Rana Muhammad Sajid Imran Khan
Kabindra Man Shrestha

Master of Science Thesis 11/02
Division of Soil and Rock Mechanics
Department of Civil and Architectural Engineering
Royal Institute of Technology (KTH)
SE-100 44 Stockholm
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Stability Analysis of Shallow Tunnel of Norra Länken

Rana Muhammad Sajid Imran Khan
Kabindra Man Shrestha

Graduate Students
Infrastructure Engineering
Division of Soil and Rock Mechanics
School of Architecture and the Built Environment
Royal Institute of Technology (KTH)
SE- 100 44 Stockholm

rmsikhan@kth.se
kmsh@kth.se

Abstract: Shallow Tunneling through highly populated areas of big cities is a challenging task. Almost in all the tunneling projects of shallow tunnel some or the other types of tunneling problems have been encountered and are still being faced. The stability of shallow tunnel is also influenced by many factors, primarily the in situ stress, geological structures, groundwater, rock mass quality, shape of tunnel etc. The design of shallow tunnel in past was almost purely a matter of experience. During last decades computational methods have been introduced as powerful design aids tool to arrive at safe and economical shallow tunnel structure. The purpose of this thesis work is to provide technical criteria and guidance for the design, and stability of Norra Länken shallow tunnel in rock for civil works projects. The design of shallow tunnels in highly dense areas is an iterative process. A good starting point is essential to the process and facilitates safe and economic design. Currently there are many practical two and three-dimensional software tools available for carrying out the task. This master thesis provides an overview of a methodology being used by tunneling experts, which captures the three-dimensional essentials of tunnel behaviour with two-dimensional analysis tools, PLAXIS. Though it is not a full and final situation and conclusion, but there is a lot to learn from such conditions.

KEY WORDS: FEM Calculation, Shallow Tunnel, PLAXIS, Norra Länken
Preface

The work resulting in this master’s thesis has been carried out at the Division of Soil and Rock Mechanics, Royal Institute of Technology (KTH), Stockholm Sweden in duration of Feb 2010 to Feb 2011. Thesis work is done on Norra Länken in Stockholm. All data was provided by TRAFIKVERKET. Numerical Analysis is done at KTH and technical support was provided by SWECO.

Acknowledgements

First of all we would like to say our deep thanks to Professor Håkan Stille, Head of Soil and Rock Mechanics Division Royal Institute of Technology for his support and encouragement as well as performing tiresome proof-readings. The efforts rendered by our project advisor were very vital in the completion of this project. His guidance and encouragement played a key role in the planning and completion of this project. We are proud to do work with Håkan Stille, as he is reputable scientist and expert in rock mechanics and tunneling.

We would like to thank Dr. Thomas Dalmalm for giving this opportunity to perform this analysis works with TRAFIKVERKET and co supervising our work. We are grateful for his support and valuable suggestions how to improve our work and for providing all necessary data and information. We could not forget his guidelines during site visits and performing Numerical analysis.

Special thanks go to Nancy Bono form SWECCO for her continuous guidelines for numerical modeling in PLAXIS. We appreciate her enthusiastic approach towards this work. Without her help we were not able to get better understanding of numerical modeling.

Finally we want express our gratitude to our parents, siblings and friends for their continuous inspirations.

Stockholm, February 2011
List of symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma )</td>
<td>Rock density in t/m(^3)</td>
</tr>
<tr>
<td>( Q_c )</td>
<td>Normalized rock mass quality rating</td>
</tr>
<tr>
<td>( Q )</td>
<td>Rock mass quality rating</td>
</tr>
<tr>
<td>( E_{\text{ma}} )</td>
<td>Rock mass deformation modulus</td>
</tr>
<tr>
<td>( \rho )</td>
<td>Density of rock mass kg/m(^3)</td>
</tr>
<tr>
<td>( g )</td>
<td>Gravity acceleration m/s(^2)</td>
</tr>
<tr>
<td>( h )</td>
<td>Depth below the ground surface (m)</td>
</tr>
<tr>
<td>( \nu )</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>( \sigma_{\text{h}} )</td>
<td>Horizontal stress</td>
</tr>
<tr>
<td>( \sigma_{\text{v}} )</td>
<td>Vertical stress</td>
</tr>
<tr>
<td>( Q_w )</td>
<td>Inflow or leakage rate in m(^3)/s</td>
</tr>
<tr>
<td>( K )</td>
<td>Specific permeability in m(^2)</td>
</tr>
<tr>
<td>( \mu_w )</td>
<td>Dynamic viscosity of water</td>
</tr>
<tr>
<td>( K_w )</td>
<td>Hydraulic conductivity</td>
</tr>
<tr>
<td>( q )</td>
<td>Specific leakage</td>
</tr>
<tr>
<td>( K_{\text{grout}} )</td>
<td>Conductivity of the grouted zone</td>
</tr>
<tr>
<td>RQD</td>
<td>Rock mass designation</td>
</tr>
<tr>
<td>( J_n )</td>
<td>Joint set number</td>
</tr>
<tr>
<td>( J_t )</td>
<td>Joint roughness number</td>
</tr>
<tr>
<td>( J_a )</td>
<td>Joint alteration number</td>
</tr>
<tr>
<td>( J_w )</td>
<td>Joint water reduction factor</td>
</tr>
<tr>
<td>SRF</td>
<td>Stress reduction factor</td>
</tr>
<tr>
<td>RMR</td>
<td>Rock mass rating</td>
</tr>
<tr>
<td>L</td>
<td>length</td>
</tr>
<tr>
<td>D</td>
<td>Tunnel diameter</td>
</tr>
<tr>
<td>( \varepsilon )</td>
<td>Skin factor</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>--------------------------------------------------</td>
</tr>
<tr>
<td>$t$</td>
<td>Thickness</td>
</tr>
<tr>
<td>$\sigma_i$</td>
<td>Uniaxial compressive strength of intact rock</td>
</tr>
<tr>
<td>$V_b$</td>
<td>Block volume in cubic meter</td>
</tr>
<tr>
<td>$j_c$</td>
<td>Joint condition factor</td>
</tr>
</tbody>
</table>
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Chapter 1

Introduction

1.1 Shallow Tunnels

Multiple use of land in spatial planning is in high demand. The tunnels are the underground passages for vehicles, trains and water, but tunnels could also be used for storage, they may have one end open or both. Every passing day gives new ideas to convert road, motorways, water channels and high voltage cables with underground passage with buildings and houses on top. On the other hand issues like noise control, air pollution and external safety risks (the risk of a person close to the road) and internal safety risks (the risk to the road user) lead to give more attention towards underpasses especially in urban and highly populated areas. In urban areas of SWEDEN there are lot of tunneling projects like Norra Länken in Stockholm, Citytunneln in Malmö and Götatunneln in Göteborg.

Shallow tunneling projects always have some uniqueness regarding engineering problem. Stability and behavior of shallow seated tunnel depends on many factors. Some of them are difficult stress conditions, rock break due to low overburden, geological structures, water ingress, and high safety factor regarding urban areas. Mechanical properties and state of stress play an important role in tunnel design and stability; in addition these are not easy to find out. More accurate investigation and conceptual study require establishing better understanding about these engineering problems in shallow tunneling.

Tunneling is directly related to the geology or the rock formations. Geology of the area greatly helps in the selection of location and certainly the tunnel layout depends upon the geologic information. The geology controls the design and construction methods, and plays the main role in the tunneling techniques. It has been analyzed in detail that the 3-10% of the total expenditure for project cost is to be catered for and should not be less than this.

In design and stability, stress plays a vital role that is fundamental to rock mechanics, principles and applications. There is high demand to understand the pre-existing state of stress in ground while applying to analysis and design. There can be a dramatic change in stress during excavation due to load re-distribution. Moreover it’s a tensor quantity and tensors are not encountered in everyday life. In case of shallow tunnel if overburden is highly weathered then even low absolute stress may cause failure and instability.

The settlement and subsidence is another big issue in underground excavation at shallow depth. It can be affected by overlying building and surface loads that may increase this settlement. Better understanding about these loads along with appropriate optimum dimension of the underground opening leads to engineering solutions.

1.2 Objective and Approach

Main objective is to analyze the stability of Shallow Tunnel of Norra Länken. This work will focus on numerical analysis of two sections depending on unique problems like geological structure, ingress water, fault line and shallow depth. This analysis will identify the factors that have most impact on the stability of shallow tunnels and constructions in week rock. This work will also provide understanding of which factors are important and which combinations are favorable for the stability.
1.3 Outline of Thesis

Chapter 1 An introductory chapter and provides general overview of the importance of shallow tunnel in highly dense areas with respect to population and traffic. Overview of influence factors on stability of tunnel is included.

Chapter 2 Literature study is presented in this chapter. It will deals with the influence of different factors on stability of shallow seated tunnel, going to focus on RMQ and its relation to stability. Also include the literature about state of stress and its influence at shallow seated tunnel. One part of this chapter will enlighten the reader of the importance of geological conditions and shape of shallow seated tunnel openings.

Chapter 3 First part of this chapter will give brief knowledge about the design of shallow tunnel along with functional requirements and loading criteria. Remaining part consist of different analysis methods, empirical methods and numerical methods. Introduction, applications and criteria use in numerical analysis is presented afterwards. Lastly, different FEM tools that are going to use now a days with respect to their advantages and limitations will be valuable source of information here.

Chapter 4 is dedicated to the Norra länken case study. All the details of corresponding analysis sections like rock mass condition, geology, loads, ingress water, and shape of tunnel are included.

Chapter 5 Conceptual Numerical Analysis for two selected sections is presented in this chapter. This chapter starts with brief introduction about PLAXIS that was used for numerical analysis tool, followed by input data and FEM outputs like deformed mesh, vertical displacement, axial force and bending moment etc. This analysis has been done with different excavation approaches and construction stages.

Chapter 6 Discussion on Continuous & Discontinuous Approach for Stability Analysis.

Chapter 7 Discussion on FEM output towards conclusion and recommendations for future work are presented.
Chapter 2

Factors Affecting the Stability of Shallow Tunnel

According to Hoek and Brown (1980), the stability of the underground excavation is totally depends up on the structure condition in the rock mass, degree of the weathering of the rock mass and the relationship between rock stress and the rock mass strength. The rock is a natural material and is the combination of the rock mass, intact rock and discontinuity, are major influencing factor for the stability of the shallow tunnel. In fact the stability of the tunnel is influenced by rock mass quality, mechanical process acting on the rock mass, geological structures, tunnel size and location and the surface loads. Moreover the rock mass quality is governed by the rock mass strength, rock mass deformability, strength anisotropy, presence of discontinuities and weathering effect. Similarly the mechanical processes are affected by the rock stresses and ground water (k k panthi, 2006).

Figure 2.1 Factors affecting to the stability of the shallow tunnel
2.1 Rock mass Quality

The rock mass quality depends upon the rock mass strength, Rock mass deformability, anisotropy, discontinuity, weathering and alteration effects. (k.k Panthi, 2006). There are many ways to classified rock mass and Most widely used are Q-system(Barton et al.1974) and RMR-system (Bieniawski,1974).In recent development of rock mass classification provided RMi-system(Palmstrom,1995)and GSI-system(Hoek, 1994).

2.1.1 Rock mass strength

It is the capacity of the rock mass to withstand stress and deformation which is directly influenced by the discontinuities, foliation and the orientation of these features. The intake mass never represent the whole rock mass strength, since the intake rock mass is usually strange and homogenous with few discontinuities. According the Bieniawski and van Heerden (1975), the rock mass strength deformation is quite different from the intake rock mass specimen. Since the rock mass strength is really difficult to estimate in the field, the following some of the most used empirical formulae are developed.

