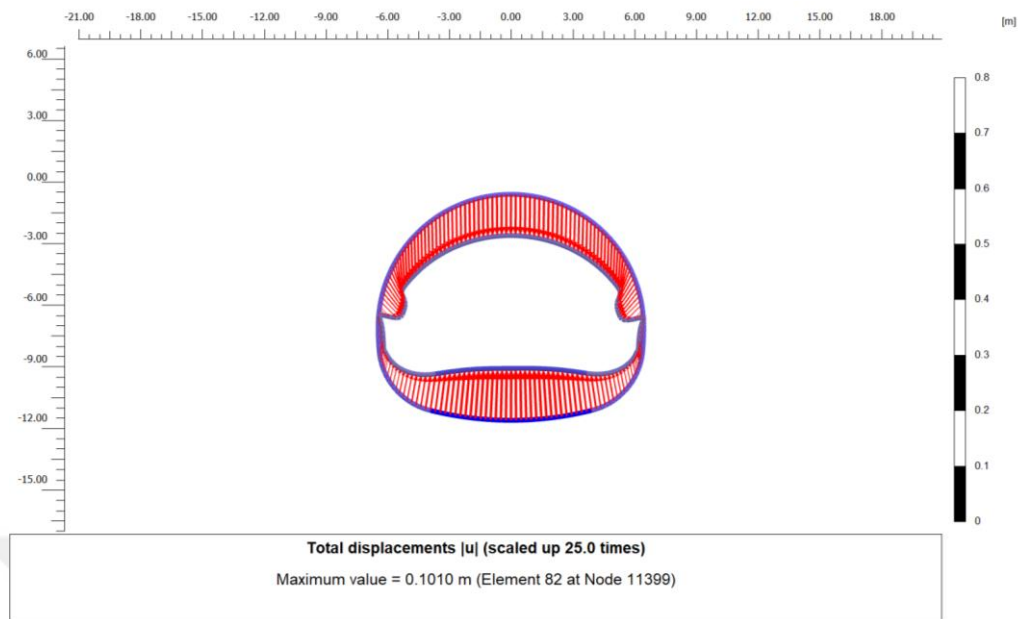


Figure 5.41 Total Displacements at the Final Lining in Analysis 3-Alternative B

In FE analysis, the radius of the plastic zone is determined as 10.95m, which results in a 13.37% error with the preliminary assessment. Additionally, in the FE analysis, the displacements in the primary lining are calculated as 98.3mm, which gives a highly consistent result with the preliminary observations with only 3.11% deviation.

Although the assessment of the plastic radius zone relatively deviates in preliminary analyses and in FE analyses, the obtained final displacements at the primary lining results in considerably compatible outcomes. As previously mentioned, this margin in plastic radius zones can be caused by the presence of the water table, the applied relaxation factors in FE analyses as in the analyses performed in RocSupport, the latter mentioned factors are not taken into account. Therefore, the verification of the preliminary assessment and the FE results are satisfied. The further evaluation of the statical verification of the cross-section is conducted in FE analyses.

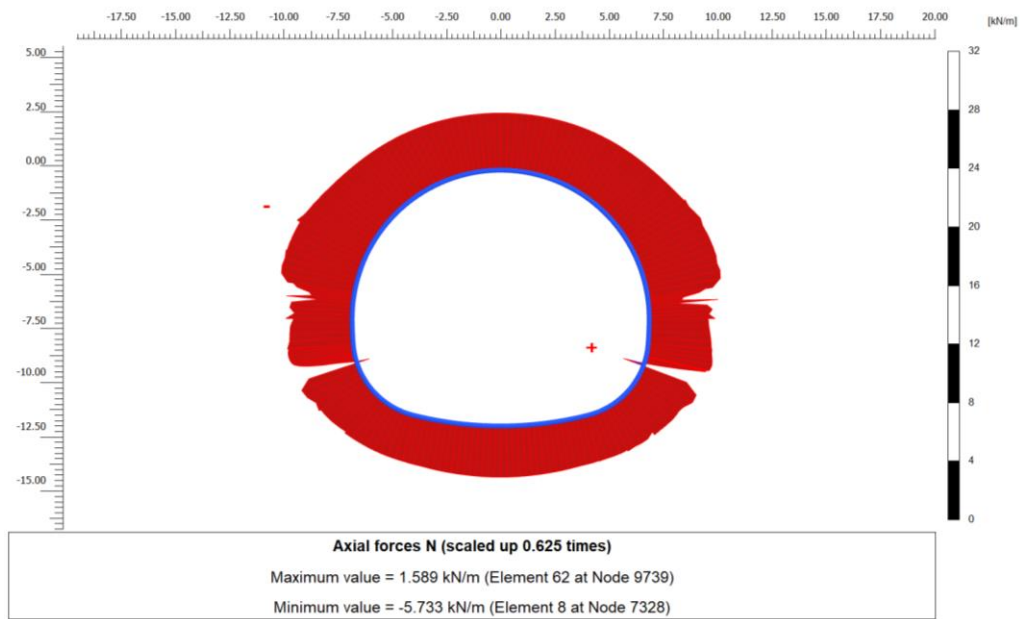


Figure 5.42 Axial Forces at the Primary Lining in Analysis 3-Alternative B

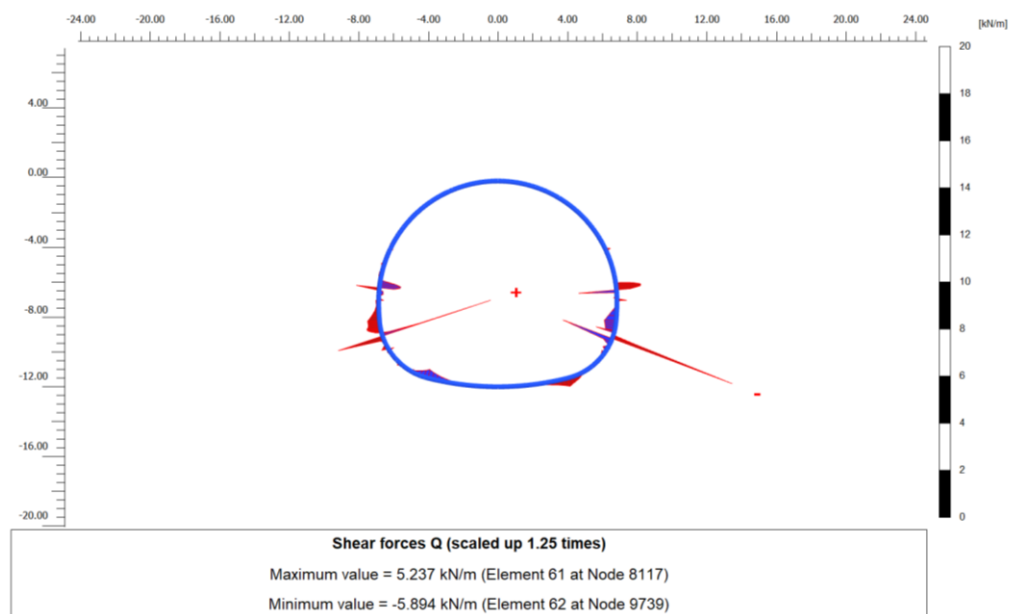


Figure 5.43 Shear Forces at the Primary Lining in Analysis 3-Alternative B

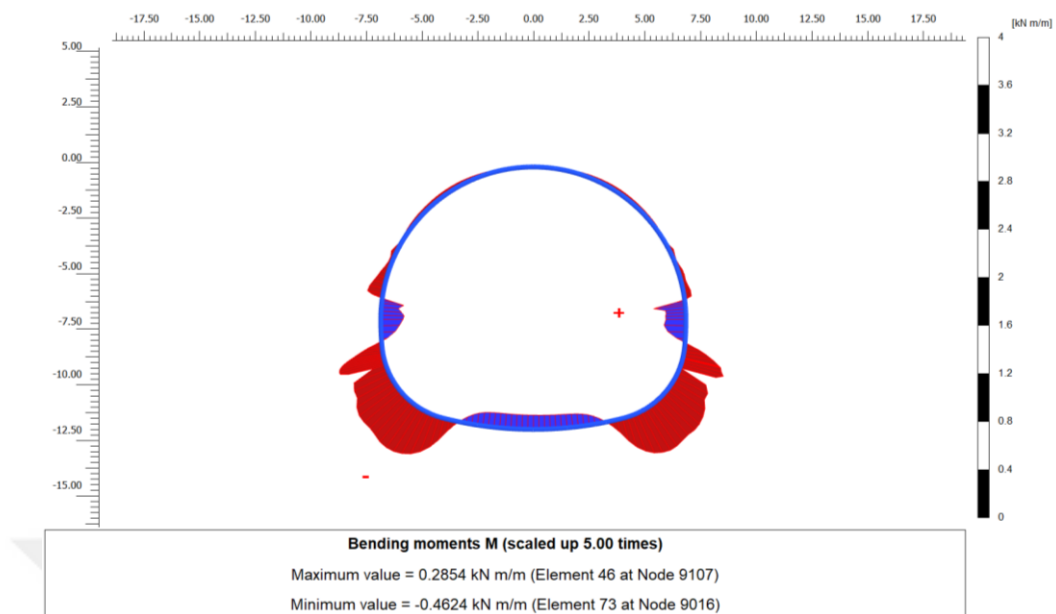


Figure 5.44 Bending Moments at the Primary Lining in Analysis 3-Alternative B

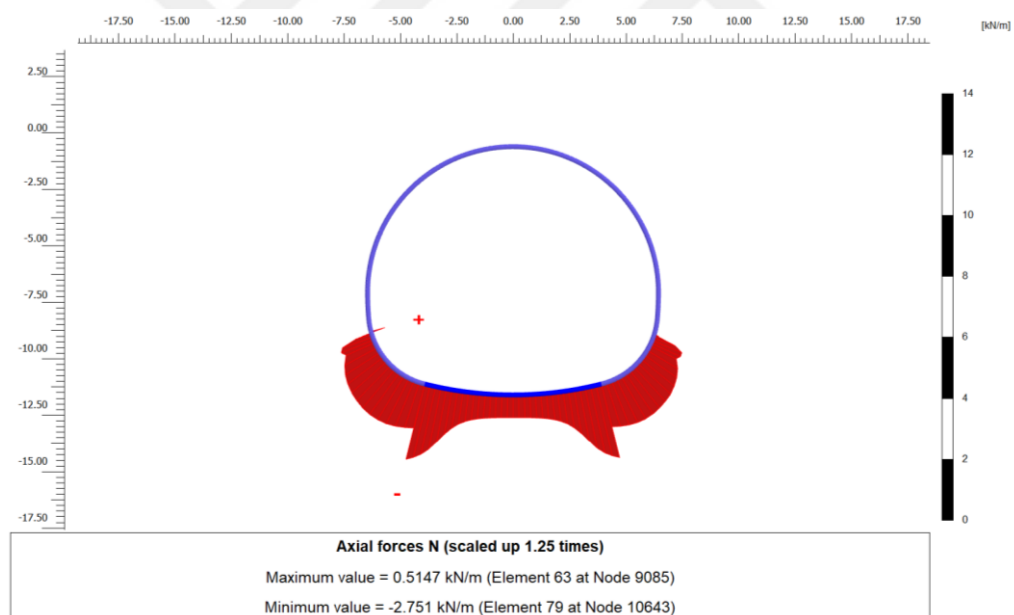


Figure 5.45 Axial Forces at the Final Lining in Analysis 3-Alternative B

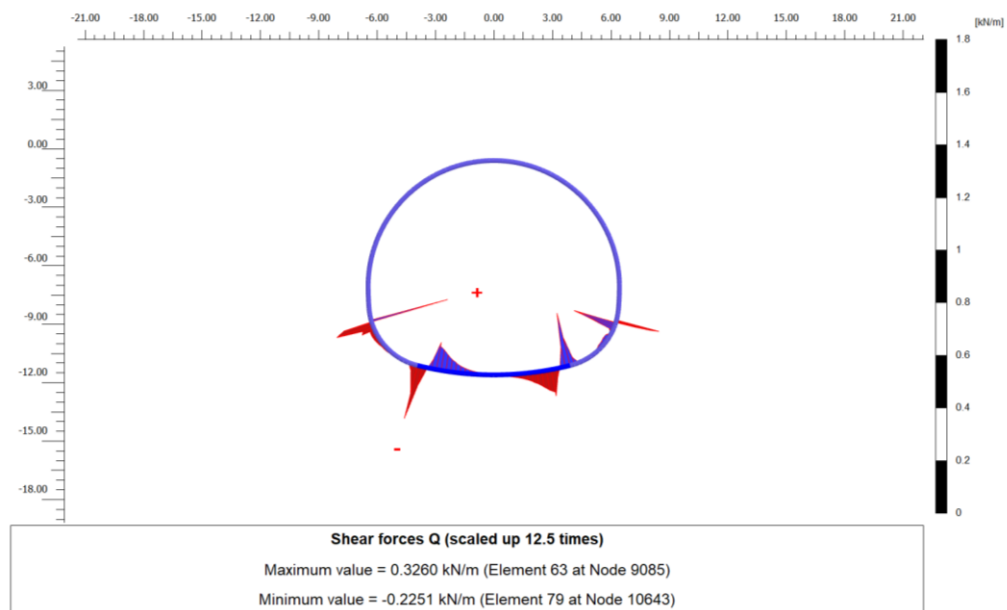


Figure 5.46 Shear Forces at the Final Lining in Analysis 3-Alternative B

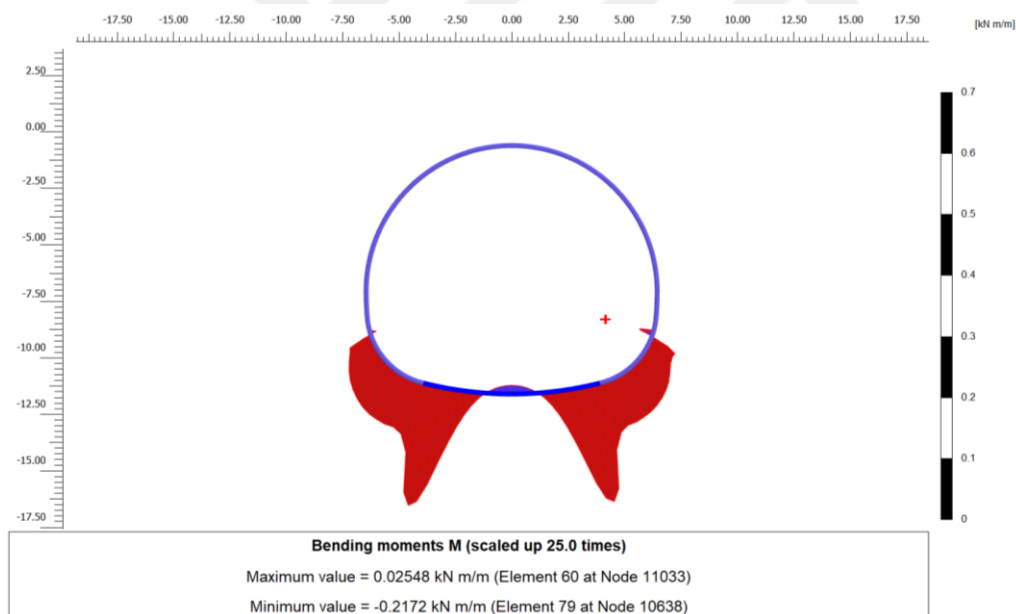


Figure 5.47 Bending Moments at the Final Lining in Analysis 3-Alternative B

According to the resulting axial force, shear force, and bending moment diagrams, the static verification of the primary lining and the final lining was performed, and the interaction diagrams are provided in Figure 5.48 and Figure 5.49.

