

AN INVESTIGATION ON THE APPLICABILITY OF THE CONVERGENCE
CONFINEMENT METHOD TO A DEEP ROCK TUNNEL FOR DIFFERENT
CONDITIONS

by

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ABSTRACT

AN INVESTIGATION ON THE APPLICABILITY OF THE CONVERGENCE CONFINEMENT METHOD TO A DEEP ROCK TUNNEL FOR DIFFERENT CONDITIONS

Numerical modelling with finite element programs, allows users to model an underground opening almost exactly in relation to its actual situation in reality. However, the engineering analyses that are performed using analytical methods may be more economic and less time consuming. The overall objective of this research was to investigate application of the convergence confinement method for the design of a deep tunnel, with the ultimate goal of providing a better understanding of the method. First, a case study of Chamoise Tunnel (France) which is reported in the literature, is regarded as case study to create synthetic data sets for this research. Different analyses are conducted by using the finite element approach and analytical approach to establish a comparative study between the obtained results. The numerical models that are created to simulate uniform loading ($K_o=1.0$) conditions for a deep circular tunnel were altered to study different geometry, non-uniform loading conditions ($K_o=0.6$ and $K_o=1.2$) and a different number of excavation stages. As a result of investigating the above mentioned variables; occurring convergences around the tunnel contour were found to be increased as the tunnel shape changes from a fully circular tunnel to a circular-arc shaped tunnel and as the initial ratio of the horizontal stresses to the vertical stresses (K_o) increases in $[0.6, 1.2]$. A good match has been detected between the numerical and analytical approaches for the excavation induced radial displacements for the case where tunnel is unsupported. However, it is not possible to conclude the same for the radial displacements developed in the case where the tunnel is supported. A further research may be appreciated to comprehend the behaviour of the supported rock-mass.

ÖZET

KAPANMA-SINIRLAMA YÖNTEMİNİN KAYA ORTAMDA YER ALAN DERİN BİR TÜNEL İÇİN UYGULANABİLİRLİĞİNİN ARAŞTIRILMASI

Sonlu elemanlar programları ile yapılan sayısal modellemeler, kullanıcıların yer-altı kazılarını neredeyse gerçekte olduğu gibi modelleyerek gerçekçi tasarımlar oluşturmalarına olanak sağlamaktadır. Ancak bazı durumlarda analitik yöntemler kullanılarak yapılan tasarımlar mühendislik hizmetleri açısından daha ekonomik ve daha az zaman alıcı olabilmektedir. Bu tez çalışmasının asıl amacı, convergence-confinement (kapanma-sınırlama) yöntemini bir derin tünel tasarımı örneğinde irdeleyerek, bu yöntemin daha iyi anlaşılmasını sağlamaktır. Öncelikle, bir veri dizisi oluşturmak için literatürde yer alan Chamoise Tüneli'ne ait veriler derlenerek kullanılmıştır. Analitik yaklaşım ve sonlu elemanlar yaklaşımıyla gerçekleştirilen analizlerden elde edilen sonuçların karşılaştırılması amacıyla farklı analizler yapılmıştır. Üniform gerilme koşullarında ($K_o = 1.0$); derin dairesel tünel kazısını benzeştirebilmek edebilmek amacıyla oluşturulan sayısal modeller esas alınarak, farklı geometri, üniform olmayan gerilme koşulları ($K_o = 0.6$ ve $K_o = 1.2$) ve farklı sayıda kazı aşamaları için analizler alternatifli olarak çoğaltılmıştır. Analizler ile bu çalışmada yer alan kaya birimi ve tünel çapı için, tünel çeperinde oluşan hacimsel deformasyon değerlerinin, tünel kazısının dairesel kesit yerine dairesel yay şeklinde yapıldığında ve başlangıç durumundaki yatay gerilme değerlerinin düşey gerilme değerlerine (K_o) oranının artmasıyla $[0.6, 1.2]$ arttığı sonucuna varılmıştır. Destek sistemlerinin kullanılmadığı durum için analitik ve sayısal yaklaşımla yapılan analizler benzer sonuçları vermiştir. Bununla birlikte, aynı değerlendirmenin destek sistemlerinin de kullanıldığı tünel tasarımı için de yapılabilmesi mümkün olmamıştır. Destek sistemleri ile kaya kütlelerinin birlikte davranışını irdelleyecek bir bilimsel çalışma yapılarak bu konu daha derinlemesine incelenebilecektir.

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LIST OF SYMBOLS

a	Rock mass constant for HB criterion
A	Cross-sectional area of the lining m ²
D	Disturbance Factor in HB Criterion
d_b	Diameter of the bolt m
d_{rb}	Diameter of the rockbolts
E	Young's modulus kPa
EA	Axial stiffness N/m
E_c	Young's modulus of the shotcrete or concrete kPa
EI	Flexural rigidity Nm ² /m
E_{rm}	Young's modulus of the rock mass MPa
G_{ref}	Elastic shear modulus kPa
GSI	Geological strength index
J_a	Joint alteration number
J_n	Number of joint sets
J_r	Joint roughness number
J_w	Joint water reduction number
K_0	Initial stress ratio
K_s^{max}	Maximum support stiffness m/MPa
l	Free length of the bolt m
L	Distance from the tunnel face m
m_b	Reduced value of intact rock parameter m_i
m_i	Intact rock parameter in HB Criterion
p_i	Initial support pressure MPa
p_i^{cr}	Critical support pressure MPa
P_{ref}	Reference stress
p_s	Support pressure MPa
p_s^{max}	Maximum support pressure MPa
Q	Deformation-load constant for the anchor and head m/MN
R	Radius of the tunnel m

R_{pl}	Radius of the plastic zone m
s	Rock mass constant for HB criterion
s_c	Circumferential spacing m
s_l	Longitudinal spacing m
T_{bf}	Ultimate load obtained from a pull-out test MN
t_c	Thickness of the concrete/shotcrete shell mm
u_r^{el}	Radial deformation in elastic region mm
u_r^{max}	Maximum elastic deformation mm
u_r^{pl}	Radial deformation in plastic region mm
β	Load Reduction factor
γ	Unit weight kN/m ²
ν	Poisson's ratio of intact rock
ν_c	Poisson's ratio of the shotcrete or concrete
ν_{rm}	Poisson's ratio of the rock mass
σ_o	In-situ stress KPa
σ_1	Confining stress applied to the rock mass KPa
σ_3	Axial stress applied to the rock mass KPa
σ_{cc}	Uniaxial strength of the shotcrete or concrete KPa
σ_{ci}	Uniaxial compressive strength of intact rock KPa
σ_h	Horizontal stress KPa
σ_v	Vertical stress KPa
ψ	Dilatancy angle degrees

LIST OF ABBREVIATIONS

CCM	Convergence Confinement Method
FEM	Finite Element Method
GRC	Ground Reaction Curve
GSI	Geological Strength Index
HB	Hoek-Brown
LDP	Longitudinal Deformation Profile
NATM	New Austrian Tunneling Method
NGI	Norwegian Geotechnical Institute
RMR	Rock Mass Rating
SCC	Support Characteristic Curve
SRF	Stress Reduction Factor
TBM	Tunnel Boring Machine
UCS	Uniaxial Compressive Strength

1. INTRODUCTION

1.1. Background

At the moment, the application of the analytical approach in tunnel design is limited in worldwide scale. The pre-design of the tunnels is made based on the empirical approach followed by a detailed design using the numerical methods.

However, in some cases, the design using analytical solutions may be more economic and less time consuming. Even though the analytical solutions rely on several assumption, they still may be used as a practical tool for pre-design. Furthermore, analytical solutions may represent more accurate results compared to ones that are determined by empirical methods in the stage of pre-design.

This research represents the analytical CCM which is developed for the design of underground openings in rocks. Although the finite element models are generally used to solve the design related problems for tunnels, regardless of simplifications, analytical solutions should not be overlooked as they reflect both tunnelling tradition and design experience (Simanjuntak, 2015). The applications of convergence confinement method using both analytical and numerical approach are represented in this research as a comparative study.

1.2. Aim and Research Questions

Overall, this thesis covers a series of comparative investigations between the applications of convergence confinement method using analytical approach and finite element approach. The comparative study, which is performed by Putz-Perrier *et al.*, investigates the north and the south tube drives of Chamoise Tunnel (France) across the Oxfordian marls is regarded as a case study for this thesis. The difference between the driving mechanisms of the north and the south tube for Chamoise Tunnel is regarded as case study which is used to create synthetic data for this research.

CCM is a simplified semi-analytical method to analyse the two-dimensional interaction between the ground and the support. The simplicity of the method, is a characteristic that has led to its widespread use since the end of the seventies (Oreste, 2009). The method is derived from two important assumptions:

- The tunnel cross section is circular,
- The far-field principal stresses normal to the long axis of the tunnel are uniform ($K_o=1.0$).

Chamoise Tunnel case allows the application of plain strain, two-dimensional finite element models since it is deep and long tunnel, moreover the shape of the tunnel is assumed as circular.

Although the driving of its first tube (North tube) was carried out by three-staged mechanical excavation, driving of the second tube (South tube) was performed by full-face drill-and-blast excavation. Therefore, three different models are used as a basis to illustrate the full faced and three-staged excavations using finite element method (Chapter 5). Additionally, two-staged excavation has been introduced into the models to be able to scan the transition.

The following research questions arise:

- It is a known fact that the horizontal stresses in a rock depend on the geological history and may be different than the vertical stresses. As a consequence K_o may be different than 1. Therefore, how much does a change in the horizontal pressure effect the occurring radial displacements on tunnel wall in an elasto-plastic isotropic rock mass subjected to non-uniform in-situ stresses compared to those subjected to uniform in-situ stresses?
- How different are the calculated radial displacements on tunnel wall in the analytical approach compared to two-dimensional finite element approach?
- What is the effect of conducting the excavation in stages on convergences occurring on the tunnel wall and on the radius of plastic zone?
- What are the limitations of the convergences confinement method?

- When using two-dimensional finite element models, how the convergences can be determined? What are the limitations of the application of convergence confinement method in two-dimensional finite element method?
- What is the efficiency of the supports used in the models?
- What is the influence of the geometry of the excavation on occurring radial convergences?

1.3. Research Outline

In each chapter, a general information about the convergence confinement method may be repeated, however with different emphases depending on the topic discussed. Chapter 1 introduces scope of this research. The outline of the research is presented with an overview of content and the structure. Additionally, the research questions and objectives are listed.

Chapter 2 starts with a flow chart which summarizes the followed path during literature review. This chapter mainly focuses on rock-mass behaviour and design methods for underground openings and used support systems. The theoretical background of the convergence confinement method for both analytical approach and finite element approach have been presented.

Chapter 3 introduces the case study: Chamoise Tunnel Project with general information about the geology of the site, characteristics of the surrounding rocks and geometry of the tunnel. The derivation of the synthetic data is explained step by step and the designated rock-mass parameters for the analyses are presented.

Chapter 4 deals with the convergence confinement method applications to the synthetic data sets using semi-analytical method. The simulations for the Chamoise Tunnel case are demonstrated in this part. The application of rock mass classification systems on created data sets are presented in this chapter. Q System and RMR system are selected for this purpose. Moreover, the semi-analytical calculation results are verified using the specific software for convergence confinement applications, called

RocSupport.

Chapter 5 presents the convergence confinement method applications to the synthetic data sets using two-dimensional finite element approach. The geometry and the mesh characteristics of the numerical models are introduced. Used structural elements and the application details are explained. Furthermore, the set-up of the models for each calculation phase are defined and the assumptions has been made are listed.

Chapter 6 focuses on comparison of the convergence confinement method between semi-analytical and two-dimensional finite element method.

Chapter 7 summarizes the main findings of the research and draws conclusions.

2. LITERATURE REVIEW

This chapter briefly presents the behaviour determination of the rock-mass, accordingly the the Hoek-Brown failure criterion and the convergence confinement method. Furthermore, it also provides information about the rock-mass classification systems based on the design principles for rock excavation and supports. Below illustrated chart summarizes the followed path for literature review.

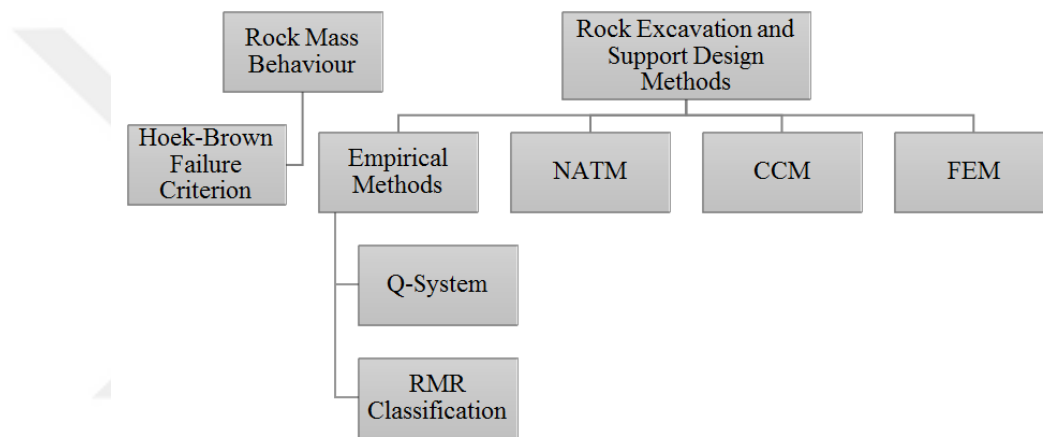


Figure 2.1. Flow Chart Summarizing the Literature Review.

2.1. Rock-Mass Behaviour

Behaviour determination of the ground surrounding a rock excavation is considered as a fundamental step in the design procedure of rock excavation. The behaviour around an underground opening mostly characterised by the combination of following parameters:

- Rock mass composition:

Looking at different rock engineering classification systems, the following rock mass characteristics have mostly been used (Palmstrom and Stille, 2010):

- rock material characteristics such as homogeneity, schistosity, strength and special properties or behaviour,
- weathering and alteration of rocks,
- joint and jointing features such as joint conditions, jointing pattern, block size, degree of jointing, density of joints,
- weakness zones, faults.

	Occur between weakness zones and faults			Occur often in weakness zones and faults	
	Massive rocks	Jointed rocks or blocky materials		Particulate materials	Special materials
	Class A	Class B	Class C	Class D	Class D
	Weak to strong rocks	Rocks intersected by joints and partings	Jointed rocks intersected by seams or weak layers	Highly jointed or crushed rocks, and soil-like materials	Soft and weak materials
1	 Brittle, homogeneous and foliated rocks (granite, gneiss, quartzite)	 Jointed homogeneous foliated and bedded rocks	 Jointed rocks intersected by seams (filled joints) (seamy and blocky ground)	 Highly jointed or crushed rocks with clay seams or shears	 Alternating soft and hard layers (e.g. clay schist-sandstone-clay schist)
2	 Schistose (deformable) rocks with high content of platy minerals	 Jointed, schistose rocks	 Prominent weathering along joints	 Highly jointed or crushed rocks (sugar-cube, etc.) little clay	 Rock fragments with few contacts, in a matrix of soft (clayish) material
3	 Rocks with plastic properties (soapstone, rocksalt, many weathered rocks)	 Layered and bedded rocks with frequent partings (slate, flagstone)	 Jointed rocks with weak bedding layers	 Soil-like materials with friction properties (poorly cemented sandstones, etc.)	 Soft or weak materials with plastic properties (mudstone, clay-like materials)

Figure 2.2. Typical Rock Mass Compositions (Palmstrom and Stille, 2010).

The rock type which is investigated within the scope of this research, is defined as marl. Even though it usually exhibits soft or weak material behaviour with plastic properties and low strength, it shall be classified as massive and moderately strong rocks with plastic properties (A3) according to the rock mass compositions given in Figure 2.2 since it represents relatively high mean UCS values (20 - 26 MPa) and low water content ($\leq 5\%$) confirmed by laboratory test results. However, locally measured low compressive strength with convergences greater than 70 mm also verifies the existence of soft or weak materials with plastic properties on site (Putz-Perrier *et al.*, 2014).

- Stress state:

The initial stress condition at around an underground opening may have significant impact on the stability, especially if the strength of the rock mass is less than a fourth

of occurring stresses. On the other hand, low stresses may also decrease the stability in jointed rock masses. Rearranging of the stresses around an underground opening is dependent on the magnitudes and the directions of the principal stresses and the geometry of the opening. Although, there are some exceptions, the initial stress condition is often highly anisotropic. That means the tangential stress will vary around the periphery of the opening, even for a circular shape. Rock stress measurements indicate that the stresses in the rock stabilise at a constant level outside, approximately, a tunnel width from the tunnel contour (Palmstrom and Stille, 2010). Lastly, deformation properties of the rock mass have also significant impacts on the magnitude of the tangential stresses around an opening.

Stress Reduction Factor (SRF) in Q system (Barton *et al.*, 1974) describes the relation between stress and rock strength around and underground opening. SRF values represents the stress related problems in tunnels such as spalling, slabbing, deformation, squeezing, dilatancy and block release (NGI, 2013).

- Groundwater conditions:

The groundwater pressure usually decreases close to the vicinity of the underground opening due to the drainage through the joints. Therefore, the failures that are directly related to water pressure are rarely encountered. However, in general, it can be said that the groundwater pressure may cause the instability of the underground opening due to its reduction effect in the strength of rock mass and the shear strength of discontinuities.

- Shape and geometry of the excavation:

The shape, size and geometry of an underground opening have great influence on the behaviour of surrounding rock mass. In theory, the occurring displacements will increase with the radius of the opening. Many investigations have also indicated that the strength of the rock mass decreases with increasing size of the loaded area (Palmstrom and Stille, 2010).

- Excavation effects:

The excavation process may have changed the composition of the rock mass since it may lead to formation of new joint sets or cracks in surrounding rock mass with respect to the rock mass quality. The damage from blasting is mainly dependent on various features such as strength and density of the rock material, the amount of explosive, the pattern and number of the blasting holes etc. (Hudson and Harrison, 1997).

- Installation of rock support:

The more the installation of the support is postponed, the pressure on the support will be reduced. On the other hand, it is also important to install the support before the failure has occurred. This sensitive relation is controlled by the stand-up time of the rock mass. The stand-up time is an empirical relation, which in principle indicates that a rock mass of better rock quality will have a longer stand-up time (Palmstrom and Stille, 2010).

It is also crucial to understand the individual ground behaviour in order to be able to evaluate the behaviour of the whole system using the flow chart as shown in Table 2.1. Therefore, it is necessary to carefully follow each step in flow chart in order to interpret the ground response accurately.

Table 2.1. Flow Chart for Determining The Ground Behaviour (Price, 2009).

Rock Material Properties	+	Rock Mass Fabric	=	Rock Mass Properties
Rock Mass Properties	+	Environment	=	Ground Behaviour
Ground Behaviour	+	Changes by Engineering Work	=	Engineering Ground Response

2.1.1. Hoek-Brown Failure Criterion

A rock mass can exhibit various types of behaviour by the change in the load states. Therefore, a failure criterion must be selected to determine the behaviour of the rock mass in different load states. The presence of joints and related in-situ geological effects can remarkably decrease the strength and the stiffness of the rock mass compared to the properties of the intact rock specimen.

A non-linear empirical peak strength criterion for rocks and rock masses has been developed by Hoek and Brown (1980). The criterion uses the uniaxial compressive strength of the intact rock material as scaling parameter, and introduces two dimensionless strength parameters, m and s (Hoek and Brown, 1980).

Hoek-Brown failure criterion which is most commonly used in the design of rock excavations, is a practical tool to define the stress state under which a rock mass will deform inelastically. According to Hoek-Brown, the failure criterion for a rock mass is defined by following equation (Hoek *et al.*, 2002).

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a \quad (2.1)$$

where σ_3 is the the confining stress applied to the rock mass (e.g., in MPa), σ_1 the axial stress that produces failure of the rock mass (e.g., in MPa), σ_{ci} the unconfined compressive strength of the intact rock (e.g., in MPa), m_b a reduced value of the intact rock parameter m_i which also depends on the

Geological Strength Index (GSI) and the Disturbance factor (D):

$$m_b = m_i e^{(GSI-100)/28-14D} \quad (2.2)$$

s and a are the constants for the rock mass (Hoek *et al.*, 2002).

$$s = e^{(GSI-100)/9-3D} \quad (2.3)$$

$$a = 0.5 + 1/6 \left[e^{(-GSI/15)} - e^{(-20/3)} \right] \quad (2.4)$$

GSI which relates the criterion of failure to geological observations in the field, provides a system for estimating the reduction in rock mass strength for different geological conditions. Figure 2.3 illustrates the relationship between empirical variable 'a' and GSI.

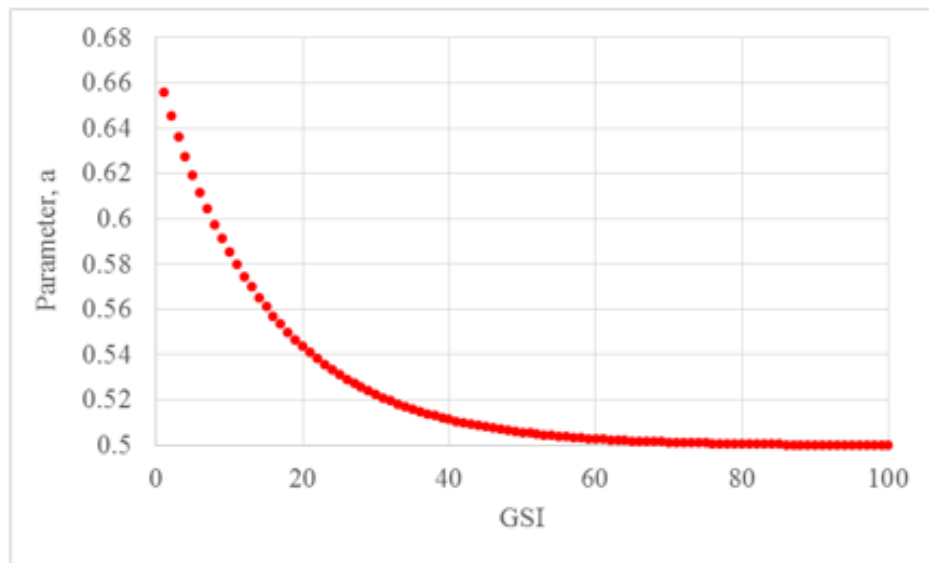


Figure 2.3. Parameter 'a', Varying with GSI.

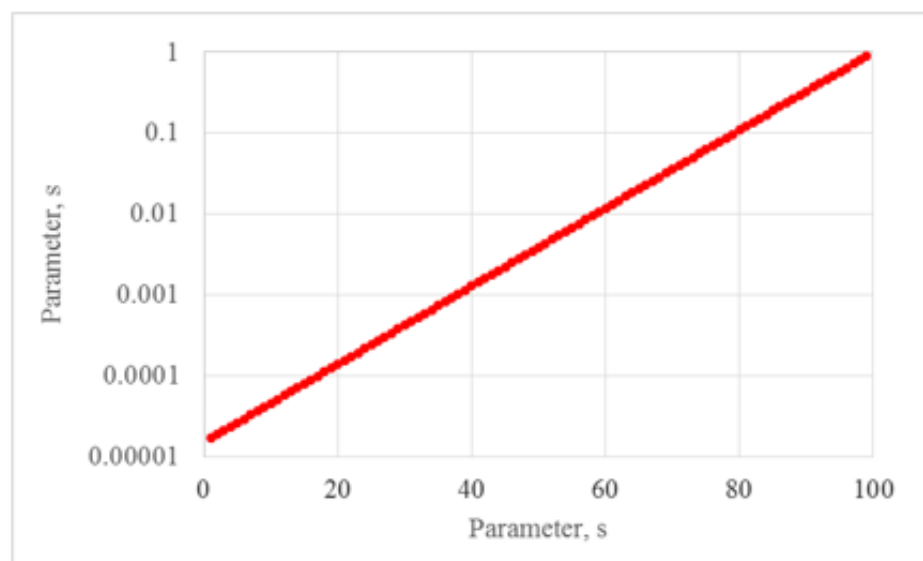


Figure 2.4. Parameter 's', Varying with GSI.

The change in the failure envelope with different m values is illustrated in Figure 2.15. The physical meaning of the parameter ‘ m ’ can be determined by the change in the shape of the downward curved failure envelope. It can be appreciated that the parameter m represents the generalization of the internal friction angle.

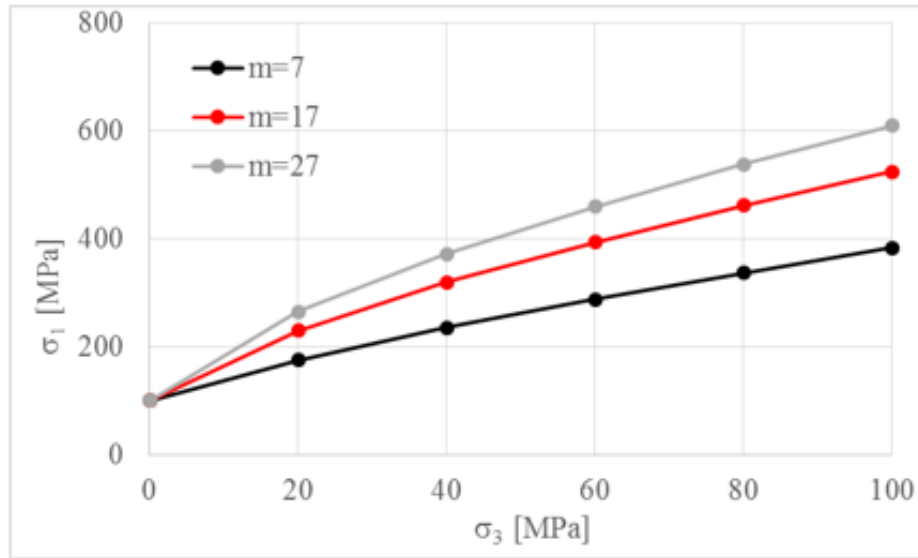


Figure 2.5. Failure Envelopes for Different m Values.