Table 2.1 Empirical formulae for estimation of rock mass strength

<table>
<thead>
<tr>
<th>Proposed by</th>
<th>RMR and its relation with rock mass classifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bieniawski (1993)</td>
<td>( \sigma_{cm} = \sigma_{cr} \times e^{\exp \left( \frac{RMR - 100}{18.75} \right)} )</td>
</tr>
<tr>
<td>Hoek et al (2002) and Hoek (1994)</td>
<td>( \sigma_{cm} = \sigma_{cr} \times s^{d} = \sigma_{cr} \times \left[ \exp \left( \frac{GSI - 100}{9} \right) \right] = \sigma_{cr} \times \left[ \exp \left( \frac{RMR - 105}{9} \right) \right]^{d} )</td>
</tr>
<tr>
<td>Barton (2002)</td>
<td>( \sigma_{cm} = 5 \gamma \times Q_{c}^{1/3} = 5 \gamma \times \left[ \frac{\sigma_{ci}}{100} \times Q \right]^{1/3} = 5 \gamma \times \left[ \frac{\sigma_{ci}}{100} \times 10 \right]^{15} \times \left( \frac{RMR-50}{10} \right)^{1/3} )</td>
</tr>
</tbody>
</table>

Where: \( \sigma_{cm} \) is the unconfined compressive strength of the rock mass in MPa, \( \sigma_{cr} \) is the axial compressive strength of the intact rock mass having 50 mm core diameter in MPa.RMR is the Bieniawski’s rock mass strength, \( s \) and \( a \) are the material constant belongs to the Hoek_ Brown failure criteria, GSI is the geological strength index, \( \gamma \) is the rock density in t/m3. QC is the normalized rock mass quality rating and Q is the rock mass quality rating.

2.1.2 Rock mass deformability

The modulus of deformation (Em) is defined as the ratio of stress to the corresponding strain during the loading of the rock mass including the elastic and plastic behavior. The deformation modulus could be measured in the field by using the methods like Plate Jacking Test (PLT), Goodman Jack Test (GJT), Flat Jack Test (FJT), Cable Jack Test (CJT), Radial Jack Test (RJT) and Dilatometer Test (DT) (Palmstrom and singh, 2001). Since these methods are more time consuming, cost and operational difficulties, the following empirical equations are developed to estimate the rock mass deformation modulus.
Table 2.2 Relationship to estimate rock mass deformation modulus

<table>
<thead>
<tr>
<th>Introduced by</th>
<th>Relationship to estimate rock mass deformation modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bieniawaski (1978)</td>
<td>$E_m = 2RMR - 100$</td>
</tr>
<tr>
<td>Serafim and Pereira (1983)</td>
<td>$E_m = 10^{(RMR-10)/40}$</td>
</tr>
<tr>
<td>Palmstrøm (1995)</td>
<td>$E_m = 5.6 \times RMi^{0.375}$</td>
</tr>
<tr>
<td>Hoek and Brown (1997)</td>
<td>$E_m = \sqrt{\frac{\sigma_{ci}}{100}} \times 10^{(GSI-10)/40}$</td>
</tr>
<tr>
<td>Barton (2002)</td>
<td>$E_m = 10 \times Q_c^{1/3} = 10 \times \left(\frac{Q \times \sigma_{ci}}{100}\right)^{1/3}$</td>
</tr>
</tbody>
</table>

Where, $Rmi$ is the Palmstrøm’s rock mass index and $E_m$ is the rock mass deformation in GPa. For isotropic, homogeneous and massive rock mass, the rock mass deformation modulus ($E_m$) could be calculated by the following relation,

$$E_m = E_{ci} \times \left(\frac{\sigma_{cm}}{\sigma_{ci}}\right)$$

According to the Palmstrøm and Singh (2001), for massive and isotropic rocks, the deformation modulus should be considered as the fifty percent of the elasticity modulus.

2.1.3 Strength anisotropy

It is the property of the rock mass having the unequal physical properties along different axes. The strength anisotropy of the rocks can be classified into the five categories on the basis of $Ia$ index and Tsidzi (1987) foliation index which are given below in table.

Table 2.3 Classification of rock strength anisotropy (after Palmstrøm, 1995 and Tsidzi, 1987).

<table>
<thead>
<tr>
<th>Class</th>
<th>Descriptive class</th>
<th>Strength anisotropy index (Ia)</th>
<th>Typical rock types</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Isotropic or close to Isotropic</td>
<td>1.0 – 1.2</td>
<td>Rocks having platy/prismatic minerals &lt; 10% with shape factors &lt; 2 and platy minerals in random orientation. Rock Types: Most of the igneous rocks and very high grade metamorphic rocks, i.e. diorite, granite, gabbro, quartzite, granitic gneiss, granulite etc.</td>
</tr>
<tr>
<td>II</td>
<td>Slightly anisotropic</td>
<td>1.2 – 1.5</td>
<td>Rocks having platy/prismatic minerals 10 – 20 % with shape factors 2-4 and platy minerals in compositional layering. Rock Types: High grade metamorphic rocks and some strong...</td>
</tr>
</tbody>
</table>
Factors Affecting the Stability of Shallow Tunnel

<table>
<thead>
<tr>
<th>III</th>
<th>Moderately anisotropic</th>
<th>1.5 – 2.5</th>
<th>Rocks having platy/prismatic minerals 20 – 40 % with shape factors 4-8 and foliation plane distinctly visible. Rock Types: Medium-high grade metamorphic rocks, i.e. mica gneiss, quartzitic schist, mica schist, biotite schist, etc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>IV</td>
<td>Highly anisotropic</td>
<td>2.5 – 4.0</td>
<td>Rocks having platy/prismatic minerals 40 – 60 % with shape factors 8-12 and very closely foliated. Rock Types: Low - medium grade metamorphic rocks such as phyllite, silty slate, etc.</td>
</tr>
<tr>
<td>V</td>
<td>Extremely anisotropic</td>
<td>&gt;4.0</td>
<td>Rocks having platy/prismatic minerals &gt;60 % with shape factors &gt;12 and fissile rocks. Rock Types: Low grade metamorphic and argillaceous sedimentary rock, i.e. slate, carbonaceous phyllite, shale, etc.</td>
</tr>
</tbody>
</table>

According to the Amedei (1983), the anisotropy can influence both the magnitude and orientation of local principle in situ stress so it should not be ignored in the elevation of the stress measurement data.

### 2.1.4 Discontinuity

Discontinuity is the changes of the homogeneity in the rock mass which are formed due to the movement in the rock mass caused by geological events at different times and at different stress level. It is the mechanical discontinuity in the rock mass having zero or low tensile strength (ISRM 1978). According to the Nilsen and Palmstrom, it is the collective term for most types of the joints, bedding planes, foliation planes, schistocity planes, weakness zones and fault zones. The mechanical characteristics of the discontinuities surface are represented by roughness, alternation, weathering, spacing and persistence (Barton et al, 1985 and Hudson, 1989). According to ISRM (1978), ten parameters are to be considered for describing the discontinuity characteristics in the rock mass which could be easily understood by the following figure.

![Figure 2.2 Discontinuity characteristics in the rock mass (Hudson and Harrison, 1997)](image-url)
2.1.5 Weathering and alteration effects

Weathering is the natural process of disintegration and decomposition of the materials according with changing environments. The weathering effect is maximum at the surface and decreasing with the increasing of the depth of the earth surface. Rock may be disintegrated by physically and chemically. The physical weathering is controlled by discontinuities, grain boundaries and mineral cleavages. According to ISRM (1978), both mechanical and chemical weathering act together depending on the environment and climatic regime, one or other of these aspects may be dominant. Weathering is divided into physical, chemical and biotic weathering (Loberg, 1993).

The weathering is the major factors which decreased the strength and stiffness of the rock mass. The stress is frequently redistributed from the ground surface downward where the rock mass has not been affected by the weathering to the same degree. So the state of the stress at the shallow depth is continuously redistributing.

![Figure 2.3 Typical rock weathering profile from the surface (after Rhardjo et al. 2004)](image)

2.2 Stresses on the Rock Mass

The underground geological Rock mass has different kind of the stress depending on the overburden, geological structures, geological location and type of the rocks etc. All type of stress play an important role in the stability of the underground excavated structures. The weathering, irregular topography, residual stresses, erosion and melting of the land ice are more influencing factors in the virgin state of the stress at shallow depth rather than greater depth. The ratio between horizontal and vertical stress is greater at greater depth rather than shallow.
The rock excavation at shallow depth may lead to more problems due to presence of the horizontal or absence of the horizontal stress (Amadei and Stephansson, 1997). The stress in the rock can be considered as virgin and induced stress as disturbance in the rock mass due to excavation. \( \sigma_v, \sigma_h, \) and \( \sigma_H \) are the virgin stress in the rock mass and can be considered by following relations

\[
\sigma_v = \rho g h
\]

Where \( \rho \) is the density of the rock mass \((\text{kg/m}^3)\), \( g \) is the gravity acceleration \((\text{m/s}^2)\), and \( h \) is the depth below ground surface \((\text{m})\).

\[
\sigma_H = \frac{\nu}{1 - \nu} \sigma_v
\]

Where \( \sigma_H \) is horizontal stress, \( \nu \) is Poisson’s ratio ranging from 0.15 to 0.35 for most rock types, with a common value of 0.25.

\( K \) is the relation between horizontal and vertical stress found by Hoek and Brown (1980). He developed the chart for the relation figure 3.3.

**Figure 2.4** Variation of ratio of average horizontal to vertical stress with depth below surface (after Hoek and Brown, 1980)
2.3 Geological Structure

The geological structure is the critical part of the tunnel engineering which concerned with rock type origin (igneous, metamorphic, sedimentary). Rock hardness (hard, medium, soft and decomposed) and geological discontinuities (massive slightly faulted/folded, moderately faulted/folded, intently faulted/ folded, foliations, joints, bedding planes, shear zones, dykes) which affect the stability of the human engineered structure like tunnel, dam.

2.4 Dimension and Shape

During the designing process, the shape of the shallow tunnel is governed by geological structure of that locality, orientation of in situ stresses, selected construction method, strength of lining material and estimated ground load including its distribution. Generally, the shape of the tunnels is circular, horseshoe, modified horseshoe, trapezoidal, elliptical and rectangular. Specially, the circular tunnel is used when there is the soft ground, squeezing, swelling problem. A tunnel excavated by the tunnel boring machine will be circular. Likewise, the horseshoe or circular shape is used in the location where side pressure is expected to be exerted by host media. The location where the principle stresses of the host media are unequal, the shape will be elliptical. The major axis of the ellipse is usually parallel to the direction or the major principle stresses.

As similar to the shape of the shallow tunnel, the size of the shallow tunnel is also influenced by the functional requirements of its capacity, geological setting, host media characteristics, and the selected construction methodology.

If shape is not suitable, low stress levels at shallow depth will be more critical and will lead to unnecessarily low stress in the roof of tunnel. The risk of distressed roof will minimize with the help of “stream line” section in the direction of major principal virgin stress. On the other hand, tunnel section with a flat roof will help to redistribute the stresses away from boundary into the rock, see Figure 2.5

![Stress distributions](image)

**Figure 2.5** Stress distributions around a) an arched roof and b) a flat roof

2.5 Surface loading

The surface load is also affected the stability of the shallow seated tunnel. The stress developed due to the surface load at the ground surface is measured by using the linear elastic theory and solution by Boussinesq (1883) for a point load applied to a semi-finite body.
2.6 Method of the excavation

The selection of the excavation method during the constructions is one of the challenging jobs with respect to the stability of the tunnel. The rough and careless blasting during excavation destroys the reinforcement of the interlocking between individual rock pieces which eventually causes of the instability of the tunnel. According to Hoek (1982), carefully controlled blasting does not hamper more to amount of the reinforcement between to rock mass and that reduces the overall cost of the excavation and support. Now days different kind of the excavation machines are used to minimize the disturbance to the surrounding rock mass.