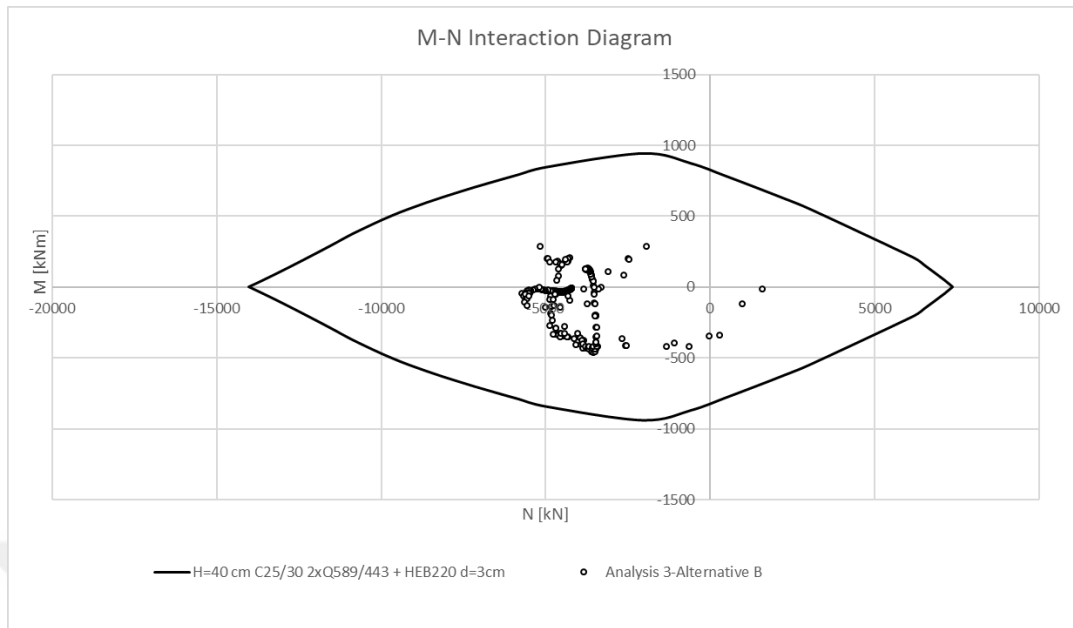


Figure 5.48 Interaction Diagram of Analysis 3-Alternative B – Primary Lining

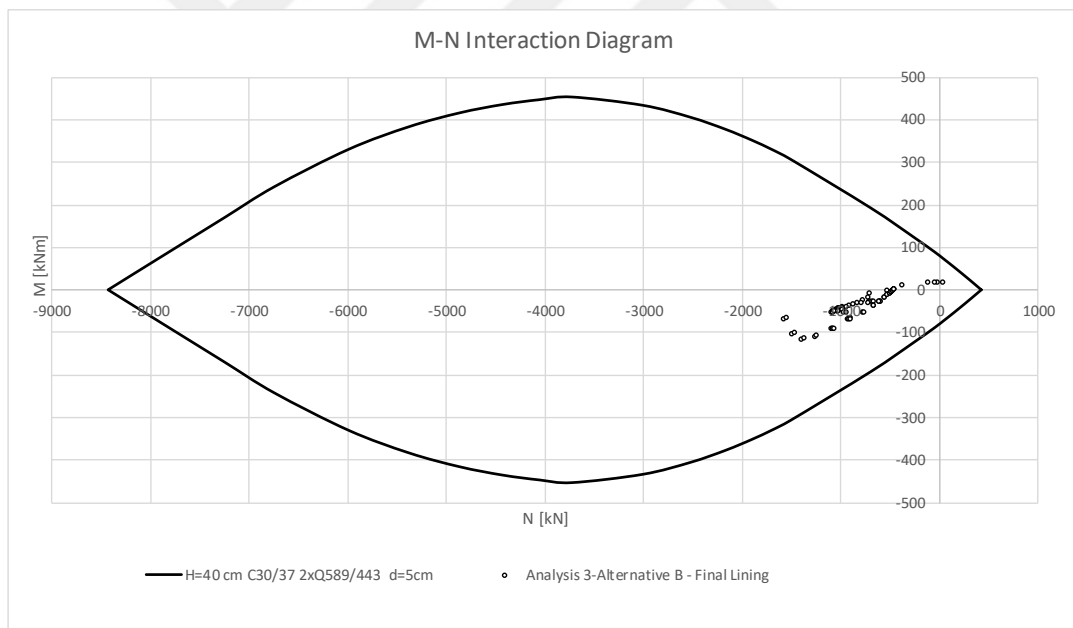


Figure 5.49 Interaction Diagram of Analysis 3-Alternative B – Final Lining

The outcomes of Analysis 3-Alternative B, as expected, are not tended to result in an occurrence of a collapse. The deflections at the tunnel support are verified with numerical models and result compatibly with the monitoring data. The static verifications also show that there is no evidence of a failure throughout the primary

and final lining in this section. Therefore, the tunnel advancement with the altered construction technique is safe throughout the section.

## 5.5 Analysis 4-Alternative A

Analysis 4-Alternative A is devoted to assessing the tunnel performance in TBM excavation in KM:217+395. The analysis is based on the monitoring data, in which the total deformations in KM: 217+395 was determined as 231mm. In this section, the observed deformations reached their peak, and the partial collapse occurred. The Analysis 4-Alternative A is conducted according to geotechnical parameters of the soil determined from the back analysis.

Figure 5.50 and Figure 5.51 indicate the ground-support reaction curve and the tunnel section view. The maximum displacement at the walls is calculated as 255.85mm, and the radius of the plastic zone is calculated as 13.44m. Additionally, the factor of safety is calculated as 0.95. As the terminology of the factor of safety suggests, the provided support measures are not able to carry the load applied to them. Therefore, according to the preliminary assessment, the provided support system is not safe to resist the loads applied to it.

As demonstrated in Figure 5.50, the support installation took place earlier than the optimal relaxation rate of the host ground. The capacity of the precast segments was not sufficient to counterbalance the pressure applied from the host ground at the time the *equilibrium*, in which point of junction of the ground reaction curve and the support reaction curve, was reached.

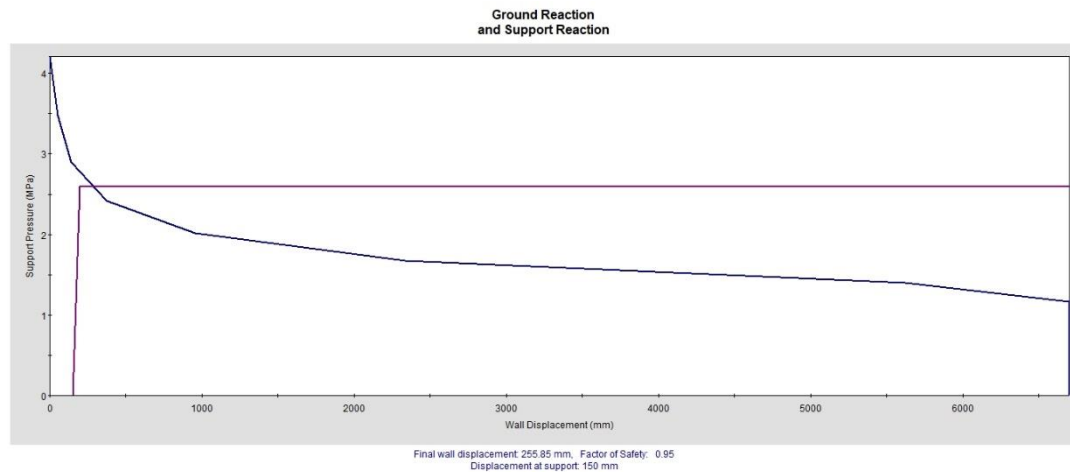


Figure 5.50 Ground-Support Reaction Curve of Analysis 4-Alternative A

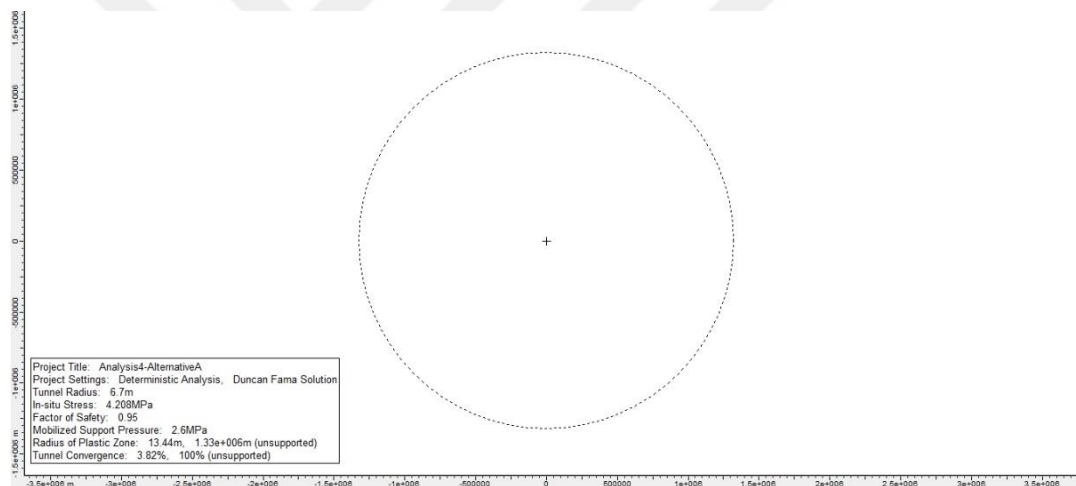


Figure 5.51 Tunnel Section View of Analysis 4-Alternative A

Following the analyses performed on RocSupport, 2D finite element analyses are performed for further evaluation of the deformations, stress distributions, and support reactions. The radius of the plastic zone, principal stress contours around the tunnel boundary, expected deformations at the segments, and compatibility of the support reactions are compared in the analyses conducted in Plaxis 2D. The finite element analysis of Analysis 4-Alternative A was performed according to the procedures described in Chapter 4.2.