The parameter ‘ s ’ which depends on the structure and the surface conditions of joints in the rock mass can vary between 0 and 1. It may be assumed as $s=0$ for rock masses of very poor quality. The variation of parameter ‘ s ’ is given in Figure 2.4 for minimal disturbance of the rock mass surrounding a tunnel.

The charts used estimating the GSI and D values are represented in Figure 2.6 and Figure 2.7. According to GSI chart for general rock mass types, each box represents the different relative weights assigned to each combinations of structural and surface conditions (Hoek and Marinos, 2000). The value for GSI varies between 10 for extremely poor rock mass, to 100 for intact rock. Additionally, in order to introduce the heterogeneous rock masses into GSI system, another chart has also been developed particularly for “flysch rocks” by (Hoek and Marinos, 2000). However, it is not presented in this report since it does not apply to the rock mass of the study case.

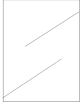
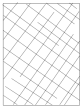
GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek and Marinos, 2000) From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. <u>Note that the table does not apply to structurally controlled failures.</u> Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.		SURFACE CONDITIONS				
STRUCTURE		DECREASING SURFACE QUALITY →				
 DECREASING INTERLOCKING OF ROCK PIECES	INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities	VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
		90			N/A	N/A
		80	70			
			60	50		
				40	30	
					20	
		N/A	N/A			10

Figure 2.6. Guide Chart for Determining GSI Value (Hoek, 1999a).

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	$D = 0$
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	$D = 0$ $D = 0.5$ No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	$D = 0.8$
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

Figure 2.7. Guide Chart for Determining GSI Value (Hoek, 1999a).

2.1.2. Hoek-Brown Material Model

The relatively stiff and strong behaviour of rocks necessitates the utilization of a material model which is different from the models that are used for soils. HB Model is a linear-elastic perfectly-plastic model with Hooke's law for the elastic part and the HB failure criterion implements in a plasticity framework (Brinkgreve, 2015).

The Hoek-Brown model is a continuum approach which is applicable to weathered rocks. The criterion which is a better non-linear approximation of the strength of rocks, involves shear strength as well as tensile strength in a continuous formulation (Brinkgreve *et al.*, 2016).

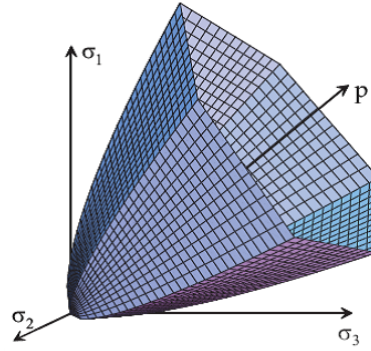


Figure 2.8. The Hoek Brown Failure Contour in Principal Stress Space.

The Hoek-Brown model is an elastic perfectly plastic model with non-associated flow rule. Deformation prior to yielding is assumed to be linear elastic governed by the elastic parameters Young's Modulus (E) and Poisson's ratio (ν) (Schweiger and Nasekhian, 2009). The Hoek-Brown failure contour is curved in the direction of the isotropic stress axis.

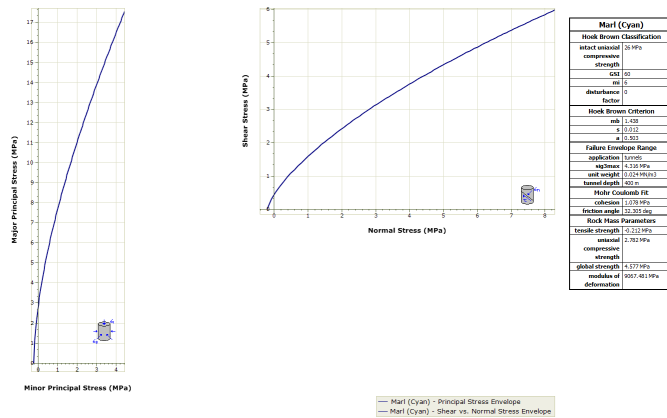


Figure 2.9. Hoek-Brown Failure Contour in a Shear Stress vs. Normal Stress Diagram (Rocscience, 2015).

As can be seen from Figure 2.9, the criterion defines both the shear strength and the tensile strength of the rock. The model parameters are listed in Table 2.2.

Table 2.2. Parameters for the HB Model.

Parameter	Unit	Description
G_{ref}	[kN/m ²]	Elastic Shear Modulus
ν	-	Poissons's Ratio
σ_{ci}	[kN/m ²]	Unconfined Compressive Strength
m_i	-	Intact rock parameter
GSI	-	Geological Strength Index
P_{ref}	-	Reference Stress
D	-	Disturbance Factor

2.1.3. Determination of Rock Mass Properties for Hoek-Brown Criterion

- Modulus of Deformation, E_{rm}

The value of the modulus of deformation is often estimated indirectly from observations of relevant rock mass parameters that can be acquired easily and at low cost (Palmström and Singh, 2001). These parameters are used to estimate the modulus of deformation using approximate equations, such as:

$$E_{rm} = 2RMR - 100 \text{ (for } RMR > 50 \text{) } (Bienawski, 1978) \quad (2.5)$$

$$E_{rm} = 10^{((RMR-10)/40)} \text{ (for } RMR > 50 \text{) } (Serafim\&Pereira, 1983) \quad (2.6)$$

$$E_{rm} = 25 \log Q \text{ (Grimstad\&Barton, 1993)} \quad (2.7)$$

$$E_{rm} = \frac{\sqrt{q_c}}{10} 10^{((RMR-10)/40)} \text{ (Hoek\&Brown, 1997)} \quad (2.8)$$

It is usually suggested to use more than one indirect calculation procedure by

many authors to reach more reliable results. Therefore, after having checked the other possible indirect procedures, the equation is given in Equation 2.8 proposed by Hoek and Brown (1997) has been selected in the modulus of elasticity determination for the synthetic data sets which are created for this research. The main reason for selecting the equation proposed by Hoek and Brown (1997) is the Hoek-Brown material model is employed in the numerical models that are performed within the scope of this study.

- Poisson's ratio, ν

Poisson's ratio value is generally ranges between 0.1 and 0.4. The typical values for specific rock types are listed in Figure 2.10.

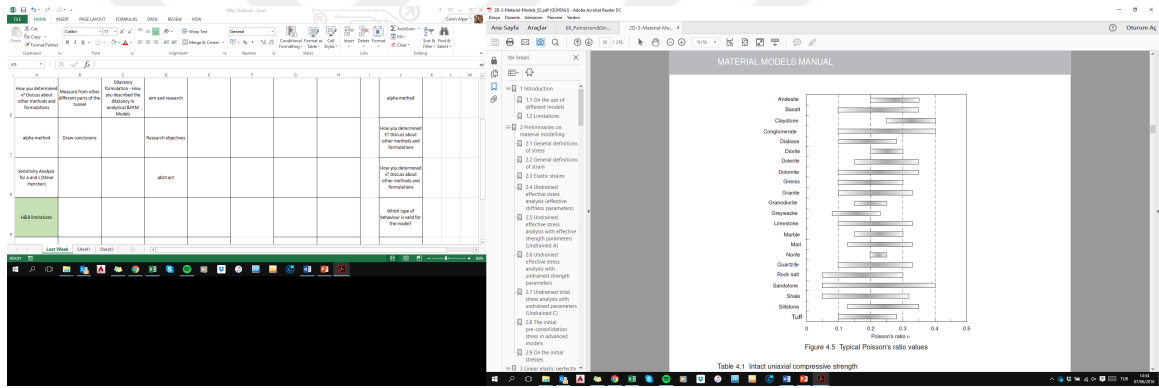


Figure 2.10. Determination of Poisson's Ratio for Rock Masses (Gercek, 2007).

However, as shown in Equation 2.9, a correlation proposed by (Lorig and Pierce, 2000) has been taken into account in the analyses that are performed for synthetic data sets.

$$\nu_{rm} = 0.32 - 0.00.15xGSI \quad (2.9)$$

- Dilatancy, ψ

There are two types of behaviour that an elasto-plastic rock mass may display in terms of dilatancy: non-dilatant or dilatant material. Non-dilatant behaviour means that the rock mass displays no change in volume during plastic shear strains, contrary to this,

in case of dilatant behaviour it does.

At larger confining stress, dilatancy may be assumed as suppressed since evolution of the mobilised dilatancy controlled by σ_3 as shown in Figure 2.11 (Brinkgreve *et al.*, 2016).

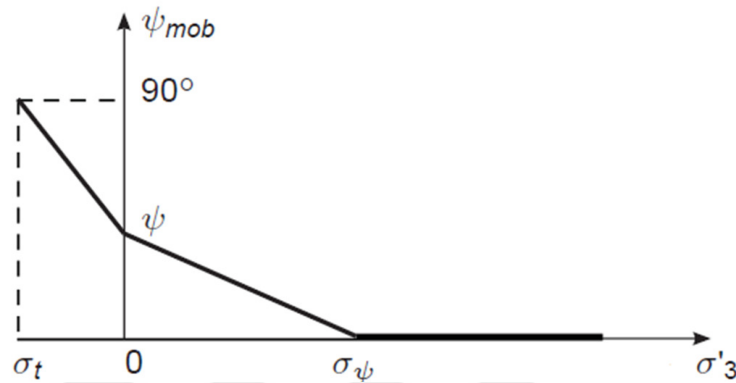


Figure 2.11. Evolution of Mobilised Dilatancy Angle (Brinkgreve *et al.*, 2016).

Therefore, it can be assumed that rock may behave as dilatant material when subjected to shear under relatively low confining stress. However, for this research the behaviour of the surrounding rocks are assumed as non-dilatant due to existence of larger confining stress. This behaviour is modelled using a special value of ψ for to provide $\sigma_3=0$.

2.1.4. Limitations of Hoek-Brown Criterion

Even though Hoek-Brown failure criterion is most frequently used tool to design of rock excavations, there are several conditions in which the failure process that controls the stability of underground excavations is dominated by special circumstances. The Hoek-Brown failure criterion should not be applicable for the following failure conditions which occurs (Marinos *et al.*, 2005).

- along a discontinuity,
- in massive rocks at great depths due to the susceptibility of blasting,
- at soils in which discontinuities and structures have been lost.

Additionally, the HB Criterion is not applicable for stratified and jointed rocks. The application of the criterion to the real cases requires to take into account the assumptions have been made. For instance, the assumption of elastic perfectly plastic isotropic behaviour of the material can be violated easily in the nature due to the anisotropy in rocks.

2.2. Rock Excavation and Support Design Methods

The approaches that are today used in tunnel design can be classified into three different categories: the empirical, rational and observational approaches.

- The empirical approach

characterised by the application of the experience acquired from previous projects to the conditions anticipated at a proposed site. Set of recommendations and schemes used to determine the geo-mechanical properties of the rocks and soils encountered in the tunnel construction.

- The rational approach

depends on the utilization of analytical methods and numerical calculation methods to evaluate the stress and strain relationship in the rock, in the support and reinforcement structures.

- The observational approach

is mainly related to the monitoring of a work during its construction. The required adjustments are defined based on the interpretation of the monitoring results and therefore on the basis of the behaviour of the surrounding ground conditions.

The three previously mentioned approaches are not considered as alternatives but rather as being complementary to each other. Each approach allows one to add useful

information to the geo-mechanical design of a tunnel (Oreste, 2009).

2.2.1. Empirical Methods

2.2.1.1. Q System. The Q-system is introduced by Barton *et al.*, (1974) at Norwegian Geotechnical Institute (NGI). In 2002, an updating was made based on more than 900 new examples from underground excavations in Norway, Switzerland and India (NGI, 2013).

The Q-value is proposed to be used for classification of the rock mass around and underground opening, as well as for field mapping (NGI, 2013).

The concept of Q-system mainly depends on following fundamental requirements:

- Classification of the rock-mass,
- Determination of the optimum dimensions for the underground opening,
- Estimation of the required support type for the stabilization of the underground opening.

High Q-values suggest more stable underground openings compared to low Q-values. Quantification of the rock-mass gives a Q-value which is dependent on following six parameters and calculated using the following equation:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (2.10)$$

where RQD is rock quality designation, J_n is number of joint sets, J_r is joint roughness number, J_a is joint alteration number, J_w is joint water reduction number, SRF is stress reduction factor.

The numerical values of the index Q varies on logarithmic scale from 0.001 to a maximum of 1000. The paired terms in the equation presents the factors that are used to describe the stability in the underground opening (NGI, 2013).

- $\frac{RQD}{J_n}$ is the degree of jointing
- $\frac{J_r}{J_a}$ is joint friction
- $\frac{J_w}{SRF}$ is active stress

The support chart shown in Figure 2.12 gives an average of the empirical data from examined cases. The Q-values are plotted along the horizontal axis and the equivalent dimension along the vertical axis on the left hand side.

In addition to Q-value, two other factors are important for the support design in underground openings. These factors are safety requirements and the dimensions. Generally, the need for support increases as the height of the wall or the span increases. To express safety requirements, a factor called ESR (Excavation Support Ratio) is used. The low values of ESR indicate the need for a high level of safety while the high values of ESR indicate that a lower level of safety will be efficient (NGI, 2013).

$$\frac{\text{Span or height in m}}{ESR} \text{ is equivalent dimension} \quad (2.11)$$

The support chart indicates what type of support is used in terms of the centre to centre spacing for rock bolts and the thickness of the sprayed concrete. It also indicated the energy absorption of the fibre reinforced sprayed concrete, as well as the bolt length and design of reinforced ribs of sprayed concrete (NGI, 2013).

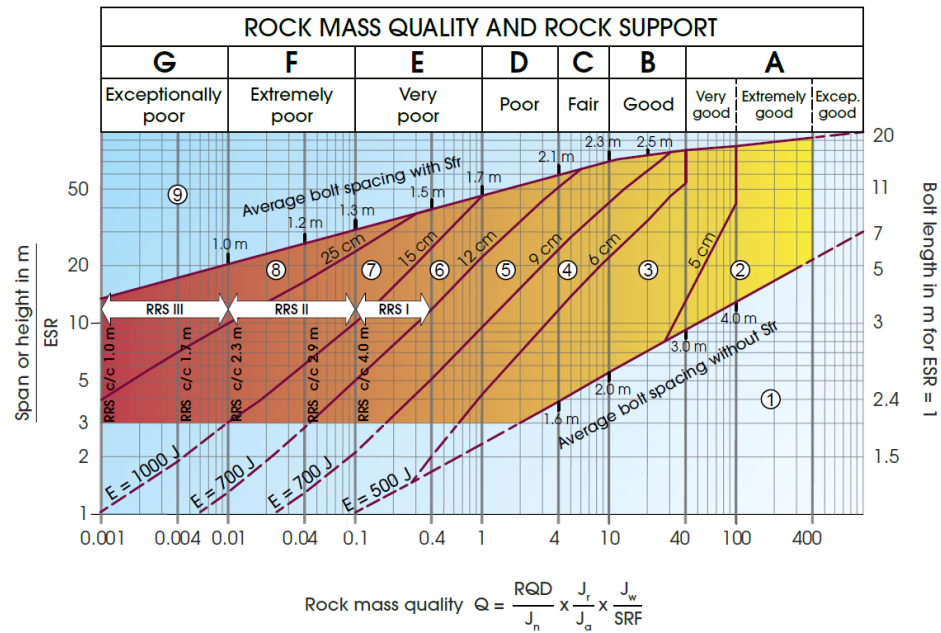


Figure 2.12. Q System Chart (NGI, 2013).

2.2.1.2. Geomechanics Classification (RMR). The Geomechanics Classification or the Rock Mass Rating (RMR) system was first published by Bieniawski (1973). Several refinements have been made over the years, however the description of the classification is based on its 1989 version (Bienawski, 1989). The following six parameters are used to classify a rock mass:

- Uniaxial compressive strength of rock material,
- Rock Quality Designation,
- Groundwater conditions,
- Spacing of discontinuities,
- Orientation of discontinuities,
- Condition of discontinuities.

Table 2.3. Rock Mass Rating System (Bienawski, 1989).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point load strength index	> 10 Mpa	4-10 MPa	2-4 Mpa	1-2 Mpa	For this low range uniaxial compressive		
		Uniaxial comp. strength	> 250 Mpa	100-250 Mpa	50-100MPa	25-50MPa	5-25 MPa	1-5 MPa	< 1 Mpa
	Rating		15	12	7	4	2	1	0
2	Drill core Quaity RQD Rating		90%-100% 20	75%-90% 15	50%-75% 10	25%-50% 8	< 25% 3		
3	Spacing of discontinuities		> 2 m	0.6-2 m	200-600 mm	60-200 mm	< 60 mm		
	Rating		30	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surface Not continuous No separation Unweathered wall rock	Slightly rough surface Speation < 1 mm Slightly weathered walls	Slightly rough surface Separation < 1 mm Highly weathered walls	Slickensided surface of Gouge < 5mm thick or Speration 1-5 mm Continuous	Soft 5mm thick or Sparation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)							
		(Joint water press)							
		(Major principal o)	0	< 0.1	0.1-0.2	0.2-0.5	> 0.5		
		General Conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

Table 2.3. Rock Mass Rating System (Bienawski, 1989).

Parameter	Range of values				
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)					
Strike and dip orientations	Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Tunnels & mines	0	-2	-5	-10	-12
Foundation	0	-2	-7	-15	-25
Slopes	0	-5	-25	-50	
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS					
Ratings	100-81	80-61	60-41	40-21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	very poor rock
D. MEANING OF ROCK CLASSES					
Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300-400	200-300	100-200	< 100
Friction angle of rock mass (deg)	> 45	35-45	25-35	15-25	< 15
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS ^{1 3}					
Discontinuity length (persistence)	< 1 m	1-3 m	3-10 m	10-20 m	> 20 m
Ratings	6	4	2	1	0
Separation (aperture)	None	< 0.1 mm	0.1-1.0 mm	1-5 mm	> 5 mm
Ratings	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
Ratings	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Ratings	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly Weathered	Decomposed
Ratings	6	5	3	1	0
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING ^{2 4}					
Strike perpendicular to tunnel axis	Strike parallel to tunnel axis				
Drive with dip-Dip 45-90o	Drive with dip-Dip 20-45o	Dip 45-90o	Dip 20-45o		
Very favourable	favourable	Very favourable	Fair		
Drive against dip-Dip 45-90o	Drive against dip-Dip 20-45o	Dip 0-20o - Irrespective of strike	Fair		
Fair	Unfavourable	Fair			
* Some condition are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4. directly.					
** Modified after Wickham <i>et al.</i> , (1972).					
*** Instead of A.1, A.2, and A.3 use the charts A-D given in Figure A.1. included included in Application A.					
**** Section E is used to calculate basic RMR.					

2.2.2. New Austrian Tunnelling Method (NATM)

The New Austrian Tunneling Method (NATM) was introduced by Rabcevicz, Müller and Pacher between 1957 and 1965 in Austria (Bienawski, 1989).

NATM is an approach which relates the behaviour of rock masses under loading conditions with monitoring performance of the underground opening during construction. It also consists combination of many established ways of excavation and tunnelling, however the difference of the method is derived from the continuous monitoring of the potential movement of rock material during the construction of an underground opening. Moreover, it requires continuous revision of the design of used stabilization techniques during construction in order to achieve most economical engineering design.

There are seven important features on which the method relies (Bienawski, 1989):

- Mobilization of the strength of the rock mass,
- Shotcrete protection,
- Measurements,
- Flexible support,
- Closing of invert,
- Contractual arrangements,
- Rock mass classification determines support measures.

NATM is essentially a descriptive system, the documentation of the ground conditions is unclear, as it is based largely on subjective observations. Therefore, it is difficult to correlate NATM results with the other empirical methods (Palmstrom and Stille, 2010).

2.2.3. Convergence Confinement Method

Occurrence of the confinement loss during construction of a tunnel leads to redistribution of the stresses around the excavation and deformations. One of the methods

to consider three dimensional tunnel advance and pre-relaxation ahead of the tunnel face in a plane strain analysis is the convergence-confinement method (Panet and Guenot, 1982). The convergence-confinement method (CCM) is a simplified method of analysing the interaction between the ground and the support. It is based on the two-dimensional analysis of the interaction between the support and the ground (Panet *et al.*, 2001).

The simplicity of the method, is a characteristic that has led to its widespread use since the end of the seventies (Oreste, 2009). The method requires two important assumptions:

- The tunnel cross-section is circular,
- The far-field principal stresses normal to the long axis of the tunnel are uniform (hydrostatic).

CCM is a practical tool for estimating the stress and strain state in the rock mass and in the support system at various distances from the excavation face and ahead the face. It also provides the appropriate location for the support installation and determines the structural forces in the support system.

2.2.3.1. Description of the Method. The convergence-confinement method (CCM) allows the ground pressure to be assessed by means of closed-form solutions, and is widely used in engineering practice for preliminary dimensioning of the support (Panet, 1995). The method investigates the interaction between the tunnel support and the ground by plotting on the same graph.

The main objective of the method is to determine the load that the ground will transmit to the support from the time of installation until the time when equilibrium pressure has been reached for the ground and the support system.

The basic elements of the method are illustrated in Figure 2.13 which presents

the construction of a circular tunnel through a rock mass which is subjected to uniform stress field. When a support is installed at a distance L away from the tunnel face, it does not carry the full load immediately. A portion of the load is carried by the tunnel face itself in first place. As the tunnel advances, the amount of load carried by the face decreases and support starts to carry majority of the load that the face had carried earlier. When the face has moved enough away from the installed support, the support starts to carry the maximum design load. Figure 2.14 illustrates the change in the support pressure depending on the advancement of the tunnel face. During this attraction between support and the face, the radial displacement of the support must be equal to the radial displacement of the rock u_r as shown in Figure 2.13 II for compatibility of deformations (Carranza-Torres and Fairhurst, 2000).

Figure 2.14 which illustrates the size of the plastic zone changes with the equivalent support pressure p_i . In an unsupported tunnel the value of p_i reduces from the in situ stress value p_o to zero with distance from the face. The rock starts reacting to the oncoming tunnel about one-half tunnel diameter ahead of the face. At the face about one third of the total deformation has occurred and the final total deformation and complete formation of the plastic zone occurs about 1.5 tunnel diameters behind the face (Hoek, 1999a).

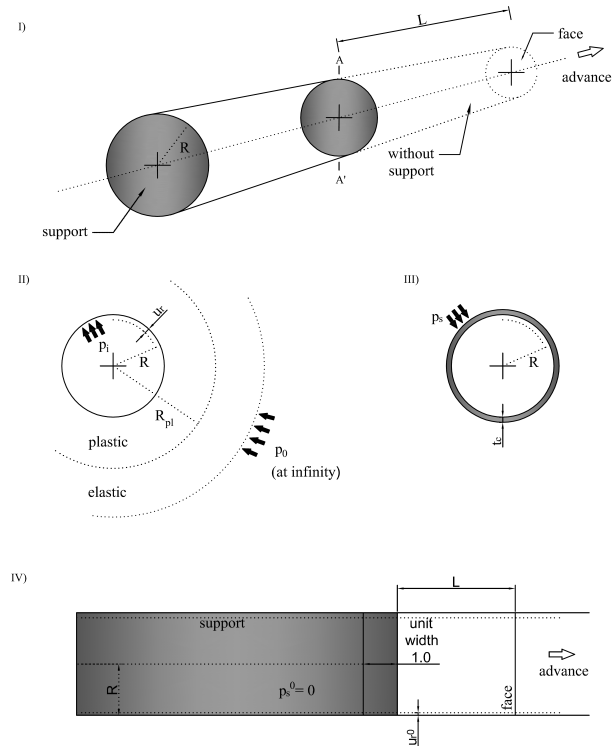


Figure 2.13. I) Construction of A Circular Tunnel With A Radius R In Rock-Mass. II) Cross-Section of The Rock-Mass at A-A'. III) Cross-Section of The Support Installed At A-A' (Carranza-Torres and Fairhurst, 2000).

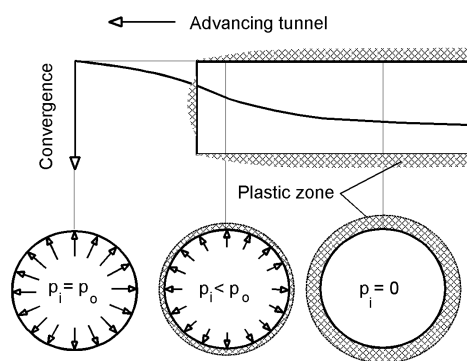


Figure 2.14. Support Pressures p_i at Different Positions during Advance of the Tunnel (Hoek, 1999a).