2.7 Supports measures

It is one of the most important factors which should be considered during construction and long term stability of the tunnel. There are two systems which applied in the support measurements of tunneling are rock reinforcement and rock support. Rock reinforcement is used to improve the strength and deformation behavior of the rock mass. The reinforcement generally consists of the steel bars such as bolts or cables which provides the additional strength to weak rock mass. In addition to the steel bars in the weak rock mass, mesh and shotcrete also play a vital role in preventing progressive raveling of the small pieces of the rock mass that are not confined by the reinforcement. In case of the very weak rock mass, the support system should be used rather than reinforcement. Generally the support consists of the shotcrete or concrete lining whose primary function is to limit the deformation of the rock or soil mass surrounding the tunnel. In practice two system ASSM (American Steel Support Method) and NATM (New Austrian Tunneling Method) are used for the tunneling operation works. In ASSM system, heavy steel or arches are installed in the tunnel to provide support to the rock mass with the help of the heavy machine. But in case of the NATM, easier systems which are handled by the men and less machinery are preferred. During the excavation process, the virgin stress of the rock mass get disturbed and release, making rock mass weak and has tendency to collapse.

The NATM system is the simple method that provides a kind of the invisible arch behind the crown with the help of the bolts which are installed in the tunnel crown. The thickness of the arch depends upon the length of the bolts that are installed in the crown. In common practices the pattern and spot bolting are more popular. According to the geological conditions such as joints, type of the rocks, fractures, tunnel dimensions, the pattern and the number of the bolts are determined.

2.8 Ground water

The ground water flow is very important factors which cause the underground structure unstable by decreasing the effective stress, by swelling and raveling of the ground, settlements of the ground surface due to consolidation from lowered ground water level, drainage of existing wells, corrosion and deterioration of installation and rock support, toxic gases from ingress water. The investigation of the ground water situation is mostly carried out by pumping tests in wells and boreholes, pressure measurements, hydrological and hydro geological investigations.

According to the Nilsen and Thideman, 1993 and Karlsrud 2002, the most of the water leakage occurs in the part of the un grouted tunnels which is the closest to the surface and it is mainly confined in fractures, faults and weathered zones. The ground water flow is generally blocked by using the grouting at the tunnel face which reduces the seepage force and increase the stability of the tunnel face. Some projects which have excessive water leakage problems during and after the constructions are Chivor II (Columbia), Whatshan (Canada), Askora and Bjerka (Norway) and Kihansi (Tanzania), according to Kassana and Nilsen (2003). According to Panthi and Nilsen, 2005a, the systematic pre injection grouting improves both rock mass quality and the hydraulic
conductivity of the rock mass near to the tunnel periphery. The hydraulic conductivity depends on the jointing and character of the joint surfaces. The rock mass has high hydraulic conductivity if the joint sets are interlinked to each other and have wide aperture and are opened or filled with permeable materials. The degree of the jointing, spacing between joints and wideness of aperture in the rock mass are depending on the depth. As the depth increases, the joints become tighter with reduced aperture. As a result, the hydraulic conductivity of the rock mass decreases with the increase of the depth of the rock mass, Figure 2.6.

![Hydraulic conductivity as a function of depth for Swedish test site in Precambrian rocks](after Carlsson and Olsén, 1977)

Figure 2.6 Hydraulic conductivity as a function of depth for Swedish test site in Precambrian rocks (after Carlsson and Olsén, 1977)

The Hydraulic conductivity of the rock mass is also depending on the characteristic of rock mass which could be easily understood by following figure.
The estimating of possible inflow and leakage in underground structure is really a very tough work. According to Nilson and Palmstrom (2000), the scale conversion of test result to large scale condition is the most difficult work. The equation developed by Tokheim and Janbu (1984) is mostly used in the literatures which are expressed as below.

\[
Q_w = \frac{2\pi \times K \times L \times p}{\mu_w \times G}
\]

Where \(Q_w\) is the inflow or leakage rate in \(m^3/s\), \(K\) is the specific permeability in \(m^2\), \(L\) is the length of the tunnel in meters, \(p\) is the active head in Pa, \(\mu_w\) is dynamic viscosity of the water (9.81 x 10^-3 N / m. s), \(G\) is the geometry factor describing flow pattern relative to the geometry of the tunnel which is expressed as follows,

\[
G = \ln \left( \frac{(2D - r) \times (L + 2r)}{r \times [L + 2 \times (2D - r)]} \right)
\]

Where, \(D\) is the distance between the length axis of the excavation and ground water table in meters and \(r\) is the equivalent radius in meters (the radius of the cylinder with a surface area equivalent to the surface area of the tunnel). Similarly the specific permeability of the rock mass is calculated by following relations,
Factors Affecting the Stability of Shallow Tunnel

\[ K = \frac{k_w \times \mu_w}{\rho_w \times g} = \frac{k_w \times \mu_w}{\gamma_w} \]

Where; \( k_w \) is the hydraulic conductivity in m/s, \( g \) is the acceleration due to gravity in m/s\(^2\), \( \rho_w \) is the density of water in kg/m\(^3\) and \( \gamma_w \) is the specific weight of water in N/m\(^3\). The substituting the value of \( k \) in \( Q_w \) then the specific leakage (\( q \)) for the unlined or shotcrete lined tunnel is expressed in the following form whose units is l/min/m.

\[ q = \frac{60000 \times 2\pi \times k_w \times p}{\gamma_w \times G_1} = \frac{120000 \times \pi \times k_w \times \gamma_w \times h_{static}}{\gamma_w \times G_1} = \frac{3.77 \times 10^5 \times k_w \times h_{static}}{G_1} \]

Where; \( q \) is specific leakage in l/min/m tunnel, \( h_{static} \) is the static head in meters and \( G_1 \) is the geometry factor for one meter tunnel length \( (L = 1 \text{ in}) \), \( K_w \) is the hydraulic conductivity in m/s which depends upon the degree of jointing as well as type of the rock mass.

Lugeon test, water pressure measurements and water inflow registration through exploratory drilling are the alternative approach which depends upon the fields measurements, are basically used during the excavation process. The Lugeon value is defined as the loss of water in litres per minute and per meter borehole at an over-pressure of 1 MPa, was formulated by a Swiss geologist named Maurice Lugeon in 1933, is used to quantify the water permeability of bedrock and the hydraulic conductivity resulting from fractures. The ingress to the tunnel can be expressed as,

\[ Q(l/s, m) = \frac{2\pi \times H \cdot k}{\ln \left( \frac{4H}{D} + \varepsilon \right)} \approx H \cdot k \]

In case the tunnel is grouted then the equitation is modified as,

\[ Q = \frac{2\pi \cdot H \cdot k}{\ln \left( \frac{4 \cdot H}{D} + \left( \frac{k}{k_{grout}} - 1 \right) \cdot \ln \left( \frac{D + 2t}{D} \right) \right)} + \varepsilon \approx 2\pi \cdot H \cdot k_{grout} \]

Where \( H \) is the depth under the ground water level, \( D \) is the tunnel diameter and, \( \varepsilon \) is the skin factor and \( k_{grout} \) is the conductivity of the grouted zone and \( t \) is the thickness.
Chapter 3

Design of Shallow Tunnel

3.1 General

The designing of the shallow tunnel consists of the evaluation of the functional requirements and its environmental conditions before and after constructions of shallow tunnel. The developed stress in the elements during the construction should be in the allowable limits otherwise it will hamper function of the whole structure. So it is extremely necessary to model the structure in the analyzable format. Mathematical or computer model which are more popular these days are generally less costly and less time consuming than a photo elastic model or an actual three dimensional scaled physical model. The structures are analyzed in the different engineering systems according to the geometry, loading and material properties of the structures.

Evaluation and classification of rock mass is leading step to the design and stability of shallow tunnel. In rock engineering, most common classification systems are RSR (Wickham et al., 1972), RMR (Bieniawski, 1973, 1975, 1989) and Q-system (Barton et al., 1974). All these traditional systems were based on fixed rate for each rating factor. Ultimately use of this traditional system will ignore regional and local geological conditions and rock characteristics related to that specific site. All these methods show certain degree of deviation in results even at same location by different investigators. New investigation procedure to develop rock mass quality and classification is in demand for any particular rock tunneling. Like for assessing rock mass rating a new proposed method based on combination of analytical hierarchy process (Saaty, 1980) and the fuzzy Delphi method (Kaufmann and Gupta, 1988). In this evaluation model, concept of hierarchy structure is used for evaluation which display relation of each parameter and can propose feasible model for specific geological conditions and aim. To lower the uncertainties in expert opinion fuzzy logic theory of weighting calculations is applied.
Tunneling in heterogeneous ground is considered to be one of the most difficult tasks in subsurface engineering. It is impossible to establish a reliable ground model during the design phase. The ground model needs updating during construction. Possible refinement methods are: probing ahead, geophysical measurements, and evaluation of data. All these methods lead to stop excavation. Even more probe drilling gives pinpoint information and needs further difficult interpretation. By using advance displacement monitoring data, a lot of money and time can be saved for short-term prediction of rock mass instead of using conventional methods (Moritz et al., 2004).

Prediction of ground deformation in shallow tunnels is always of great interest with respect to stability. In general, gap parameters play a vital role in ground movements, depending on three-dimensional deformations at tunnel face, workmanship, and physical gap of the perimeter of excavation. Horizontal movements are smaller compared to vertical ones. The distance of three to four radii is always critical with respect to ground deformation (Chou, 2001). Horizontal movements are smaller as compared to vertical ones.

Settlements at surface are generally estimated using an empirical method such as the Schmidt-Peck method (Peck, 1969; Schmidt, 1969).

\[ S = S_{\text{max}} \exp \left( - \frac{x^2}{2i^2} \right) \]

S is settlement, \( S_{\text{max}} \) is the maximum settlement above the tunnel centerline, \( x \) is the horizontal distance in the transverse direction from the tunnel centerline and \( i \) is the distance from the centerline to the inflexion point of the curve.

A generalization of the expression proposed by Cording (1991) is also used:
This expression also represents settlement: \( S(x,h) \) is vertical displacement at point \( x \), \( w(h) \) is width of half settlement, \( c \) is empirical factor introduced by Chen and Peng (1981), for flat trough: \( c = 2 \) in classic trough and \( c = 4 \) in flatter troughs (Celestino et al. 2000) and \( S_{h\text{max}} \) is max settlement (in negative) over crown. For moderate depth trough width reduction (Figure 3.2b)

\[
S(x,h) = S_{h\text{max}} \exp[-(x/w(h)/2)^c]
\]

D is the diameter of the tunnel, H depth and \( \beta \) is distribution angle. In subsidence problems, Rodriguez and Toraño (2000), the distance at which the surface settlements can be neglected is approximately \( w \) (\( h=0 \)). In clayey soils, \( \beta \) is usually in the range 45°-50°; nevertheless Schmitter et al. (1981) reported a case in which \( \beta \) was about 60°. In sandy soils, the angle \( \beta \) is more frequently in the range 40°-45°; a lower value of \( \beta \), about 35°, has been reported by Chi et al. (2001).

In agreement with several authors, i.e. González and Sagaseta (2001), Rodríguez-Roa (2002), the maximum settlement at the surface \( S_{\text{max}} \) (when \( h=0 \)) increases with the ground loss \( e \) and with the tunnel diameter \( D \) and decreases with the depth \( H \). Here the following expression is proposed:

\[
S_{\text{max}} = -\exp\left[ a \frac{D^2}{H} + 1.5 \right]
\]

In which it introduced a characteristic parameter of the soil, \( a \), that determines the magnitude of the ground loss \( e \), taking values between 0.25-0.50 in stiff clays (\( e \approx 1\% \)) and between 1.0-2.0 in soft clays (\( e \approx 3\% \)). This relationship (Figure 2a) was deduced from cases reported by Clough et al. (1983), Deane and Basset (1995), El Nahas et al. (1997), Ledesma and Romero (1997), Sagaseta et al. (1999), Chi et al. (2001), Chou and Bobet (2002). In all cases \( e=0.5\%-3\% \) and \( eD^2/H < 6 \).

![Figure 3.2 Settlement pattern](image)
It is empirically proved that increase in settlements over tunnel crown increases with depth of point, $h$, as given relation:

\[
\frac{S_{h_{\text{max}}}}{S_{\text{max}}} = \exp \left[ b \frac{H}{D} \left( \frac{h}{H - D/2} \right)^2 \right]
\]

$S_{h_{\text{max}}}$ indicates maximum settlement at depth $h$, whereas $b = 0.16$ is an empirical parameter that is almost constant. Croc et al. (1984), Lee et al. (1992), and Deane and Bassett (1995) also gave this relationship as shown in figure 3.3(b).

