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**ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**

**Optimization of Rock Bolt and Concrete Lining Combination
in case of Awash-Kombolcha-Haragebeya Railway Tunnel Project**

A Thesis in Civil Railway Engineering

BY

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A Thesis submitted to School of Graduate Studies of Addis Ababa Institute of Technology in partial fulfillment of the requirements for the Degree of Master of Science in Civil Railway Engineering.

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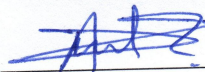
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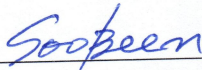
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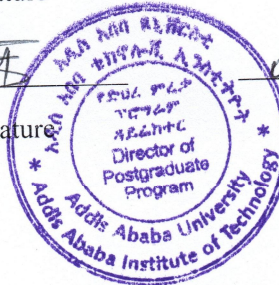
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ABSTRACT

Tunnels are crucial infrastructures used for passage of humans, cars, trains or utility lines. Among this, tunnels used for road or rail purpose needs a great attention. The tunnel to serve its intended function, rock bolt and concrete lining are provided based on the ground condition. These two support systems are worthy and need great consideration as their impact remain on tunnel cost and convergence. Thus the optimum uses of these support systems are a challenge which demands the best design solution. Consequently, the study on tunnel rock bolt and concrete lining combination optimization which need study and improving, worth researching a lot.

In this paper which is a case study, based on the design of data for optimization, the most influential support elements are selected. For different parameter combinations, the finite element methods have been performed followed by forming displacement parameter correlation. Likewise, the cost is represented by using polynomial model. Considering displacement and cost functions, the optimum combination of rock bolt and concrete lining has been found. The result show that the optimum combination is found if the number of rock bolt is 15 and concrete lining thickness 25 cm with in the constraint bolts [7,23] and lining thickness [10cm,40cm].

The optimized combination is modeled using numerical model and analysis along with the verification. The finite element analysis result gives total displacement of 6.34cm. But the observed total displacement inside the tunnel is about 60mm which is less than the predicted displacement of optimum combination using numerical analysis. The induced shear on concrete lining is 96.85 KN/m which is lower than the shear capacity of concrete lining. The total cost of rock bolt & concrete lining accounted for the optimum combination is \$30,700 per 34 m² tunnel rock bolt and concrete lining construction.

Parametric study performed on pre stress rock bolt shows that increasing number of rock bolt would slightly increase stress and strain. Conversely, the convergence of tunnel is reduced. Similarly the plastic points are greatly reduced because of the pre stress rock bolt.

Key words: Rock Bolt, Concrete Lining, Optimization, Combination, Equation.

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NOMENCLATURE

The nomenclatures in this report comprise of abbreviations and symbols used in equations, longer naming's, acronyms, representation of quantities and parameters.

Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
T-20	Technical Committee for Tunnels
AKH	Awash-Kombolcha-Haragebeya
T04	Tunnel Four
FEM	Finite Element Method
2D	Two Dimensional
NATM	New Australian Tunneling Method
FEA	Finite Element Analysis
RMR	Rock Mass Rating
3D	Three Dimensional
RQD	Rock Quality Designation
TBM	Tunnel Boring Machine
SCL	sprayed concrete lining
ULS	ultimate Limit State
DBB	Decision-Based Design
DoE	Design of Experiment
GA	Genetic Algorithms
MC	Mohr Coulomb
k	At Rest Earth Pressure Coefficient

CHAPTER 1 INTRODUCTION

1.1 Introduction

Railway or highway tunnels as defined by the American Association of State Highway and Transportation Officials (AASHTO) Technical Committee for Tunnels (T-20), are enclosed roadways with vehicle access that is restricted to portals regardless of type of the structure or method of construction (AASHTO, 2008).

Tunnels play a key role in urban transportation, water, sewerage transportation and in hydroelectric power & in many other sectors. Because of the land scarcity especially in urban areas and terrain difficulty, for constructing roads and railways, tunnels are preferable. The support systems like bolt and lining enhance the rock to be self-supporting. The pre stress bolts connects the dis connected rocks and prevent rock blocks from sliding.

The design of tunnel support system cannot be considered as a structure being subjected to well defined values of loading due to no absolute certainty of the actual ground behavior following excavation. However ground investigation is a primary task of tunneling. Similarly, there is no universally accepted method on how to assess the acceptability of the tunnel design; rather engineering judgment and experience plays a great role. Besides the engineering judgment, the support system could be designed carefully to get optimum solution in all aspects in order to control the convergence of the tunnel by providing resistance (Kersten, 2008).