CCM involves three main components which are the ground reaction curve (GRC), the support characteristic curve (SCC) and the longitudinal deformation profile (LDP).

- Construction of the Ground Reaction Curve

The GRC illustrates the relationship between the internal support pressure and the displacement has occurred on the tunnel wall. The deformations in a rock mass are made up of two components, namely the elastic and the plastic component.

If the internal support pressure p_i is greater than the critical support pressure p_i^{cr} , failure does not occur and the rock-mass shows elastic behaviour. The amount of occurring radial elastic displacement of the tunnel wall is calculated by:

$$u_r^{el} = \frac{R(1+v)}{E_{rm}} (p_o - p_i) \quad (2.12)$$

If the internal support pressure p_i is less than the critical support pressure p_i^{cr} , failure occurs and the rock-mass shows plastic behaviour. A plastic zone with a radius R_{pl} is formed around the tunnel. The radius of the plastic zone is defined by (Carranza-Torres and Fairhurst, 2000):

$$R_{pl} = R_{exp} \left[2\sqrt{p_i^{cr} - \sqrt{P_i}} \right] \quad (2.13)$$

In an underground excavation, the magnitude of the occurring volumetric change is characterized by a dilation angle such that if $\psi = 0^\circ$, there is no change in rock mass volume during plastic deformation; if $\psi > 0^\circ$, the volume increases during plastic deformations.

In some cases, Hoek and Brown (1997) suggests the assumption of no change in volume during plastic deformation may be more appropriate. The excavation of an underground opening at larger confining stresses may be considered as one of them. Therefore, this assumption can also be applied to the Chamoise Tunnel case with a thick overburden of 400 m.

The radial deformation in plastic region can be calculated using (Carranza-Torres and Fairhurst, 2000).

$$\begin{aligned}
 \frac{u_r^{pl}}{R} &= \frac{E_{rm}}{(1+\nu)(\sigma_0 - p_i^{cr})} \\
 &= \left[\frac{1-2\nu}{2} \frac{\sqrt{P_i^{cr}}}{\left(\frac{\sigma_0}{m_b \sigma_{ci}} + \frac{s}{m_b^2} \right) - p_i^{cr}} + 1 \right] \\
 \left(\frac{R_{pl}}{R} \right)^2 &= \frac{1-2\nu}{4 \left[\left(\frac{\sigma_0}{m_b \sigma_{ci}} + \frac{s}{m_b^2} \right) - p_i^{cr} \right]} \left[\ln \left(\frac{R_{pl}}{R} \right) \right]^2 \\
 &\quad - \frac{1-2\nu}{2} \frac{\sqrt{P_i^{cr}}}{\left(\frac{\sigma_0}{m_b \sigma_{ci}} + \frac{s}{m_b^2} \right) - p_i^{cr}} \left[2 \ln \frac{R_{pl}}{R} + 1 \right]
 \end{aligned} \tag{2.14}$$

An example of a ground reaction curve is illustrated in Figure 2.15.

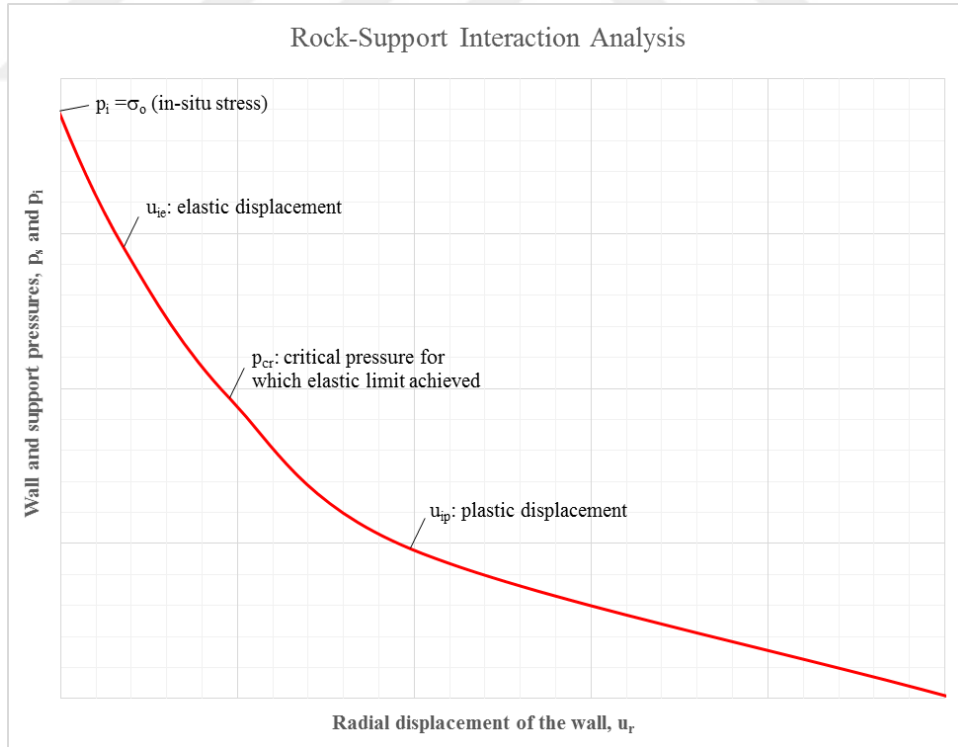


Figure 2.15. Schematic Representation of the Ground Reaction Curve.

The shape of the GRC depends on the failure criterion which is related to rock

mass properties, tunnel radius and in-situ stress conditions. As the rock becomes more fractured, its strength decreases and its reaction curve becomes flatter.

- Construction of the Support Characteristic Curve

After the installation of the support which is in full contact with the rock mass, the support starts to produce deformations. SCC represents the relationship between the developing pressure on the support and the occurring support displacement depending on the mechanical characteristics and geometry of the support.

In this research two types of support that are used in tunnels are examined. First, a shell of concrete has been taken into consideration as the lining of the tunnel. Lastly, its combination with rockbolts has been introduced into the models that are used in the analyses.

- Shell of shotcrete

The maximum pressure that a shell of concrete or shotcrete (Figure 2.16) shall provide is calculated using the formula in Eq. 2.14 and the elastic stiffness of the support can be calculated by Equation 2.16 (Carranza-Torres and Fairhurst, 2000):

$$p_s^{max} = \frac{\sigma_{cc}}{2} \left[1 - \frac{(R - t_c)^2}{R^2} \right] \quad (2.15)$$

$$K_s = \frac{E_c}{(1 - \nu_c) R} \frac{R^2 - (R - t_c)^2}{(1 - 2\nu_c) R^2 + (R - t_c)^2} \quad (2.16)$$

Where σ_{cc} uniaxial compressive strength of the shotcrete or concrete [MPa], E_c Young's Modulus of the shotcrete or concrete [MPa], ν_c Poisson's ratio of the shotcrete or concrete, R radius of the tunnel [m], t_c thickness of the concrete/shotcrete shell [m].

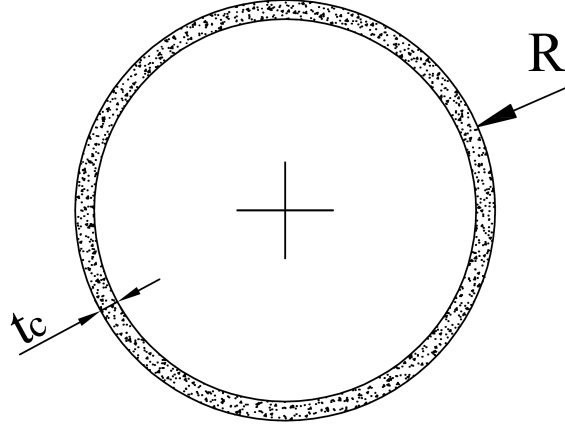


Figure 2.16. Schematic Representation of Concrete Shell.

- Grouted Rock Bolts

The maximum support pressure that is provided by mechanically anchored rockbolts (Figure 2.17) and the stiffness can be calculated using the formula in Equation 2.17 and Equation 2.18 (Carranza-Torres and Fairhurst, 2000):

$$p_s^{max} = \frac{T_{bf}}{s_c s_t} \quad (2.17)$$

The stiffness is

$$\frac{1}{K_s} = s_c s_t \left[\frac{4l}{\pi d_b^2 E_b} + Q \right] \quad (2.18)$$

where T_{bf} is the ultimate load obtained from a pull-out test for rockbolt [MN], Q is the deformation-load constant for the anchor and head [m/MN], E_b is the Young's modulus of the rockbolt [MPa], s_c is the circumferential spacing for rockbolts [m], s_t is the longitudinal spacing for rockbolts [m], l is the free length of the bolt [m], d_b is the diameter of the bolt [m].

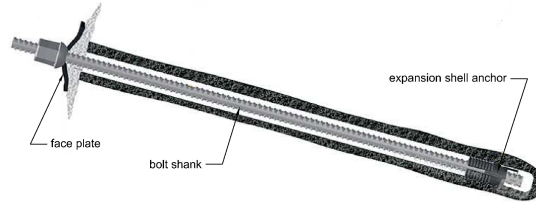


Figure 2.17. Components of UngROUTED Mechanically Anchored Rockbolt Adapted From (Hoek, 2000).

- Combined Support Systems

In case of installation of more than one support system at the same location, their effect can be defined by summing the stiffness of each individual supports as shown in Equation 2.19.

$$K_s^{max} = K_{s,concrete}^{max} + K_{s,rockbolts}^{max} \quad (2.19)$$

This value can be applicable until one of the supports reaches its maximum elastic deformations u_{max}^r . Therefore, the support with the lowest elastic deformation determines the maximum support pressure available for the combined support system. The elastic deformation of the support can be computed from:

$$u_{max}^r = p_s K_s \quad (2.20)$$

An example of a support characteristic curve is illustrated in Figure 2.18.

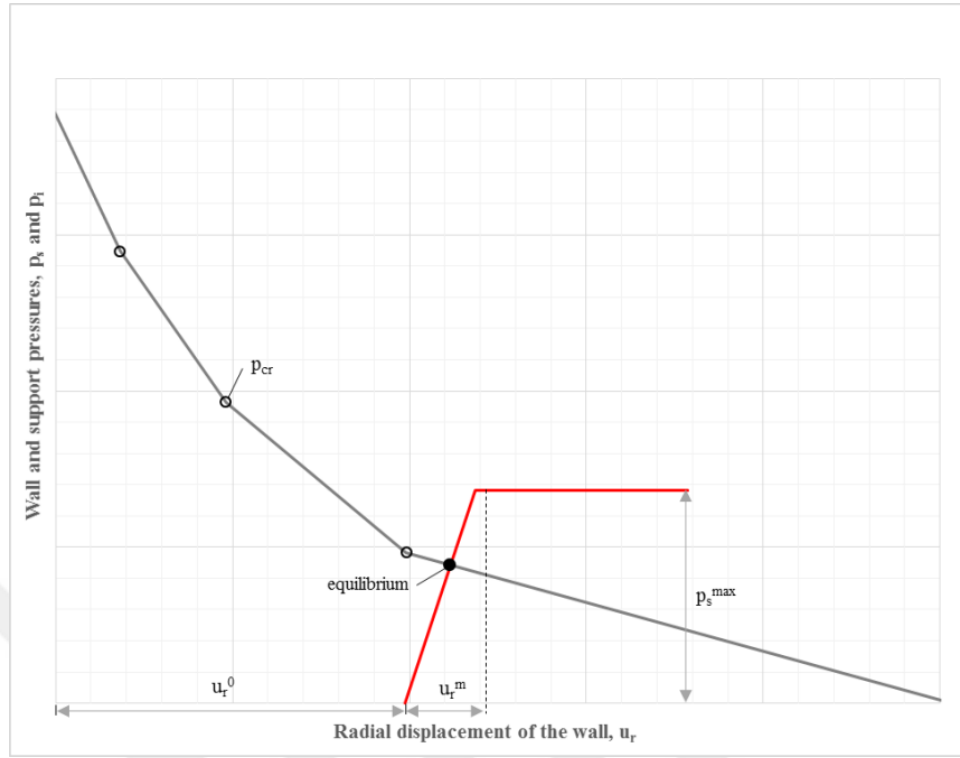


Figure 2.18. Schematic Representation of the Support Characteristic Curve.

- Construction of the Longitudinal Deformation Profile

LDP provides insight into how quickly the support begins to interact with the rock mass behind the face of the tunnel (Carranza-Torres and Fairhurst, 2000). LDP curve gives information regarding the radial displacements developing around an unsupported underground opening.

As illustrated in Figure 2.19, when the distance x is large enough, the radial displacement reaches the maximum.

The empirical relationship between the distance from the tunnel face and the radial displacement established by Hoek (1999b):

$$\frac{u_r}{u_r^M} = \left[1 + e^{\left(\frac{-x/R}{1.10} \right)^{-1.17}} \right] \quad (2.21)$$

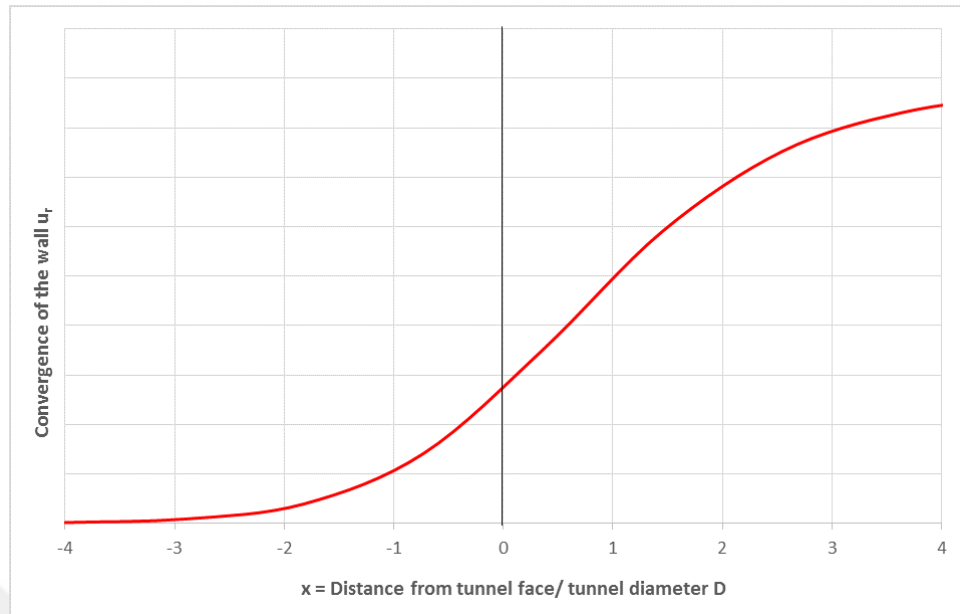


Figure 2.19. Schematic Representation of the Longitudinal Deformation Profile.

Ideally, for tunnels designed according to CCM, the LDP should be constructed from measured data. Where such information is not available, the LDP can be constructed from the numerical models (Carranza-Torres and Fairhurst, 2000).

2.2.3.2. Limitations of the Method. As discussed below, CCM relies on the several simplifying assumptions. The accurate application of the method to the real cases requires to take those assumption into consideration. For instance, if the tunnel geometry is not circular or the in-situ stress conditions are non-uniform, bending moments will also produce in the support due to non-uniform loading. However, for situations in which those assumptions are violated, it is still a practical tool to determine the magnitude of loads that are transmitted to the supported and the unsupported of the tunnel. It is useful to perform quick comparisons of the mechanical response of the different support alternatives, hence to decide on convenient support system (Carranza-Torres and Fairhurst, 2000).

2.2.4. Finite Element Method (FEM)

Finite element method is a numerical technique for finding approximate solutions to boundary value problems. The calculus of variations is used to minimize an error function and produce a stable solution. Analogous to the idea is connecting many simple element equations over many small subdomains, named finite elements, to approximate a more complex equation over a larger domain. These calculations are hard to solve without computer aid especially for larger structures (Reddy, 2005).

Development of this method dates back to 1950's as a computer-based computation technique for the stress based analysis of continuous structures. Known with the abbreviation FEM, this technique has become the most popular analysis approach over the years for the behaviour of deformable structures in the geotechnical engineering.

FEM computations can be used in various fields of engineering and other areas. The core of finite element computation is minimizing the calculation error of the subject structure which needs to be analysed. So the formulations and the computational procedure aim to be as feasible as possible while minimizing the error. The error percentage and the achievability of solution are dependent on the iteration step count, the dimensions and count of finite elements.

2.2.4.1. Modelling Principles with PLAXIS. There are mainly two different ways of modelling a tunnel using the finite element method by PLAXIS 2016. The first one is used to simulate the construction of a tunnel which is mainly excavated by tunnel boring machine (TBM). In this procedure the soil is generally over-excavated, which means that the cross sectional area occupied by the final tunnel lining is always less than the excavated soil area (Brinkgreve *et al.*, 2016).

The second one is, generally employed to simulate the construction of tunnel excavated by NATM technique in which the ground is stabilized using a temporary lining. This method is also known as β -method.

2.2.4.2. β -Method (Load Reduction Method). The β -method is based on the concept of the initial stresses acting on an underground opening can be applied to the unsupported and the supported parts of the opening. The idea originally comes from the main objective of the CCM which is to determine the pressure changes around the tunnel until the equilibrium pressure has been reached.

Two-dimensional finite element models are not able to address properly the three-dimensional natural arching effect that occurs on the tunnel roof. In order to be able to simulate tunnel excavation in two-dimensional finite element analyses, it is required to take into account the natural arching effect by introducing an artificial support pressure which is defined by load reduction factor times the initial stress around the tunnel ($\beta \cdot \sigma_o$). According to β -method, the acting initial stresses are divided into two parts:

- (i) $(1-\beta) \sigma_o$ is the part that is applied to the unsupported tunnel
- (ii) $\beta \sigma_o$ is the part that is applied to the supported tunnel.

As shown from the above given explanations, the β -method has been selected to apply the convergence confinement method with numerical approach for the case of Chamoise Tunnel driven by NATM technique.

3. SYNTHETIC DATA

To compare the analytical results with the numerical results, first a well-established set of material properties are required. For this purpose, a case study reported in the literature by Putz-Perrier *et al.*, (2014) was used. First, the deformations measured in supported tunnel were tried to be simulated by the analytical method. After achieving the real deformations on site, the material has been deteriorated to produce more deformations by decreasing the strength parameters of the material.

Following step is to conduct analyses based on the idea of the convergence confinement method (so called β -method in finite element program PLAXIS with the created data sets. The reason to perform the analysis by using the finite element approach and analytical approach is to establish a comparative study between the obtained results.

This chapter presents how synthetic data has been created using the Chamoise Tunnel project. First, The Chamoise Tunnel project is presented as a case study with general information about the geology of the site, characteristics of the surrounding rocks and geometry of the tunnel. More explanations have been given about the excavation of the tunnel. Lastly, the derivation of the synthetic data is explained step by step and the designated rock-mass parameters for the analyses are represented.

3.1. Case Study: Chamoise Tunnel

The Chamoise tunnel is the core piece of the A40 motorway connection between Macon and Geneva and takes place in France (Figure 3.1). This double tube tunnel, with a length of 3,300 m and excavated cross sections of 90 to 145 m², traverses the Mont d'Ain Monocline, which consists of a sequence of carbonates and marls as shown in Figure 3.2. Construction of the tunnel is performed in 1980s for the first tube and 1990s for the second tube, respectively.

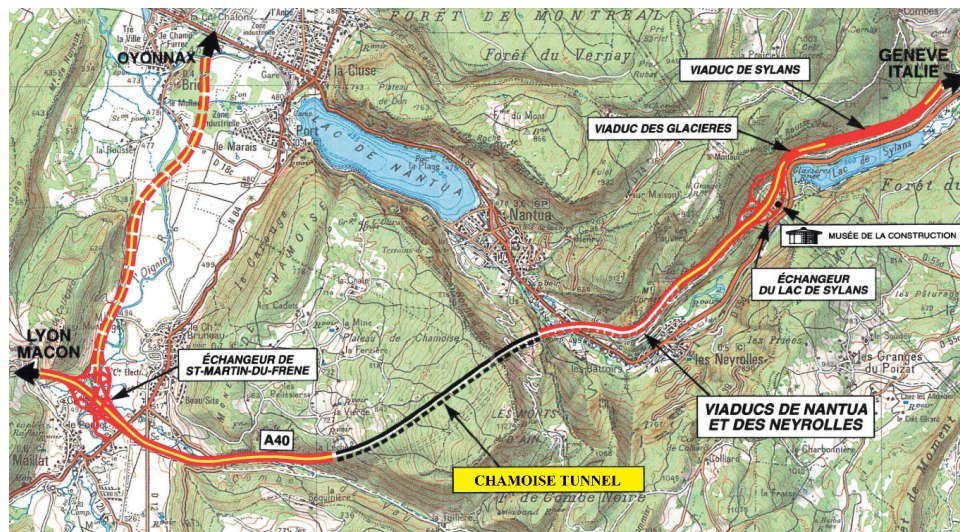


Figure 3.1. Location Map of Chamoise Tunnel.

Although the both tubes of the Chamoise Tunnel traverse the exactly the same geological units, driving of the tubes has been carried out with different methods. Driving of first tube (North tube) was carried out by three-staged mechanical excavation of top-heading, bench and invert using ground support applications such as rock-bolting, shell of shotcrete with welded wire mesh. On the other hand, driving of the second tube (South tube) was performed by full-face drill-and-blast excavation.

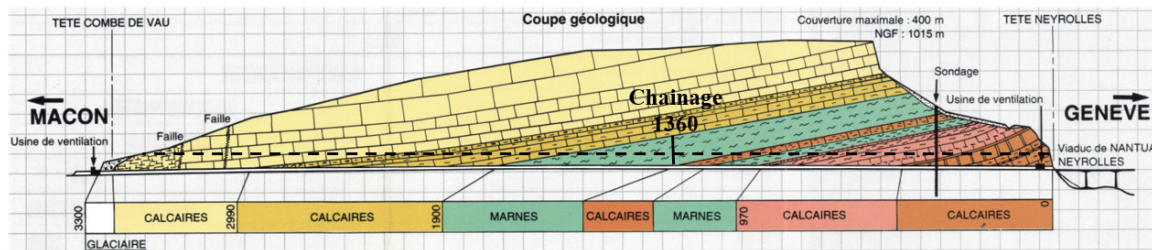


Figure 3.2. Geological Longitudinal Profile of Chamoise Tunnel; the Oxfordian Marls are Shown in Green (“Marnes”) and Separated by a Bed of Marly Limestone Highlighted in Orange (“Calcaires”) (Putz-Perrier *et al.*, 2014).

Comparative study which was performed by Putz-Perrier *et al.*, (2014), investigates the North and the South tube drives of Chamoise Tunnel across the Oxfordian marls is regarded as a case study for this research. The North and South tubes have a radius of 5.8 m and a height of 8.35 m as shown in Figure 3.3. Measured radial

displacements during the construction of the South Tube are considered as the basis to create the synthetic data. Accordingly, three different models will be used as a basis to illustrate the full faced, two-staged and three-staged excavations using finite element method.

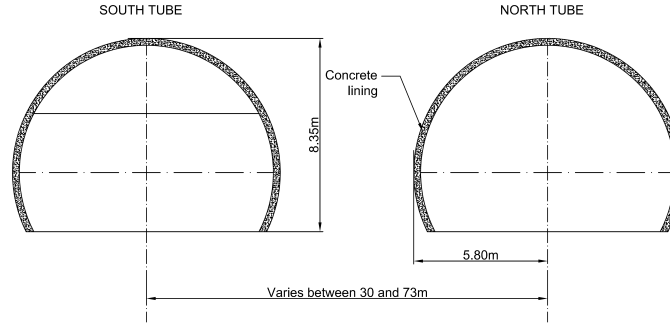


Figure 3.3. Schematic Cross Section of Chamoise Tunnel with Reconnaissance Gallery (Centre), North Tube (Right) and South Tube (Left) (Putz-Perrier *et al.*, 2014).

Ten years after the construction of the first tube, a need for a second tube is required to meet the demand of increasing traffic volumes. During the construction, the water use was strictly forbidden on site since the Oxfordian Marls exhibit high water sensitivity, therefore the problems related to dust generation occurred during the application of staged excavation. Thus, in contrast to three-staged excavation of the first tube, the second tube is driven by full-face excavation.

According to the low water content and relatively high uniaxial compressive strength of the marls, it can be stated that the Oxfordian Marls displayed rock like behaviour during excavation works (Putz-Perrier *et al.*, 2014).

3.2. Derivation of Synthetic Data Sets

During the construction of the South Tube, a section has been selected for measurements. Borehole extensometers were installed perpendicular to the tunnel axis for this section which is located at chainage 1360 m.