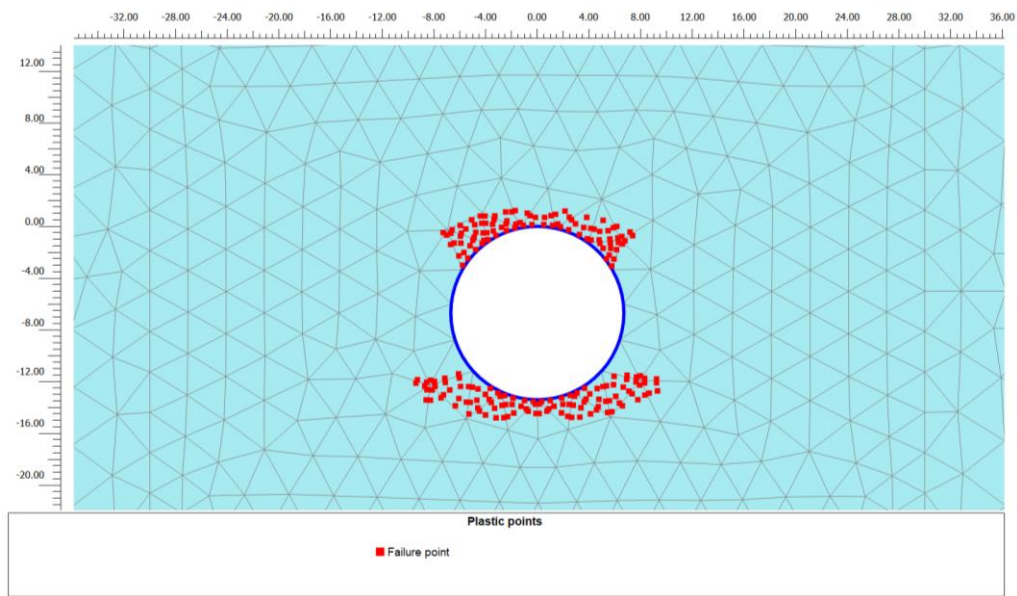


Figure 5.52 Plastic Zone Around the Tunnel in Analysis 4-Alternative A

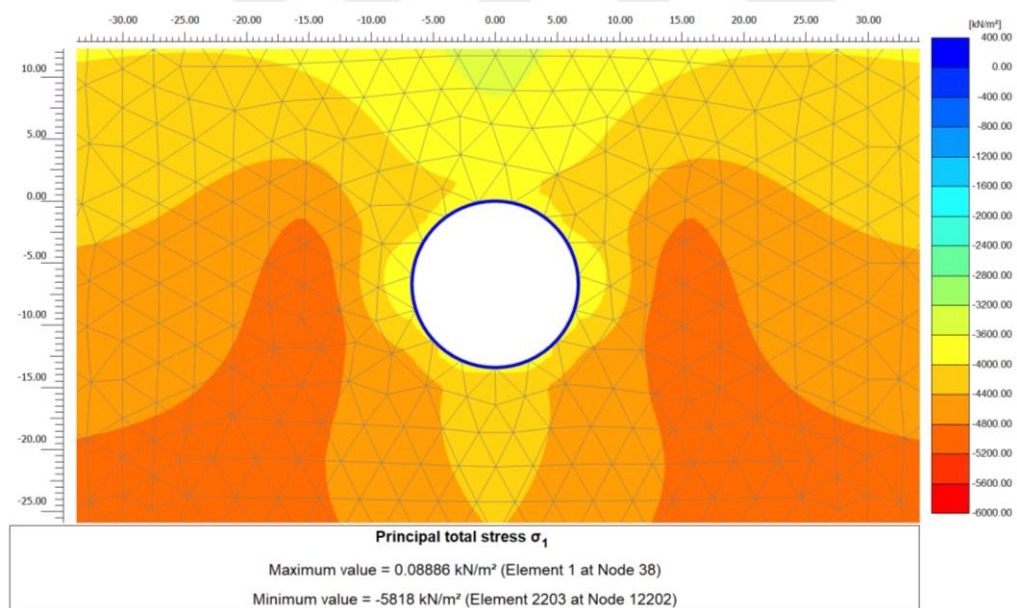


Figure 5.53 Major Principal Stress Contours Around the Tunnel in Analysis 4-Alternative A

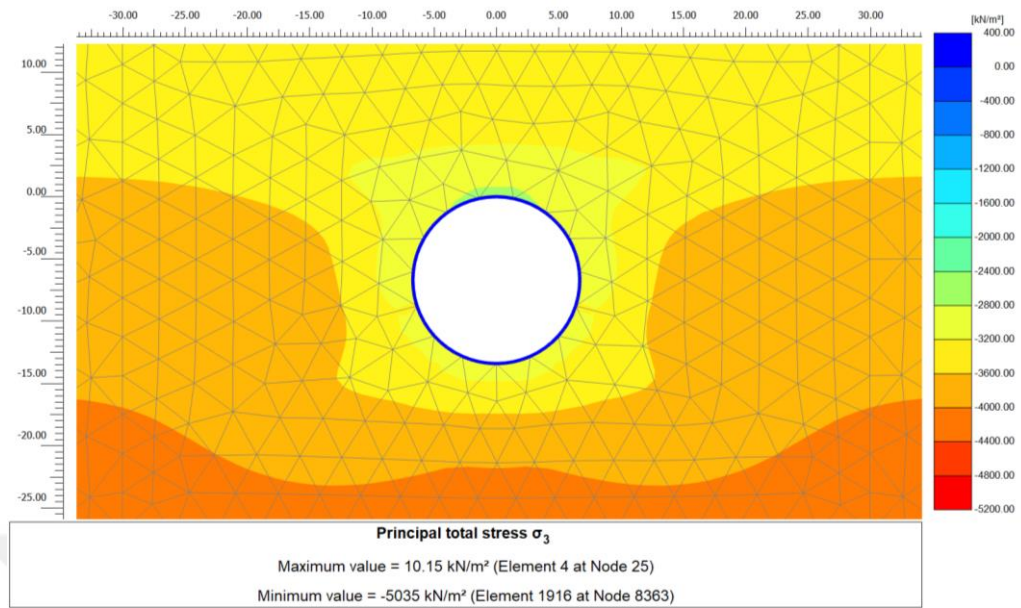


Figure 5.54 Minor Principal Stress Contours Around the Tunnel in Analysis 4-Alternative A

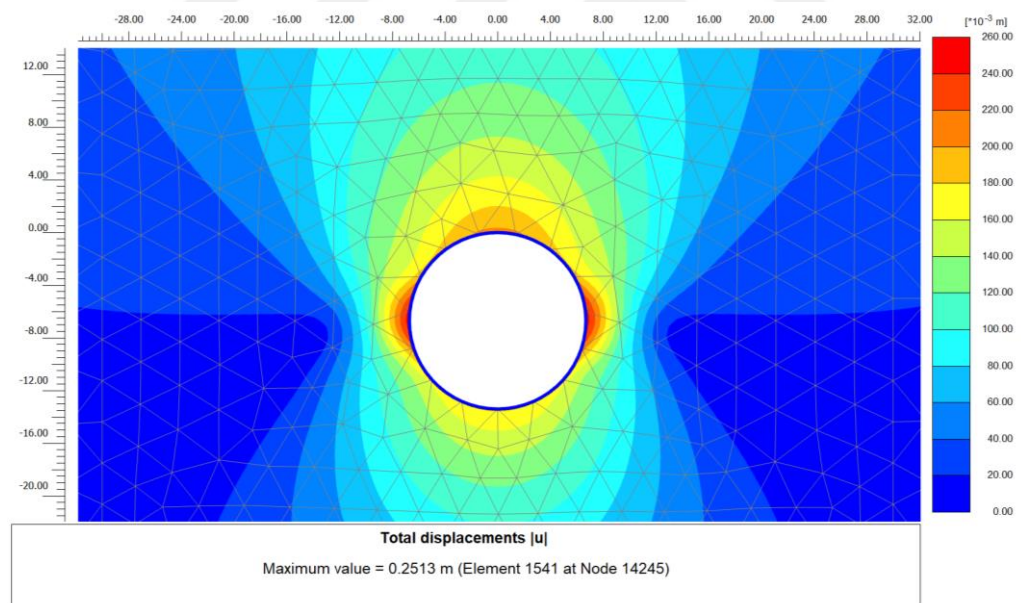


Figure 5.55 Total Displacements Around Tunnel Boundary in Analysis 4-Alternative A

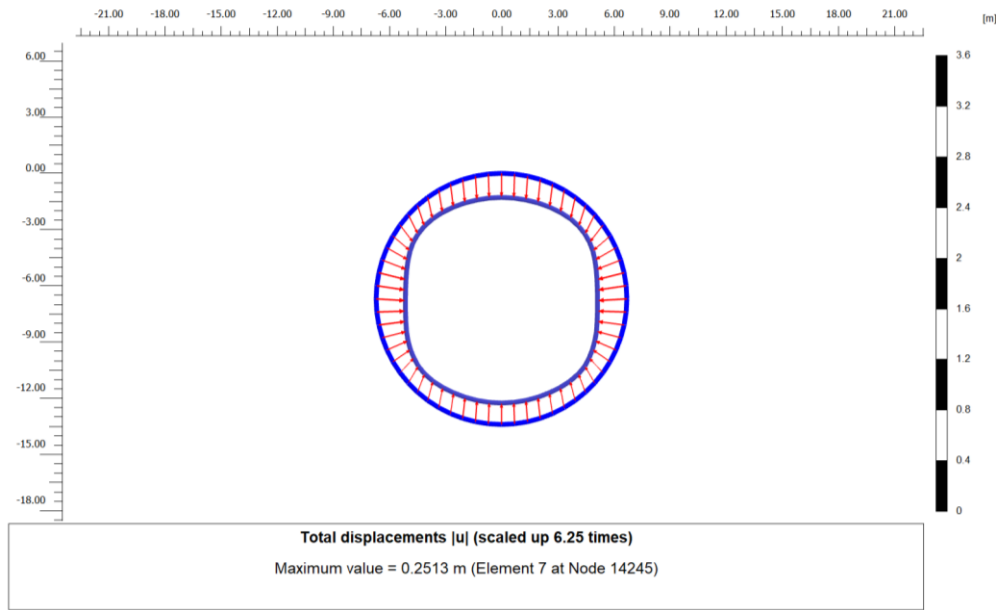


Figure 5.56 Total Displacements at the Precast Segments in Analysis 4-Alternative A

In FE analysis, the radius of the plastic zone is determined as 11.17m, which results in a 12.87% error with the preliminary assessment. Additionally, in the FE analysis, the displacements in the precast segments are calculated as 251.30mm, which gives a highly consistent result with the preliminary observations with only 1.78% deviation. Although the assessment of the plastic radius zone relatively deviates in preliminary analyses and in FE analyses, the obtained final displacements at the precast segments result in considerably compatible outcomes. As previously mentioned, this margin in plastic radius zones can be caused by the presence of the water table, the applied relaxation factors in FE analyses as in the analyses performed in RocSupport, the latter mentioned factors are not taken into account. Therefore, the verification of the preliminary assessment and the FE results are satisfied. The further evaluation of the statical verification of the cross-section is conducted in FE analyses.

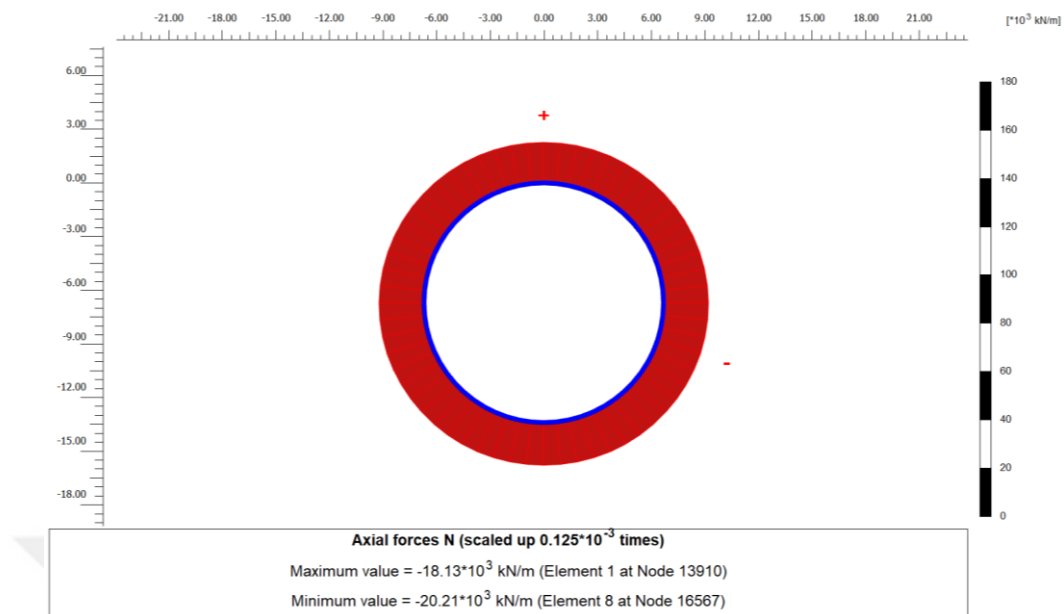


Figure 5.57 Axial Forces at the Precast Segments in Analysis 4-Alternative A

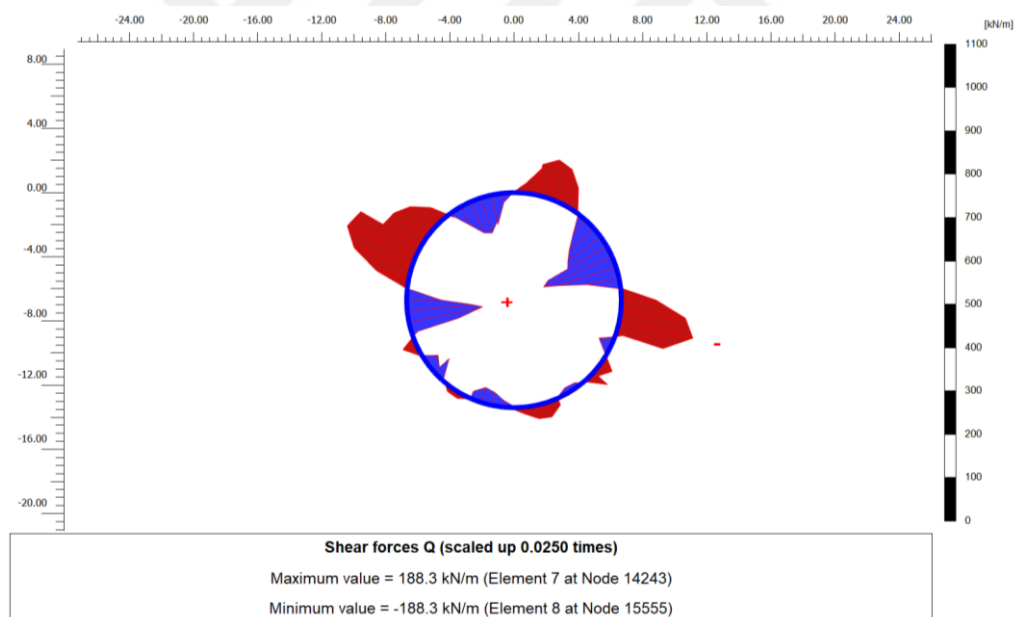


Figure 5.58 Shear Forces at the Precast Segments in Analysis 4-Alternative A

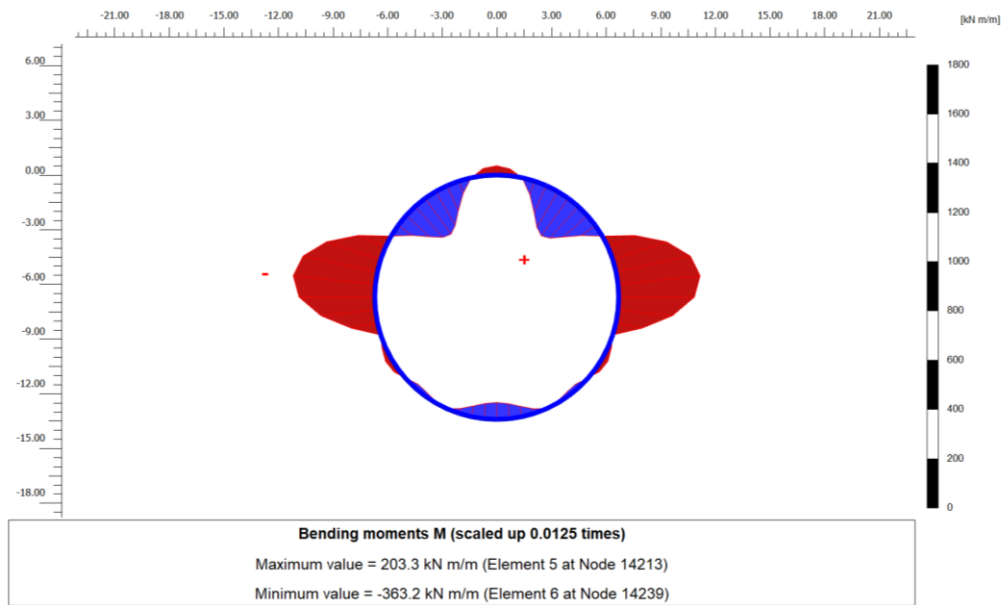


Figure 5.59 Bending Moments at the Precast Segments in Analysis 4-Alternative A

According to the resulting axial force, shear force, and bending moment diagrams, the static verification of the sections was performed, and the interaction diagram is provided in Figure 5.60.