In Ethiopia there are tunnels for railway and road purpose; some of which being completed and others are under construction. Under the Ethiopian Railway Network Program, many tunnels are going to be constructed in the coming years. The Awash-Kombolcha-Haragebeya (AKH) is one of the routes which contain 14 tunnels including the Karakore tunnel, on which this study is focusing.

This study focuses on optimizing tunnel support combination for curvilinear shape of tunnel by considering different combination of rock bolt and concrete linings. The study also focuses on parametric study of pre stress rock bolt. This has been studied by using finite element method using the ground data taken from AKH railway tunnel project.

1.2 Statement of Problem

The steel bolt reinforces the rock as reinforcement does in concrete. Because of the natural cycling, the rock will fail and the situation adds load on the tunnel primary lining. Due to the opening for tunnels, the rock found near to the opening is susceptible to be in plastic state. This mainly causes the tunnel to converge. If the rock around the opening is weak, then the overburden may induce high stress on lining. Consequently, the passive support, lining will be loaded more and susceptible for failure.

Both bolt and lining differs in terms of mechanism in supporting the tunnel. Bolt function is to reinforce the rock and binds broken rocks together. The tensile capacity of the rock is highly improved by introduction of rock bolt. And also the ground become stiffens. But the later one support the tunnel by carrying the imposed load and its involvement is passive. The two support unit's bolt and lining become integrated and designed in optimal to carry the loads, then the tunnel convergence is prevented and can sustain all the loads coming. The good anchoring capacity of the bolt carries the loads and bind together. Consequently the load on lining becomes small. If this is not the case the lining will fail.

The overall cost is a major consideration in the support of tunnels as it is essential that operations can be fully justified on an economic basis. Very often, the margin between economic and uneconomic is quite small. Therefore, it is essential that all items of the tunnel cost are optimized in terms of cost and due consideration taken in conjunction with their intended use. The major cost of tunnel support system is cost of primary linings which approximately account 40 percent of total cost. This brings into perspective role of optimum tunnel support system design.

1.3 Purpose of the Study

In tunneling work, based on the ground condition the support units are installed. From those support systems rock bolt and concrete lining are amongst. The rock bolt reduces the tunnel convergence by enhancing the rock self-supporting system. Significantly, the cost of bolt installation is found in fewer amounts but contributes a lot for reduction of tunnel convergence.

Therefore, the main ultimate goal and recently valuable concept in tunnel support design is to come up with minimum cost and as same time durable and serviceable tunnel. Lining and bolt, being support systems of tunnel, are selected. However, Lining and bolt are the support elements; they differ in the method of keeping tunnel. Lining carry compressive loads passively that comes from the soil overburden. Nonetheless, the bolt actively keeps the fractured rock into one by binding and reinforcing them.

Therefore, being the important element of the tunnel and having the key in enhancing the tunnel functional, bolts and lining need a big attention and study for new innovation in optimizing them a head.

The basic motivation to do the research is that, the lining cost being the major and the pre stress rock bolt efficiency in stiffening as well reinforcing the ground with significant tunnel displacement reduction, in demand

to do optimization of the combination. This gives understandings in increasing the number of bolt could be a preference for optimization as it minimizes the cost and the displacement also.

Therefore, by doing this research, multitude of benefits and advantages are realized through the better understanding of increasing the number of bolt could keep the tunnel convergence and the minimum cost could be attained. Therefore, the significance of research accounted in:

- ✓ Better understanding of the parameters, their correlation & their effect on tunnel convergence and cost, and suggesting best combination of rock bolt and concrete lining in tunnel support system

Therefore, the main purpose of the study is to come up with optimum combination of rock bolt and concrete lining by using the numerical analysis for specific ground condition.

1.4 Objective of the Research

The study of rock bolt and concrete lining design optimization is so important that the cost of lining gets higher this time and the bolts have significant effect on reduction of tunnel convergence. To accomplish the study the following general and specific objectives are set:

1.4.1 General Objective

The main objective of this study is to come up with optimum combination of rock bolt & concrete lining by performing numerical analysis for different combination of rock bolt and concrete lining.

1.4.2 Specific Objectives

This study covers the following specific objectives:

- Perform parametric study on pre stress rock bolt
- Both less deformable and cost effective rock bolt & concrete lining combination which is better than the Yepi Merkezi design combination

1.5 Research Area

The research address the optimum combination of tunnel supports' rock bolt and concrete lining, taking a case of tunnel 04 (Karakore Tunnel) found on Awash-Kombolcha-Haragebeya route which is located near Kara kore town in Amhara Region. T04 tunnel project is stretched between km 178+090 to 179+620 along the new Awash-Kombolcha-Hara Gebaya single track railway line. T04 has a length of about 1530m and it is excavated by using conventional method. Its maximum cover is about 320 m. For this research, in particular, the overburden is about 80m which is found at 178+300 where the research proceeds doing the objective at this section.