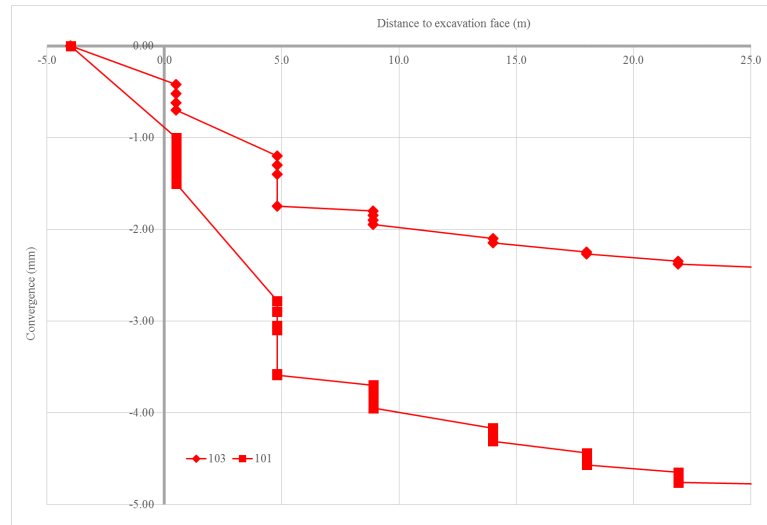


Figure 3.4. Radial Convergences Occurred at Selected Chainage (Putz-Perrier, *et al.*, 2014).

Radial convergences that are obtained in the measurements are plotted in Figure 3.4. The data sets (101 and 103) in the above given figure represent the change in the radial convergences occurred in the tunnel face from the available extensometers located at 1.5 m and 7.5 m away from the tunnel margins. Additionally, the monitoring ahead of the front face was also possible since the instrumentation has been installed in already excavated North tube.

Several steps have been followed to create the synthetic data sets from the measured convergence values. Initially, rock-mass properties for a material which give the radial convergences similar to the convergence values shown in Figure 3.4 are determined. m_i value for the marls which are the dominant rock type on site, is selected 7.0 from the table given by Hoek and Brown (1997) since there is no available lab test to determine the m_i value. Following step was to subtract the already occurred convergences behind the tunnel face from the measured convergence values. Lastly, the rock-mass properties were arranged with small refinements in order to obtain the same inclination with elastic part of the convergence curves obtained from measured data sets.

The convergence values for the measured data sets are the displacements occurred

after the support installations. Accordingly, the designated rock-mass properties are representing the characteristics of the supported rock material which exhibits stiffer and stronger behaviour since the displacements are prevented. For the scope of this research, the designated rock-mass properties are assumed as intact rock properties.

As stated in Putz-Perrier *et al.*, (2014), the locally encountered Oxfordian marls which show high squeezing behaviour produce convergence values greater than 70 mm. Starting from this statement, as the last step of the synthetic data creation, the designated properties of the marls were deteriorated by decreasing the stiffness of the material in order to reach the mentioned 70 mm of convergence after installation of the supports.

Table 3.1. Designated and Deteriorated Rock-Mass Properties for Data Set 1.

Rock-mass properties	Symbol	Units	Value
Data Set 1			
Intact Uniaxial Compressive Strength	σ_{ci}	MPa	24
Geological Strength Index, GSI	GSI	-	65
Material constant for intact rock	m_i	-	7
Young's Modulus	E	MPa	11617
Poisson's ratio	ν	-	0.22
Dilatancy angle	ψ	o	0
Unit weight	γ	kN/m3	24
Data Set 1 (deteriorated)			
Intact Uniaxial Compressive Strength	σ_{ci}	MPa	24
Geological Strength Index, GSI	GSI	-	36
Material constant for intact rock	m_i	-	7
Young's Modulus	E	MPa	2188
Poisson's ratio	ν	-	0.22
Dilatancy angle	ψ	o	0
Unit weight	γ	kN/m3	24

Table 3.1 and Table 3.2 illustrate the intact rock properties and deteriorated rock-mass properties for Data Set 1 and Data Set 2 respectively. Some of those properties are estimated using the methods that are explained in Section 2.1.3.

Table 3.2. Designated and Deteriorated Rock-Mass Properties for Data Set 2.

Rock-mass properties	Symbol	Units	Value
Data Set 2			
Intact Uniaxial Compressive Strength	σ_{ci}	MPa	29
Geological Strength Index, GSI	GSI	-	68
Material constant for intact rock	m_i	-	7
Young's Modulus	E	MPa	15177
Poisson's ratio	ν	-	0.22
Dilatancy angle	ψ	o	5
Unit weight	γ	kN/m3	24
Data Set 2 (deteriorated)			
Intact Uniaxial Compressive Strength	σ_{ci}	MPa	29
Geological Strength Index, GSI	GSI	-	35
Material constant for intact rock	m_i	-	7
Young's Modulus	E	MPa	2271
Poisson's ratio	ν	-	0.22
Dilatancy angle	ψ	o	5
Unit weight	γ	kN/m3	24

Figure 3.5 illustrates the GSI ranges for the rock mass that are used in this study. Area I presents the GSI values for the designated rock mass properties, area II represents the GSI values for the deteriorated rock mass properties.

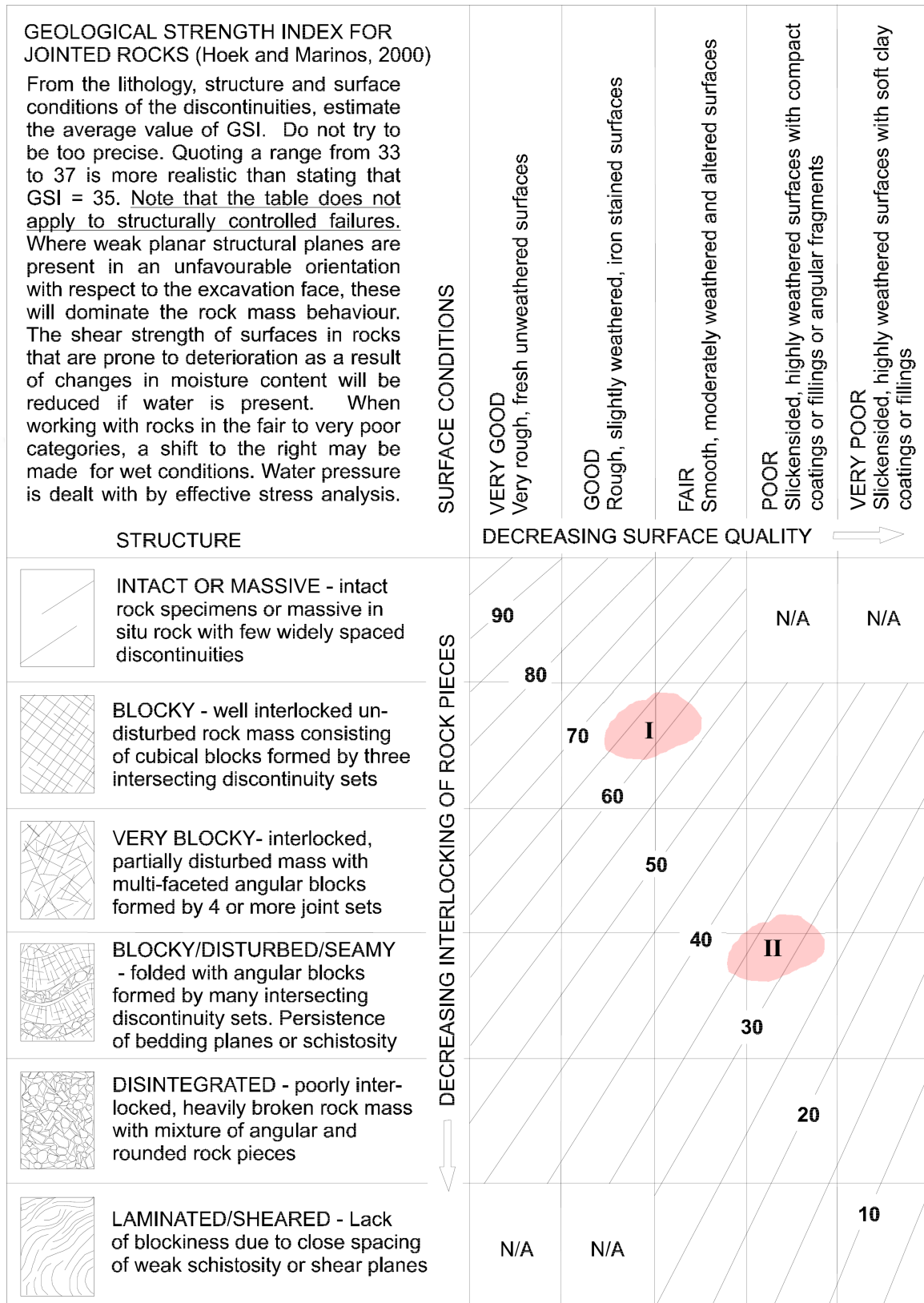


Figure 3.5. GSI Ranges for the Designated and Deteriorated Rock Mass Properties.

4. CONVERGENCE CONFINEMENT METHOD APPLICATION USING SEMI-ANALYTICAL METHOD

Analytical convergence confinement method simulations are illustrated in this part with regard to the circular shaped tunnel excavation in uniform loading conditions and support design for the stabilization of the underground opening. The spread sheet belongs to calculations is given in Appendix A.

4.1. Simulations of the Semi-Analytical Calculations

- Ground Reaction Curve (GRC)

The ground reaction curve defines the relationship between the internal support pressure and the convergence has occurred in tunnel wall. However, it is also possible to define this relationship between the internal support pressure and roof of the tunnel or floor of the tunnel since gravitational loading differs for each of these locations around the tunnel. Figure 4.1 and Figure 4.2 represent the ground reaction curves by taking into account the convergences occurred on the side wall of the tunnel, for the data sets derived from the case of Chamoise Tunnel.

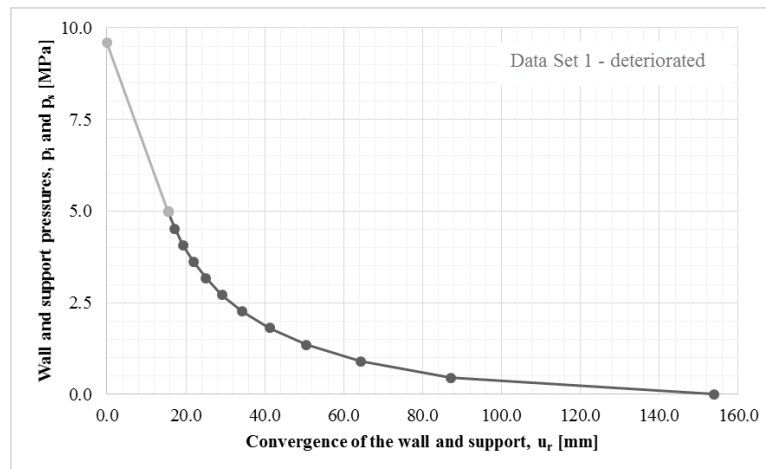


Figure 4.1. Ground Reaction Curve for Data Set 1 (Deteriorated).

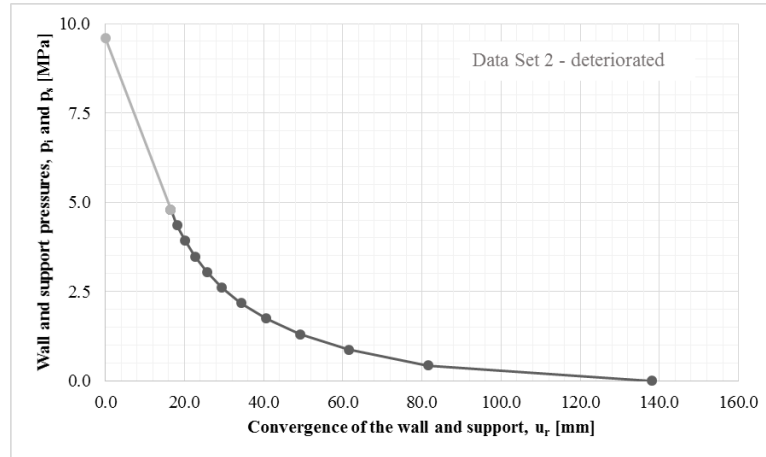


Figure 4.2. Ground Reaction Curve for Data Set 2 (Deteriorated).

The occurring convergence on the tunnel side wall is calculated by means of semi-analytical convergence confinement method. The calculations show that the occurring convergence on the tunnel side wall is in the order of 0.15 m for Data Set 1 and 0.13m for Data Set 2.

The point where the ground reaction curve passes from linear to non-linear form, demonstrates the transition from elastic behaviour to the plastic behaviour. This value, so-called critical support pressure p_i^{cr} which shows that the elastic limit of the rock is reached, equals to 4.98 MPa for Data Set 1 and 4.79 MPa for Data Set 2, respectively. As the strength of the rock decreases the curve becomes flatter. It is obvious that the ground reaction curve of Data Set 1 is more flat than the Data Set 2 due to the strength difference between the deteriorated rock-mass properties of data sets.

- Longitudinal Deformation Profile (LDP)

The LDP graphically demonstrates the radial displacements that occur along the centreline of the tunnel during an unsupported excavation. The horizontal axis represents the distance from the section analysed to the tunnel face; the vertical axis indicates the corresponding radial displacement.

In order to determine the appropriate timing for the installation of support, it is important to establish the longitudinal closure or displacement profile for the tunnel (Vlachopoulos and Diederichs, 2009).

As explained in Chapter 2.2.3, the radial displacement reaches a maximum when the distance from the tunnel face increases. Therefore, it can be assumed that at some distance beyond and ahead of the tunnel face, the advance of the tunnel has no influence in the rock-mass, in other words the radial displacement is zero.

Figure 4.3 and Figure 4.4 represent the occurring radial displacement along the centreline of the tunnel with respect to the distance from the tunnel face for Data Set 1 and Data Set 2, respectively. The LDPs of the data sets are plotted with regard to the designated and deteriorated rock-mass properties.

The calculations show that the radial displacement belonging to Data Set 1 for designated rock-mass properties is in the order of 5 mm. However, for the deteriorated rock-mass properties the radial displacements reaches a maximum, away from tunnel face at three times of the tunnel diameter. The maximum radial displacement is in the order of 150 mm.

The calculations show that the radial displacement belongs to Data Set 2 for designated rock-mass properties, is in the order of 2.5 mm. However, for the deteriorated rock-mass properties, the radial displacements reaches a maximum, away from tunnel face at three times of the tunnel diameter. The maximum radial displacement is in the order of 130 mm.

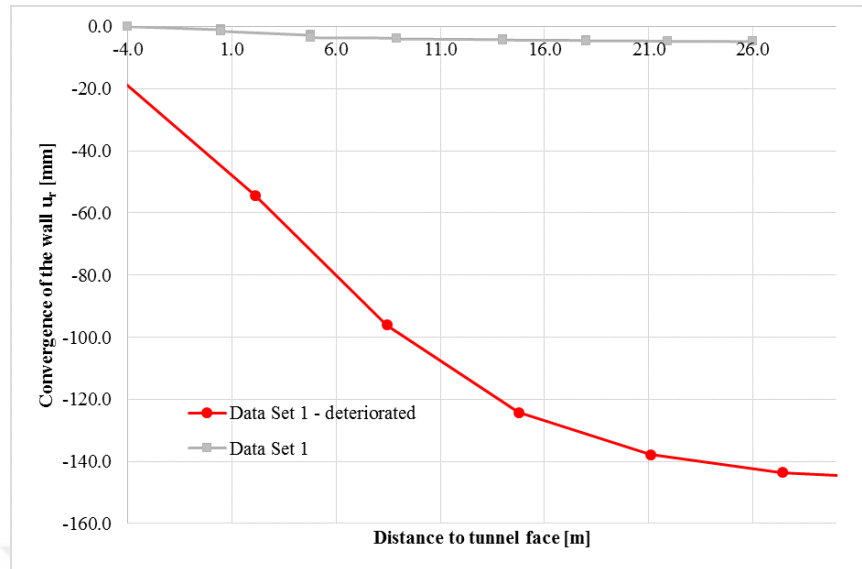


Figure 4.3. Longitudinal Deformation Profile for Data Set 1(Deteriorated).

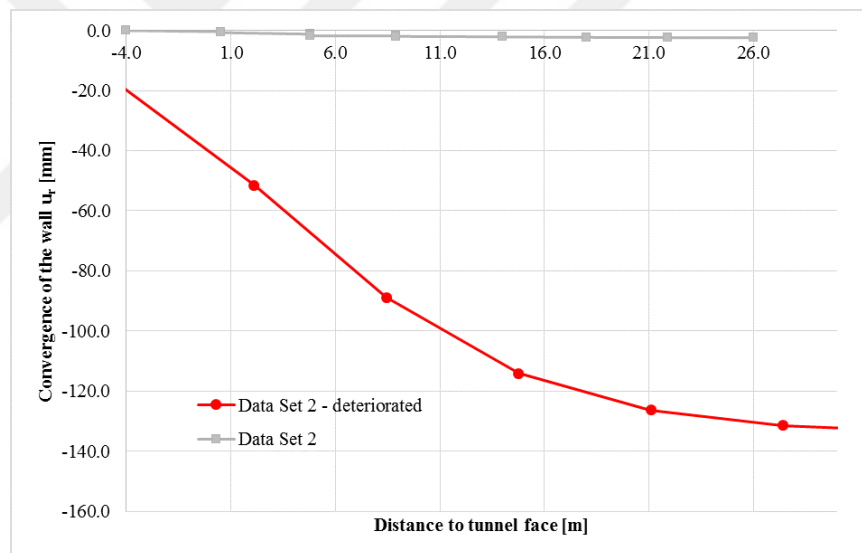


Figure 4.4. Longitudinal Deformation Profile for Data Set 2 (Deteriorated).

- Support Characteristics Curve (SCC)

Support characteristics curve demonstrates the relationship between the oncoming pressure on the support and the subsequently increasing radial displacement of the support depending on the geometry and the characteristics of the support.

The geometrical and mechanical characteristics of the used support systems are represented in the following part.

- Shotcrete Lining

A closed shell of concrete with a thickness of 300 mm is considered for the model. The typical values of σ_{cc} and E_c for dry shotcrete are listed in Table 4.1. Poisson's ratio of the shotcrete is assumed as $\nu = 0.15$.

Table 4.1. 1-Day Strength Parameters Used For Dry Shotcrete Mixture (Singh and Bortz, 1975).

Support Type	Symbol	Units	Value
Shotcrete			
Uniaxial Compressive Strength	σ_{cc}	[MPa]	20.3×10^2
Young's modulus	E	[kN/m ²]	13.6×10^6

- UngROUTED Rockbolts

As illustrated in Figure 4.6, mechanically anchored rockbolts have been installed in the surrounding rock mass. The circumferential spacing is taken into account as 1.5m therefore, the number of bolts installed in the tunnel cross-section is equal to 24.

Table 4.2. Support Characteristics for Rockbolts.

Support Type	Symbol	Units	Value
Rockbolts			
Pull-out resistance	T_{bf}	[MN]	0.267
Diameter	d_{rb}	[mm]	25
Circumferential spacing	s_c	[m]	1.5
Longitudinal spacing	s_l	[m]	1
Free length	l	[m]	4

- Combined Supports

The rockbolts and the shotcrete lining were used at the same location for the case of Chamoise Tunnel. Therefore, their combined effect has been taken into consideration

in the calculations.

Table 4.3. Support Characteristics for Combined Support System.

Support Type	Symbol	Units	Value
Combined Support			
Maximum support pressure	p_s^{max}	[MPa]	1.10
Stiffness of the support	K_s	[MPa/m]	0.145×10^{-3}
Individual Supports			
Shotcrete Lining			
Maximum support pressure	p_s^{max}	[MPa]	1.02
Stiffness of the support	K_s	[MPa/m]	0.135×10^{-3}
Rockbolts			
Maximum support pressure	p_s^{max}	[MPa]	1.52
Stiffness of the support	K_s	[MPa/m]	0.009×10^{-3}

Figure 4.5 and Figure 4.6 demonstrate the rock support interaction by plotting the main components of the method in the same graph. It should be noted that the ground reaction curve represents the radial displacements from the roof of the tunnel since it creates higher radial displacements compared to the side wall and the floor of the tunnel due to gravitational loading.

The change of the maximum support pressure in between the individual supports and the combined supports obviously shows that the rock-bolts does not have a significant role for the stabilization of the underground opening for this case. The individual support pressures are 1.02 MPa and 1.52 MPa for shotcrete lining and the rockbolts, respectively. However, the combined support pressure is calculated as 1.10 MPa. It can be said that the total support pressure is slightly improved with the usage of combined supports. The main reason for this can be evaluated as the significantly low stiffness of the rockbolts.

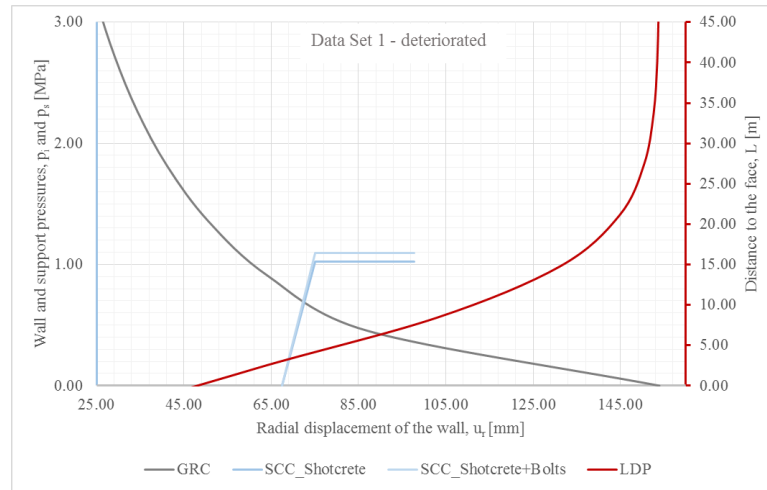


Figure 4.5. Rock-Support Interaction Analysis for Data Set 1 (deteriorated).

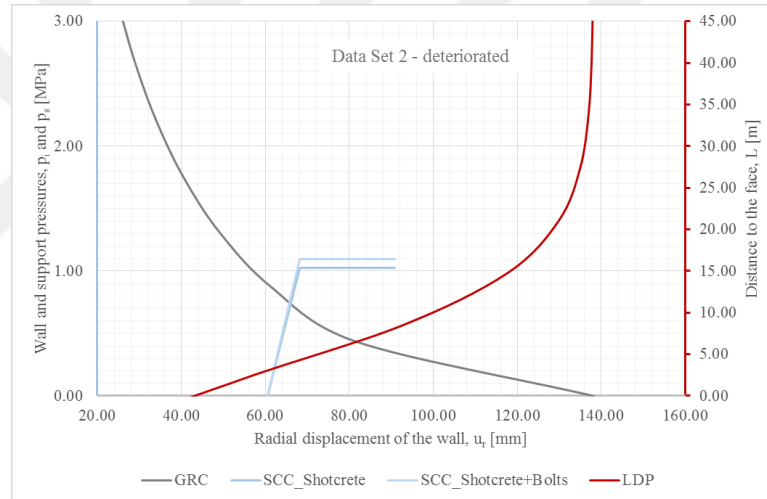


Figure 4.6. Rock-Support Interaction Analysis for Data Set 2 (deteriorated).

4.2. Validation of Semi-Analytical Calculations using RocSupport

Prior to commencement of modelling with two-dimensional FEM, the expected convergence values were examined by using RocSupport Software (Rocscience Inc.) which investigates the rock support interaction and deformation analysis for tunnels in rocks. The main purpose of this is to validate the analytical method that is used to simulate the CCM.

Figure 4.7 and Figure 4.8 present the used tunnel sections in RocSupport for

different types of support combinations. The rock-mass properties used in the RocSupport models are the similar values used in conventional calculation methods as listed in Table 3.1. Furthermore, the support characteristics are listed in Table 4.1 and Table 4.2.

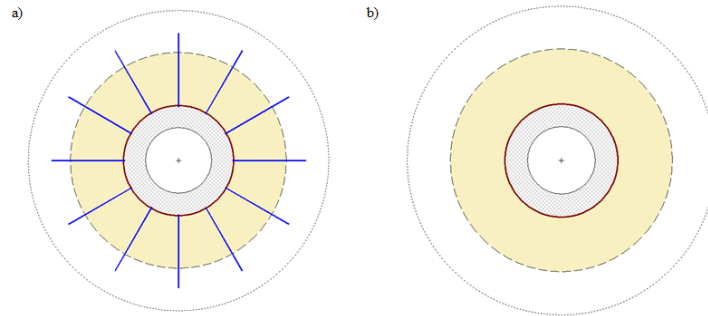


Figure 4.7. Tunnel Sections Used in Rocsupport for the Support a) Combination of Rockbolts and Shotcrete Lining, b) Shotcrete Lining.

The results of the analyses conducted with the use of RocSupport are listed in Table 4.4 for deteriorated Data Set 1 and Data Set 2, respectively. It should be noted that the software calculates the convergences for a circular one-staged excavation under uniform stress conditions ($K_o=1.0$).

Table 4.4. Determined Radial Displacements by Analytical Method.

Support System	Convergence (%)	Radial Displacements (mm)
Data Set 1 (deteriorated)		
Rock Bolts + Shotcrete Lining	1.29	75.13
Shotcrete Lining	1.31	75.89
Without Support	3.02	146.8
Data Set 2 (deteriorated)		
Rock Bolts + Shotcrete Lining	1.22	71.5
Shotcrete Lining	1.24	72.24
Without Support	2.47	134.4

The radial displacements may be determined by the conventional analytical cal-

culations by finding the horizontal coordinate of the intersection point of the GRC and SCC from rocksupport interaction graphs (Figure 4.5 and Figure 4.6). Those displacements can be found as approximately 75 mm for Data Set 1 and 72 mm for Data Set 2.