![Graph showing relationship between $S_{\text{max}}$ and $e$, $D$, $H$](image1)

![Graph showing relationship between $S_{h_{\text{max}}}/S_{\text{max}}$ and $h$, $D$, $H$](image2)

**Figure 3.3 (a) Relationship between $S_{\text{max}}$ and $e$, $D$, $H$ (b) Relationship between $S_{h_{\text{max}}}/S_{\text{max}}$ and $h$, $D$, $H$**

The maximum settlement $S_{\text{max}}$ is related to volume loss and position of inflexion point.

\[ S_{\text{max}} = \frac{Vs}{2.5i} \]

Oteo & Sagassetta (1982) gave following expression for $i/R$:

\[
\frac{i}{R} = \eta \cdot \left( 1.05 \cdot \frac{H}{D} - 0.42 \right)
\]

Where $\eta$ is a function of the constructive procedure and in-situ ground conditions vary between 0.70 and 1.30, $R$ is the excavation radius, $H$ the overburden at axis level and $D$ is the excavation diameter. Angular distortion is also a governing factor in permissible deformation (Bjerrum, 1963).
Table 3.1 *Damage criteria based on angular distortion*

<table>
<thead>
<tr>
<th>Angular distortion</th>
<th>Damage assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/100</td>
<td>Limit where structural damage is to be feared. Safe limit for flexible brick wall h/L &lt;0.25. Considerable cracking in panel walls and brick walls.</td>
</tr>
<tr>
<td>1/250</td>
<td>Limit where tilting of high rigid buildings may become visible.</td>
</tr>
<tr>
<td>1/300</td>
<td>Limit where difficulties with overhead cranes can be expected.</td>
</tr>
<tr>
<td>1/500</td>
<td>Safe limit for building where cracking is not permissible.</td>
</tr>
<tr>
<td>1/600</td>
<td>Danger limit for frames with diagonals.</td>
</tr>
<tr>
<td>1/750</td>
<td>Lower limit for sensitive machinery.</td>
</tr>
</tbody>
</table>

3.2 Functional requirements

The minimum functional requirements for the shallow tunnel (highway), is proper ventilation, exhaustion of the vehicle fuel fumes, provide proper lighting, provides the grades and curves that are easy to communicate by the intended vehicles.

3.3 Loading

The loading mechanism in the shallow tunnel is quite complex. In the shallow tunnel the crucial load comes from the host ground itself. The distribution of the pressure is dependent on the several factors such as the relatives stiffness of the structures and host ground, the elapsed time between the excavation and installation of the supports, the characteristics of the host ground, the in situ pressures, the size of the opening, the location of the water table and the adopted method of the constructions (R.S. Sinha, 1981). Discontinues such as bedding planes, joints, folds, shear zones, seams, gauges, dykes and fractures are more responsible for the loosening the magnitude of loads. Rock, having such discontinues generally exerts more loosening load than a rock component which contains less discontinuities. Water table is another component which affects the strength of the rock mass. An intact rock that is fully saturated by water looses 50 percent of its strength (R.S. Sinha, 1981).

Drill and blast method creates more loosening rock loads than excavation by tunnel boring machine (TBM). The amount of the loosening rock load during the drill and blast is governed by several factors such as powder factor, pattern of drilling, sequence of loading, use of delay system, type of the explosive used and characteristics of the rock where as in tunnel boring machine depends on the thrust of the machine, the type of the cutter used, the rotational speed of the tunnel boring machine and the characteristics of the rock. Anyway, it could be stated that a drill and blast excavation will disturb a zone about three to six times the disturbed zone of a tunnel boring machine (R.S. Sinha, 1981). Likewise in heading and bench method of construction will create lesser loads on the supports than full face excavations. Now we are discussing about the method of designing of shallow seated tunnel.
3.4 Empirical methods

3.4.1 Terzaghi’s rock load

Terzaghi developed a simplified type of rock load on roofs of tunnels in 1946. It is based on the nine types of the rock and the width and height of the opening. This rock load is usually used for the long tunnels and basically considers the loosening load. If the genuine rock pressures exist that are much larger than the loosening loads, then this method will not be applicable (R.S. Sinha, 1981).

3.4.2 The Q-system

Q-system is a good approach to make the correlation between Q-value and the tunnel rock support, was developed by Norwegian Geotechnical Institute (NGI) on the analysis of the 200 tunnel cases. Q-system is based on the following six parameters.

\[ Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \]

Where

RQD= Rock quality designation

\( J_n \) = Joint Set number

\( J_r \) = Joint roughness number

\( J_a \) = Joint alteration number

\( J_w \) = Joint water reduction factor

SRF= Stress reduction factor

RQD/Jn is the relative block size (useful for distinguishing massive, rock-burst-prone rock), Jr/Jn is the relative frictional strength (of the least favorable joint set or filled discontinuity), and Jw/SRF is the relative effect of water, faulting, strength / stress ratio, squeezing or swelling.

The Q-System correlates the actual rock supports in the tunnel. In 1993, Grimstad and Barton modified the Q-system, particularly its support chart and inclusion of squeezing conditions on the SRF rating. The most recent version of the chart is shown in the figure 3.1.
3.4.3 The rock mass rating (RMR)

The Rock mass rating was introduced by Bieniawski in 1973 also known as geo mechanics classification. In recent time, it has been modified in many times, end up with last modification Bieniawaski, 1989. In RMR-system, the following six parameters are considered to evaluate the rock strength.

1. Unaxial compressive strength of the rock
2. Rock quality designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Ground water conditions and
6. Orientation of discontinuities
Table 3.1 Geomechanical classifications (RMR) of rock masses (Bieniawski, 1974, Hoek and Brown, 1980) and Q-value and rock mass quality (Barton et al., 1974)

<table>
<thead>
<tr>
<th>Sum of rating increments</th>
<th>Class</th>
<th>Description of rock mass</th>
<th>Q-value</th>
<th>Rock mass quality for tunnelling</th>
</tr>
</thead>
<tbody>
<tr>
<td>81 - 100</td>
<td>I</td>
<td>Very good rock</td>
<td>0.001 - 0.01</td>
<td>Exceptionally poor</td>
</tr>
<tr>
<td>61 - 80</td>
<td>II</td>
<td>Good rock</td>
<td>0.01 - 0.1</td>
<td>Extremely poor</td>
</tr>
<tr>
<td>41 – 60</td>
<td>III</td>
<td>Fair rock</td>
<td>0.1 - 1.0</td>
<td>Very poor</td>
</tr>
<tr>
<td>21 - 40</td>
<td>IV</td>
<td>Poor rock</td>
<td>1 - 4</td>
<td>Poor</td>
</tr>
<tr>
<td>&lt;20</td>
<td>V</td>
<td>Very poor rock</td>
<td>4 - 10</td>
<td>Fair</td>
</tr>
<tr>
<td>10 - 40</td>
<td></td>
<td></td>
<td></td>
<td>Good</td>
</tr>
<tr>
<td>40 - 100</td>
<td></td>
<td></td>
<td></td>
<td>Very good</td>
</tr>
<tr>
<td>100 - 400</td>
<td></td>
<td></td>
<td></td>
<td>Extremely good</td>
</tr>
<tr>
<td>&gt;400</td>
<td></td>
<td></td>
<td></td>
<td>Exceptionally good</td>
</tr>
</tbody>
</table>

In later, Rutledge (1978) shows a correlation of Bieniawski’s ‘RMR’ to Wickham et al. ‘RSR’ and Barton et al. ‘Q’ system as shown in the following equations.

\[
RMR = 9 \log_e Q + 44 \\
RSR = 0.77RMR + 12.4 \\
RSR = 13.3 \log_e Q + 46.5
\]

In 1978 Bieniawski developed correlation between RMR and in-situ modulus of the rock deformations in GPa.

\[
E_{\text{in}} = 2 \times RMR - 10 \quad \text{for the value greater than 50.}
\]

3.4.4 RMi-System

This system was developed by Palmstrømin 1995 based on the strength of the rock relevant to intact joints. This system is used to find the compressive strength of the intact rock mass by using the following equation.

\[
\text{RMi} = \sigma_{\text{ci}} \times JF = \sigma_{\text{ci}} \times 0.2 \sqrt{jC} \times Vb^D \Rightarrow (D = 0.37 \times jC^{-0.2})
\]

Where;

- \( \sigma_{\text{ci}} \) = the uniaxial compressive strength of intact rock measured on 50mm Diameter sample
- \( Vb \) = the block volume in cubic meters that can be measured at site by observation
- \( jC = jL \times jR / jA \) is the joint condition factors (a function of joint size and continuity factor \( jL \), joint roughness factor \( jR \) and joint alteration factor \( jA \))
3.4.5 Geological strength index (GSI)

The geological strength index is the system introduced by Hoek et al in 1995 was used for the characterization of the rock mass strength and the deformation. GSI is mostly concentrates on the rock structure and the block surface conditions. In case of the poor rock quality, GSI is the good approach rather than RMR. GSI is based on the RMR<sub>76</sub> system. There are different ways to calculate the GSI.

For $RMR_{76} \geq 18$

$$GSI = RMR_{76}$$

For $RMR_{89} \geq 23$

$$GSI = RMR_{89} - 5$$

For both relations, the Rock should be assumed as Dry. For $RMR_{76}$, the rating for the ground water conditions should set at 10 where as $RMR_{89}$ it should set at 15.

In case of very poor rock quality, RMR is not a good approach, Hoek et al., (1995) proposed the use of the Q-system (Barton et al., 1974) instead of RMR.

$$GSI = 9\ln Q' + 44$$

Hoek and Brown (1997) introduced a chart for the GSI value considering the degree of interlocking figure 3.2.
Figure 3.2 Chart for the determination of GSI (Hoek et al, 1997)
Till today, the empirical methods which are mostly used for the classification of the rock mass are used in the tunneling and underground excavations. The most commonly used rock mass classification systems are RSR (Wickham et al., 1972), RMR (Bieniawski, 1973, 1975, 1979, 1989), and Q-system (Barton et al., 1974) which only provides the quantitative data and guidelines for the engineering purpose. The main demerits of this system are to ignore the regional and local geological features and rock properties, fixed rate for each rating factors and the certain degree of the rating deviation in the same case by the different investigators. To overcome such kind of demerits, a new methodology is developed by Chao-shi Chen and Ya-ching Liu in 2006 for the evaluation and the classification of the rock mass quality. This evaluation method is based in the analytic hierarchy process (AHP) and Fuzzy Delphi method (FDM). According to the variation of the geological conditions, the weighting of each rating parameters are also changed which is the main advantage of this method.

3.5 Numerical method

The numerical method represents the most versatile and complex group of the computational methods used for the tunnel engineering. The numerical methods are mostly used to study practical problems. According to the material assumptions, the differences numerical methods have been developed for the continuous and discontinuous problems. (Jing, 2003). Both these models have the ability to model a varying topography, different virgin states of stress, different overburden, and loading applied to the ground (Jing, 2003).