Figure 1.5-1: Tunnel study area location

1.6 Methodology

To perform this research, multiple cases of virtual experiments has been done, based on design of data for optimization approach to come up with an optimal parameter combination, using numerical modeling and then a finite element simulation. Finally based on optimum parameter combinations, the combination has been checked along with interpretations. Finally the parametric study on bolt has been presented. The procedures to follow and resources to use are discussed in detail in the following paragraphs.

i. Data Collection

- ✓ Tunnel geometry, ground condition , and support data were taken from the AKH project;
- ✓ The Sleeper, ballast and rail data were taken from literature;

ii. Methodology

The methodology used to implement the research and come up with good result was by developing Finite Element Method analysis with different parameter combination based on design of data for optimization. Data was analyzed using numerical analysis method. Then the optimum parameter combination was identified and the FE analysis has run to validate and check the best parameter combination. And also a parametric study on pre stress rock bolt has been performed.

The step-by-step procedure of the approach used is outlined as follow:

1. FE modeling and analysis using Plaxis 2D for parameter correlation

- ✓ Modeling the tunnel supports bolt and lining
- ✓ Running FE analysis Based on the design of data for optimization approach by varying number of rock bolt and thickness of concrete lining

- ✓ Extracting displacements from the FE analysis output which has been used as the fitness function for the optimization process
- ✓ Developing polynomial model for parameter correlation
- ✓ Finding the optimum parameter combination

2. FE modeling for the optimum design parameter combination

- ✓ Model the optimized parameter combination with Plaxis 2D
- ✓ Extract important FE analysis outputs like, forces and displacement
- ✓ Check the FE results using the analytical equations
- ✓ Make interpretation of the results