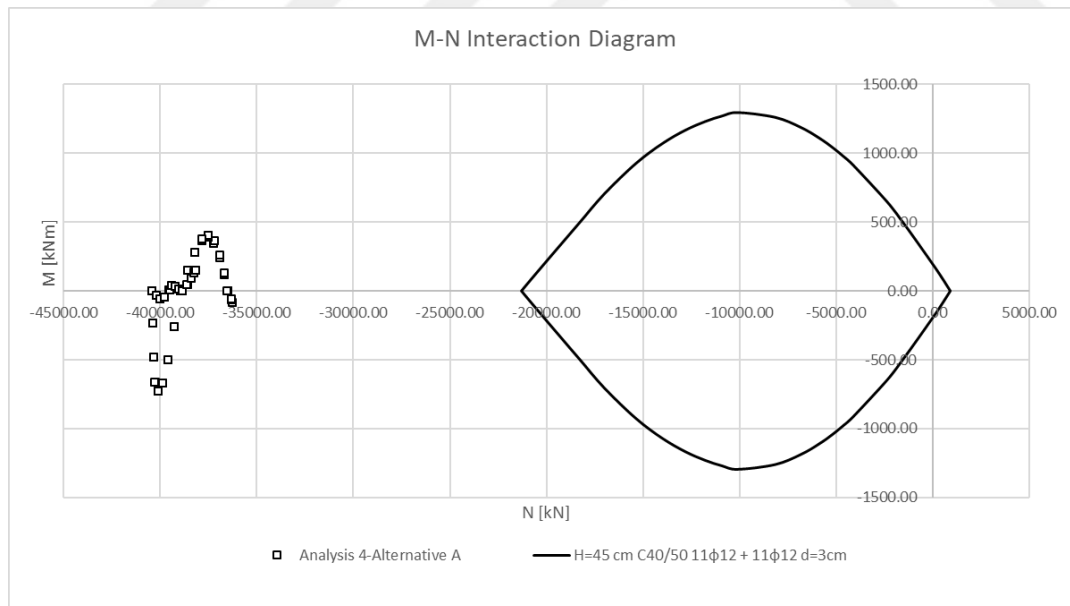


Figure 5.60 Interaction Diagram of Analysis 4-Alternative A

The deflections at the tunnel support are verified with numerical models and result compatibly with the monitoring data. However, the outcomes of Analysis 4-Alternative A show that the tunnel's stability is not satisfied with the proposed support system, both in the preliminary analyses and in the static verification of the segments.

## 5.6 Analysis 4-Alternative B

Analysis 4-Alternative B is devoted to assessing the tunnel advancement in NATM after the partial collapse in KM: 217+385. Based on the monitoring data, the expected maximum deformation in KM:217+385 with the NATM technique is 186mm. Analysis 4-Alternative B is composed of an altered tunnel construction after the partial collapse incident demonstrated in Analysis 4-Alternative A.

As seen in Figure 5.61 and Figure 5.62, the radius of the plastic zone is calculated as 15.58m, the maximum displacement expected at the tunnel walls are determined as 172.16mm. In addition, the factor of safety is calculated as 4.65. Hence, the support system interacts adequately with the specified ground conditions, according to the preliminary analyses.

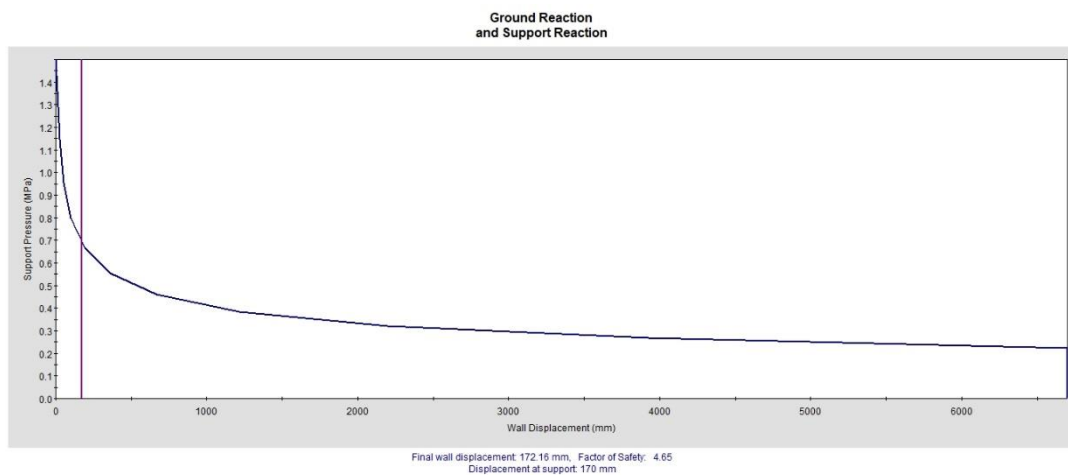


Figure 5.61 Ground-Support Reaction Curve of Analysis 4-Alternative B

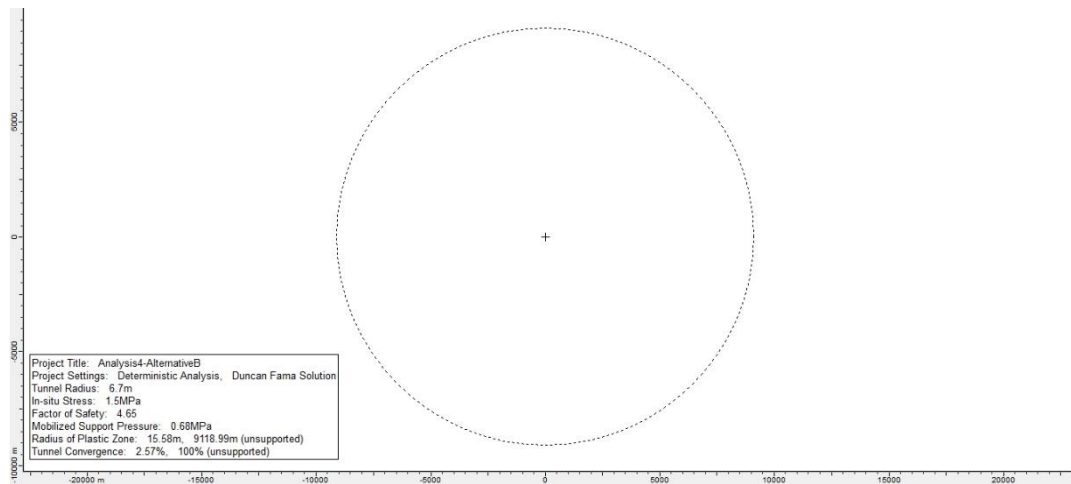


Figure 5.62 Tunnel Section View of Analysis 4-Alternative B

Following the analyses performed on RocSupport, 2D finite element analyses are performed for further evaluation of the deformations, stress distributions, and support reactions. The radius of the plastic zone, principal stress contours around the tunnel boundary, expected deformations at the segments, and compatibility of the support reactions are compared in the analyses conducted in Plaxis 2D. The finite element analysis of Analysis 4-Alternative B was performed according to the procedures described in Chapter 4.3.

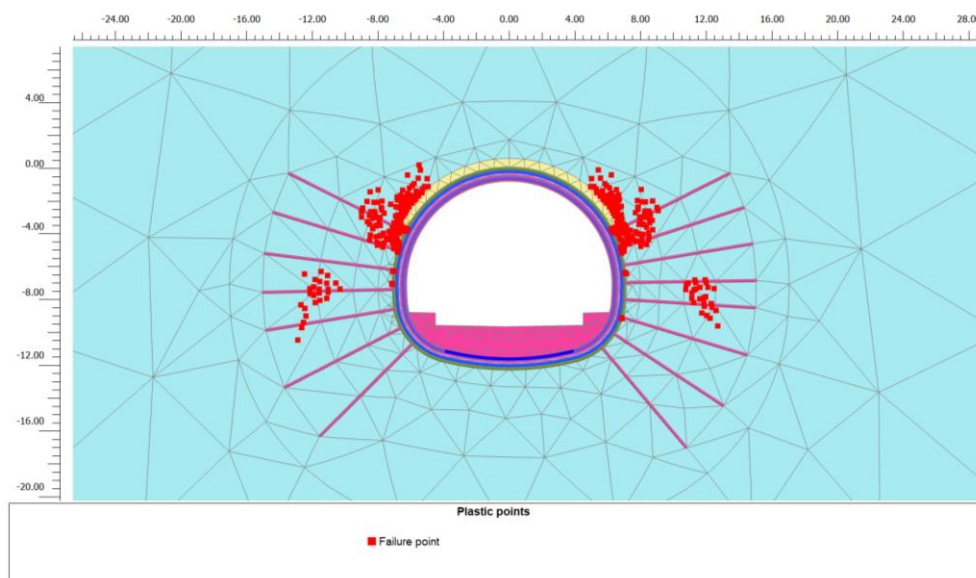


Figure 5.63 Plastic Zone Around the Tunnel in Analysis 4-Alternative B

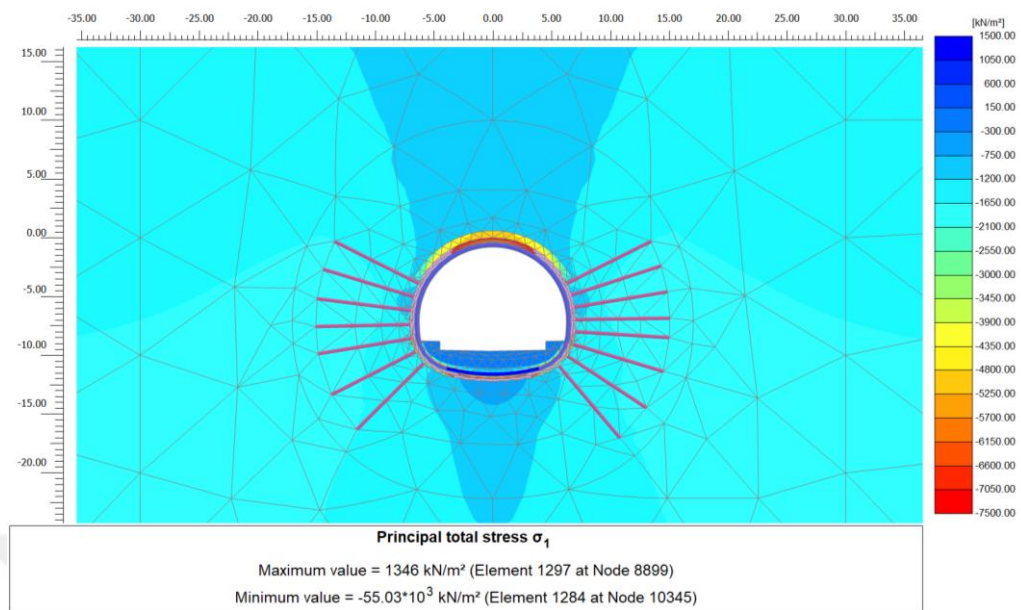


Figure 5.64 Major Principal Stress Contours Around the Tunnel in Analysis 4-Alternative A