An outline of the steps recommended for performing numerical analysis for tunneling is as follow:

1. Define the objective of numerical analysis
2. Selection of appropriate software and of 2D or 3D approach
3. Conceptual drawing of the analysis layout
4. Create geometry and finite element mesh
5. Application of boundary condition, initial condition and external loading
6. Apply material properties
7. Simulation of the objective proposed construction sequence
8. Check the results
9. Interpretation of the results

3.5.1 Continuous methods

This method is the most popular to study of the behavior of a closely jointed rock mass, the effects of weathering of the rock mass and of course the damage zone around the tunnel. Depending on the problem to be solved, it can divide as following groups:

i. The finite element method (FEM)
ii. The finite difference method (FDM)
iii. The boundary element method (BEM)

i. The finite element method (FEM)

In this method the subsurface is predominantly modeled as a continuum in which the host ground is discretized into a limited number of the elements that are interconnected at the nodal points. Each element in this method is considered as finite and also the discontinuities are also modeled individually. The stress, strain and deformation which are to be analyzed in the tunnel are due to the changing of original subsurface conditions and these
induced stress, strain and deformation in single element impacts the neighboring elements and so on. The main
strength of finite element method is to able to analyze the highly complex underground conditions and tunnels,
the simulation of complex constitutive laws, non-homogeneities, and the impact of the advance and time depend
characteristics of the constructions methods. The main weaknesses point of this method is to require the large
computer processing and storage capacity.

ii. The finite difference method (FDM)
In this method the subsurface is predominantly modeled as a continuum in which the host ground is divided into
a number of elements which are interconnected at their nodes. This method is based on the explicit method
which implies that the time step is smaller than the time that the disturbance take place propagates between two
adjacent points. In this method, the required processing and storage capacity of the computer is relatively small
since there is no matrix formed. This method is most efficient for the dynamic computations.

iii. Boundary element Method (BEM)
As similar to the finite element and finite difference method, the subsurface is modeled as a continuum. This
method is used for the linear, non linear static, dynamic and thermal analysis of the solids. As compared to the
other method, the data input and output are comparably simple and are easily processed. This method is more
efficient and economical for the two or three dimensional problems when the boundaries are of the great

3.5.2 Discontinuous Methods

Actually the rock mass consists of some form of the discontinuities which are due to the faults, major joints,
sedimentation, schistossity, tectonic jointing.

i. The Discrete element method(DEM)
ii. The Discrete fracture network method(DFN)

i. The discrete element method (DEM)
In this method, the ground mass is considered as the dis-continuum. The dis-continuum model as considered
when the rock mass consists of the number of discrete, interacting block. In these models the rock mass
movements are described with deformation of intact rock, slips along the joints surfaces, separation and rotation.
This method is especially used when there is highly jointed rock masses around the tunnel are presented. The
main demerit of that method is that the parameters studies are performed by assuming various joints
configurations.

3.6 Applicability and the use of numerical methods to tunnel engineering
The main application of the numerical methods in the field of the tunnel engineering is to analysis of the stress,
strain and deformations. The purpose and the goal of the numerical computations vary according to the results
which are discussed below.
3.6.1 Qualitative Analysis

In this analysis the impact on the certain parameters are describing according to the nature of the surrounding such as stress strain, deformations and the results are never expressed in the form of the absolute numbers. The numerical analysis can be divided into two groups according to the purpose of discussing the qualitative approaches.

\( a. \) Parameters studies: The main aims of these parameters studies are to analysis the impact of possible effects on subsurface conditions.

\( b. \) Sensitivity studies: In this studies the impact on civil structure parameters such as tunnel geometry, size and depth of the tunnel, relative location of the underground structure etc are analyzed.

\( c. \) Basic Principle studies: The Basic principle which determines the design requirements, are performed with both parameters of the tunnel structure and the surrounding medium.

3.6.2 Quantitative Analysis

This analysis is expressed in terms of the absolute number. It is described as design analysis and back analysis.

\( i. \) Design analysis

The tunnel excavation supports lining, anticipated strains in the surrounding medium, surface settlement etc are determined from this design analysis.

\( ii. \) Back Analysis

According to Zeng et al., 1988, the back analysis is mostly used when the surrounding parameters follow the complex constitute law which cannot be described easily. In this analysis, the input parameters are measured during the construction period. The main purpose of the back analysis is to validate the quantitative results obtained from the numerical analysis and to get the realistic input parameters for the numerical analysis.

3.7 Stability analysis of tunnels at shallow depths

\( i. \) Assessing failure mode

Understanding about failure mechanism of rock mass and also its surrounding is essential in design and support of a tunnel. During last half century there is great development in rock mechanics as before it was dealing as sub-set of soil mechanics. Failure mechanism will greatly depend on in-situ stress conditions and rock mass characteristics. If rock mass is blocky and jointed as in case of shallow depth, stability problems will be related to gravity fall of wedges from side wall and also from roof because of low confinement of rock mass. With increase of depth failure will be in result of increasing stress, ultimately can produce rock burst, slabbing and spalling. Unweathered massive rock mass can be most ideal situation, paired with relatively low stress result in minimal rock support.
Stereographic projections techniques are widely used to analyze the failure modes. By drawing the great circles of the main joints planes on stereo net, which is a stereographic projection of a set of references planes and lines within hemisphere, eventual existing wedges detected (Goodman, 1989). Failure in sidewall is more likely to occur in same way as roof wedge failure having a difference that the falls are not possible and involve in sliding on plane (Hoek and Brown, 1980).

ii. **Wedge Failure**

In case of continuous material, tunnel stability depends on the intrinsic strength and deformation properties of that material. On other hand in case of discontinuous material it depends on spacing and character of those discontinuities. Surrounding rock tends to act as discontinuous material due to size of tunnel opening relative to joint spacing. In tunnel excavation, rock is forced to build “ground arch” to redistribute the forces around the opening in a way that ground can carry most of the load.

In block or wedges stability analysis following joint conditions are quite important:

- Roughness of joint
- Number of joints
- Joint alteration
- Joint water condition
- Joint stress condition

Analysis can be done for block or wedge and support also while doing a careful study of these parameters. In case of small tunnel having ordinary geometry, initial analysis can be done by a simple free-body approach.

In case of large tunnel having complicated geology and joint conditions, recommendations are for computer base analysis such as UNWEDGE.

The concept of “solid rock” is a general misconception, except for a very small tunnel in very massive rock. Ultimately, ground around rock tunnel is a combination of blocky medium and a continuum. Loads that apply on tunnel supports are non uniform and erratic. This point is making contrast to soft ground tunneling, in which ground can be assume as elastic or elastic-plastic assumptions.

“Key blocks” (Goodman, 1989) cause most of tunnel failure. So in general words challenge in supporting a tunnel is to natural tendency of the rock to unravel. When “key blocks” succeeds to come out, others become lose and tend to follow the first one, ultimately as result of whole tunnel collapse or until the stress conditions and geometry come into equilibrium. To attain the stability, first block should be supported and held in its place so that stress rearranges them into ground arch around tunnel opening. Figure 3-3 illustrates the consecutive block failure behavior while Figure 3-4 indicates that how it can be stabilize with the help of support (After Deere 1969).
Following steps will indicate wedge failure in rock:

**Step 1** Drop down of block A  
**Step 2** Counter clockwise rotation of block B and drops out  
**Step 3** Counter clockwise rotation of block C and drops out  
**Step 4** Drop of block D followed by block E  
**Step 5** Drop of block E followed by block F  
**Step 6** Block F rotates clockwise and drops

**Figure 3.4 Progressive Failures of Blocks in Unsupported Rock**

**Step 1** Rock bolt holds block A and C in place  
**Step 2** Block B remain in place due to tight holding of block A and C  
**Step 3** Block D remain in place due to tight holding of block A, B and C
Step 4 Block E and F are held in place due to block A, B and D as result of Shotcrete and rock blot

**Figure 3.5 Prevention of Progressive Failure of Blocks in Unsupported Rock**

While using UNWEDGE, it is assume that discontinues are ubiquitous, mean block can occur anywhere in rock mass. This program considers joints, bedding and other geological structure features as planar and continuous. Ultimately this program will always detect the largest wedge and it seems to be conservative. Anyhow this program will allow wedges to scale down to more realistic size if required. Consideration of density of the rock, joint cohesion and friction angle will give safety factor of that wedge failure.

iii. **Stress Induce Failure**

Failure occurs when the stress exceeds the strength of the rock mass. This failure can range from minor spalling or slapping to rock burst in which significant failure of rock mass occur. Depending on induce stress, different kind of stability problem can arises during excavation. Different kind of failure mechanism can occur depending on sate of induce stress, Table 3.2.

**Table 3.2 Failure Mode in different state of stress (Jimmy, 2004)**

<table>
<thead>
<tr>
<th>Low Induce Stresses</th>
<th>Intermediate Induce Stresses</th>
<th>High Induce Stresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Beam Failure</td>
<td>• In general, normally stable stress condition for shallow tunnel</td>
<td>• Rock burst</td>
</tr>
<tr>
<td>• Wedge Failure</td>
<td>• Formation of bearing arch</td>
<td>• Stress induce failure</td>
</tr>
<tr>
<td>• Block Failure</td>
<td></td>
<td>• Spalling</td>
</tr>
<tr>
<td>• Ravelling</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Stress induces failure can be investigated by using the strength factor (SF) against shear failure define as \( (\sigma_{1f} - \sigma_3) / (\sigma_1 - \sigma_3) \), where:

\[
(\sigma_{1f} - \sigma_3) \text{ is strength of rock mass} \\
(\sigma_1 - \sigma_3) \text{ is induce stress} \\
\sigma_1 \& \sigma_3 \text{ are major and minor principal stresses} \\
\sigma_{1f} \text{ is major principal stress at failure}
\]

SF having value of 1 will indicate that rock mass strength is greater than the induce stress; mean no overstress in rock mass. In case of, SF less than 1 will indicate induce stresses are greater than rock mass strength, result of overstress rock mass and likely to behave in plastic region.
3.8 Numerical Modeling Programs used in Tunnel Design and Analysis

Different kind of numerical modeling programs used in tunnel design and analysis now days. Use and selection of that design tool depends on applications and case study requirements. There is brief description of different tools regarding their descriptions and applications, Table 3.3.

Table 3.3 *Comparison of Different Numerical Programs used in Tunnel Design and Analysis*