As a result, it can be stated that the obtained radial displacements by means of conventional analytical calculations are in good match with results that are obtained from Rocsupport.

4.3. Discussion on the Results of Semi-Analytical Analyses

4.3.1. Limitations of the Semi-Analytical Analyses

As explained in this chapter, the semi-analytical convergence confinement method built upon several assumptions, the most important of which are the circular geometry of the tunnel and uniform in-situ stresses. Therefore; alternative in-situ stress conditions ($K \neq 1.0$), the different tunnel shapes and the staged excavations are not included in the analytical application of the convergence confinement method.

4.3.2. Efficiency of the supports used in tunnel excavation

The alternative support systems that are used in the tunnel stabilization are considered as 300 mm of shotcrete lining as individual support and a combined support system consists of 300 mm of shotcrete lining with $L=4$ m, 25 mm of rockbolts.

According to the analytical rock-support interaction analyses performed (Figure 4.5 and Figure 4.6), it can be stated that the rockbolts that are installed as a member of combined support system in the tunnel construction do not show a significant effect on maximum support pressures.

In case where the tunnel is not been supported, the radial displacement that has occurred along the tunnel wall has been calculated in the order of 160 mm and 135

mm for Data Set 1 and Data Set 2, respectively. If the same calculation procedure is also been followed for the supported tunnel, the calculated radial displacements on the circular tunnel wall (for full face excavation in uniform stress conditions) are determined as in Figure 4.8.

In case where only shotcrete is used as an individual support, radial displacement is found as 76 mm. However; in case where the tunnel is supported with a combined system including shotcrete lining and rockbolts, the radial displacement is calculated as 75 mm for Data Set1.

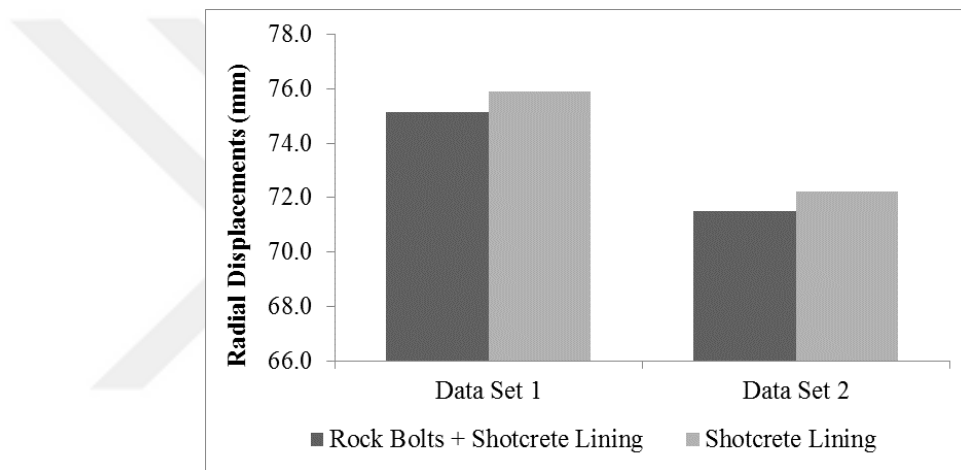


Figure 4.8. Radial Displacements along the Tunnel Wall Calculated by Analytical Method.

Similarly, for Data Set 2, In case where only shotcrete is used as an individual support, radial displacement is found as 72 mm, however in case where the tunnel is supported with a combined system including shotcrete lining and rockbolts, the radial displacement is calculated as 71 mm.

It is noticeable that the installed rockbolts does not produce a significant effect on the occurring radial displacements along the tunnel wall for this case study.

4.4. Application of Rock Mass Classification Systems to the Synthetic Data Derived From Chamoise Tunnel

For a preliminary tunnel design, at least two classification systems should be applied (Bienawski, 1989). For the preliminary design of the Chamoise Tunnel, RMR and Q classification systems were used.

4.4.1. Application of Q System

The application of Q system is performed for Chamoise Tunnel. As listed Table 4.5, the ratings related to joint conditions are estimated based on the information given in case study established by Putz-Perrier *et al.*, (2014). According to the Q system, rating of the marls which are the dominated rock material on site, changes between 0.75 and 6.0. These values correspond to Class C and D which represents the poor quality rock and fair rock, respectively.

Table 4.5. RMR Ratings for Synthetic Data Sets.

Q System Ratings			
1	RQD	Poor	25-50
2	Joint set number, Jn	One joint set plus random joints	0.5
3	Joint roughness number, Jr	Rough or irregular, undulating	3
4	Joint alteration number, Ja	Silty or sandy clay coatings, small clay-faction	0.75
5	Joint water reduction factor, Jw	Dry excavations or minor inflow	1
6	Stress reduction factor, SRF	$\sigma_c/\sigma_1=2-3$, $\sigma_\theta/\sigma_1=0.6$	50-200
7	ESR-values	Very important tunnel with a long lifetime	0.5
8	Q value	$0.1 < Q \leq 10$	0.75 - 6.0

For 3 m of span as in the Chamoise Tunnel case, the stability of the underground opening can be ensured by a systematic bolting with shotcrete. This support requirements, which are determined for the stability of the underground opening using Q system, are matching well with the application on site during the construction of Chamoise Tunnel.

4.4.2. Application of RMR System

The RMR system is applied based on the Bienawski's 1989 version. As summarized in Chapter 2, the required six parameters were determined and by summing those parameters RMR value was computed as illustrated in Table 4.6.

Table 4.6. RMR Ratings for Synthetic Data Sets.

RMR Ratings				
A. Classification parameters and their ratings				55-62
	1	Strength of intact rock material	20 - 26 MPa	2-4
	2	RQD	25-50 %	8
	3	Spacing of discontinuities	> 2 m	20
	4	Condition of discontinuities	Separation < 1 mm	10-15
	5	Groundwater	Completely dry	15
B. Rating adjustment for discontinuity orientations			Fair	-5
C. Rock mass classes determined from total ratings			III - Fair Rock	50-57
D. Meaning of rock mass classes for tunneling works (stand-up time)			III - Fair Rock	1 week for 5 m span

The computed RMR values for the marls are changing between 50 and 57. These values correspond to Class III which represents the fair quality rock. The rock masses which belong to Class III, are allowed stand one week for 5 m span without support. In Chamoise Tunnel, the supports are installed 3 m away from the tunnel face. Therefore, it can be stated that the support requirements, which are determined for the stability of the underground opening using RMR, are matching well with the application on site during the construction of Chamoise Tunnel.

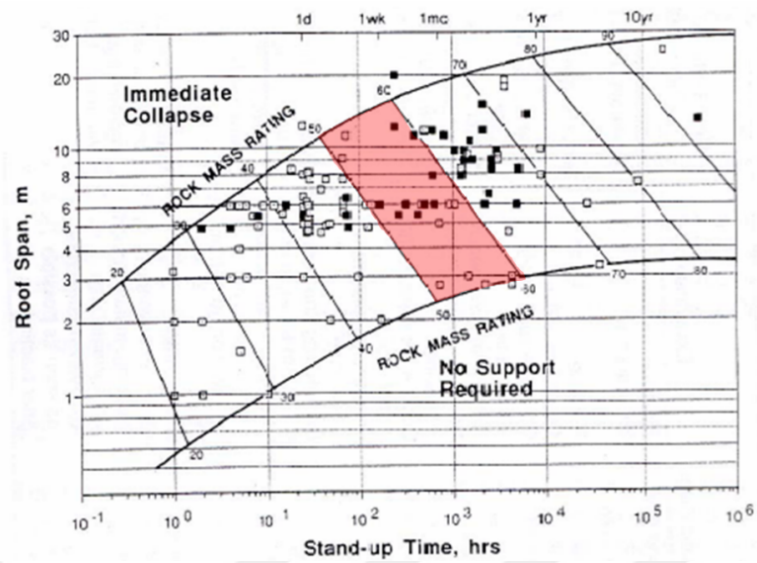


Figure 4.9. RMR Range for Chamoise Tunnel on Unsupported Span-Time Chart of the RMR System.

The red area in Figure 4.9 represents the RMR range for Chamoise Tunnel on unsupported span time chart of the RMR system.

5. CONVERGENCE CONFINEMENT METHOD APPLICATION USING NUMERICAL METHOD

In this chapter, the numerical method simulations are illustrated with regard to the case study: Chamoise Tunnel, using finite element program PLAXIS. A tunnel excavation supported with shotcrete lining and rockbolts, is represented with respect to the geometry and rock-mass properties of the Chamoise Tunnel. The Hoek-Brown material model is employed to model the behaviour of the rock material as CCM method validates the material which is in accordance with Hoek-Brown failure criterion.

In Chamoise Tunnel Project, the construction of the first tube (North tube) was carried out by three-staged mechanical excavation and the driving of the second tube (South tube) was performed by full-face drill-and-blast excavation. Therefore, three different models are used as a basis to illustrate the full faced and three-staged excavations using finite element method. Additionally, two-staged excavation has been introduced into the models to be able to scan the transition.

The scope of this chapter is to summarize how the geotechnical models have been created and to simulate CCM using a finite element program. Furthermore investigations are performed on the influence of the variation of the selected parameters related to excavation, on the extent of rock behaviour. The outputs of numerical analyses are given in Appendix B.

5.1. Set up of the Geotechnical Model

To be able to apply CCM on numerical models; first, the ratio between the loads transmitted to the support and to the ground must be determined for each excavation phase on the models. Therefore, the ratio which produces the same convergences with the ones calculated by analytical methods, is determined by β -method (Section 2.2.4.2). For this purpose, first, the convergences that are listed in Table 4.4 (Section 4.2) tried

to obtain in the model which represents circular tunnel excavation conducted in one stage under uniform stress conditions ($K_o=1.0$).

After determining the ratio between the loads applied to the support and to the ground, following steps are followed to answer some of the research questions:

- Changing the number of stages (to simulate full-faced, two-staged and three-staged tunnel excavation)
- Changing the initial stress ratio (under uniform and non-uniform stress conditions),
- Changing the selected support system (shotcrete lining as individual support system and combined support system consisting of shotcrete lining and rockbolts)
- Changing the shape of the tunnel (fully circular shaped and circular arc shaped opening)
- Finite Element Mesh

The geotechnical model used for all simulations is shown in Figure 5.1. The plane strain conditions has been assumed since the tunnel is long. Accordingly, only the cross section of the tunnel has been considered for the analyses. The domain extends four times of the tunnel diameter both above and below the tunnel centreline in the vertical direction and extends three times of the tunnel diameter for both right and left side in the horizontal direction.

The finite element mesh (from a depth of 90 m to a depth of 0 m) consists of fifteen noded triangular elements. The required mesh coarseness to model the tunnel has been determined by plotting the computed radial displacements on the tunnel wall as the number of elements change as shown in following figure.

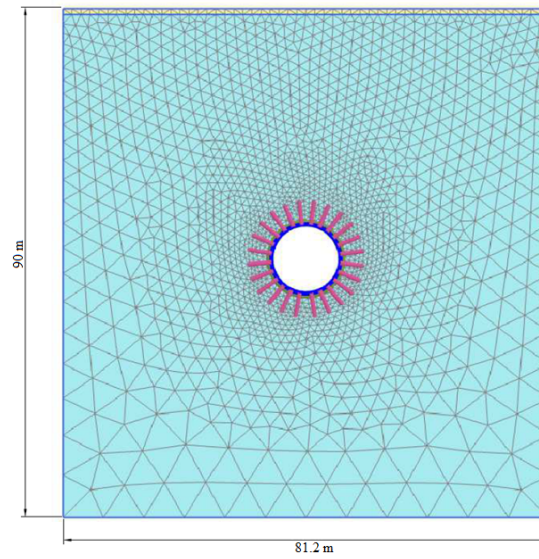


Figure 5.1. Geotechnical Model and FEM Mesh Used for the Analyses.

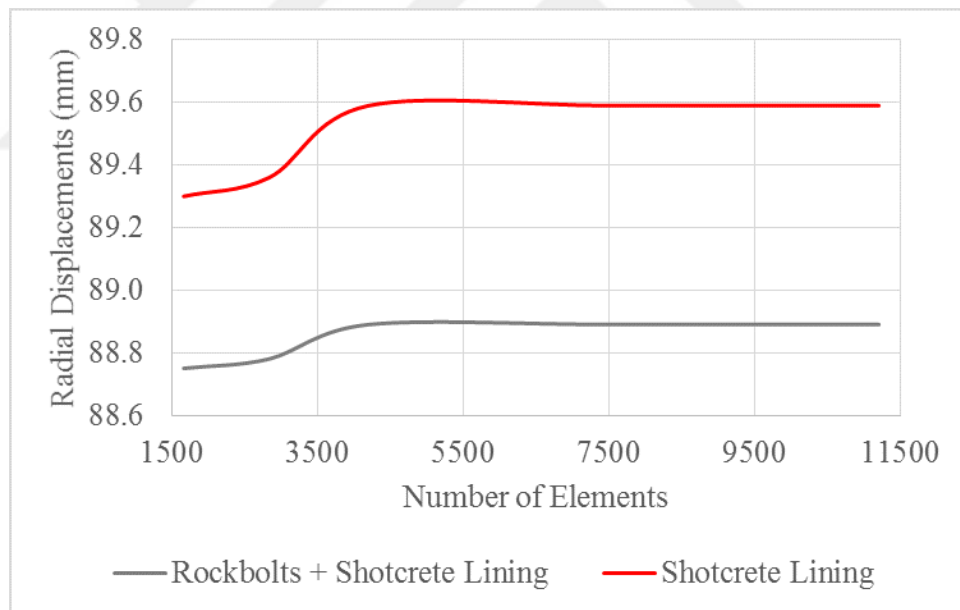


Figure 5.2. Change in Radial Displacements by Number of Elements in Mesh.

As shown in Figure 5.2, the curves exhibit a plateau after reaching the required number of element for this numerical model. Additionally, the required average element size is also determined using the same approach with the one used in determination of required number of elements in the model.

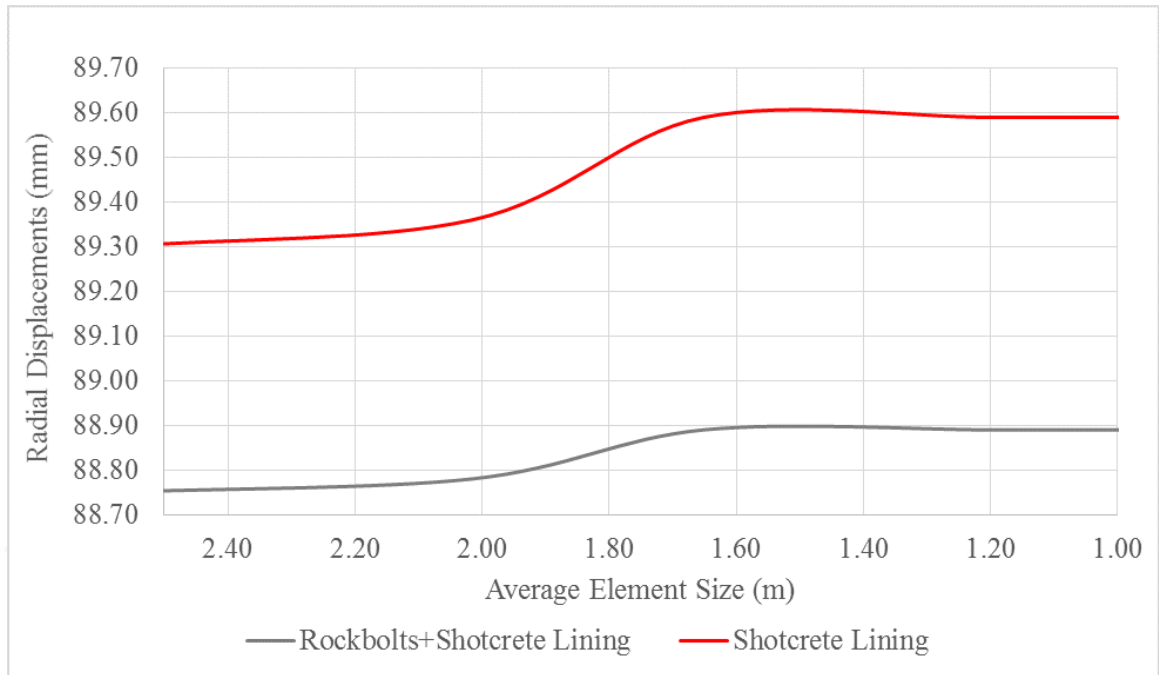


Figure 5.3. Change in Radial Displacements by Mesh Element Size.

Ultimately, the coarseness factor used for the generation of the mesh varies along the model in order to achieve higher accuracy in the vicinity of the excavation and the supports where stress and strain gradients are expected to be higher. The coarseness factor has been considered as 0.25 for the vicinity of the excavation, 0.10 for the vicinity of supports (rockbolts and shotcrete). For the rest of the model, the mesh has been generated with a coarseness factor of 1.0.

The following mesh characteristics have been considered adequate for the models which are employed for the application of numerical method. The mesh characteristic which have been determined using the Data Set 1, have also been used for Data Set 2.

Table 5.1. Mesh Characteristics.

Number of elements	Average element size (m) ¹
7561	1.2

¹The average element size does not represent the real average element size considering the refinements around the tunnel.

- Tunnel Geometry and Overburden Pressure

The cross section of the tunnel is considered as circular with a radius of 5.8 m and plane strain conditions have been assumed for the model since the tunnel is very long. The overburden depth for the Chamoise Tunnel is 400 m, therefore, the overburden pressure has been introduced into model using a 1m thick rock layer with an exaggerated unit weight. For this case, the unit weight of marls which are the dominant rock type on site is assumed equal to $\gamma_{marl}=24 \text{ kN/m}^3$. The overburden pressure is equal to $\gamma_{marl} \times \text{depth}=8640 \text{ kPa}$. Therefore, a unit weight of 8640 kN/m^3 has been taken into account for the layer which presents the overburden pressure.

- Shotcrete Lining Properties

A liner with a thickness equal to 0.3 m is used and the liner behaviour is assumed as linear-elastic. The cross sectional area (A) of the shotcrete lining per meter out of plane is equal to $A=0.3 \text{ m}^2/\text{m}$. The moment of inertia of the shotcrete lining is calculated using the Equation 5.1.

$$I = \frac{1}{12}d^3 \quad (5.1)$$

Table 5.2. Characteristics of the Shotcrete Used in the Model.

Support Type	Symbol	Units	Value
Shotcrete			
Thickness	d	[m]	0.3
Young's modulus ¹	E	[kN/m ²]	13.6x10 ⁶
Axial stiffness	EA	[kN/m]	4.08x10 ⁶
Bending stiffness	EI	[kN.m ² /m]	3.06x10 ⁶

The calculated characteristics for shotcrete lining is listed in Table 4.1.

¹Taken from Singh and Bortz (1975).

- Rockbolt Properties

Rockbolts are introduced to the model as embedded beam rows with a length of $L=4$ m. The following mechanical geometrical characteristics haven been used in the model are listed in Table 5.3.

Table 5.3. Characteristics of the Rockbolts Used in the Model.

Support Type	Symbol	Units	Value
Rockbolts			
Stiffness	E	[kN/m ²]	207×10^6
Unit weight	γ	[kN/m ³]	78
Diameter	D_{rb}	[m]	0.025
Longitudinal spacing	$L_{spacing}$	[m]	1
Skin resistance	T_{skin}	[kN/m]	66.75

- Assumptions for the Model

Following assumptions have been made for the development of the models:

- The excavation is performed under plane strain conditions,
- The initial stress fields are assumed as $K_o=0.6$, $K_o=1.0$ and $K_o=1.2$ for the analyses,
- The rock-mass is isotropic and homogenous.

5.2. Calculation Phases

The calculation process consists of mainly four different phases. However, the number of the phases varies with regard to the number of excavation stages that are performed for tunnel construction. Following calculation steps are generated using the similar procedure in PLAXIS 2016 for excavation of an NATM tunnel, namely β -method.

- (i) Initial stress field was created using the K_0 procedure for the model which is 90 m in height and 81.2 m in width to calculate the initial stresses. A finite element

mesh was generated. The coarseness of the mesh varies along the model to ensure the higher accuracy in calculations.

- (ii) The tunnel geometry was introduced into the model. The related parts of the tunnel was deactivated in order to simulate the excavation for the top heading, the bench and the invert respectively. For one-staged excavation, this phase contains the complete excavation of the tunnel.
- (iii) Later, the supports that are used in tunnel construction are introduced into model. Even though the real application of the shotcrete lining usually followed by the installation of the rockbolts, both support types are activated in the same phase for this model in order to simplify the application of the β -method.
- (iv) Ultimately, the change in the tunnel radius is measured at the end of the analyses as the last phase of calculations. The convergence value has occurred in tunnel is calculated by means of difference between the deformed mesh and the original mesh. The measurements are performed between the roof and the floor of the tunnel.

$$Convergence (\%) = \frac{D_{original} - D_{deformed}}{D_{original}} \times 100 \quad (5.2)$$

5.3. Discussion on the Results of the Numerical Analyses

5.3.1. Influence of the in-situ stress conditions

The distribution of deformations on the rock mass are investigated by means of a two-dimensional finite element model. Additionally, the influence of the initial stress ratio of convergence values occurring on the tunnel wall were explored.

The numerical analyses are divided into three parts, consecutively dedicated to tunnel excavation; creation of initial stress field distribution, excavation of the tunnel, installation of support systems. The non-linear Hoek-Brown criterion is employed to examine the rock-mass behaviour.

The tunnel being considered as embedded in an elasto-plastic rock mass which is surrounded by uniform in-situ stresses for this case. To be able to determine the convergences in numerical models; first, the ratio between the pressures transmitted to the support and to the ground must be determined for each excavation phases in the models by the application of β -method (Section 2.2.4.2). Lastly, the pressure ratios which produce the same convergences with the ones calculated by analytical methods (Table 4.4), were tried to obtain in numerical models.

It is a known fact that the horizontal stresses in a rock depend on the geological history and may be different than the vertical stresses. As a consequence K_o may be different than 1.0. To examine the influence of change in initial stress conditions, the obtained pressure ratio under uniform stress conditions has been used in the following models which are used to determine the convergences under non-uniform stress conditions.

The effects of the change in the initial stress conditions on the convergences which were occurred on the tunnel wall were examined. Two different non-uniform in-situ stress ratio were taken into consideration in the numerical analyses. First, the initial stress ratio is equal to $K_o=0.6$ which is used to simulate the site conditions for Chamoise Tunnel case. Later, an alternative value of $K_o=1.2$ is utilized to simulate the initial stress ratio in Turkey. This value is obtained by taking the average of the vertical and horizontal stresses determined by Acoustic Emission method for various sampling locations in Turkey by Ulusay *et al.*, (2003).

Table 5.4. Vertical Stress (σ_v), Horizontal Stress (σ_h), and Initial Stress Ratio Measured for Various Settlements in Turkey (Ulusay *et al.*, 2003).

Location	σ_v (MPa)	σ_h (MPa)	K_o
Eynez	4.43	1.93	2.30
Izmir	2.41	1.64	1.47
Cayirhan	4.22	4.1	1.03
Küre	4.64	3.39	1.37
Dodurga	4.9	4.26	1.15
Zonguldak	11.83	9.7	1.22
Istanbul	1.45	1.16	1.25
Ankara	0.37	0.21	1.76
Bayburt	0.87	0.65	1.34
Denizli	0.72	0.76	0.95
Kirsehir	0.87	0.9	0.97
Sivas	1.15	1.11	1.04
Bigadic	1.49	1.55	0.96
Kestelek	1.07	1.34	0.80
Eskisehir	0.33	0.34	0.97
Seydisehir-Dogankuzu	2.98	2.44	1.22
Seydisehir-Mortas	2.54	2.38	1.07
Ordu	5.53	4.58	1.21
Emet	1.07	1.06	1.01
Kayseri	0.59	0.6	0.98
Average	2.67	2.21	1.20

Finally, the influence of the change in initial stress conditions on the occurring convergences are plotted in Figure 5.4 for both deteriorated data sets. According to the histograms, the occurring convergences around the tunnel contour are increasing as the initial ratio of the horizontal stresses to the vertical stresses (K_o) increases.

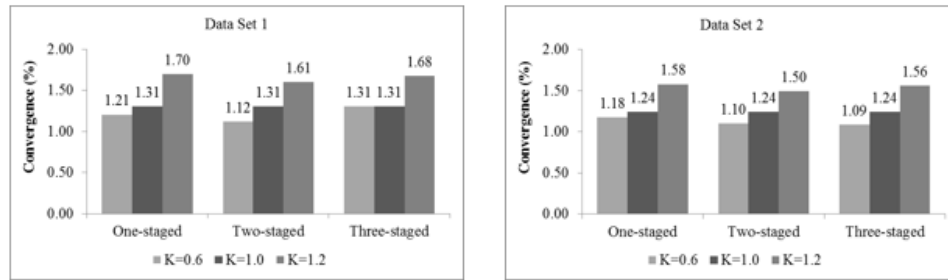


Figure 5.4. Determined Convergence Values of Deteriorated Data Sets for Different In-Situ Stress Conditions.

Figure 5.5 illustrates the occurring radial displacements between side walls. Although it represents an increase when the initial stress conditions are deviated from the uniform stress conditions ($K_o=1.0$), it is not possible to come to same conclusion as in the case of occurring convergences.