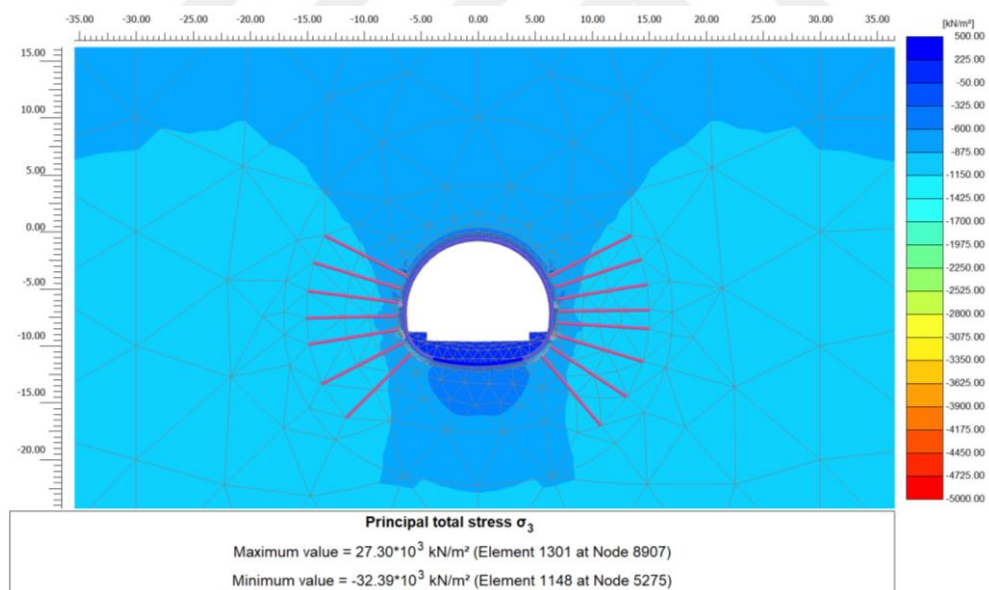


Figure 5.65 Minor Principal Stress Contours Around the Tunnel in Analysis 4-Alternative A

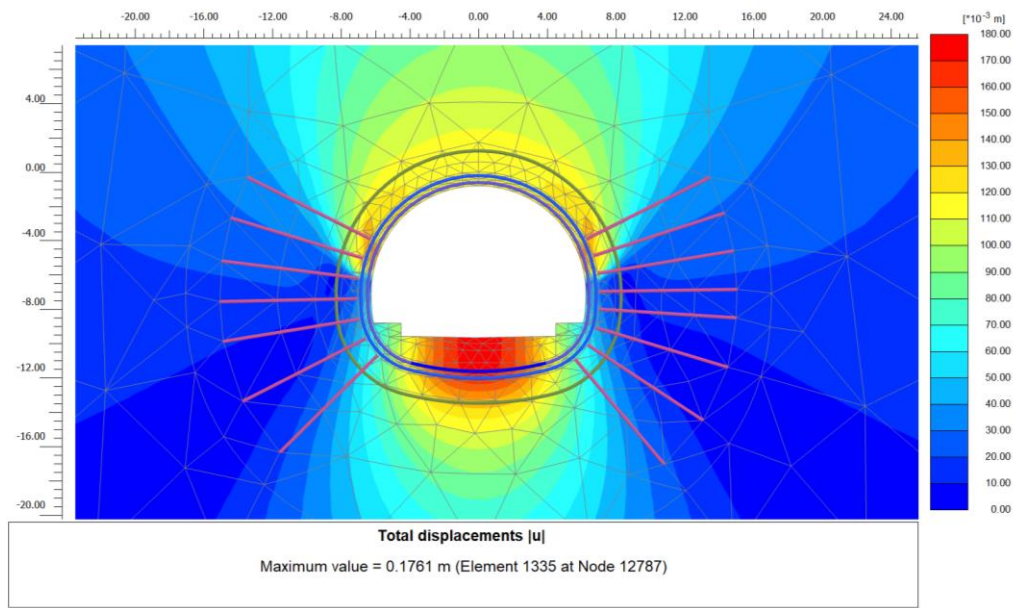


Figure 5.66 Total Displacements Around Tunnel Boundary in Analysis 4-Alternative B

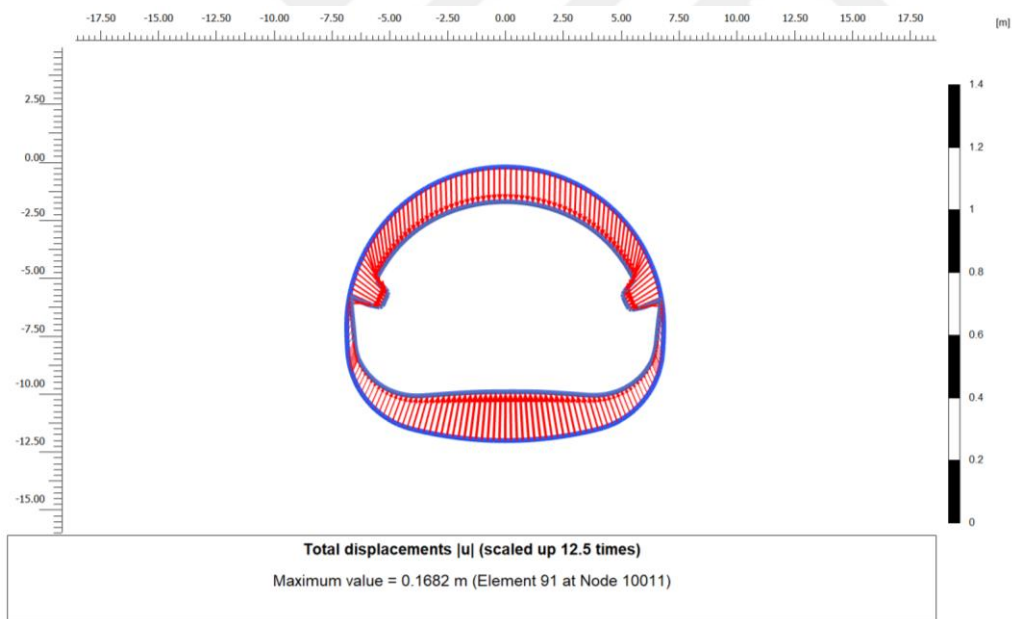


Figure 5.67 Total Displacements at the Primary Lining in Analysis 4-Alternative B

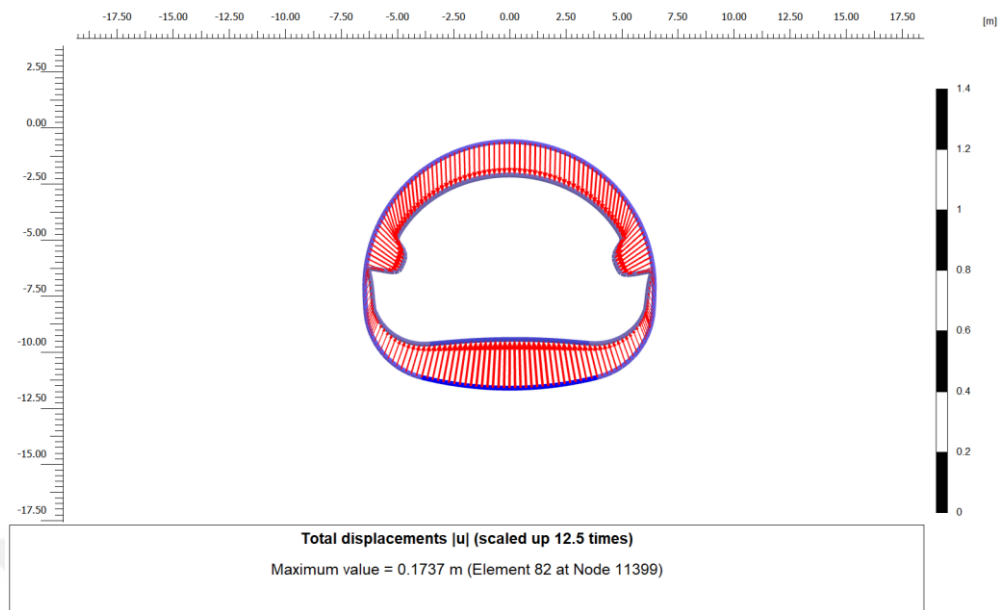


Figure 5.68 Total Displacements at the Final Lining in Analysis 4-Alternative B

In FE analysis, the radius of the plastic zone is determined as 13.43m, which results in a 13.8% error with the preliminary assessment. Additionally, in the FE analysis, the displacements in the final lining are calculated as 168.2mm, which gives a highly consistent result with the preliminary observations with only a 2.3% deviation.

Although the assessment of the plastic radius zone relatively deviates in preliminary analyses and in FE analyses, the obtained final displacements at the primary lining results in considerably compatible outcomes. As previously mentioned, this margin in plastic radius zones can be caused by the presence of the water table, the applied relaxation factors in FE analyses as in the analyses performed in RocSupport, the latter mentioned factors are not taken into account. Therefore, the verification of the preliminary assessment and the FE results are satisfied. The further evaluation of the statical verification of the cross-section is conducted in FE analyses.