<table>
<thead>
<tr>
<th>Programs</th>
<th>Descriptions</th>
<th>Applications</th>
</tr>
</thead>
</table>
| FLAC (FDM) | • Two-dimensional finite difference codes  
  • Mostly used in general analysis and as a design tool applied to wide range of problems  
  • Use user defined constitutes models and FISH functions. It is suitable for modeling of several stages like placement of support, sequential excavation, loading and backfilling  
  • This program enables thermal analysis, creep analysis, dynamic analysis and two-phase flow analysis  
  • It requires high running time when complex geometry and/or sequence modeling is involved | • Coupling of hydraulic and mechanical behavior of soils  
  • More suitable for tunneling or excavation in soil  
  • Seismic analysis |
| FLAC 3D (FED) | • Three dimensional form of FLAC  
  • In complicated geometry, meshing generation software is recommended | • Suitable for interaction study for crossing tunnels  
  • Complex three dimensional behavior of geometry |
| PLAXIS (FEM) | • User friendly  
  • Finite element analysis for two-dimensional and three-dimensional work  
  • Automatic finite element mesh generator | • Coupling of hydraulic and mechanical behavior  
  • Tunneling and excavation in soil  
  • Modeling of hydrostatic and non-hydrostatic pore pressure in the soil |
<table>
<thead>
<tr>
<th>Software</th>
<th>Features</th>
<th>Applications</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PHASE2</strong></td>
<td>More suitable for rock engineering</td>
<td>Tunneling and excavations in rock</td>
</tr>
<tr>
<td>(FEM)</td>
<td>Automatic finite element mesh generator</td>
<td>Global overview of engineering solution in rock mass</td>
</tr>
<tr>
<td></td>
<td>Two dimensional elasto-plastic finite element analysis</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Easy to use</td>
<td></td>
</tr>
<tr>
<td><strong>SEEP/W</strong></td>
<td>Finite element method for analyzing ground water seepage and excess pore-water pressure dissipation problems with in porous material</td>
<td>Steady state and transient groundwater seepage analysis for tunnels and excavation</td>
</tr>
<tr>
<td>(FEM)</td>
<td>For saturated and unsaturated flow</td>
<td>Equivalent properties of the rock mass should be properly evaluated</td>
</tr>
<tr>
<td></td>
<td>Suitable for simple and saturated steady state problems to sophisticated, saturated unsaturated time-dependent problems</td>
<td></td>
</tr>
<tr>
<td><strong>MODFLOW</strong></td>
<td>Widely used and suitable for ground water flow simulation</td>
<td>Modeling of heterogeneous and anisotropic aquifer system</td>
</tr>
<tr>
<td>(FEM)</td>
<td>Modular finite difference groundwater flow model</td>
<td>Three dimensional steady state and transient flow</td>
</tr>
<tr>
<td><strong>UDEC</strong></td>
<td>Two dimensional discrete element code</td>
<td>Tunneling and excavation in jointed rock mass</td>
</tr>
<tr>
<td>(DEM)</td>
<td>Suitable for rock problems with joined rock system</td>
<td>More suitable if dominating weak planes are well identified with their properties properly quantified</td>
</tr>
<tr>
<td></td>
<td>Modeling of large deformation along the joint systems</td>
<td>Seismic analysis</td>
</tr>
<tr>
<td></td>
<td>The intact rock/block can be rigid or deformable blocks</td>
<td>In case of pressure tunnels, which requires more and details of joint flow, aperture and disclosure relationships, hydro jacking potential analysis</td>
</tr>
<tr>
<td></td>
<td>Fully dynamic capability is available with absorbing boundaries and wave inputs</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Joints data can be input by statistically based joint set generator</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Coupling of hydraulic and mechanical modeling</td>
<td></td>
</tr>
<tr>
<td>Software</td>
<td>Features</td>
<td></td>
</tr>
<tr>
<td>----------</td>
<td>----------</td>
<td></td>
</tr>
</tbody>
</table>
| **3DEC (DEM)** | - Three dimensional extension of UDEC  
- Hydro mechanical coupling is available  
- Designed for simulating the dynamic response to loading of rock mass with multiple and intersecting joint systems  
- Suitable for interaction study for crossing tunnels in jointed rock mass  
- For pressure tunnels, hydro jacking potential analysis  
- Complex three dimensional behavior of geometry |
| **UNWEDGE** | - Simple safety factor analysis  
- Three joints sets requires to form wedge  
- Conceptual analysis tool for tunnel support design  
- A parametric study for wedge loading diagrams for tunnels |
| **LSDYNA** | - Mostly use in dynamic and impact analysis  
- Coupling of Euler-Lagrange non-linear dynamic analysis  
- Transient dynamic finite element base program  
- Blast/explosion analysis  
- Seismic/vibration analysis  
- Impact analysis  
- Modeling of computational fluid dynamics |
| **AUTODYN** | - Coupling of Euler-Lagrange non-linear dynamic analysis  
- Convenient material library  
- Dynamic analysis  
- A finite difference, finite volume and finite element based Hydro code  
- Blast/explosion analysis  
- Seismic/vibration analysis  
- Impact analysis  
- Modeling of computational fluid dynamics |
Chapter 4

Case Study and Technical Descriptions

4.1 Background

The Norra Länken is one of the biggest road and tunnel Project in SWEDEN with respect to scope and budget. The geotechnical investigation is in progress from 2005. NORRA LÄNKEN will be a link in the peripheral route around the Stockholm inner city area and be part of the E20 European Highway, Figure 1.1. The part of the link that is now to be built will be around 4 km long and will run between Norrtull and Värtan, with a connection to Roslagsvägen at Stockholm University. Most of the link will be housed in tunnels. Norra Länken is designed to solve traffic problems in central Stockholm. This in turn, will reduce air pollution and will make inner city streets safer for pedestrians and cyclists. Norra Länken will also improve the infrastructure and competitiveness of the Stockholm region.

Figure 4.1 Norra Länken site

The stability of shallow tunnel is always big concern in tunnel construction and in service phase. Shallow tunnels are defined as tunnel that has an overburden of less than 0, 5 times of the tunnel span or diameter. Objective will be to gain stability with respect to construction methods and support system by using numerical analysis.

To gain the real knowledge of the stability of shallow tunnel, real 2 cases are analyzed in Norra Länken at tunnel section at 1/680-1/695 and 2/343-2/355.

Purpose to study only these two sections:

i. Very low overburden

ii. In case of large deformation, subsidence of nearby houses, buildings and rail station can occur

iii. More sensitive points regarding vibration due to metro crossing etc.
iv. Water ingress.
v. Three fault lines are crossing there.
vi. More sensitive in stability point of view.

Norrå Länken tunnel project will intersect both the metro and Roslassbana tunnel. This fact leads this project to be more sensitive in construction and support conditions that should be analyzed carefully with good engineering tools.

4.2 Section 1/680-1/695

Tunnel section 1/680-1/695, see figure 4.2 has wings Span 19.2 m. Mountain coverage area ranges from 6 to 3 m, see Figure 4.4. Jb probe do not indicate any abrupt level changes in the rock surface, but these cannot be excluded. Underground Skull lies in the area at approximately +7 to +8. The ground surface is at a level of +19 to +21. From above the earth consists of 1 to 2 m fill, 3-4 m clay and 3-4 m moraine. As most is the layer of soil to 9 m. The water table lies about 1 m below the surface.

This section contains homogeneous granite and gneiss with thin pegmatite, Single horizontal joints with direction NE and SE with dip direction less than 30 degree, see figure 4.3. Rock is in good quality having strength greater than 10 Mpa.

Figure 4.2 Traffic Tunnel IHT301’s crossing over Roslagsbanan. Plan
Figure 4.3 Longitudinal section of road tunnel

Figure 4.4 Geometrical conditions - Area of poor mountainous coverage
4.3 Section 2/340-2/342

In section 2 / 340 -2/42 rock cover ranges from 2.5 to 3.5. The calculations in section 2 / 340 and 2 / 342 contained a 340 mm temporary shotcrete construction and a 400 mm thick permanent shotcrete construction, while Section 2 / 344 contained a 340 mm temporary shotcrete construction and a 700 mm permanent shotcrete construction.

The area is shown in plan in Figure 4.6 and the various divisions in section 2 / 340, 2 / 342.
Figure 4.6 Geometric conditions – Plan

Figure 4.7 Longitudinal section of road tunnel
Figure 4.8 Geometric conditions - Section 2 / 342 Red line represents the position of temporary shotcrete construction. Green line is the rock surface.

Figure 4.9 Cross section of road tunnel.
4.4 Material Properties of rock mass

In order to make FEM model, following material properties were used in PLAXIS, provided by TRAFIKVERKET.

Table 4.2 Material properties of subsoil

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Name</th>
<th>Fyllning</th>
<th>Lera</th>
<th>Friktionsjord</th>
<th>Rock</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Model</td>
<td>Model</td>
<td>Mohr-</td>
<td>Mohr-</td>
<td>Mohr-Coulomb</td>
<td>Mohr-Coulomb</td>
<td>_</td>
</tr>
<tr>
<td>Type of material behaviour</td>
<td>Type</td>
<td>Drained</td>
<td>Undrained</td>
<td>Drained</td>
<td>Drained</td>
<td>_</td>
</tr>
<tr>
<td>Soil unit weight above p.l.</td>
<td>γ&lt;sub&gt;Unsat&lt;/sub&gt;</td>
<td>18.00</td>
<td>17.00</td>
<td>18.00</td>
<td>27.00</td>
<td>KN/m&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
<tr>
<td>Soil unit weight below p.l.</td>
<td>γ&lt;sub&gt;Sat&lt;/sub&gt;</td>
<td>19.00</td>
<td>17.00</td>
<td>19.00</td>
<td>27.00</td>
<td>KN/m&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
<tr>
<td>Horizontal Permeability</td>
<td>K&lt;sub&gt;X&lt;/sub&gt;</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>m/day</td>
</tr>
<tr>
<td>Vertical Permeability</td>
<td>K&lt;sub&gt;y&lt;/sub&gt;</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>m/day</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>E&lt;sub&gt;ref&lt;/sub&gt;</td>
<td>2.00 E+04</td>
<td>5.00 E+03</td>
<td>2.00 E+04</td>
<td>1.78 E+07</td>
<td>KN/m&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>Poision’s ratio</td>
<td>ν</td>
<td>0.320</td>
<td>0.35</td>
<td>0.306</td>
<td>0.25</td>
<td>_</td>
</tr>
<tr>
<td>Cohesion</td>
<td>C&lt;sub&gt;ref&lt;/sub&gt;</td>
<td>1</td>
<td>20.00</td>
<td>1.00</td>
<td>2800.00</td>
<td>KN/m&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>Friction angle</td>
<td>φ</td>
<td>32</td>
<td>0.10</td>
<td>34.00</td>
<td>35.00</td>
<td>•</td>
</tr>
<tr>
<td>Dilatancy angle</td>
<td>ϕ</td>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.80</td>
<td>•</td>
</tr>
<tr>
<td>Interface reduction factor</td>
<td>R&lt;sub&gt;inter&lt;/sub&gt;</td>
<td>0.67</td>
<td>1.00</td>
<td>0.67</td>
<td>1.00</td>
<td>_</td>
</tr>
</tbody>
</table>
Chapter 5

FEM Calculations

Numerical analysis is main issue of this research. As discussed before stability of a tunnel is ultimate goal in any shallow tunneling project. Background information needed for the analyses has been collected and a comprehensive review of available data has been made in previous chapter. During analysis work of this Norra Länkan shallow tunnel project, two main problematic sections were selected depending on typical problems. PLAXIS was used as analysis tool here in this project.

The results are shown in the order it was analyzed. The accuracy of results depends on the accuracy of the rock mass properties and logical approach in software tool. Behavior of the rock mass at these sections was studied without any support and with different kind of supports. Then the deformed mesh, vertical displacement, total stress, axial force and bending moment are shown. Results obtained from sequential method of excavation are also presented.

5.1 Σ-Mstage

PLAXIS 3D tunnel program has advantage to model a typical sequential tunnel excavation in accordance with the principles of the New Austrian Tunneling Method (NATM) realistically.

Pre–relaxation factor is introduced to account for 3D effects in case of PLAXIS 2D by setting Σ-Mstage < 1.0 (β–method) and thus deformations and stress redistributions take place before the shotcrete lining is put in place.

Analysis of tunnel construction can be described with the help of many methods according to NATM. One of this is known to be as β – method, see Figure 5.1. Basic concept is, the initial stress pk that is acting on that location is divided into (1- pk that will apply on unsupported tunnel face and βpk which will apply to the supported tunnel. β-value depends on practical experience and it is not an easy task. Besides of this β-value also depend on the ratio of the unsupported tunnel length to the tunnel diameter.

In PLAXIS β–value are not in use but stage construction Σ-Mstage values are in used that are correlated with β-value. In case of stage construction, when Σ-Mstage is equal to zero, this force is fully applied to the active mesh and going to decrease stepwise to zero with simultaneous increase of Σ-Mstage value towards unity.

![Figure 5.1 β – method diagram](image-url)
Σ-Mstage represents” Stage Construction”. Its value is increased from 0.0 to 1.0. This parameter is displayed on the calculation information window. As Σ-Mstage has reached the value 1.0, the construction stage is completed and the calculation phase is finished.

This analysis has been performed at Σ-Mstage is equal to 0, 8.

5.2 Analysis Pattern

First step in model, after using all inputs, full face excavation. Analysis has been done without any support. At second stage analysis is carried out with the application of shotcrete, rock bolt and both as support at full face excavation at Σ-Mstage is equal to 0.8 figure 5.2. In the last part, analysis has been done with sequential excavation method to get comparison and better understanding about output in two different cases.

Figure 5.2 Analysis pattern
5.3 Section 1/690

5.3.1 Only tunnel Excavation

According to study plan, in first step Numerical Analysis has been done without any support like shotcrete, rock bolt etc. Outputs that got from this analysis have been presented here.

![Figure 5.3 Deformed mesh, excavation only without support](image-url)
Figure 5.4 *Vertical displacements, without support*

Figure 5.5 *Vertical displacements, without support*
5.3.2 Section 1/690, Tunnel Excavation and Introducing of Shotcrete

At this stage, shotcrete is introduced with the following characteristics properties.