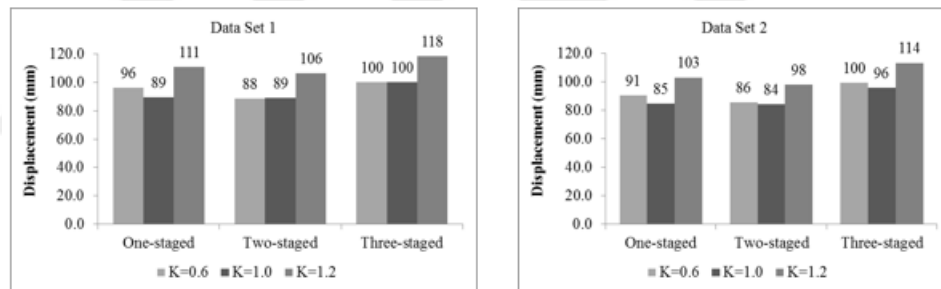


Figure 5.5. Determined Displacement between Side Walls of Deteriorated Data Sets for Different In-Situ Stress Conditions.

5.3.2. Influence of the number of excavation stages on the occurring radial displacements on the tunnel wall

To understand the effects of conducting the excavation in stages on the occurring radial displacements along the tunnel side wall, three different excavation types have been taken into account. In Chamoise Tunnel, driving of first tube (North tube) was carried out by three-staged mechanical excavation, but the second tube (South tube) was performed by full-face drill-and-blast excavation. Accordingly, three different models are used as a basis to illustrate the full faced and three-staged excavations using

finite element method. Additionally, two-staged excavation has been introduced into the models to be able to scan the transition.

The results of the model have been presented in Figure 5.6 to Figure 5.8 for different initial stress conditions.

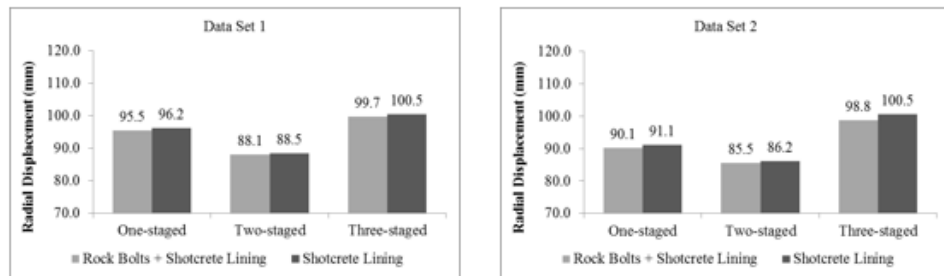


Figure 5.6. Change in Convergence Values for $K_o=0.6$ In-Situ Stress Conditions for Deteriorated Data Sets.

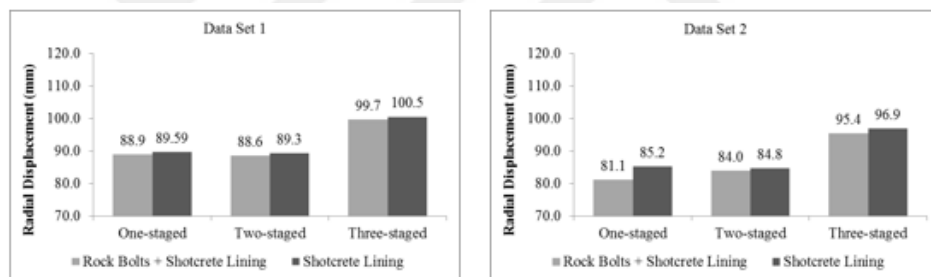


Figure 5.7. Change in Convergence Values for $K_o=1.0$ In-Situ Stress Conditions for Deteriorated Data Sets.

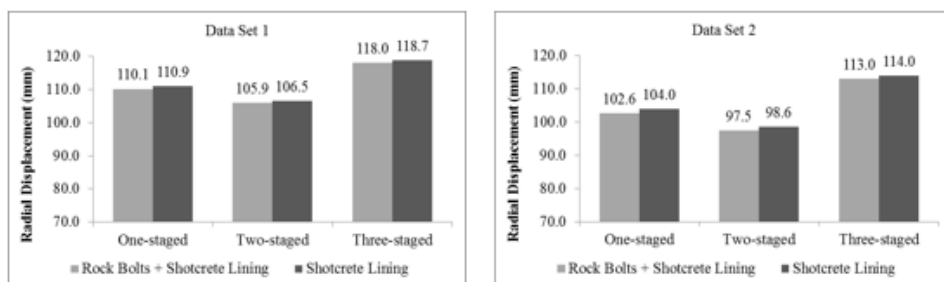


Figure 5.8. Change in Convergence Values for $K_o=1.2$ In-Situ Stress Conditions for Deteriorated Data Sets.

The calculated radial displacement values on the tunnel side wall are increased for the case in which the tunnel is excavated in three stages. This situation can be

evaluated as the loads that are applied to the supported part of the tunnel may be decreased if the tunnel is driven in three stages.

5.3.3. Efficiency of the supports used in tunnel excavatio

It can be concluded that rockbolts that are utilized as a member of combined support system in the tunnel construction do not show a significant effect on the deformations that are occurred on the tunnel wall for the studied case.

In case where the tunnel is not been supported, the radial displacement that has occurred along the tunnel wall has been calculated in the order of 150 mm and 130 mm for Data Set 1 and Data Set 2, respectively. If the same calculation procedure is also been followed for the supported tunnel, the calculated radial displacements are determined as illustrated in Figure 5.9. In case where only shotcrete is used as an individual support, radial displacements vary between 90 mm and 101 mm. However; in case where the tunnel is supported with a combined system including shotcrete lining and rockbolts, the radial displacements are changing between 89 mm and 99 mm for Data Set 1.

Similarly, for Data Set 2, In case where only shotcrete is used as an individual support, radial displacements varies between 85 mm and 97 mm, however in case where the tunnel is supported with a combined system including shotcrete lining and rockbolts, the radial displacements are changing between 84 mm and 95 mm.

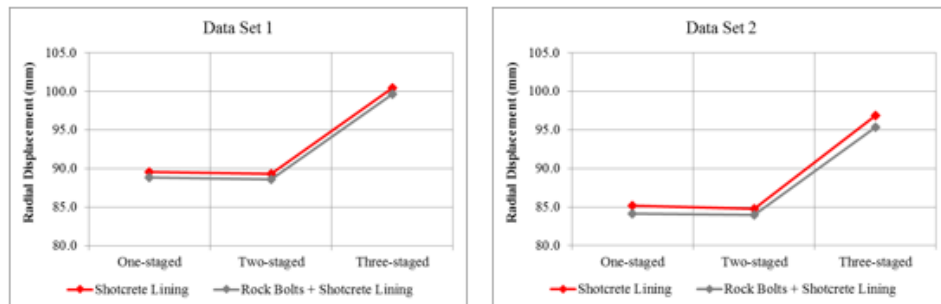


Figure 5.9. Radial Displacements along the Tunnel Wall after Support Installation in Uniform Initial Stress Conditions.

It is noticeable that the installed rock bolts does not produce a significant effect on the occurring radial displacements along the tunnel wall for this case study.

5.3.4. Influence of the Geometry of the Tunnel

The change in occurring convergences along the tunnel contour by the change in the tunnel geometry is examined. Figure 5.10 illustrates the model has been used in numerical analyses for a circular arc shaped tunnel excavation.

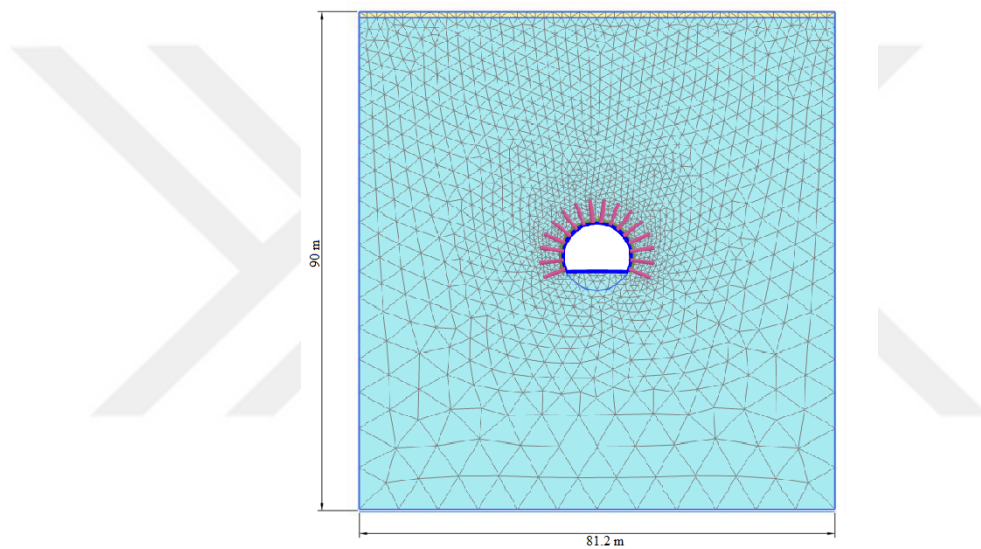


Figure 5.10. Geometrical Model and FEM Mesh for Circular Arc Shaped Tunnel Section.

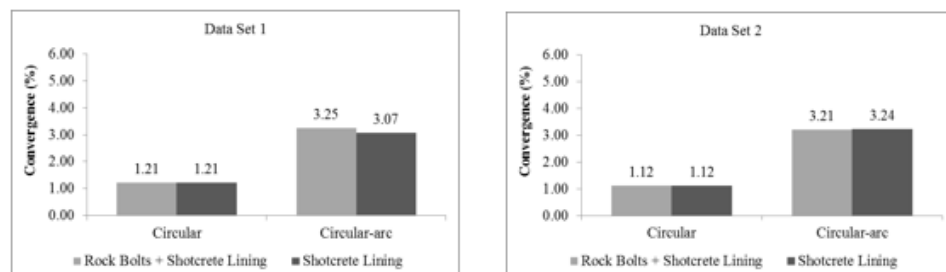


Figure 5.11. Comparison between the Occurring Convergences Circular and Circular Arc Shaped Tunnel for Data Sets ($K_o=0.6$).

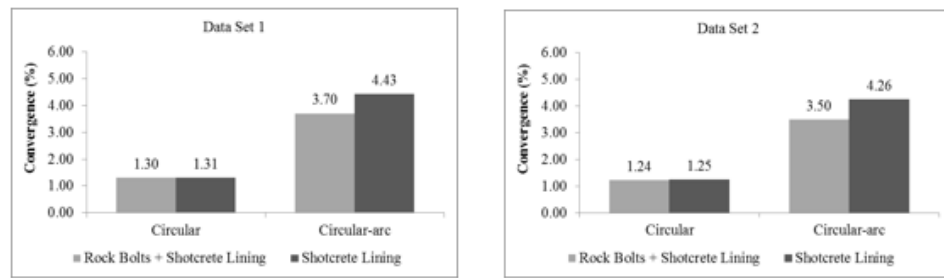


Figure 5.12. Comparison between the Occurring Convergences Circular and Circular Arc Shaped Tunnel for Data Sets ($K_o=1.0$).

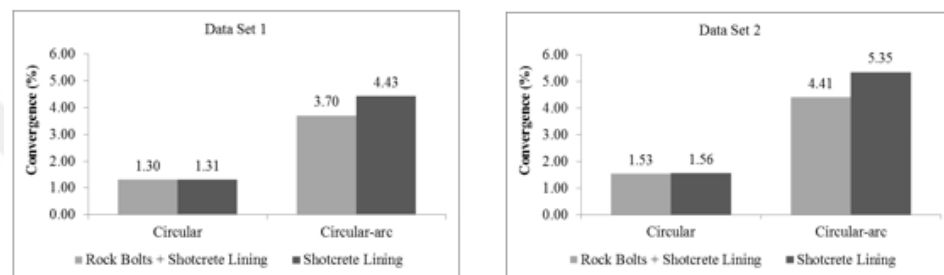


Figure 5.13. Comparison between the Occurring Convergences Circular and Circular Arc Shaped Tunnel for Data Sets ($K_o=1.2$).

As illustrated in Figure 5.11 to Figure 5.13, the convergences occurring on the tunnel contour show appreciable change between circular shaped and circular arc shaped tunnel excavation for different initial stress ratios. The convergence values increase by 1.8 - 5.0% if the tunnel excavation would performed for a circular arc shaped tunnel instead of a circular one.

This results shall be evaluated as the loads that are applied to the supported part of the tunnel may be decreased if the tunnel is driven in circular arc shape.

6. COMPARISON BETWEEN ANALYTICAL AND NUMERICAL METHOD USED FOR CCM APPLICATION

Following marks have been taken into account to be able to compare the results of the CCM applications using analytical method and the finite element method for both data sets.

- The tunnel excavation has been performed within one stage,
- The initial stress condition which is used in the application of both methods, is equal to $K_o=1.0$,
- The rock mass is assumed to behave as an elasto-plastic isotropic non-dilatant material.

The results are examined by taking into account the radial displacements and plastic radius.

6.1. Radial displacements along the tunnel wall

- Without Support

As explained in previous sections, several calculations have been made by both analytical and numerical method to determine the radial displacements along the tunnel wall. The displacements that are induced by the tunnel excavation are examined without using a support system for this case.

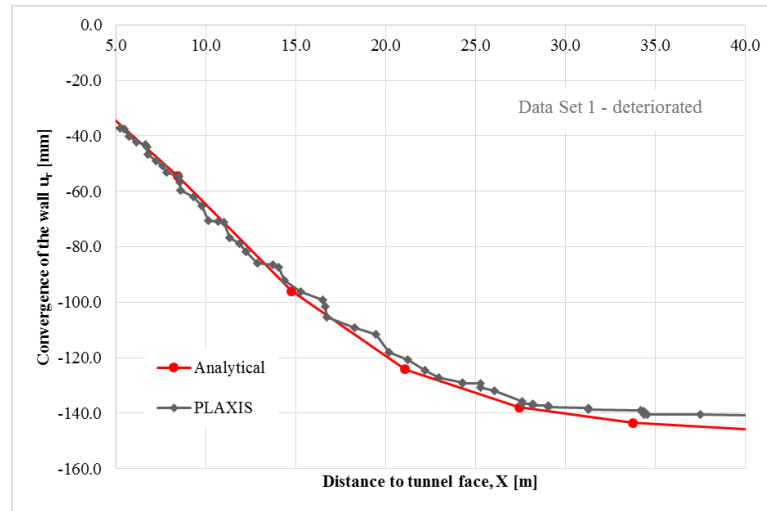


Figure 6.1. Distribution of Convergences on the Tunnel wall for Data Set 1.

The distribution of the convergences occurring on the tunnel wall for Data Set 1, is shown in Figure 6.1. The predicted radial displacements on the tunnel wall was computed as approximately 0.14 m. It is obvious that the numerical results are in good match with those obtained using the analytical approach for this case.

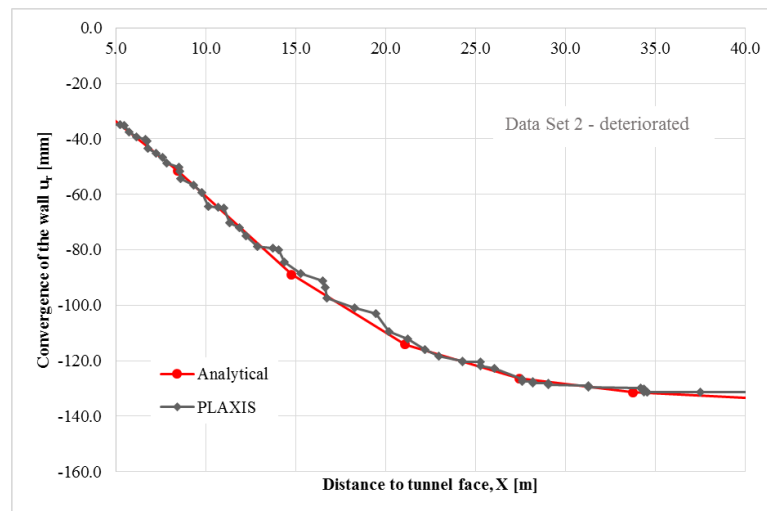


Figure 6.2. Distribution of Convergences on the Tunnel Wall for Data Set 2.

The distribution of the radial displacements occurring on the tunnel wall for Data Set 2, is shown in Figure 6.2. The predicted convergence on the tunnel wall was computed as approximately 0.13 m. It is obvious that the numerical results are in accordance with those obtained using the analytical approach for this case.

The similarity in the results obtained by numerical and analytical methods is appreciable for the case the tunnel is unsupported.

- With Support

The alternative for support systems that are used in the tunnel stabilization are considered as 300 mm of shotcrete lining as individual support and a combined support system consists of 300 mm of shotcrete lining with $L=4$ m, 25 mm of rockbolts.

Table 6.1 summarizes the radial displacements that are computed by analytical method and finite element method, separately. The radial displacement that are occurred along the tunnel wall are calculated in the order of 75 mm by analytical method, whereas the expected radial displacements are calculated in the order of 89 mm for Data Set 1.

Table 6.1. Comparison between the Radial Displacements Calculated by Numerical and Analytical Methods.

Data Set 1 (deteriorated)		
Support System	Radial Displacements (mm)	
	Analytical Method	FE Method
Shotcrete Lining	75.9	88.9
Rockbolts + Shotcrete Lining	75.1	88.6
Data Set 2 (deteriorated)		
Support System	Radial Displacements (mm)	
	Analytical Method	FE Method
Shotcrete Lining	72.2	85.2
Rockbolts + Shotcrete Lining	71.5	84.1

The radial displacement that are occurred along the tunnel wall are calculated in the order of 72 mm by analytical method, whereas the expected radial displacements are calculated in the order of 84 mm for Data Set 2.

At this point, it might be needed to repeat the procedure followed to determine the radial displacements. First, the convergences (%) that are determined by analytical approach (Table 4.4) were tried to obtain in numerical models to determine the radial displacements. This is provided by changing the ratio between loads transmitted to the supported and to the unsupported part of the tunnel (so-called β -method). As explained in Section 5.2, the convergence values in the numerical models are calculated by measuring the difference between deformed mesh diameter and original mesh diameter.

Although the analytically calculated radial displacements for the case where the tunnel is not supported, are in good match with the ones calculated by finite element method, it is not possible to make the same evaluation for the case where the supports are used.

The radial displacements that are found by numerical finite element calculations are 16-17% higher than the ones that are found by conventional analytical calculations. The reasons might be the existence of structural elements (supports) might have changed the uniform distribution of the stresses which produce displacements.

6.2. Plastic radius

If the internal support pressure p_i is less than the critical support pressure p_i^{cr} , failure occurs and the rock-mass shows plastic behaviour. A plastic zone with a radius R_{pl} is formed around the tunnel.

The plastic radius of the Chamoise Tunnel is examined by means of both semi-analytical method and the finite element method. Figure 6.3 represents the simulations for the plastic radius from PLAXIS and from Rocsupport, from left to right, respectively.

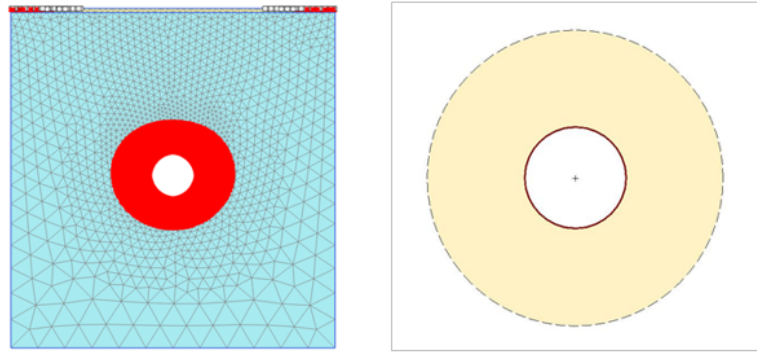


Figure 6.3. The Simulations of the Plastic Radius Measurement from FEM Program PLAXIS and CCM Program Roc Support (off-scale).

The values of the plastic radius which are determined by means of numerical method do not show a good match with the ones which are determined by analytical method.

Table 6.2. Computed Values of Plastic Radius for both Numerical and Analytical Methods.

Method	Support Type	R_{pl} (m), Data Set 1	R_{pl} (m), Data Set 2
Numerical	With Support	≈ 6	≈ 5
	Without Support	≈ 9.5	≈ 8.5
Analytical	With Support	11.6	11.6
	Without Support	16.84	16.84

The radius of the plastic zone is decreasing by introducing supports into the underground opening. Therefore, it is possible to conclude that as the internal pressure value closes to the critical support pressure value, the radius of the plastic zone decreases.

On the other hand, the plastic radius that has formed due to the excavation of the tunnel, can be easily determined using semi-analytical approach. However, as the number of stages increases in FEM, the accuracy of determining the plastic radius decreases.

6.3. Efficiency of the support systems used

As shown from the listed radial displacements from Table 6.1, the radial displacements are only decreased by one per cent with the usage of rock bolts as a member combined support system. Therefore, it may be stated that the installation of the rockbolts do not show a significant effect on the occurring convergences for this case study. This is confirmed by both numerical and analytical approach for this case study based on the calculated results.



7. CONCLUSIONS AND RECOMMENDATIONS

7.1. Conclusions

The conclusions drawn for the support system design using convergence confinement method regarding the analytical and numerical simulations performed based on the synthetic data derived from the case study of Chamoise Tunnel are given in the following subsections.

7.1.1. On the comparison between semi-analytical approach and finite element approach

It should be noted that it is not possible to use analytical approach for tunnel design under non-uniform stress conditions. Therefore, the comparison was made between the analytical approach and finite element approach, relies on some assumptions.

- The tunnel excavation has been performed within one stage,
- The initial stress condition which is used in the application of both methods, is equal to $K_o=1.0$,
- The rock mass is assumed to behave as an elasto-plastic isotropic non-dilatant material.

The radial displacements that are occurred on the tunnel wall calculated by the numerical approach are in good match with those obtained using the analytical approach for the case where tunnel is unsupported.

The radial displacements that are found by numerical finite element calculations are 16-17% higher than the ones that are found by conventional analytical calculations for the case where the supports are installed.

The plastic radius that has formed due to the excavation of the tunnel, can easily can be determined using semi-analytical approach. However, as the number of stages increases in finite element method, the accuracy of determining the plastic radius decreases.

7.1.2. On the efficiency of the used support systems

Two different approaches (numerical and analytical) have been used in the analyses that are performed for the application of convergence confinement method. The calculated radial displacements from both method are recorded. Lastly, the efficiency of the selected support systems are examined by comparing the displacements before and after excavation.

According to the results obtained by analytical and numerical calculations, shotcrete lining decreases the amount of occurring radial displacements by the percentage varies 40 to 60 as an individual support. However, a combined support system consisting of shotcrete lining and rockbolts have the same effect on the occurring radial displacements along the tunnel wall.

It may be concluded that the installed rockbolts do not produce a significant effect on the occurring radial displacements along the tunnel wall for this case study.

7.1.3. On the limitations of the research

This research has been done to provide a better understanding of the convergence confinement method and its applications to the deep tunnels by using a comparative case study between analytical approach and numerical approach.

On one hand, analytical approaches are available to determine the deformations and stresses around the tunnel, however their applicability is limited with tunnel conditions. On the other hand, numerical approaches allow users to examine the tunnel problems in detail, but the interpretation of the results is difficult due to the lack in-situ

measurements to compare the results.

7.1.4. On the number of excavation stages

Three different models are used as a basis to illustrate the full faced and three-staged excavations using finite element method. Additionally, two-staged excavation has been introduced into the models to be able to scan the transition.

According to the results of the analyses, occurring convergence values along the tunnel centreline are not affected significantly from the number of excavation stages. However, the calculated radial displacement values on the tunnel side wall are increased for the case in which the tunnel is excavated in three stages. This situation can be evaluated as the loads that are applied to the supported part of the tunnel may be decreased if the tunnel is driven in three stages.

7.1.5. On the geometry of the tunnel

An alternative shape of circular-arc has been introduced into the models in order to maintain the applicability of the convergence confinement method based on the assumption of the circular tunnel.

The change in the radial convergences that are calculated for the circular arc shaped tunnel are compared with the full-circular shaped tunnel for different initial stress conditions.

The convergences would have been increased by 1.8 - 5.0% if the tunnel excavation would performed for a circular arc shaped tunnel instead of a circular one.

Lastly, it may be concluded that the radial convergences occurring on the tunnel contour are expected to be higher if the shape of tunnel excavation deviates from the circular excavation apart from the initial stress conditions on the site.

7.2. Recommendations for further research

This research provides a better understanding of the semi-analytical convergence confinement method application.

Although the results of this research show that the results that are obtained from analytical approach are in good match with those obtained by numerical approach, some missing parts should be indicated for further future research.

- The grouting material type that is used for the support installation and the interface between the rockbolts and the surrounding rock mass, are not considered in this study. The grouting material may affect the behaviour of the supports if the research deepens. Further research in this direction can be encouraged.
- Although the development of plastic zone by analytical approach is easily determined, it is not fully covered for the numerical application of the convergence confinement method (so-called β -method) as the number of stages increases in finite element method, the accuracy of determining the plastic radius decreases for the case study.
- A good match has been detected between the numerical and analytical approaches for the excavation induced radial displacements for the case where tunnel is unsupported. However, it is not possible to conclude the same for the radial displacements developed in the case where the tunnel is supported. A further research may be appreciated to comprehend the behaviour of the supported rock-mass.