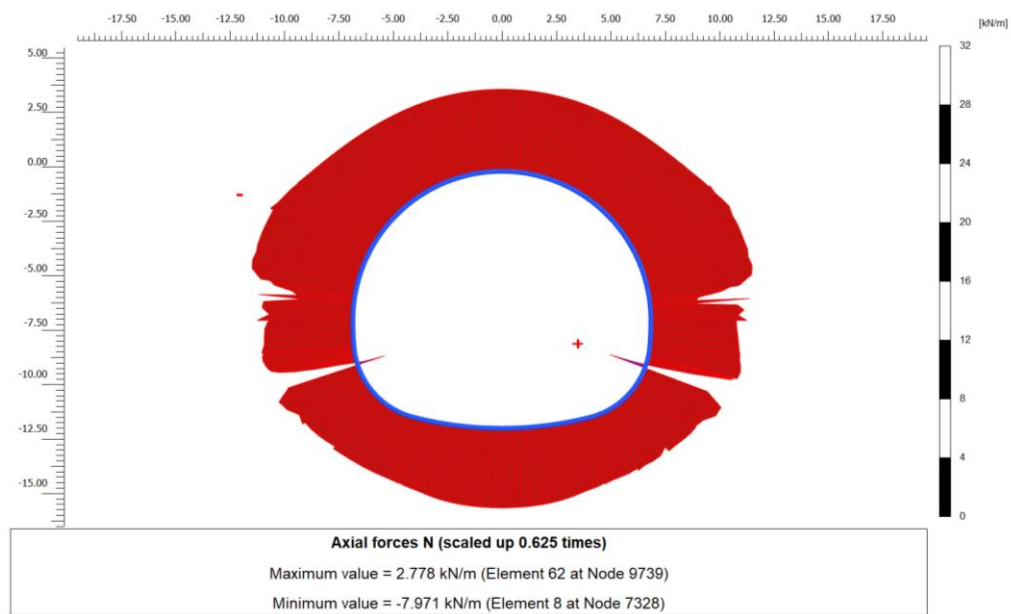


Figure 5.69 Axial Forces at the Primary Lining in Analysis 4-Alternative B

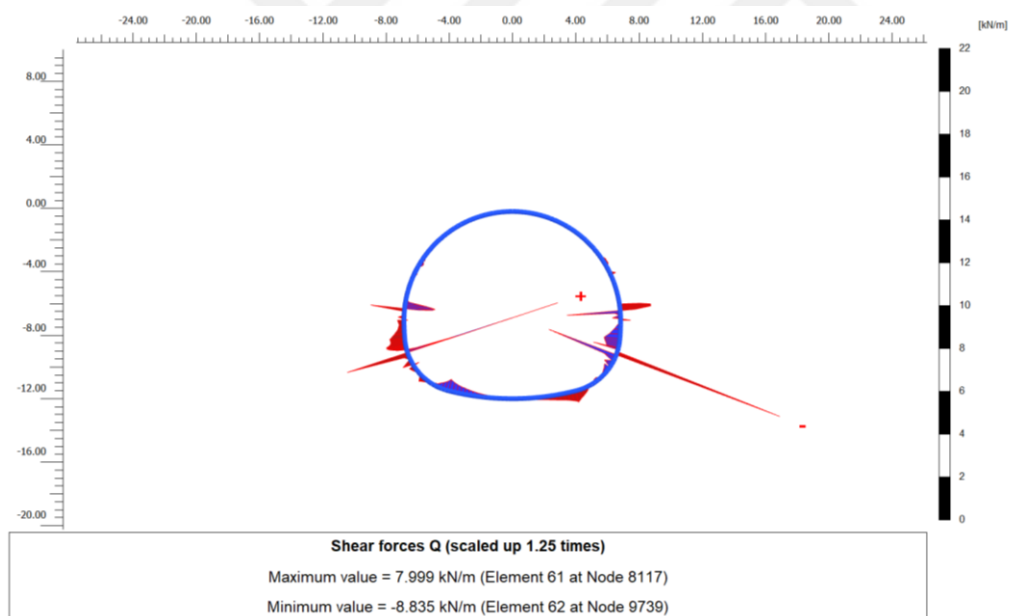


Figure 5.70 Shear Forces at the Primary Lining in Analysis 4-Alternative B

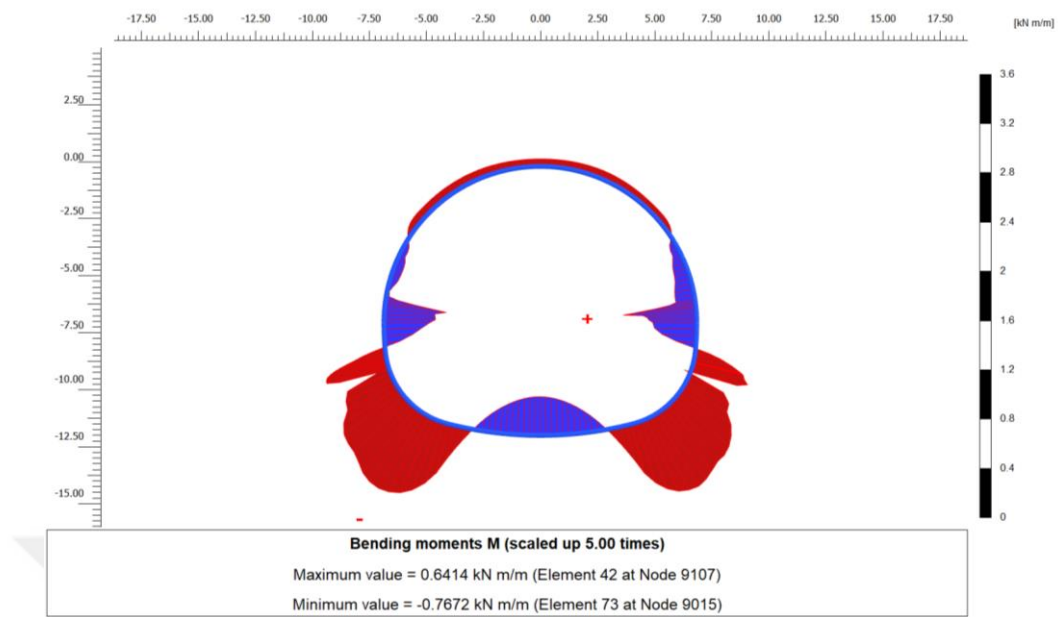


Figure 5.71 Bending Moments at the Primary Lining in Analysis 4-Alternative B

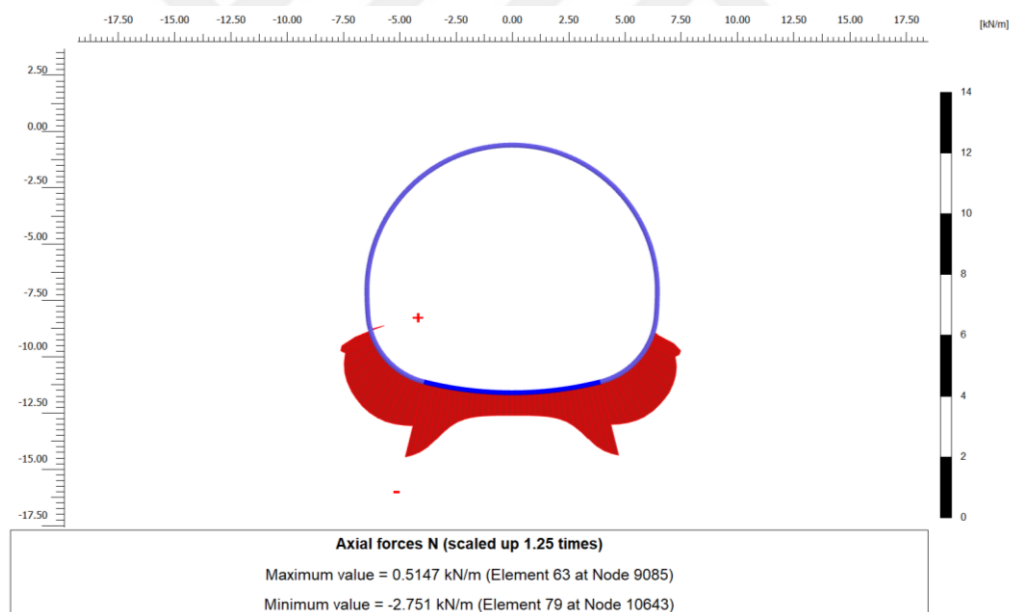


Figure 5.72 Axial Forces at the Final Lining in Analysis 4-Alternative B

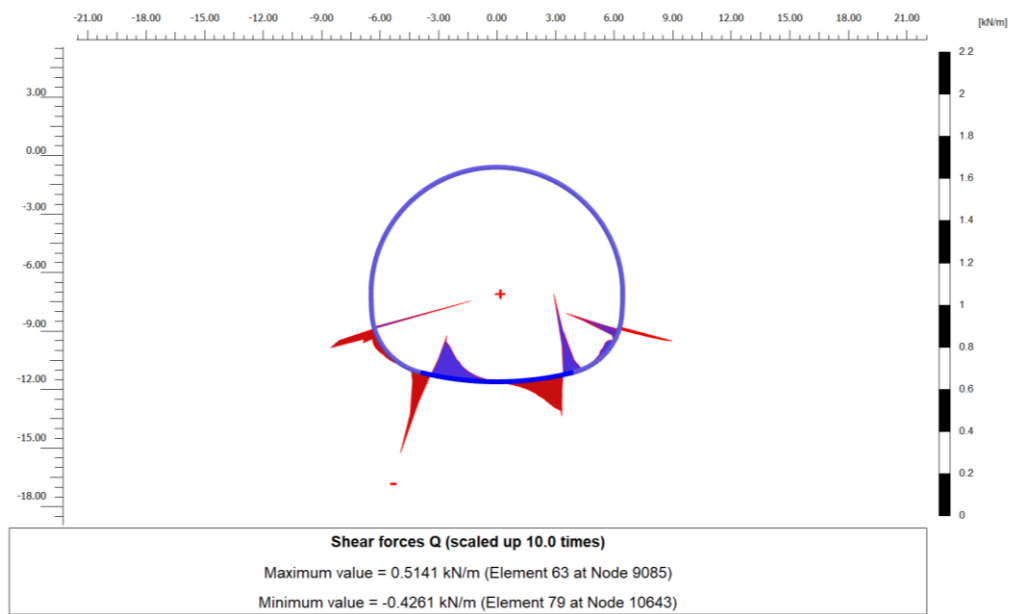


Figure 5.73 Shear Forces at the Final Lining in Analysis 4-Alternative B

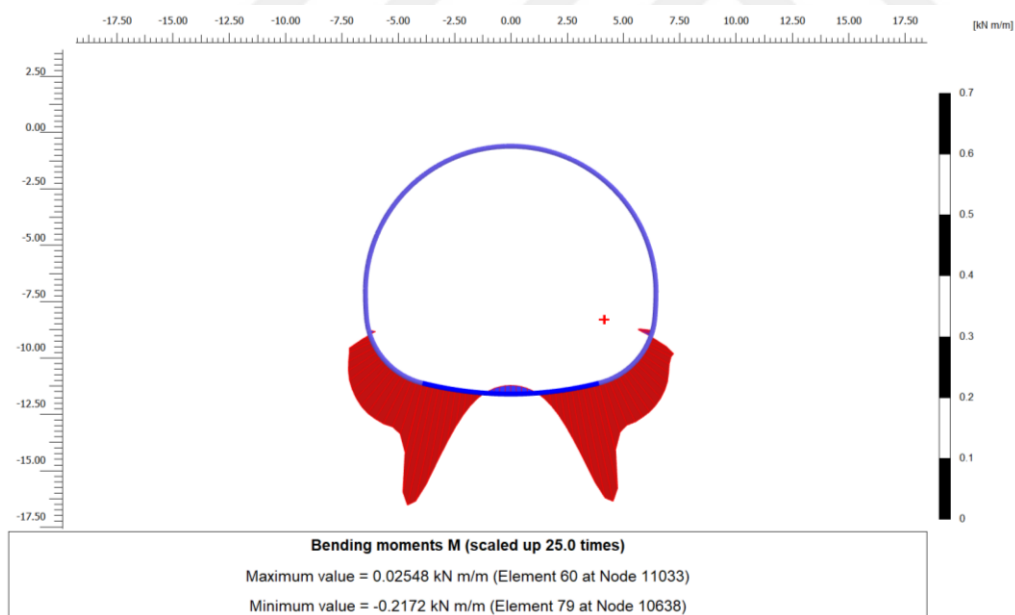


Figure 5.74 Bending Moments at the Final Lining in Analysis 4-Alternative B

According to the resulting axial force, shear force, and bending moment diagrams, the static verification of the primary lining and the final lining was performed, and the interaction diagrams are provided in Figure 5.75 and Figure 5.76.

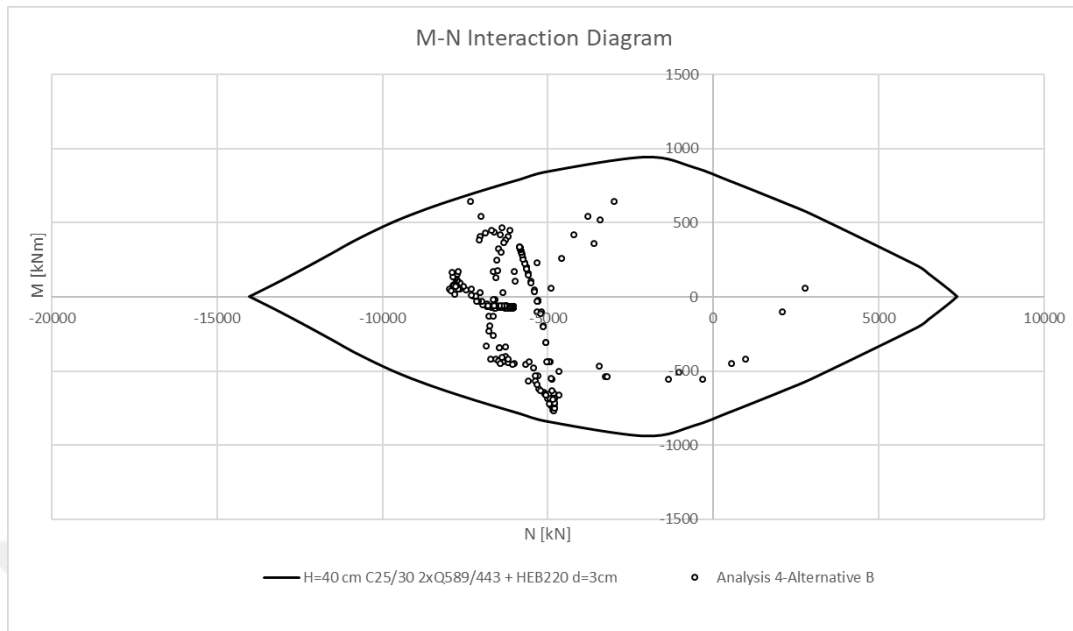


Figure 5.75 Interaction Diagram of Analysis 4-Alternative B – Primary Lining

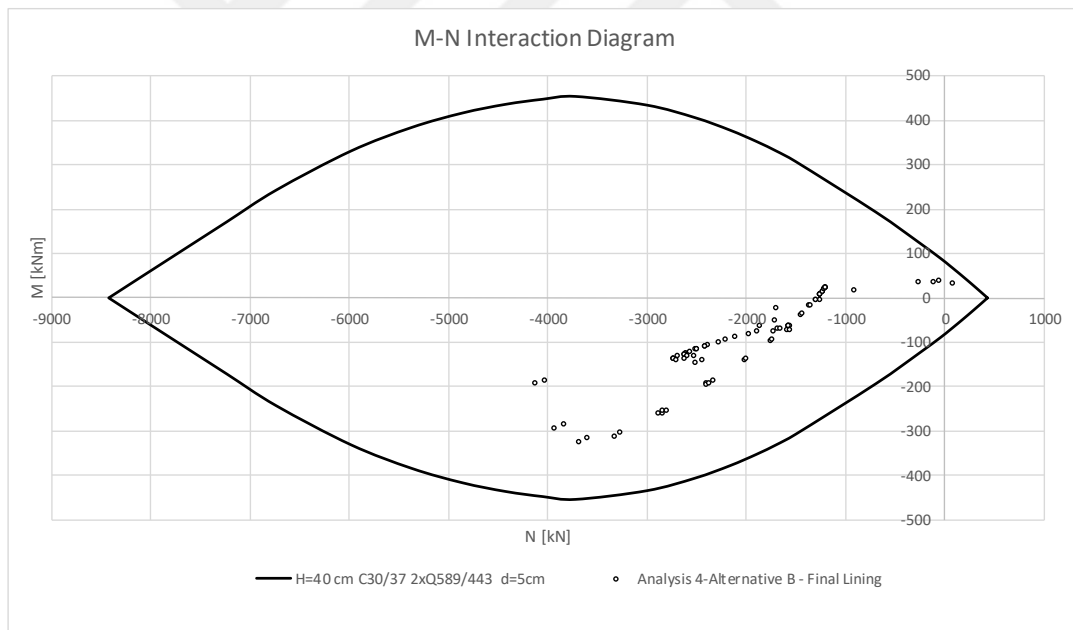


Figure 5.76 Interaction Diagram of Analysis 4-Alternative B – Final Lining

The outcomes of Analysis 4-Alternative B, as expected, are not tended to result in an occurrence of a collapse. The deflections at the tunnel support are verified with numerical models and result compatibly with the monitoring data. The static verifications also show that there is no evidence of a failure throughout the primary

and final lining in this section. Therefore, the tunnel advancement with the altered construction technique is safe throughout the section.



## CHAPTER 6

### CONCLUSION

The tunnel advancement of Tunnel A93 in the collapsed area was achieved by two different construction methods, namely TBM and NATM. The influences of several factors, including the tunnel advancement conforming to the principles of TBM and NATM, ground characteristics, soil stiffness, strength parameters, are investigated using RocSupport and Plaxis 2D software. A total of seven cases with different soil stiffnesses and strength parameter sets are investigated. These cases are summarized as given in Table 6.1.