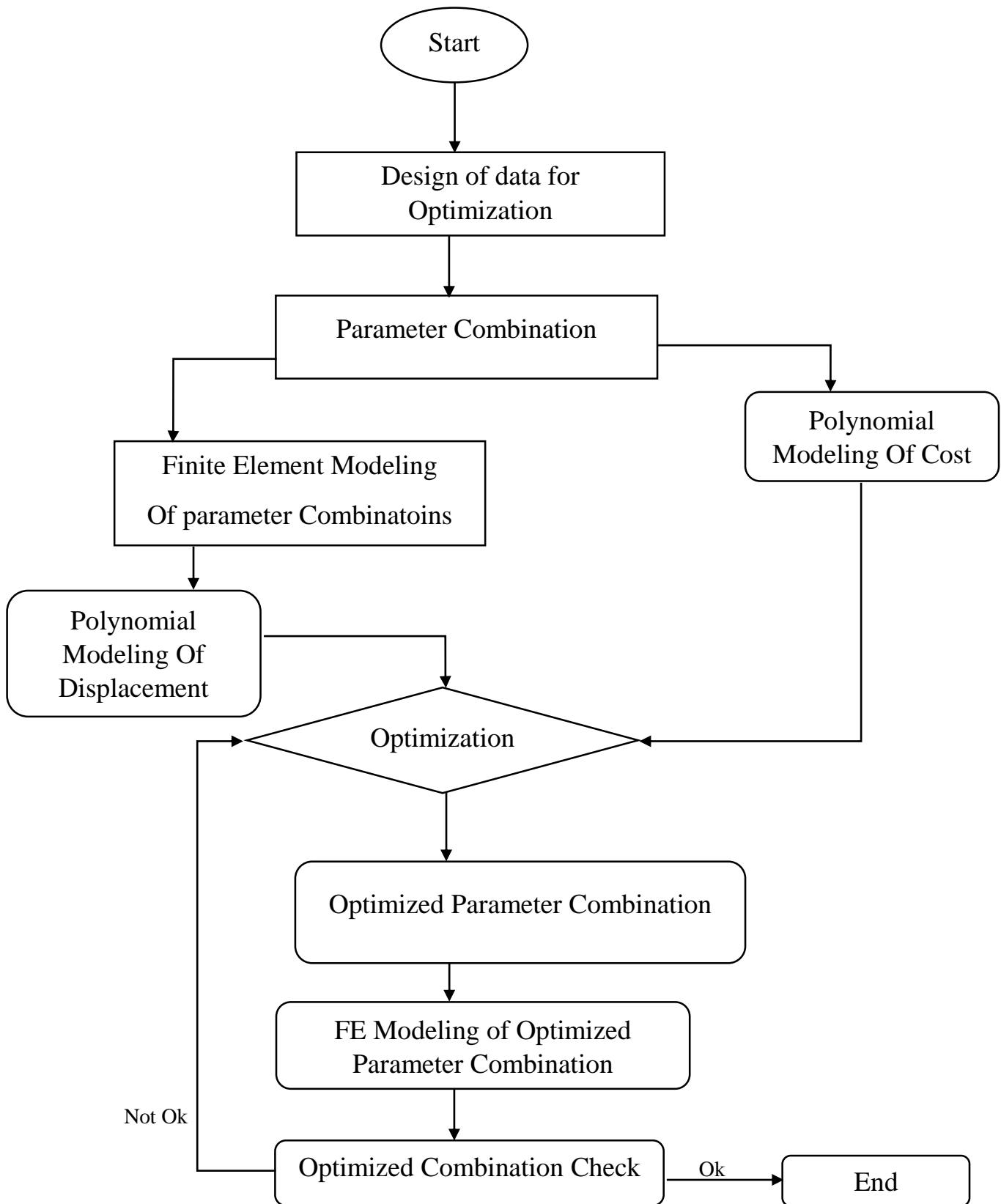


Figure 1.6-1: Flow chart-algorithm for conducting optimization

iii. Materials/ resources

The materials and/ or resources been used include hardware and software for the purpose of doing the virtual experiments of the developed model, FE approaches, either in the case of the conceptual/ preliminary models or the final in detail established models. The resources required in achieving this research included:

- ✓ Computing hardware/ Personal computer(Laptop);
- ✓ Computing/ application software which are Plaxis 2D for FE simulation and Mat lab and Excel for optimization, academic versions;

1.7 Scope and Limitation

The study focuses on the optimal parameter combination of rock bolt and concrete lining for curvilinear shape of tunnel by considering different combination of lining and of bolts. However, there are many support options like ribs and beams besides rock bolt & concrete lining, the study focuses on the two support types, as they differ in terms of supporting the tunnel.

The study is done using numerical analysis based on the computer modeling of rock bolt and concrete lining. Therefore, experimental investigations of the phenomenon were not conducted either to validate or verify. Besides, the resource limitation and research duration make it impossible undertaking. So the work is drawn only from numerical analysis outcomes and outputs.

With the above limitations stated, the research has been performed to be worthwhile and give insight for the way how the optimization of rock bolt and concrete lining combination be achieved.

1.8 Outline of the Research

The thesis is presented in six chapters. The arrangement of thesis is described below.

- ❖ Chapter 1 gives a brief introduction to the research and arrangement of the thesis. The objective, the scope and the research methodology are found in this chapter.
- ❖ Chapter 2 Review on tunnel support and design. The conceptual background related to the topic is reviewed including the recent previous studies.
- ❖ Chapter 3 deals about modeling in finite element method. The scientific approach of the numerical modeling method is discussed together with equations for optimization of the combination.
- ❖ Chapter 4 Results and discussion: this part contains the numerical modeling and an analysis result of optimized parameter combination along with verification of the optimized combination has been presented. Parametric study on pre stress rock bolt. The effect of varying number of pre stress rock bolt on tunnel displacement, stress, strain, and force on tunnel is done and presented.
- ❖ Chapter 5 Conclusion and recommendations. In the final chapter the topic has been concluded and further research area has been suggested.

CHAPTER 2 LITERATURE REVIEW

2.1 Lining Design and Analysis

Benoît D. Jones (2007) studied about design of sprayed concrete lining (SCL) tunnels in soft ground using euro codes. Nevertheless, one feature of both of these semi-empirical approaches to the design of tunnels that couldn't be avoided is the emphasis on measuring displacements of the soil and the structure, while ultimate limit state (ULS) design codes such as the Euro codes are based on the prediction of stresses. Is it really safe to assume that because a numerical model has predicted displacements correctly it has also predicted stresses in the tunnel lining correctly? The paper explored the importance of this question to the ULS design of SCL tunnels in soft ground.

The paper went through all the stages of the design of sprayed concrete lined (SCL) tunnels in soft ground. Beginning with the design philosophy, and then moved onto more detailed aspects of design with particular emphasis on numerical analysis. All examples involved the use of Euro codes, and concentrated on their application in the UK.

The limitation of the paper was that it is applied only in particular area UK. The design did not include the different soil conditions i.e. it was only for soft soil.