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APPENDIX A: SPREADSHEET FOR THE SEMI-ANALYTICAL CCM CALCULATIONS

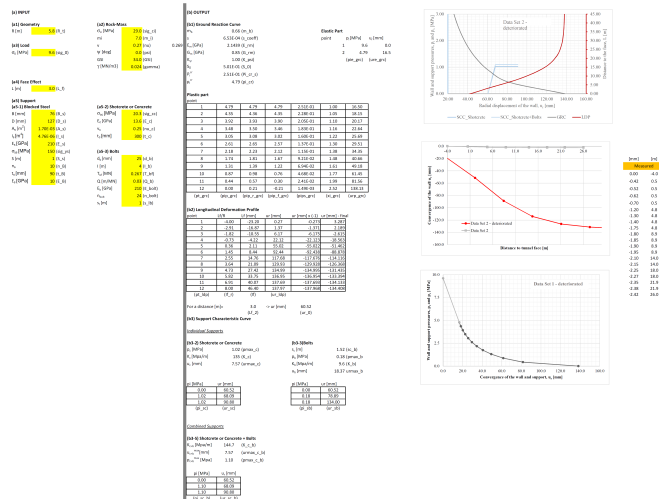


Figure A.1. Data Set 1.

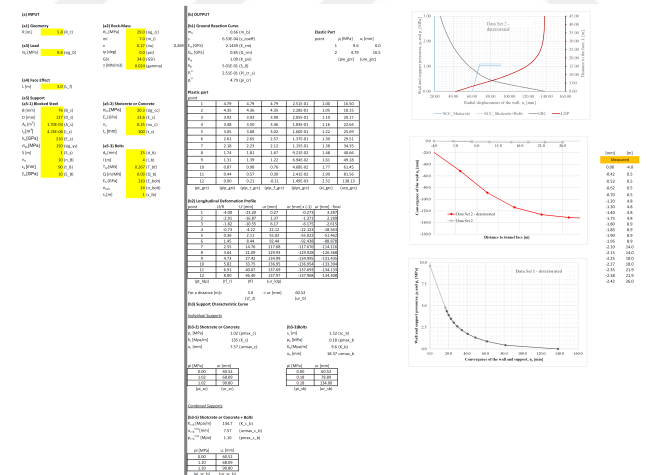


Figure A.2. Data Set 2.

APPENDIX B: RESULTS OF THE NUMERICAL ANALYSES

B.1. Displacements for Data Set 1

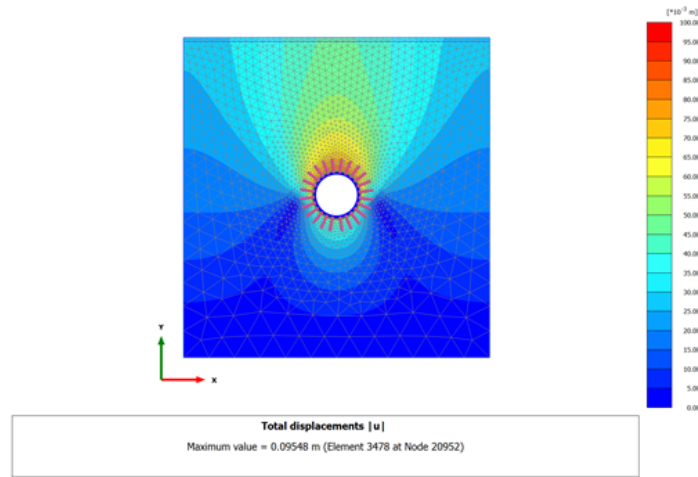


Figure B.1. One-Staged - $K_o=0.6$ - Rockbolts and Shotcrete.

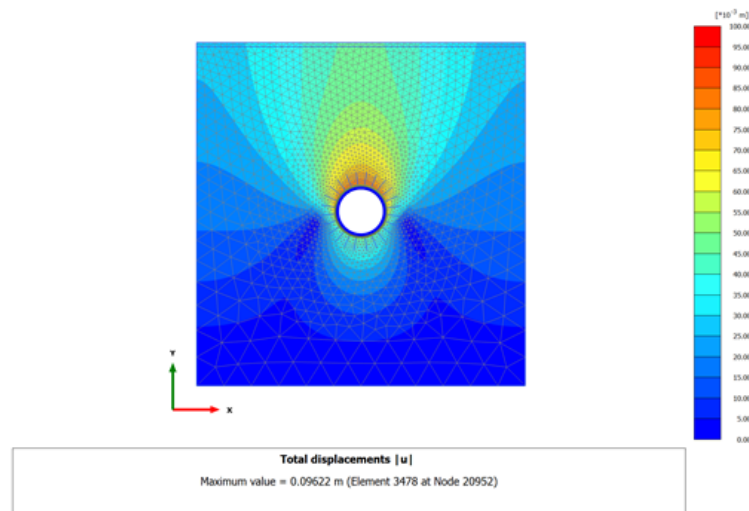


Figure B.2. One-Staged - $K_o=0.6$ - Shotcrete.

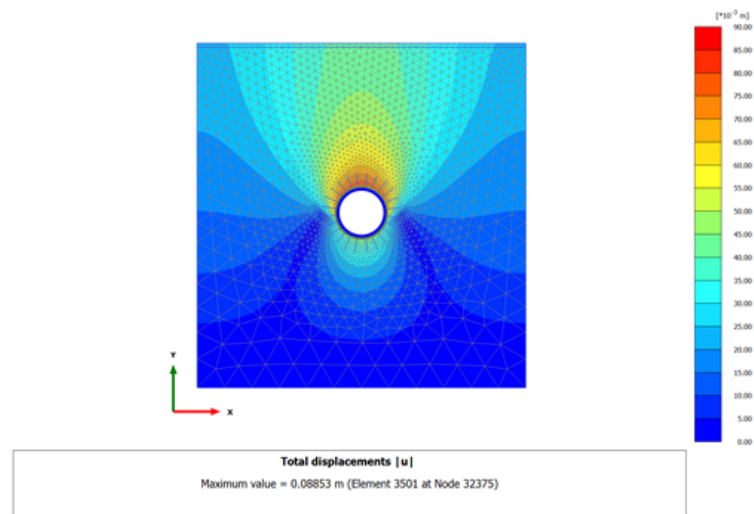


Figure B.3. Two-Staged - $K_o=0.6$ - Rockbolts and Shotcrete.

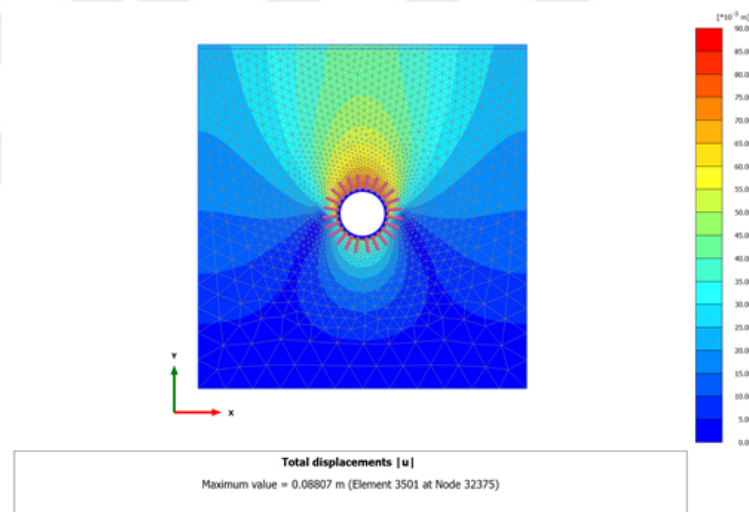


Figure B.4. Two-Staged - $K_o=0.6$ - Shotcrete.

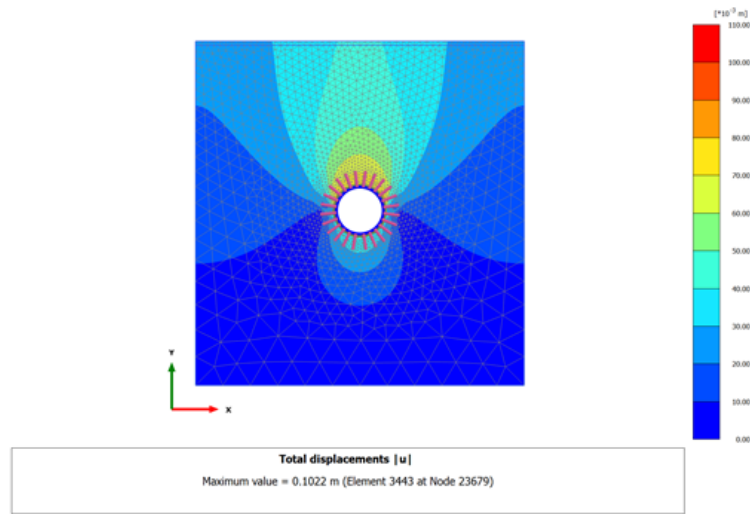


Figure B.5. Three-Staged - $K_o=0.6$ - Rockbolts and Shotcrete.

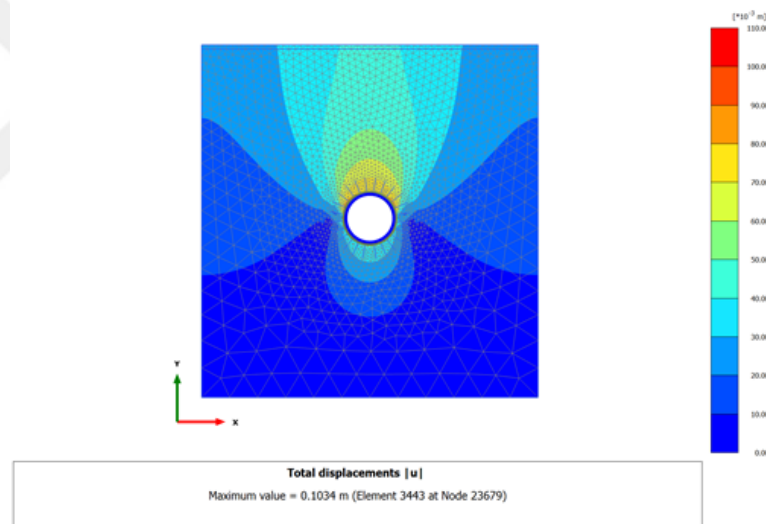


Figure B.6. Three-Staged - $K_o=0.6$ - Shotcrete.

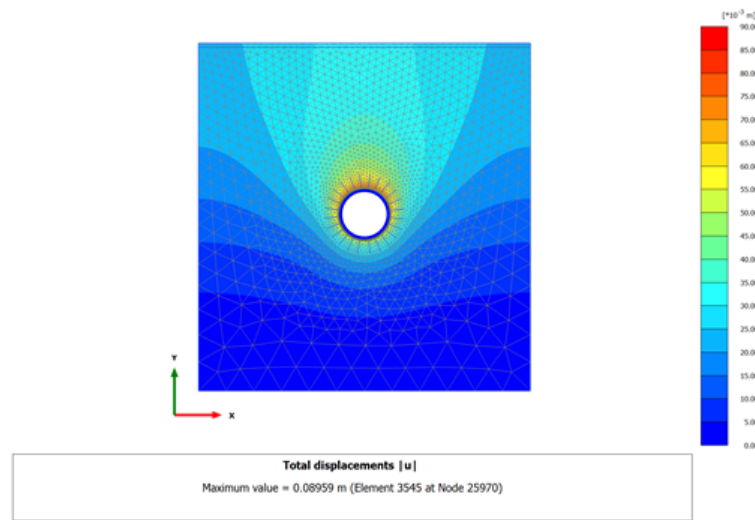


Figure B.7. One-Staged - $K_o=1.0$ - Rockbolts and Shotcrete.

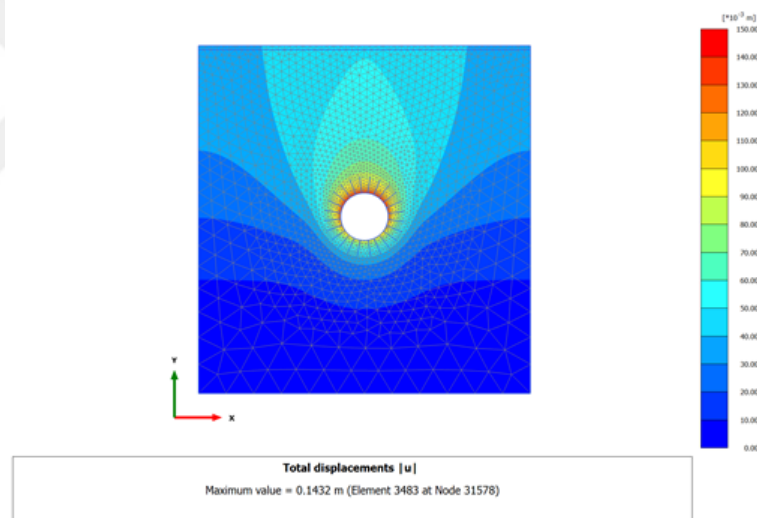


Figure B.8. One-Staged - $K_o=1.0$ - Shotcrete.

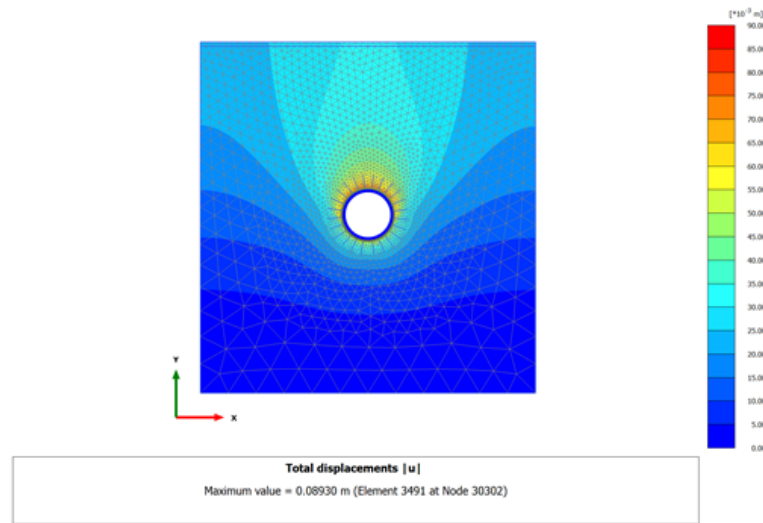


Figure B.9. Two-Staged - $K_o=1.0$ - Rockbolts and Shotcrete.

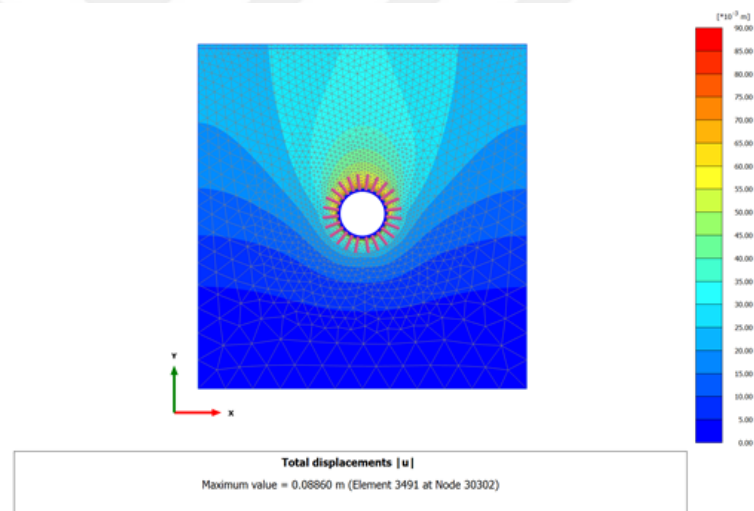


Figure B.10. Two-Staged - $K_o=1.0$ - Shotcrete.

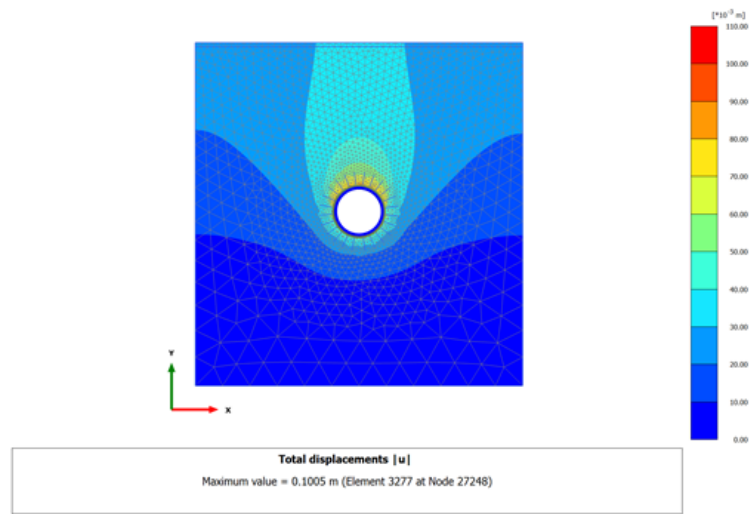


Figure B.11. Three-Staged - $K_o=1.0$ - Rockbolts and Shotcrete.

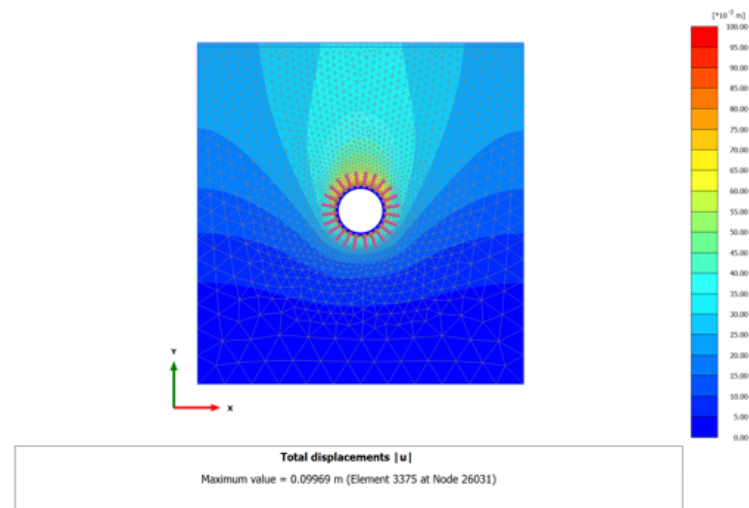


Figure B.12. Three-Staged - $K_o=1.0$ - Shotcrete.

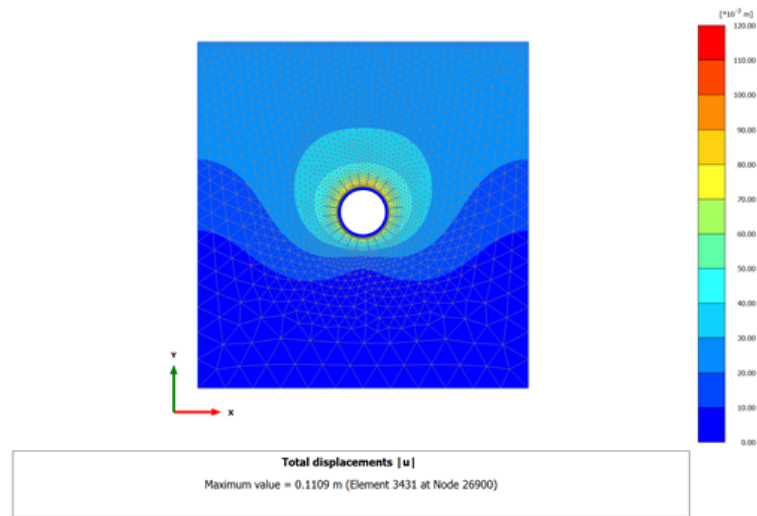


Figure B.13. One-Staged - $K_o=1.2$ - Rockbolts and Shotcrete.

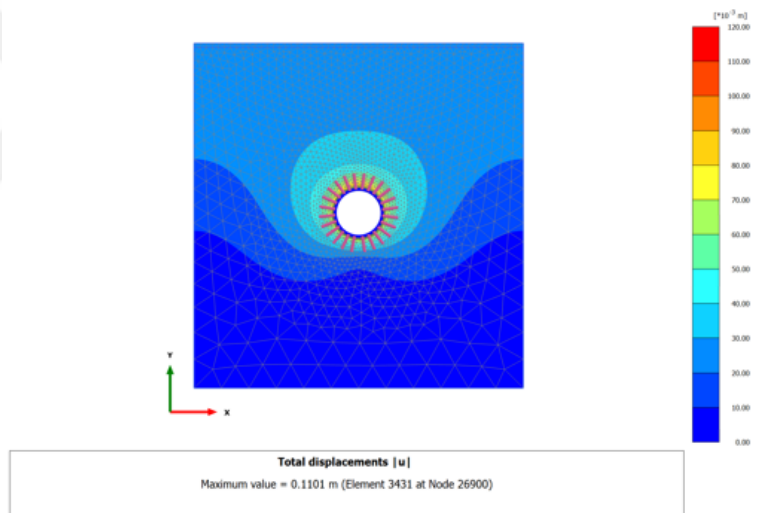


Figure B.14. One-Staged - $K_o=1.2$ - Shotcrete.

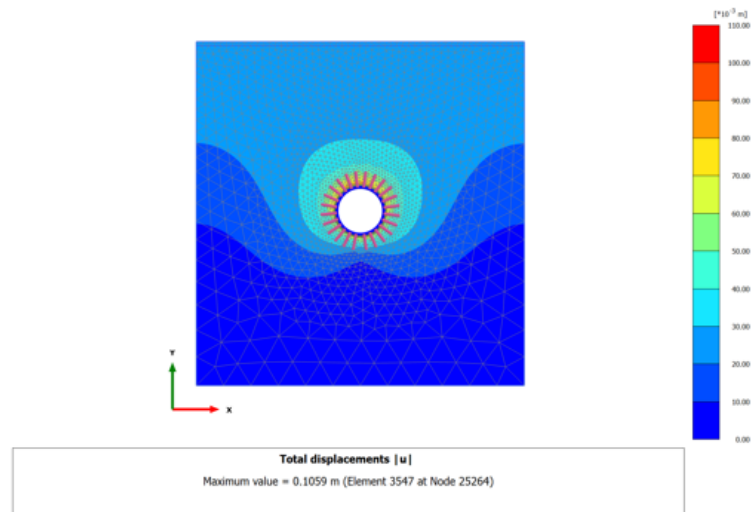


Figure B.15. Two-Staged - $K_o=1.2$ - Rockbolts and Shotcrete.

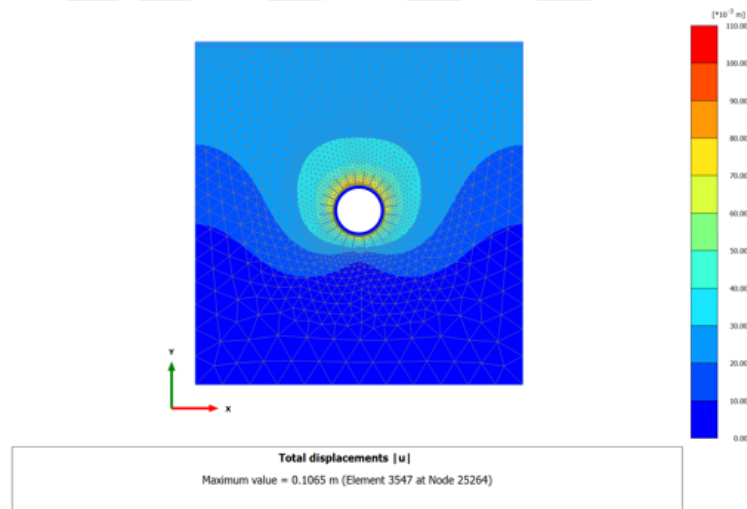


Figure B.16. Two-Staged - $K_o=1.2$ - Shotcrete.

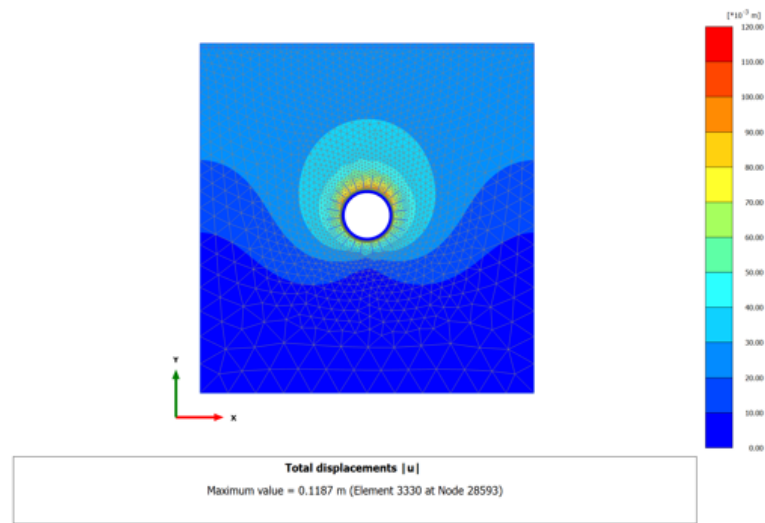


Figure B.17. Three-Staged - $K_o=1.2$ - Rockbolts and Shotcrete.