Table 6.1 The summary of the analyses

| Analysis ID | Description                                                                                                                                                                               | Predicted vs. (Monitored) Displacement | Conclusion                                                                                          |
|-------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------|-----------------------------------------------------------------------------------------------------|
| Analysis 1  | Modeling parameters were chosen to be identical to those used in the design stage. Design stage parameters were selected on the basis of site investigation and laboratory-based studies. | 19.4 (10-300) mm (at the crown)        | Parameters used in the design stage were not in conformance with the actual response of the tunnel. |

Table 6.1 (continued) The summary of the analyses

|                             |                                                                                                                                                                                                                                  |                                      |                                                                                                                                                                                                                                                  |
|-----------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Analysis 2                  | The back-calculated geotechnical parameters were determined based on monitoring data. This case represents the stable section in TBM excavation.                                                                                 | 25.0 (25.9) mm (at the crown)        | Semi-empirical 1-D assessments and static verifications demonstrate there is no evidence of a failure.                                                                                                                                           |
| Analysis 3<br>Alternative A | The geotechnical parameters were back-calculated based on available monitoring data. The displacements exhibit an increasing trend, and the partial collapse was initiated.                                                      | 150.0 (162.1) mm (at the crown)      | The outcomes of the analysis show that stability is not satisfied with the proposed support system. The construction method was changed from TBM to NATM. The altered construction method of this case was analyzed in Analysis 3-Alternative B. |
| Analysis 3<br>Alternative B | This case is composed of the altered tunnel construction method of NATM after the partial collapse demonstrated in Analysis 3-Alternative A. The geotechnical parameters specified in Analysis 3-Alternative A were implemented. | 102.2 (101.5) mm (at the side walls) | The altered construction technique reduced the total displacements, and the tunnel advancement with the proposed system was proved to be safe.                                                                                                   |

Table 6.1 (continued) The summary of the analyses

|                                |                                                                                                                                                                                                                                            |                                         |                                                                                                                                                                                                                                                                                            |
|--------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Analysis 4<br>Alternative<br>A | The geotechnical parameters were obtained based on monitoring data. The observed deformations reached their peak, and the partial collapse occurred.                                                                                       | 231.0 (255.8)<br>mm (at the crown)      | As a result of the plastic flow observed in segments, the stability is not satisfied. The construction method was changed from TBM to NATM. The altered construction method of this case was analyzed in Analysis 4-Alternative B                                                          |
| Analysis 4<br>Alternative<br>B | This case is comprised of the altered tunnel construction method of NATM after the partial collapse incident demonstrated in Analysis 3-Alternative A. The geotechnical parameters specified in Analysis 3-Alternative A were implemented. | 186.0 (172.1)<br>mm (at the side walls) | After the partial collapse observed in Analysis 4-Alternative A, the altered method of construction resulted in a remarkable reduction in stresses subjected to the support and in acquired displacements. The tunnel advancement with the proposed section was revealed to be convenient. |

As presented in Table 6.1, the first case is composed of the preliminary predictions based on the available geotechnical data in the design stage before initiating the tunnel excavation. Analysis 1 presented the ideal design-basis conditions for TBM excavation. The outcomes of the monitoring data authenticated that the tunnel

response did not comply with the design previsions. Therefore, based on the incremental deformations along the route, three individual analyses were performed. Analysis 2, Analysis 3-Alternative A, and Analysis 4-Alternative A were performed to demonstrate the performance of tunnel advancement in excavation with TBM, along with the evolving deformation trends. For this purpose, the complied monitoring data were utilized to back-calculate the strength parameters. The reference maximum deformation values obtained from monitoring data for Analysis 2, Analysis 3-Alternative A, and Analysis 4-Alternative A are 25mm, 150mm, and 231mm, respectively. The results of Analysis 3-Alternative A and Analysis 4-Alternative A suggested that the studied sections are vulnerable to failure. Thus, Analysis 3-Alternative B and Analysis 4-Alternative B were performed to trace the tunnel response in construction with NATM, subsequent to the partial collapse that occurred in TBM excavation in the sections studied in Analysis 3-Alternative A and Analysis 4-Alternative B, respectively. Based on the evaluations presented in the previous chapter, the following conclusions and recommendation for future works can be derived:

- According to the analysis performed by considering only the strength parameters obtained from the limited site investigation results with the assumption of uniform material, the tunnel advancement is estimated to be safe along the alignment with the selected construction method of TBM. In contrast, the monitoring data indicates that the segmental linings are subjected to excessive deformations.
- In Analysis 1, the obtained deformation values deviate from the measured data by more than 60%. These results suggest that site investigation results do not fully represent the rock conditions along the tunnel route. Henced, site investigation, and laboratory-based idealizations were proven to be not sufficient in anticipating the observed tunnel collapse.
- It is evident that the installation time of the support measures is crucial, especially in a squeezing tunnel environment, to maintain the stability of it. The construction with TBM restrains the optimization of installation time of

the segments since a particular advancement length is to be achieved to install the segmental linings with a certain length.

- The ground-support reaction curves of Analysis 3-Alternative A and Analysis 4-Alternative A have indicated that the segmental linings were installed too early before letting the relaxation be fully completed. Hence, linings were subjected to stresses exceeding their capacities, which lead to the plastic flow of these elements.
- It can be derived that the emergency warnings had already been provided in the monitoring data during the excavation. Early prediction and the application of appropriate mitigative measures could have prevented the partial collapse.
- The displacement values obtained in ground-support reaction analyses and FE analyses in each back-analyzed case are in conformance with the monitoring data and deviate by only less than 10%. Therefore, the optimization of the strength parameters of the back-analyzed sections seems reasonable. Although the analyses with back-analyzed strength parameters present consistent results with field deformation data, in the future, a more comprehensive back-analysis procedure can be applied to reduce the scatter of these results.
- In the FE analyses, numerous variables such as the exact tunnel geometry, presence of water, construction stages, and a more comprehensive support configuration can be presented in a constitutive model. Although 1-D assessments of the tunnel behavior deliver useful guidance in preliminary stages, the FE analyses present more realistic results as opposed to semi-empirical 1-D assessments.
- The occurrence of partial collapse sets of a reduction in the in-situ stresses. The operation of the tunnel advancement was initiated in the remainder of the ground pressures after the stress release. Hence, the required capacity of support measures to stabilize the host ground was mitigated correspondingly.

- The predicted radius of the plastic zone values around the tunnel periphery in FE analyses and ground-support reactions differ more than 13% in the sections excavated with NATM principles. The 1-D semi-empirical assessments do not fully authorize modeling tunnels conforming to NATM principles due to the adopted assumptions in the software. Therefore, the estimations conclude relatively higher results.
- It is indicated that along the tunnel alignment, the geological units consist of graphite-schists. Although the overall overburden is determined as 160m, the excavated section's poor geotechnical properties indicate that the related area might be affected by the fault zones. In the future, an evaluation of the seismic response of the tunnel could be executed to interpret the performance.

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# APPENDICES

## A. RocLab Results

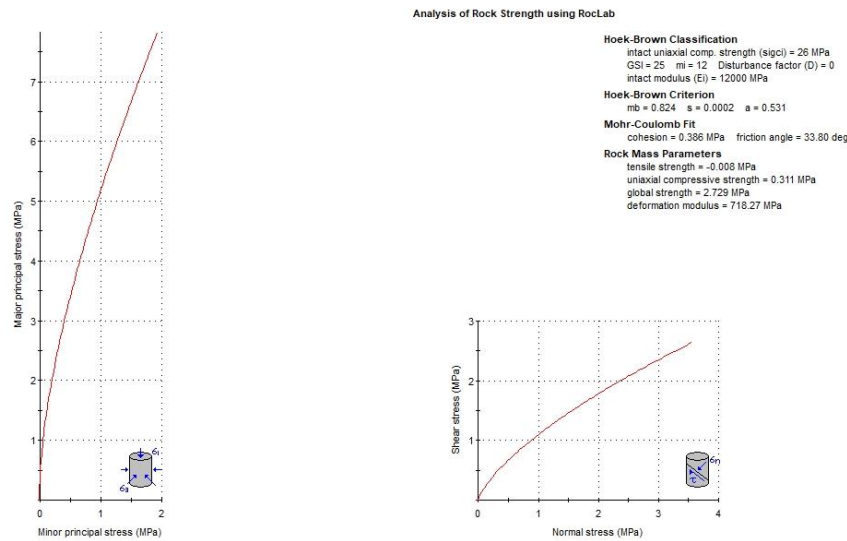
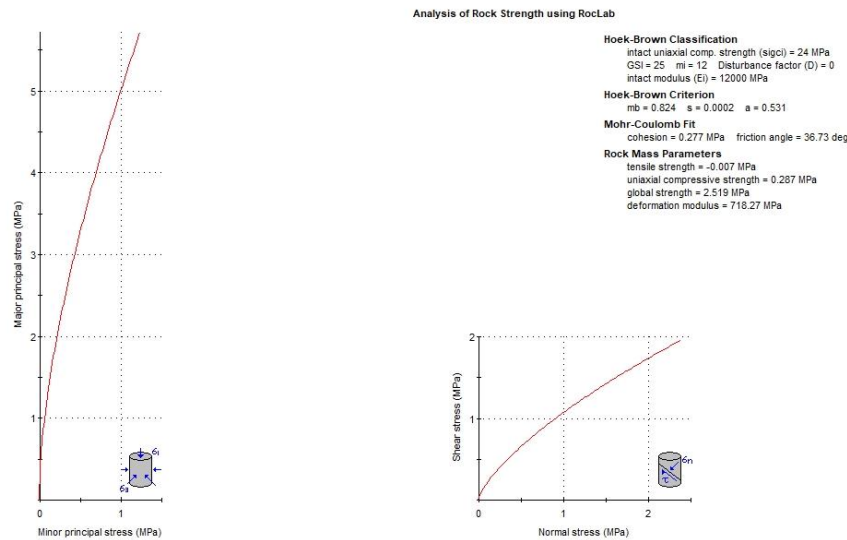


Figure A.1 RocLab Outputs of Analysis 1

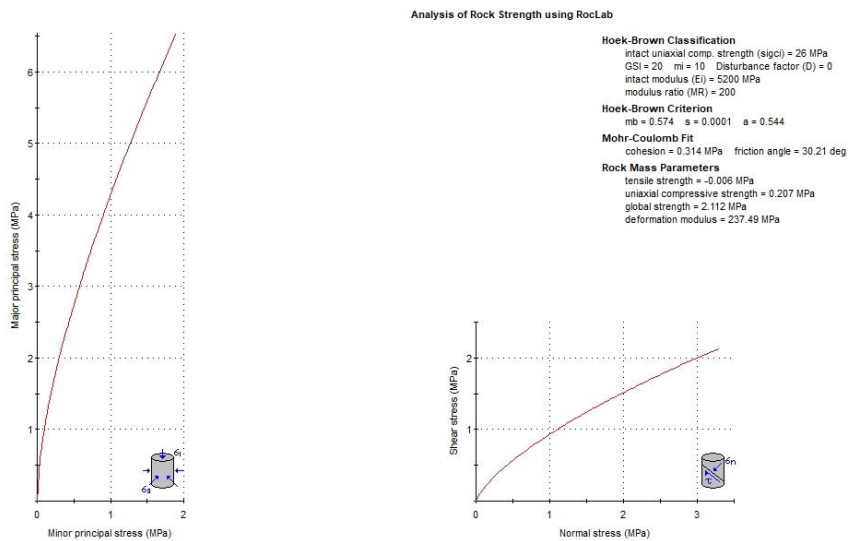
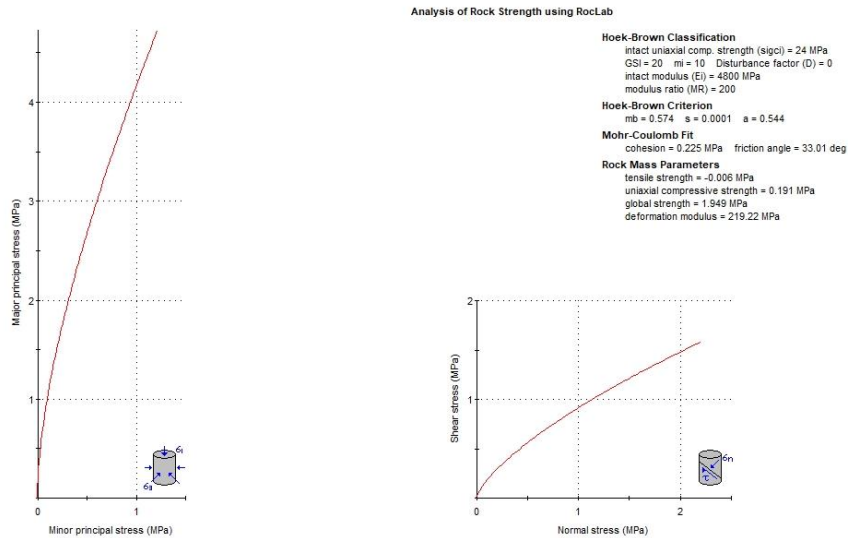


Figure A.2 RocLab Outputs of Analysis 2

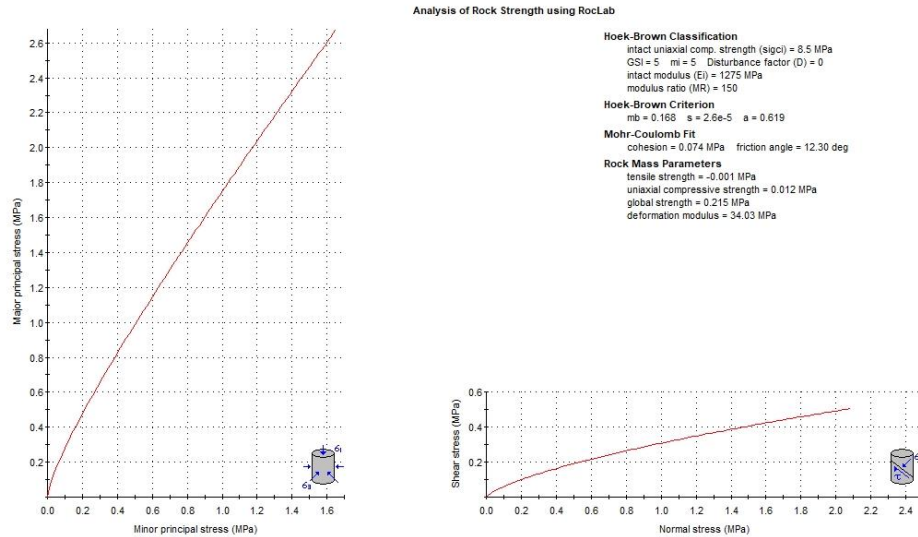
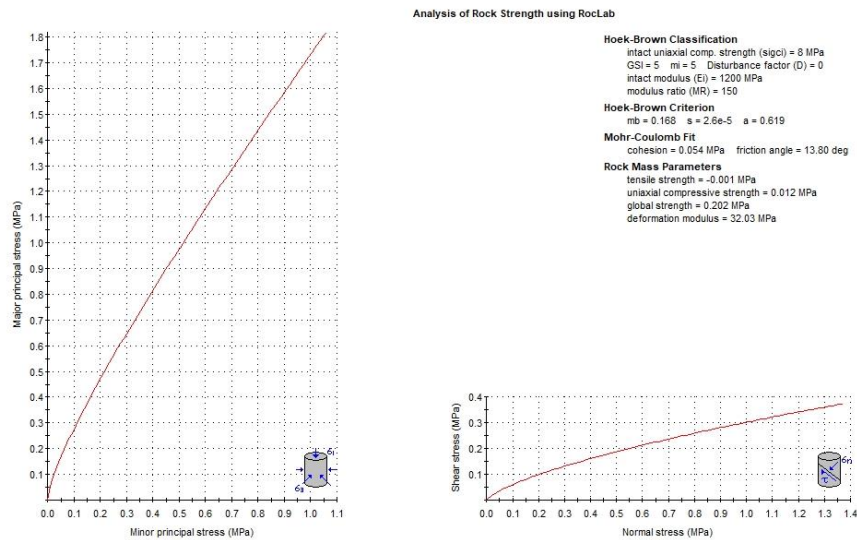