**Table 5.1 Material Properties of Shotcrete**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of behaviour</td>
<td>Material type</td>
<td>Elastic</td>
<td>_</td>
</tr>
<tr>
<td>Normal stiffness</td>
<td>$EA$</td>
<td>$1.12 \times 10^7$</td>
<td>KN/m</td>
</tr>
<tr>
<td>Flexural rigidity</td>
<td>$EI$</td>
<td>$4.57 \times 10^5$</td>
<td>KNm²/m</td>
</tr>
<tr>
<td>Equivalent thickness</td>
<td>$d$</td>
<td>0.700</td>
<td>m</td>
</tr>
<tr>
<td>Weight</td>
<td>$w$</td>
<td>1.00</td>
<td>KN/m²/m</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>$\nu$</td>
<td>0.00</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 5.7 Deformed mesh, with shotcrete support

Figure 5.8 Deformed mesh, with shotcrete support
Figure 5.9 *Vertical displacements, with shotcrete support*

Figure 5.10 *Vertical displacements, with shotcrete support*
Figure 5.11 Total Stresses, with shotcrete support

Figure 5.12 Axial Forces, with shotcrete support
Figure 5.13 Shear forces, with shotcrete support

Figure 5.14 Bending moments, with shotcrete support
5.3.3 Section 1/690, Tunnel Excavation and Introducing Rockbolt

At this stage, shotcrete is introduced with the following characteristics properties.

Table 5.2 Material Properties of Geogrid (Grout body)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of behaviour</td>
<td>Material type</td>
<td>Elastic</td>
<td>-</td>
</tr>
<tr>
<td>Normal stiffness</td>
<td>EA</td>
<td>1.00 E+06</td>
<td>KN/m</td>
</tr>
</tbody>
</table>

Figure 5.15 Deformed mesh, with rock bolt support
Figure 5.16 *Vertical displacement, with rock bolt support*

Figure 5.17 *Vertical displacement, with rock bolt support*
Figure 5.18 Total stresses, with rock bolt support

5.3.4 Section 1/690, Tunnel Excavation, Rockbolt and Shotcrete

Figure 5.19 Deformed mesh, with shotcrete and rock bolt support
Figure 5.20 Vertical displacements, with shotcrete and rock bolt support

Figure 5.21 Vertical displacements, with shotcrete and rock bolt support
Figure 5.22 Total Stresses, with shotcrete and rock bolt support

Figure 5.23 Axial forces, with shotcrete and rock bolt support
Figure 5.24 Shear forces, with shotcrete and rock bolt support

Figure 5.25 Bending moment, with shotcrete and rock bolt support
Table 5.3 FEM Results for section 1/690

<table>
<thead>
<tr>
<th>Steps</th>
<th>Vertical displacement (mm)</th>
<th>Total Stress x10^3 KN/m^2</th>
<th>Axial Force Kn/m</th>
<th>Shear Force Kn/m</th>
<th>Bending Moment KNm/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel Excavation</td>
<td>2.71</td>
<td>4.01</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tunnel Excavation with shotcrete</td>
<td>2.13</td>
<td>3.65</td>
<td>286.94</td>
<td>52.85</td>
<td>31.55</td>
</tr>
<tr>
<td>Tunnel Excavation with Rockbolt</td>
<td>2.18</td>
<td>3.97</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tunnel Excavation With Shotcrete and Rock bolt</td>
<td>2.04</td>
<td>3.64</td>
<td>278.32</td>
<td>51.75</td>
<td>30.52</td>
</tr>
</tbody>
</table>

5.4 Section 2/350

Numerical analysis has been carried out for this section. All graphical outputs are presented in Appendix A. Summary of the FEM results is presented below.

Table 5.4 FEM Results for section 2/350

<table>
<thead>
<tr>
<th>Steps</th>
<th>Vertical displacement (mm)</th>
<th>Total Stress x10^3 KN/m^2</th>
<th>Axial Force Kn/m</th>
<th>Shear Force Kn/m</th>
<th>Bending Moment KNm/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel Excavation</td>
<td>1.63</td>
<td>3.71</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tunnel Excavation with shotcrete</td>
<td>1.48</td>
<td>3.49</td>
<td>208.73</td>
<td>40.57</td>
<td>26.39</td>
</tr>
<tr>
<td>Tunnel Excavation with Rockbolt</td>
<td>1.49</td>
<td>3.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tunnel Excavation With Shotcrete and Rock bolt</td>
<td>1.41</td>
<td>3.44</td>
<td>192.29</td>
<td>39.20</td>
<td>23.01</td>
</tr>
</tbody>
</table>
5.5 Sequential Excavation

5.5.1 Section 1/690, Tunnel Excavation only left portion

**Figure 5.26** Deformed mesh, left portion excavation only without support
Figure 5.27 *Vertical displacements, left portion excavation only without support*

Figure 5.28 *Total stresses, left portion excavation only without support*
5.5.2 Excavation left portion with shotcrete

Figure 5.29 Deformed mesh, left portion excavation with shotcrete support

Figure 5.30 Vertical displacements, left portion excavation with shotcrete support
**Figure 5.31** *Total stresses, left portion excavation with shotcrete support*

**Figure 5.32** *Axial force, left portion excavation with shotcrete support*
Figure 5.33 Shear force, left portion excavation with shotcrete support

Figure 5.34 Bending moment, left portion excavation with shotcrete support
5.5.3 Excavation of left portion with rock bolt and shotcrete

**Figure 5.35** Deformed mesh, left portion excavation with shotcrete & rockbolt support

**Figure 5.36** Vertical displacement, left portion excavation with shotcrete & rockbolt support
Figure 5.37 Total stresses, left portion excavation with shotcrete & rockbolt support

Figure 5.38 Axial forces, left portion excavation with shotcrete & rockbolt support
Figure 5.39 Shear forces, left portion excavation with shotcrete & rockbolt support

Figure 5.40 Bending moment, left portion excavation with shotcrete & rockbolt support
5.5.4 Tunnel Excavation left portion with shotcrete + rockbolt And Right Portion excavation

**Figure 5.41** Deformed mesh, right portion excavation & left portion with shotcrete & rockbolt support

**Figure 5.42** Vertical displacement, right portion excavation & left portion with shotcrete & rockbolt support
Figure 5.43 Total stresses, right portion excavation & left portion with shotcrete & rockbolt support

5.5.5 Tunnel Excavation left portion with shotcrete + rockbolt And Right Portion excavation + shotcrete + rockbolt

Figure 5.44 Deformed mesh, right & left portion excavation with shotcrete & rockbolt support
Figure 5.45 *Vertical displacement, right & left portion excavation with shotcrete & rockbolt support*

Figure 5.46 *Axial force, right & left portion excavation with shotcrete & rockbolt support*
Figure 5.47 Shear force, right & left portion excavation with shotcrete & rockbolt support

Figure 5.48 Bending moment, right & left portion excavation with shotcrete & rockbolt support
5.5.6 Tunnel Excavation left portion with shotcrete + rockbolt And Right Portion excavation + shotcrete+rockbolt + mid portion excavation

Figure 5.49 Deformed mesh, mid portion excavation having right & left portion excavation with shotcrete & rockbolt support

Figure 5.50 Vertical displacement, mid portion excavation having right & left portion excavation with shotcrete & rockbolt support
Figure 5.51 Total stresses, mid portion excavation having right & left portion excavation with shotcrete & rockbolt support

5.5.7 Tunnel Excavation left portion with shotcrete + rockbolt And Right Portion excavation + shotcrete+rockbolt + mid portion excavation + shotcrete +rockbolt

Figure 5.52 Deformed mesh, mid, right & left portion excavation with shotcrete & rockbolt support
Figure 5.53 *Vertical displacement, mid, right & left portion excavation with shotcrete & rockbolt support*

Figure 5.54 *Total stresses, mid, right & left portion excavation with shotcrete & rockbolt support*
Figure 5.55 Axial forces, mid, right & left portion excavation with shotcrete & rockbolt support

Figure 5.56 Shear force, mid, right & left portion excavation with shotcrete & rockbolt support
Figure 5.57 *Bending moment, mid, right & left portion excavation with shotcrete & rockbolt support*
Sequential Excavation

**Table 5.5 FEM Results for section 1/690 with S**

<table>
<thead>
<tr>
<th>Steps</th>
<th>Vertical displacement (mm)</th>
<th>Total Stress $\times 10^3$ KN/m$^2$</th>
<th>Axial Force KN/m</th>
<th>Shear Force KN/m</th>
<th>Bending Moment KNm/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel Excavation Left Portion</td>
<td>0.419</td>
<td>2.12</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tunnel Excavation Left Portion with shotcrete</td>
<td>0.419</td>
<td>1.96</td>
<td>45.34</td>
<td>13.36</td>
<td>4.92</td>
</tr>
<tr>
<td>Tunnel Excavation Left Portion with Shotcrete and rock bolt</td>
<td>0.419</td>
<td>1.96</td>
<td>45.38</td>
<td>13.66</td>
<td>4.81</td>
</tr>
<tr>
<td>Tunnel Excavation Left Portion with Shotcrete + rock bolt And excavation right</td>
<td>0.427</td>
<td>1.56</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tunnel Excavation Left Portion with Shotcrete + rock bolt And excavation Right + shotcrete + Rockbolt</td>
<td>0.427</td>
<td>1.56</td>
<td>227.21</td>
<td>72.16</td>
<td>23.08</td>
</tr>
<tr>
<td>Excavation Left+right with shotcrete And rock bolt And mid Excavation</td>
<td>2.09</td>
<td>2.42</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excavation Left+right with shotcrete And rock bolt And mid Excavation + shotcrete + rockbolt</td>
<td>1.63</td>
<td>2.40</td>
<td>958.00</td>
<td>228.16</td>
<td>122.97</td>
</tr>
</tbody>
</table>
Chapter 6

Continuous & Discontinuous Approach for Stability Analysis

6.1 General

Stability of shallow tunnel does not only depend on tunnel stability itself but also depends on behaviour and stability of ground conditions. The mechanical behavior of the rock mass also depends on mechanical behaviour of joints and blocks because of existing natural discontinuities in rock mass. Different kind of stability issues can occur around excavation, depending on state of induced stress.

In case of jointed rock mass having low stress magnitude, the stability problems can be associated with gravity driven fallouts of blocks and wedges from roof as well as sidewalls which is controlled by three-dimensional geometry of rock structure (Hoek and Brown, 1980). To form a wedge in rock mass at least three joint surfaces must exist, Figure 6.1.

![Figure 6.1 Gravity driven wedges and blocks in the tunnel roof and wall](image)

Stress always plays vital role in stability and mode of failure. Irregular topography can cause stress-induced failure in case of stress concentration above the tunnel. On the other hand spalling and rock bursts can be a result of few joints in brittle rock. Stress induce failure can occur but not in case of intermediate stress conditions with result of bearing arch formation.

Use of different numerical methods in continuous and discontinuous problems is in practice depending on underlying material assumptions.

A continuum approach is in practice and more likely relevant in case of intact rock mass conditions and also when the discontinuities are closely spaced and pervasive in relation to the size of problem domain. In this case continuum approach can be use indicating equivalent rock mass properties, Figure 6.2.
Scale effect is also another important factor that can be correlate with this issue. The behaviour of the rock mass can be almost same in case of few joints or a large number of joints which are closely spaced.

The rock mass consist of number of discrete, interacting blocks leads towards a discontinuum problems where discontinuities and intact rock mass are described separately.

![Figure 6.2 Continuous and discontinuous rock masses (Edelbro, 2003).](image)

**6.2 Continuum and Discontinuum Approach**

Linear elastic behaviour is assumed in continuum approach with the results in term of displacements, stresses and infinitesimal strains. Linear elastic behaviour will not present relevant stresses and deformations if load will exceed the yield limit. Constitutive models simulates plastic behaviour can be used in that case. It is not possible to define plastic strain in term of current stress state. The theory of plasticity must use as incremental loading approach while summing the incremental deformations to obtain total plastic deformation.