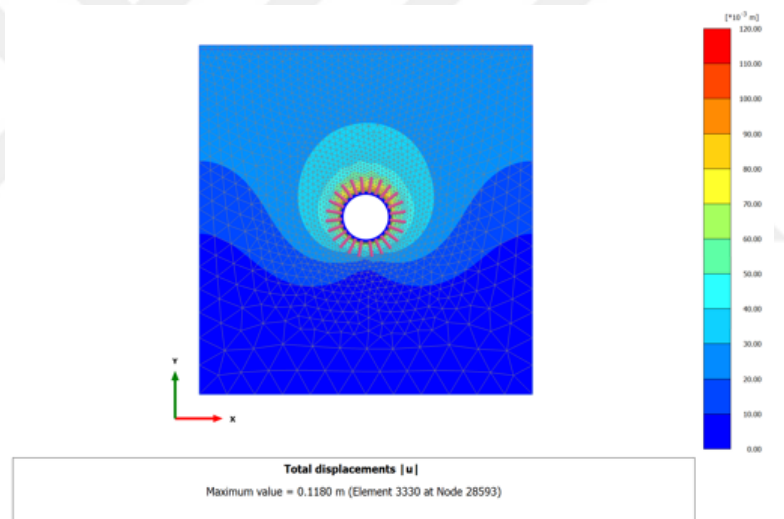


Figure B.18. Three-Staged - $K_o=1.2$ - Shotcrete.

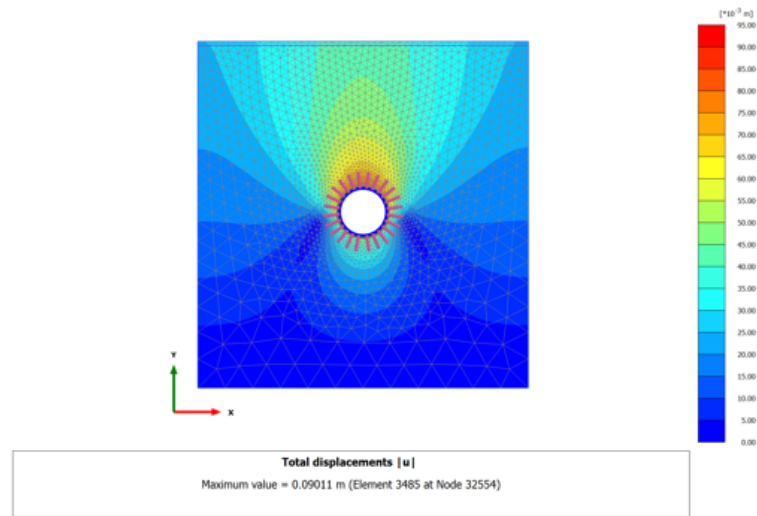


Figure B.19. One-Staged - $K_o=0.6$ - Rockbolts and Shotcrete.

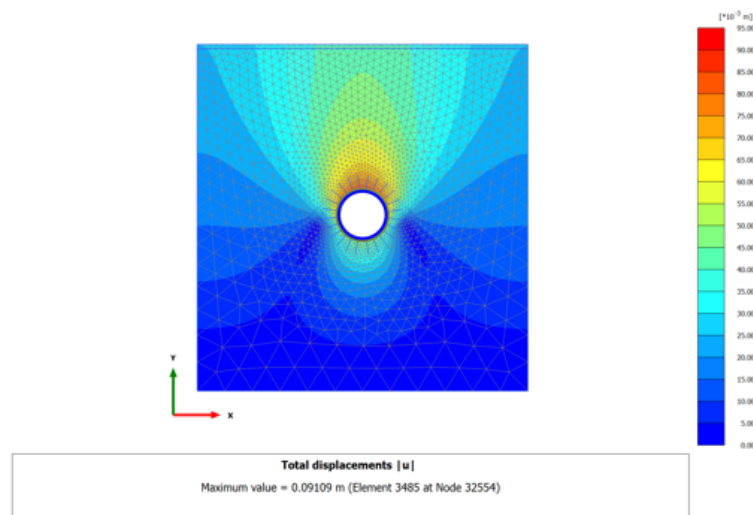


Figure B.20. One-Staged - $K_o=0.6$ - Shotcrete.

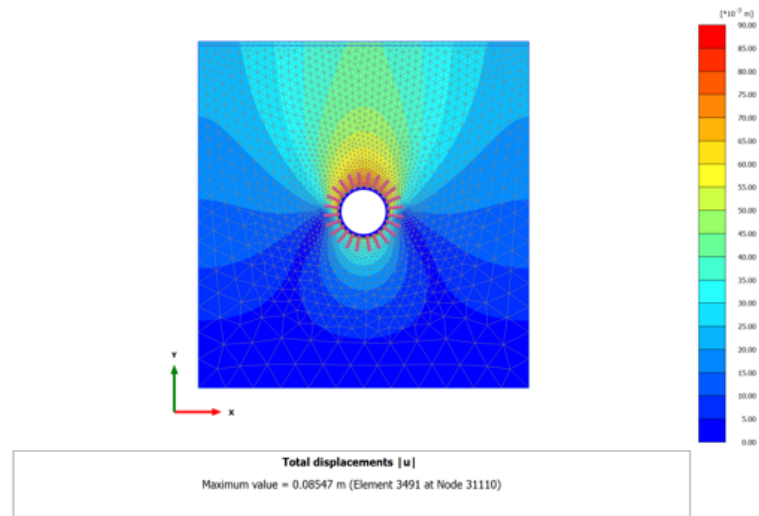


Figure B.21. Two-Staged - $K_o=0.6$ - Rockbolts and Shotcrete.

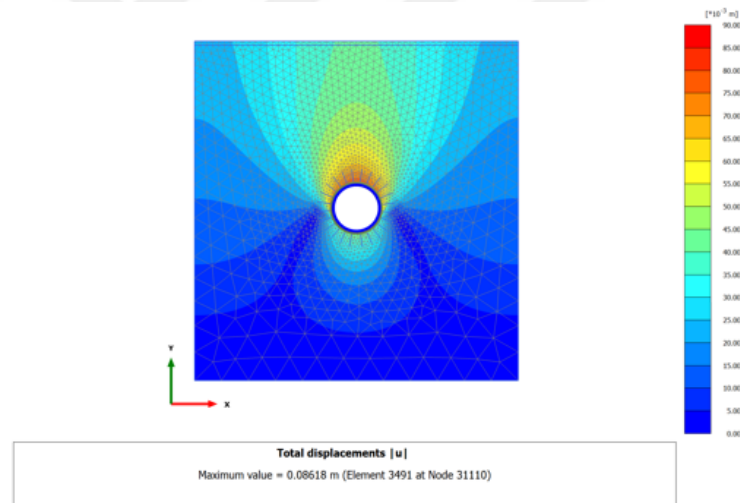


Figure B.22. Two-Staged - $K_o=0.6$ - Shotcrete.

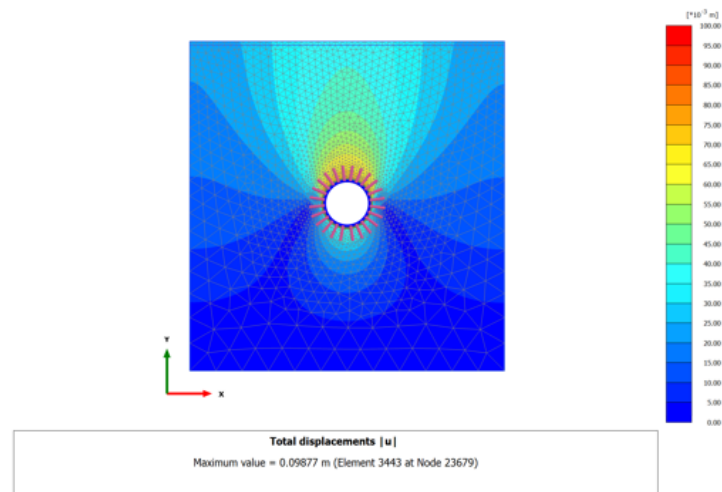


Figure B.23. Three-Staged - $K_o=0.6$ - Rockbolts and Shotcrete.

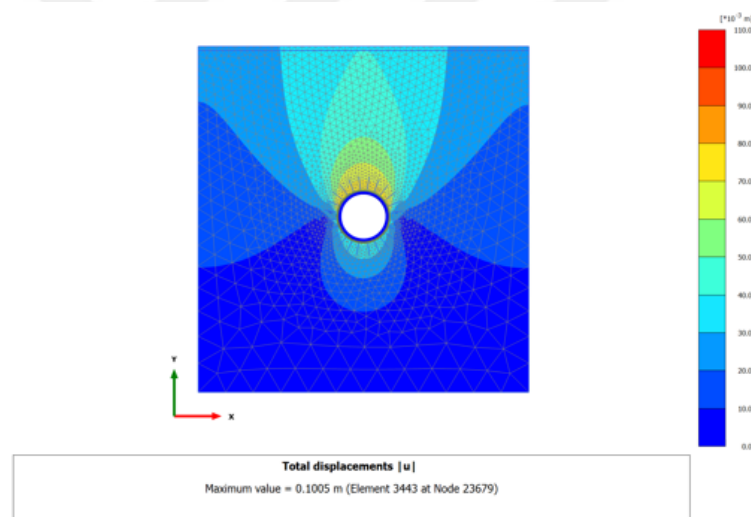


Figure B.24. Three-Staged - $K_o=0.6$ - Shotcrete.

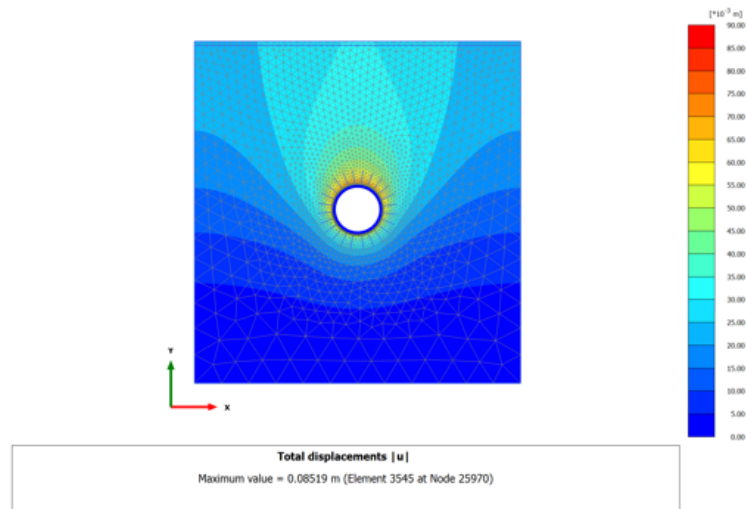


Figure B.25. One-Staged - $K_o=1.0$ - Rockbolts and Shotcrete.

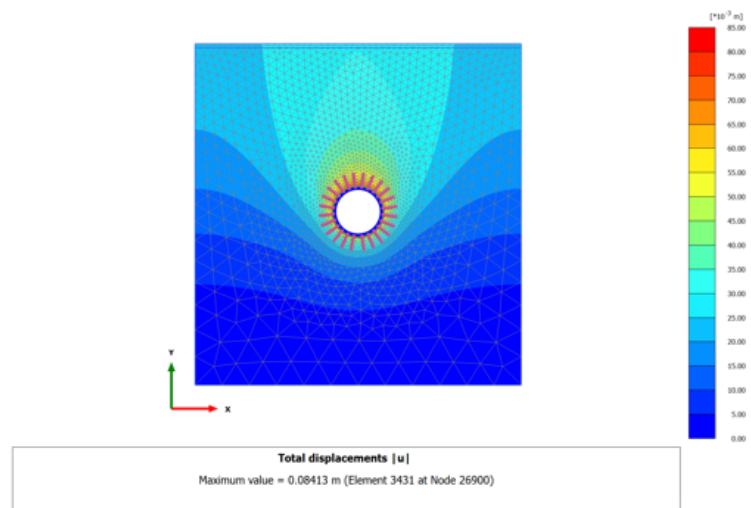
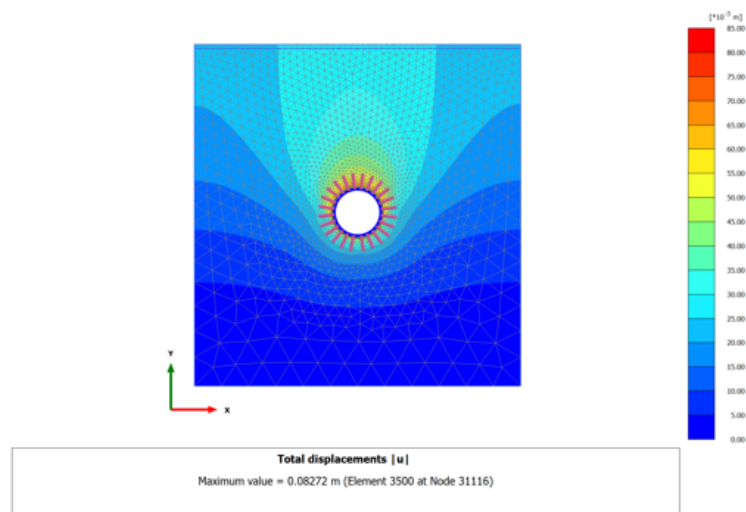
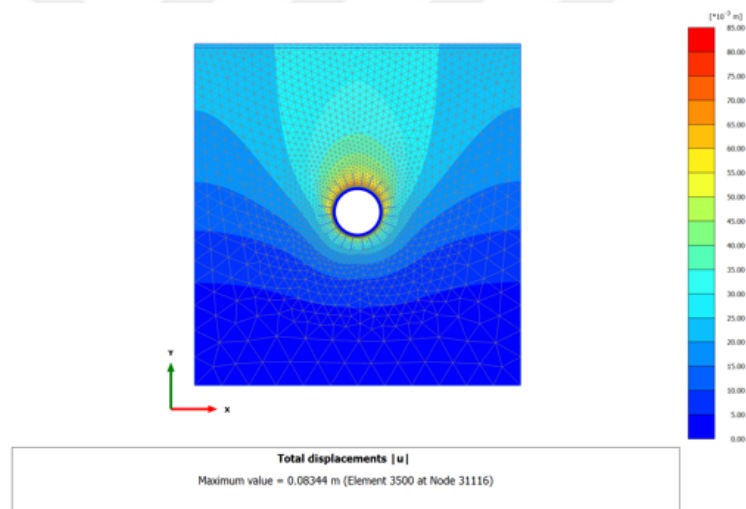
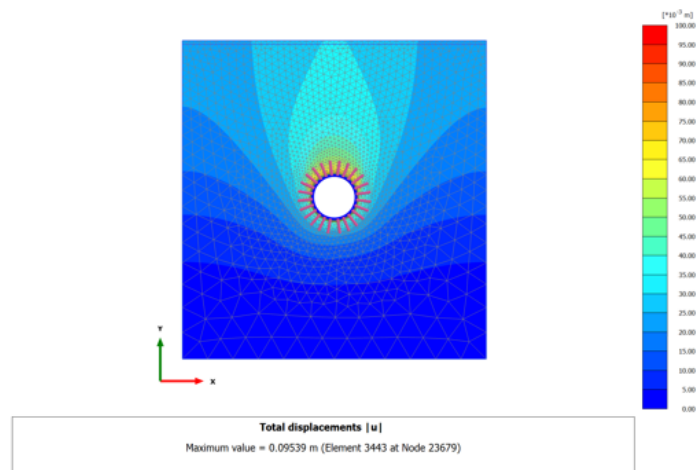
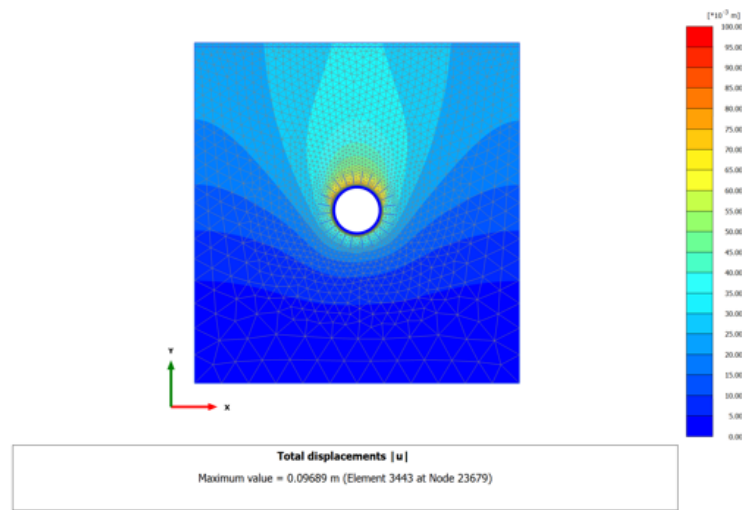
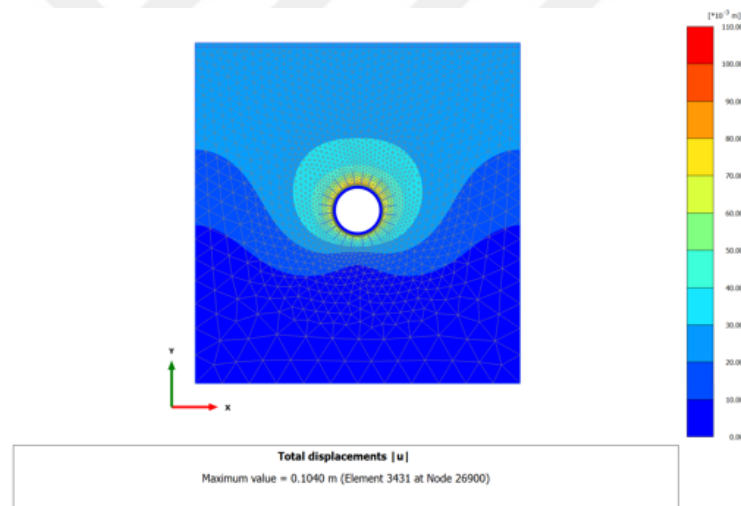


Figure B.26. One-Staged - $K_o=1.0$ - Shotcrete.

Figure B.27. Two-Staged - $K_o=1.0$ - Rockbolts and Shotcrete.Figure B.28. Two-Staged - $K_o=1.0$ - Shotcrete.Figure B.29. Three-Staged - $K_o=1.0$ - Rockbolts and Shotcrete.

Figure B.30. Three-Staged - $K_o=1.0$ - Shotcrete.Figure B.31. One-Staged - $K_o=1.2$ - Rockbolts and Shotcrete.

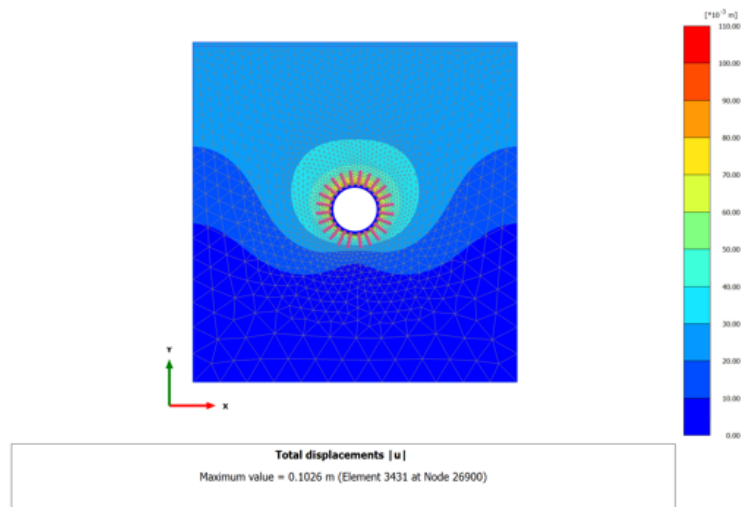


Figure B.32. One-Staged - $K_o=1.2$ - Shotcrete.

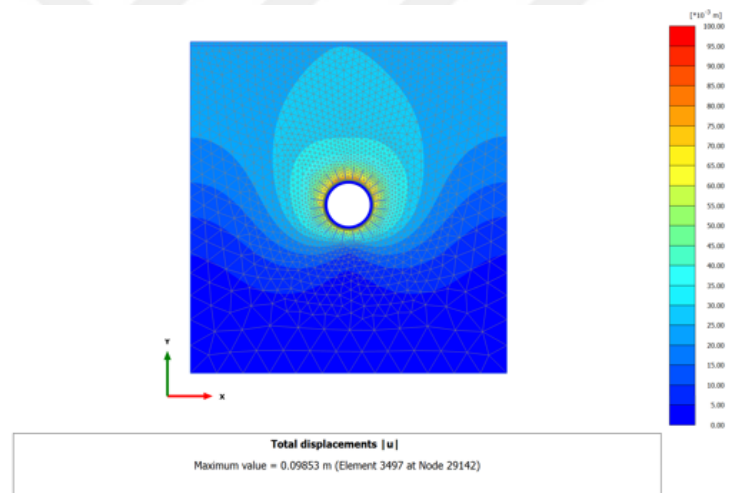
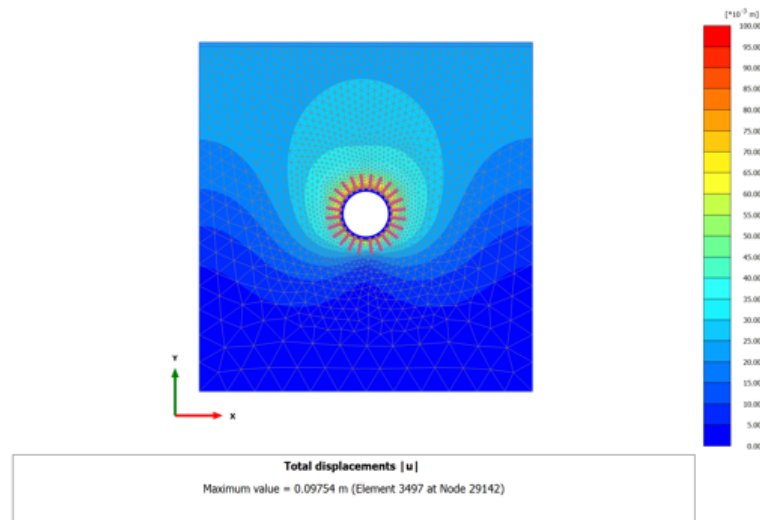
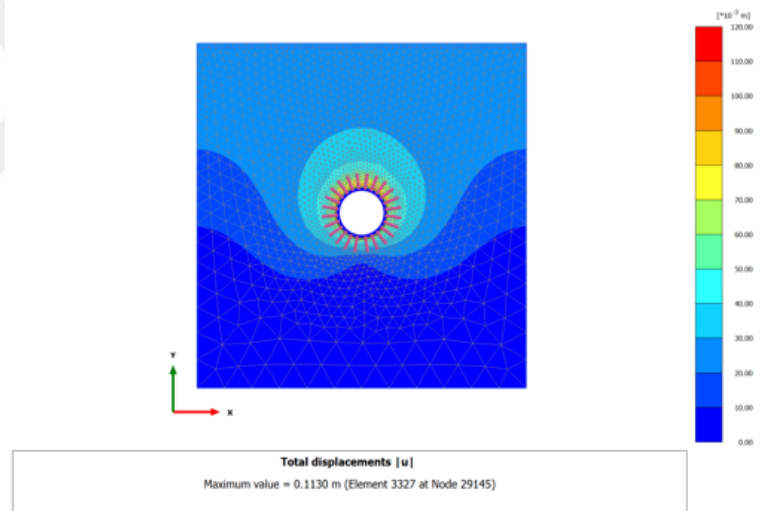


Figure B.33. Two-Staged - $K_o=1.2$ - Rockbolts and Shotcrete.

Figure B.34. Two-Staged - $K_o=1.2$ - Shotcrete.Figure B.35. Three-Staged - $K_o=1.2$ - Rockbolts and Shotcrete.

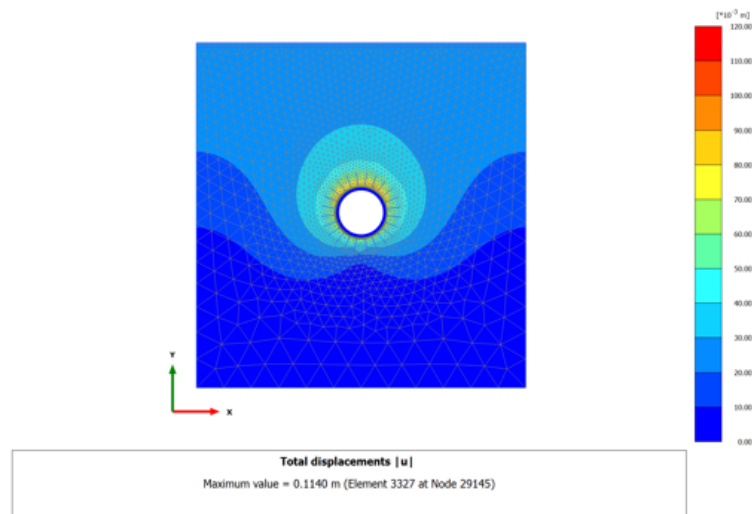


Figure B.36. Three-Staged - $K_o=1.2$ - Shotcrete.