Figure A.3 RocLab Outputs of Analysis 3-Alternative A

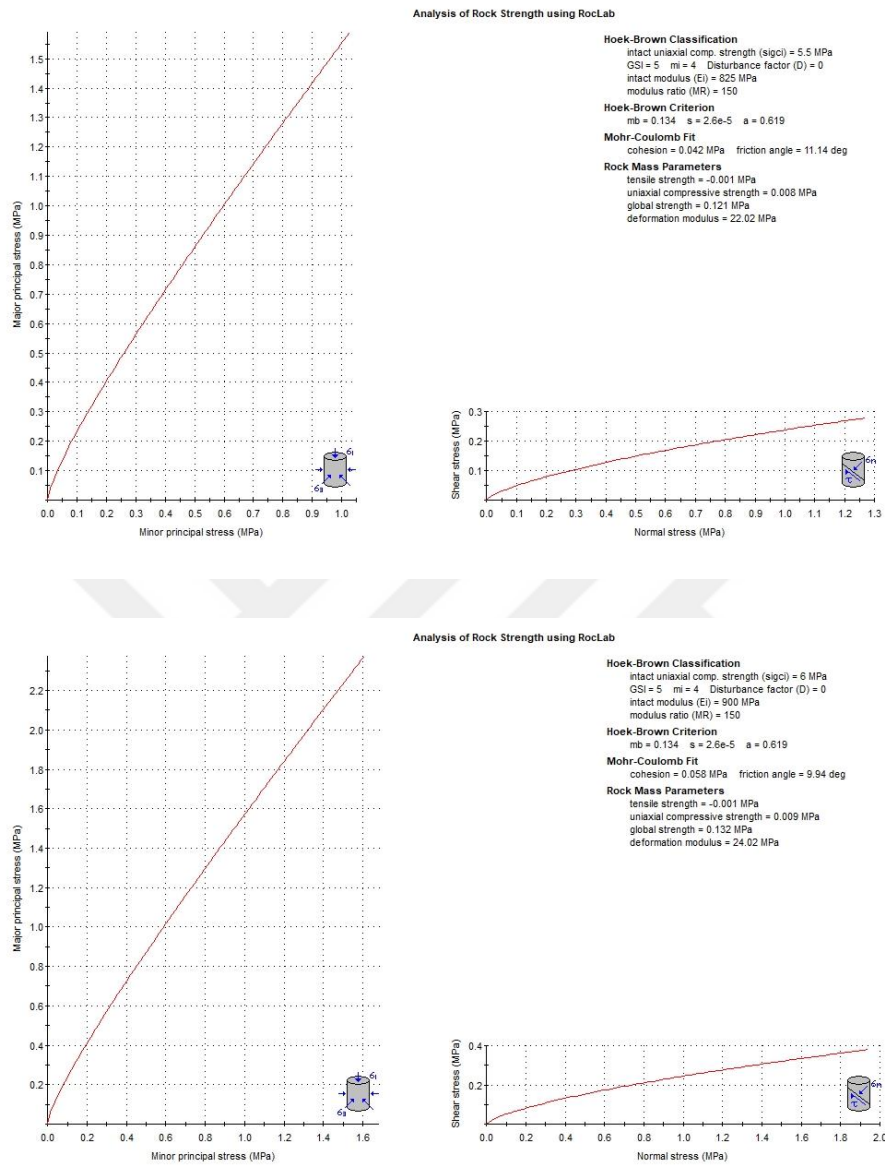
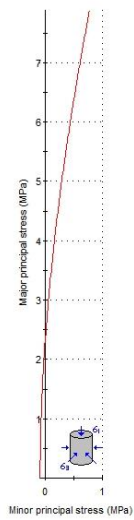


Figure A.4 RocLab Outputs of Analysis 4-Alternative A



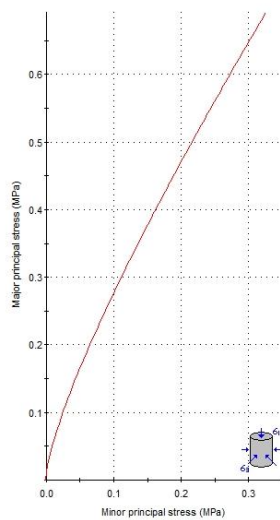
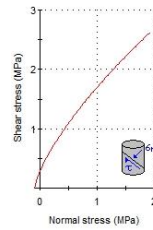
#### Analysis of Rock Strength using RocLab

**Hoek-Brown Classification**  
 intact uniaxial comp. strength ( $\sigma_{ci}$ ) = 39.94 MPa  
 GSI = 50  $m_i$  = 9 Disturbance factor (D) = 0  
 intact modulus ( $E_i$ ) = 7988 MPa  
 modulus ratio (MR) = 200

**Hoek-Brown Criterion**  
 $m_b$  = 1.509  $s$  = 0.0039  $a$  = 0.506

**Mohr-Coulomb Fit**  
 cohesion = 0.461 MPa friction angle = 50.01 deg

**Rock Mass Parameters**  
 tensile strength = -0.102 MPa  
 uniaxial compressive strength = 2.405 MPa  
 global strength = 6.625 MPa  
 deformation modulus = 2453.80 MPa



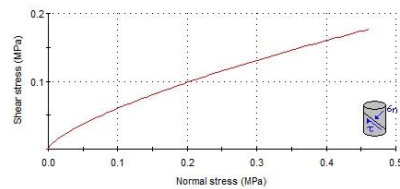
#### Analysis of Rock Strength using RocLab

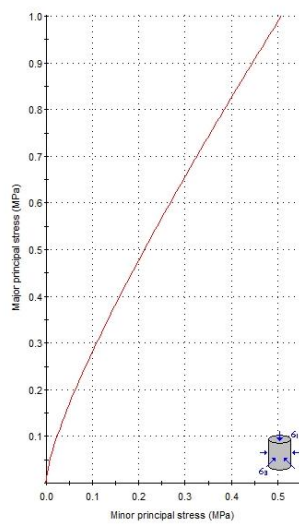
**Hoek-Brown Classification**  
 intact uniaxial comp. strength ( $\sigma_{ci}$ ) = 8 MPa  
 GSI = 5  $m_i$  = 5 Disturbance factor (D) = 0  
 intact modulus ( $E_i$ ) = 1200 MPa  
 modulus ratio (MR) = 150

**Hoek-Brown Criterion**  
 $m_b$  = 0.168  $s$  = 2.6e-5  $a$  = 0.619

**Mohr-Coulomb Fit**  
 cohesion = 0.024 MPa friction angle = 19.21 deg

**Rock Mass Parameters**  
 tensile strength = -0.001 MPa  
 uniaxial compressive strength = 0.012 MPa  
 global strength = 0.202 MPa  
 deformation modulus = 32.03 MPa





Analysis of Rock Strength using RocLab

**Hoek-Brown Classification**  
 intact uniaxial comp. strength ( $\sigma_{ci}$ ) = 8.5 MPa  
 GSI = 5  $m_i$  = 5 Disturbance factor ( $D$ ) = 0  
 intact modulus ( $E_i$ ) = 1275 MPa  
 modulus ratio ( $MR$ ) = 150

**Hoek-Brown Criterion**  
 $m_b$  = 0.168  $s$  = 2.6e-5  $a$  = 0.619

**Mohr-Coulomb Fit**  
 cohesion = 0.033 MPa friction angle = 17.32 deg

**Rock Mass Parameters**  
 tensile strength = -0.001 MPa  
 uniaxial compressive strength = 0.012 MPa  
 global strength = 0.215 MPa  
 deformation modulus = 34.03 MPa

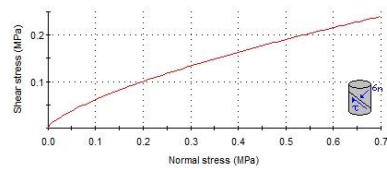
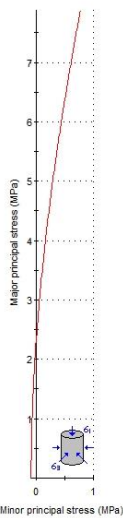


Figure A.5 RocLab Outputs of Analysis 3-Alternative B



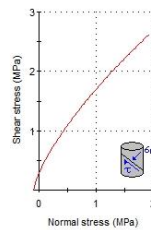
Analysis of Rock Strength using RocLab

**Hoek-Brown Classification**  
 intact uniaxial comp. strength ( $\sigma_{ci}$ ) = 39.9 MPa  
 GSI = 50  $m_i$  = 9 Disturbance factor ( $D$ ) = 0  
 intact modulus ( $E_i$ ) = 9975 MPa  
 modulus ratio ( $MR$ ) = 250

**Hoek-Brown Criterion**  
 $m_b$  = 1.509  $s$  = 0.0039  $a$  = 0.506

**Mohr-Coulomb Fit**  
 cohesion = 0.461 MPa friction angle = 50.00 deg

**Rock Mass Parameters**  
 tensile strength = -0.102 MPa  
 uniaxial compressive strength = 2.403 MPa  
 global strength = 6.618 MPa  
 deformation modulus = 3064.18 MPa



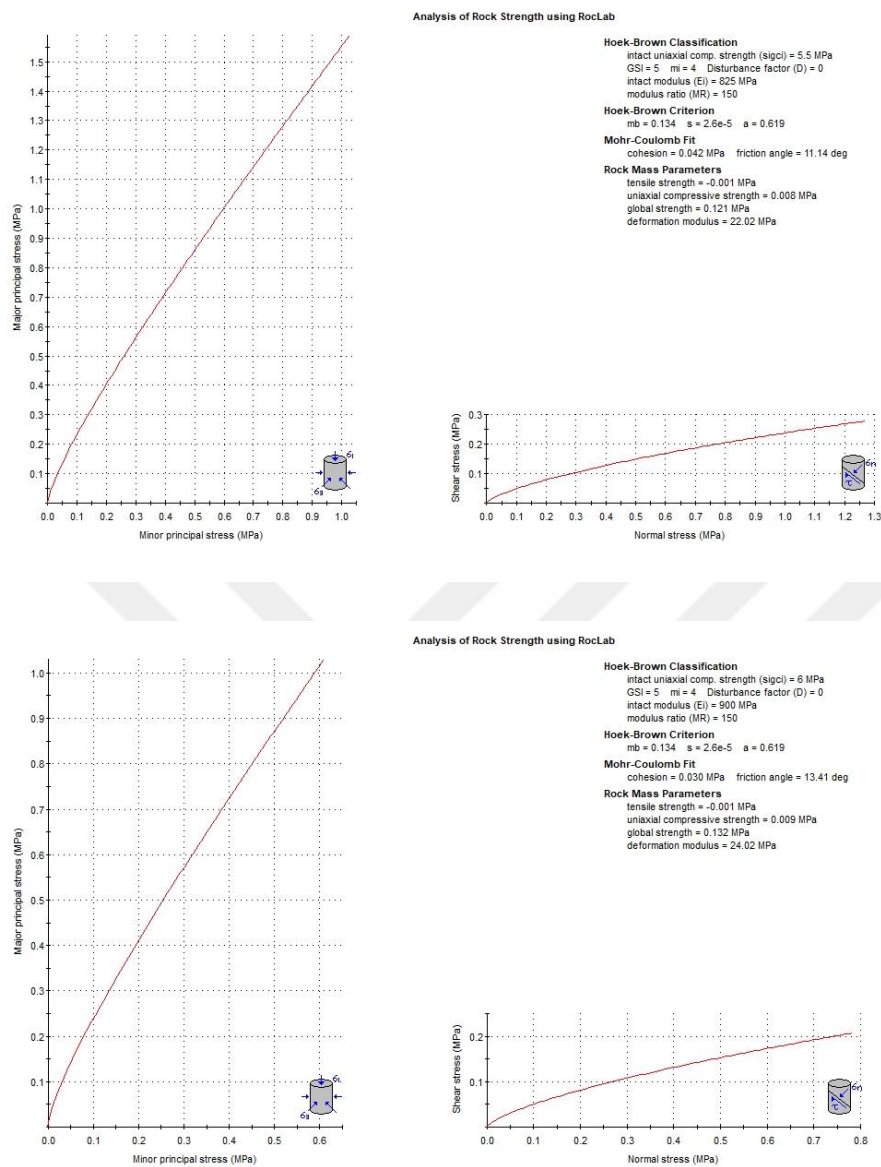


Figure A.6 RocLab Outputs of Analysis 4-Alternative B