Continuum methods that can be divided into different groups depending on the way to problem are going to solve. There are two main approaches, the boundary element approach and finite element or finite difference approach. In first approach, all boundaries are discretised and result will be exact solution on the boundary. There are limitations in boundary element method. It is not often possible to use different behaviour of material in different parts of the model. Plastic model is not always available. The whole model is discretised for a method based on finite element and finite difference formulations. The accuracy of results is dependent on size of elements with respect to the scale of problem.

In case of stability study of shallow seated tunnels, numerical methods must have ability to consider ground surface to account the factors of interest. The behaviour study of closely jointed mass, effect of weathering and damage zones around tunnels can be analyzed with the help of continuous models. On the other hand, study of the effect of individual small and large scale discontinuities can be analyzed with discontinuous models. In both model types, ability to model different state of stress, overburden, loading conditions on ground surface and varying topography should cover. It should be able to simulate non-elastic behaviour in case of stresses exceed the strength.
6.3 Assessing failure mode

Hemispherical projection technique can be used to analyze the failure due to structure. At least three joint planes must exist to form a wedge in the roof or sidewall of the tunnel. These techniques are considerable in case of kinematic conditions for block or wedge such as sliding. If the sliding resistance is greater or less than the driving force with assumptions of joint strength is purely frictional, it’s again considerable.

Block analysis

In most of the shallow tunnel, the rock around the tunnel tends to act as discontinuum. The behavior of the tunnel in discontinuous material depends on the characters and the spacing of the discontinuities. The main principal of the block analysis is to hold the rock mass together by providing the ‘ground arch’ around the opening and redistribute the force in such way that the ground itself can withstand the most of load. According to the Goodman, 1980 the most failure in the rock tunnels are initiated by the block which is more interested to loosen and come out until the tunnel completely collapses or stress conditions come to equilibrium or until the natural arching in the rock mass prevents the further unraveling. The following steps should be considered to deal the above problem,

a. Describe the structural plan such as average dip, dip direction and strike.

b. Identify the active block/wedges in the tunnel.

c. Determine the factor of safety of these block/wedges depending on the mode of the failure.

d. Calculation of the amount of reinforcement which can hold the wedge into the acceptable level.

The block analysis is carried out by determining the number, orientations and conditions of the joints. The basic information that is required for the calculations of joint set is determined from the Q-system where as the collection of the data of joints are done by oriented core boring, Borehole camera e.g. BIPS and surface mapping. By using the pole stereographic projection and rose diagram, the data of the joints are presented. With the help of those parameters the stability of the block is determined and hence determines the required support in the shallow tunnel in required level. In case of the large tunnel with complicated geometry, the computer program me UNWEDGE is used to analyses the opening. With the help of the structural discontinuity sets, friction angle, factor of safety and the density of rock, this software determines the locations and dimensions of the wedges which can be formed in roof, floor and side walls of the excavation. The in site stresses in the rock masses are largely considered in this software.

The block failure could be controlled by providing the rock bolting and shotcrete support in the tunnel opening. To stable the roof wedge, the total force which should be applied by the reinforcement, should be sufficient to hold the full dead weight, allowance for errors and poor quality installations. At the same time, a great attention should be taken on the length and location of the rock and cable bolts. For the case of the side walls, the bolts and cables are placed to cross the sliding planes than across the separations planes.

The shotcrete is also used to make the additional support of wedges in blocky grounds. It helps to prevent a reduction in support capacity by peeling –off of the rock layer.

Compressed arch action

Through an arch transfer of load is achieved as compressive load. In any classical arch, different number of blocks has been arranged in such a way that only compressive forces transfer through joints (Stille et al., 2004) Figure 6.3 & Figure 6.4.
Arch collapse can occur in three different ways:

- Sliding along joints
- Crushing of joint surface or block
- Rotation of blocks

When shear stress exceeds the joint strength, sliding will occur. If the compressive stress exceeds the compressive strength, it will end with crushing and also refers to spalling. If the load induces a moment causing a tensile stress in a joint with no tensile strength, it will end up with rotation of blocks.

For a certain load \( q(x) \), the compressed arch is obtained (Stille et al., 2004) by integration of:

\[
y'' = - \frac{q(x)}{H}
\]

Where \( H \) is the horizontal support reaction, Figure 6.5.
It can also explain with following expression:

\[ y = f \left[1 - \frac{(2x)^2}{L}\right] \]

Where \( q_{\text{max}} \) is:

\[ q_{\text{max}} = 8 f H / L^2 \]

It depends on geometry of an arch L & H and horizontal support reaction H.

Maximum bearing capacity depends on risk for sliding & rotation and some time crushing.

\[ Q_{\text{max}} = \text{minimum of} \]

\[ \begin{cases} 2Hq \tan \varphi_{\text{eff}} / L \\ 8Hq \ kB / L^2 \end{cases} \]
**Voussoir Beam Theory**

Compression arch phenomena can be explainable through voussoir beam theory. Diederichs, 1999 presented one model for this theory which considers overburden, self weight deflection, and support and water pressure. Underground excavation often encountered by parallel laminations. These laminations can be result of different factors and can also depend on formation type like sedimentary layering or fracturing due to stress parallel to excavation. It will play vital role in stability of that structure.

On the other hand these are not only parallel laminations in rock mass but also contain joint set with different orientations which will put their role to reduce ability of boundary sustain process Figure 6.6. This stability also depends on cutting angels of these joint set through the lamination and also on reinforcement (Diedrichs, 1999).

![Figure 6.6](image)

**Figure 6.7 a) Jointed rock beam b) Voussoir beam analogue (Diederichs, 1999).**

Crushing (compression failure), buckling and diagonal fracturing are modes of failure in Voussoir beam model. In case of thick beam, shear failure is more common to occur having low span and thickness ratio. In case of thinner beam, crushing can more likely to occur. There is also a relationship between cross cutting joints and normal to the lamination plane and effective friction angle of the joints (Ran et al. 1994). Voussoir beam theory can be valid if angle between cross cutting joints to the normal lamination plane is less than 30-50 % of the effective friction angle of joints. If this angle is larger than 30-50 % then shear failure of the beam can occur.
6.4 Conclusion

In order to achieve the objectives of this thesis, numerical analysis of Norra Länken has been done. Reliability & accuracy of this analysis depends on different factors starting from input boundaries to the outputs evaluation. Each analysis tool has different limitations and assumptions to generate outputs but in a perfect selection that tool should consist of reality base assumptions and model close to the actual situations. So first important question is, either this analysis model has same input boundary conditions? Second, how much reliable these outputs are according to real case stability??

In these two study sections, stresses can play a complex role at shallow depths due to not only different geological conditions but also anisotropic state of stress. An exact stress measurement at shallow depth is difficult task. Different range of stress condition will lead to different stability results. Large deformation and high tensile stresses will be as result of high horizontal stresses. Low stresses can cause fallouts. In this case study, lack of inputs may leads to unrealistic stability approach. Since both sections 1/680-1/695 and 2/343-2/355 have low horizontal forces which can also lead to the failure as a result of fallouts.

Arch action is an important factor to consider in stability analysis. We don’t have sufficient data information about geological discontinues here in both sections 1/680-1/695 and 2/343-2/355. But according to theoretical & practical point of view these discontinue are going to play vital role in the stability of these two sections 1/680-1/695 and 2/343-2/355. Each study model has different kind of instability indicators up to some extent. We consider continues model, which has instability indicators like deformation around tunnel boundary, tangential stresses around tunnel boundary, subsidence, area of plasticity etc.

There is a great risk of fallouts of wedge formation due to low confining stresses. Risk of opening of pre-existing joints or tensile failure is another important factor to deal in this case study. Deformations and subsidence are also important factors to consider in this case as high deformation or subsidence will lead to damage rock support system, installations inside the tunnel and buildings or infrastructure damage cost at the surface. Deformations will depend on angle of discontinuity. Discontinues with small dip angles have trend to open and ultimately increase slip risk. Normal stress of discontinuity will decrease with the decrease of dip angle. Steeper discontinues contain more stable conditions. In case of low normal stress conditions, friction is less important than cohesion for stability. Stiff discontinues tends to be less stable as compare to soft one. Dip angle is very important parameter in discontinuity. Steep angle tends to be more favourable for stability than shallow angles.

Continuum surrounding conditions is a big issue in stability analysis here. This factor leads to point able difference between actual measured values of deformation in tunnel and analysis calculations. In depth study of actual boundary conditions with real geological conditions covering most of prominent geological structures is very important here. It is not possible to evaluate the stability of this shallow tunnel while using continuum approach. There is need to discuss three factors here, reliable and accurate input data, suitable failure criteria and reasonable instability factors.
Chapter 7

CONCLUSION AND RECOMMENDATIONS

Based on analysis of two study sections of Norra Länken following conclusions can be made:

During the conceptual stability analysis, structural geology is one of the most important factors to consider. Stability mechanism is very sensitive to the orientation in structures and discontinuities pattern. Tensile strength of rock mass is directly related to the tensile strength of geological structure as spacing, joint set, discontinues, friction angels etc. In this case rock mass is classified as good rock which increases the stability. Good rock mass leads to very low vertical displacements in both sections. But more information regarding engineering geology and structures in this case study should be included to get exact case evaluation. This is important to include in detail as it will direct effect on design on rock support system also.

The stability and behaviour of shallow tunnel is associated to the great extent with state of stress in that area. Maximum displacement is at mid span in both sections due to gravitational stress. Under the continuum rock mass assumption, high horizontal stress will lead to large deformation and tensile stresses. Favourable condition is intermediate state of stresses. Low stress conditions can also affect the results as a result of lower tangential stresses in the boundary of tunnel and will lead to the risk of fallouts.

Purpose to do this study analysis is to get better pre-understanding of the project which will lead to low project cost and use of support as more effective way. Better understanding about displacement & failure due to Numerical Analysis will lead to the more effective support design system. Due to large span, deformation level can be high depending some another influence factors i.e. load conditions, opening shape, method of excavation etc. FEM outputs for both sections with different excavation plans gave good understanding about stability of these sections of shallow tunnel. Support capacity from FEM analysis is found to be in very safe range and this is perhaps due to quite conservative assumptions made for input data.

It is highly recommended to analyze this case with some another better analysis tools. PLAXIS is continuum model, which is not so good to deal with rock problems. More realistic model is needed for more sensitive and careful study. Those study models should cover all geological structures according to real existence of this case study. Future recommendation is to study same sections with different tools that will deal rock mechanics problems in more realistic way. Here discontinuum approach should apply.

It is also recommended to deal with extent and reduction of rock mass discontinues and rock mass strengths in detail.

Tunnel will be smart option with increasing population in big cities. Further investigation can correlate to fulfill the requirement of comprehensive economical analysis study of different projects.
Appendix A

List of References


Appendix A


Appendix A

FEM outputs obtained by numerical analysis are presented in Appendix A.

Section 2/350, only tunnel Excavation

Figure 0.1 deformed meshes, excavation only without support
**Figure 0.2** Vertical displacements, without support

**Figure 0.3** Total Stresses, without support
Section 2/350, Tunnel Excavation and Shotcrete

Figure 0.4 Deformed mesh, with shotcrete support

Figure 0.5 Vertical displacements, with shotcrete support
Figure 0.6 Total Stresses, with shotcrete support

Figure 0.7 Axial Forces, with shotcrete support
Figure 0.8 *Shear forces, with shotcrete support*

Figure 0.9 *Bending moments, with shotcrete support*
Section 2/350, Tunnel Excavation and Rockbolt

Figure 0.10 Deformed mesh, with rock bolt support

Figure 0.11 Vertical displacement, with rock bolt support
Figure 0.12 Total stresses, with rock bolt support

Section 2/350, Tunnel Excavation shotcrete and rockbolt

Figure 0.13 Deformed mesh, with shotcrete and rock bolt support
Figure 0.14 *Vertical displacements, with shotcrete and rock bolt support*

Figure 0.15 *Total Stresses, with shotcrete and rock bolt support*
Figure 0.16 *Axial forces, with shotcrete and rock bolt support*

Figure 0.17 *Shear forces, with shotcrete and rock bolt support*
Figure 0.18 Bending moment, with shotcrete and rock bolt support