Benoît D. Jones concluded the followings from 3D numerical modeling and observation; the level of deformation required to mobilize a 'ground arch' in soft ground was very small compared to a tunnel in rock, and for an open-face tunnel there was no benefit to be gained by actively allowing any additional deformation to occurred. The use of the observational method was limited in soft ground because there was too much uncertainty about the amount of time available to respond to instability (Benoît, 2007).

The analysis of bored tunnel linings in soft ground is usually carried out using continuum 2D analytical or numerical calculations. When defining the 2D load applied to the tunnel lining, the available methods can be classified into four types (AFTES, 2011):

Empirical methods indicating a rock support pressure defined directly from a classification system. The most widely used methods are the Beinafski (Bieniawski, 1989) RMR and Barton (Barton N. , 2009) Q system. These can be useful aids at the project planning stages for tunnel design in hard ground but have significant disadvantages in weaker materials which include omission of ground-structure interaction and reliance on case histories which are not always compatible.

Analytical methods for determining the loads acting on the support, regardless of support type and deformation. The most widely use effective stress method is the Terzaghi (Terzaghi, 1943) arching method which has been developed further by Szechy (Szechy, 1970). These methods assume that a collapse mechanism is formed above the crown of the tunnel which is the only load applied and the corresponding displacements are not considered.

Analytical or numerical methods collectively known as “Convergence-Confinement Methods ” which can account for elasto-plastic ground-support interaction by providing equations for calculating the ground stress-strain curve as well as the stiffness of the lining. These methods are applicable to both rock and soils. The most widely used analytical solutions are the theory presented by (Panet & Guenot, 1982) for cohesive and non-cohesive soils and the effective stress method by (Hoek & Brown , 1980) (Hoek, 2007) and (Vlachopoulos & Diederichs, 2009)for soils and weak rocks.

2.1.1 Characteristics of Lining Behavior

The term tunnel lining encompasses a broad range of concepts, materials, construction methods, and details. Several common characteristics pervade all systems:

The processes of ground pretreatment, excavation, and ground stabilization alter the preexisting state of stress in the ground, before the lining comes into contact with the ground. A tunnel lining is not an independent structure acted upon by well-defined loads, and its deformation is not governed by its own internal elastic resistance. The loads acting on a tunnel are ill defined, and its behavior is governed by the properties of the surrounding ground. Design of a tunnel lining is not a structural problem, but a ground-structure interaction problem, with the emphasis on the ground.

The most serious structural problems encountered with actual lining behavior are related to absence of support-inadvertent voids left behind the lining-rather than to intensity and distribution of load. In virtually all cases, the bending strength and stiffness of structural linings are small compared with those of the surrounding ground. The properties of the ground control the deformation of the lining, and changing the properties of the lining will not significantly change this deformation. The proper criterion for judging lining behavior is therefore not adequate strength to resist bending stresses, but adequate ductility to conform to imposed deformations. In short, the lining is a confined flexible ring (Thomas, 1996).

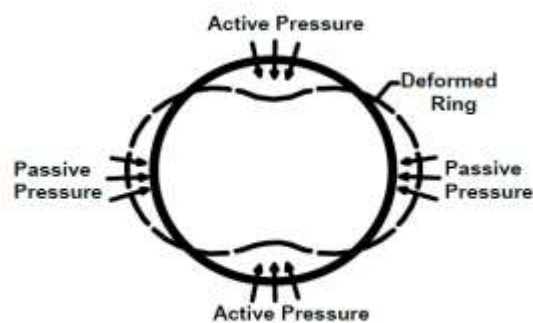


Figure 2.1-1: Flexible Lining deformation shape

2.1.2 Geotechnical Design Considerations

It is evident that the type and dimensions of a tunnel lining in the majority of civil engineering tunnels will be decided upon by the aforementioned factors prior to any consideration being given to the geotechnical environment. That is not to say that lining design in relation to the anticipated ground behavior, and the resultant loads and deformations of the support, is not a requirement as indeed it is. This is especially the case in the present-day environment of client-contractor partnerships and the implicit legal and liability aspects.

The first point to be established is that the design of a tunnel lining cannot be considered as a structure being subjected to well-defined values of loading as there is no absolute certainty of the actual ground behavior following excavation. Thus, the problem should be considered as one related to ground and structural behavior rather than simply one governed entirely by structural features.

The two lining properties which affect the generation of passive earth pressures are axial stiffness and flexural stiffness. (Kuesel, 1987) States that axial stiffness is much more important to a lining than flexural stiffness as increasing axial stiffness improves the efficiency of the compressive bulging action. Flexural stiffness, however, must also be of relevance as it is only by the deformation of the lining that earth pressures are generated, and a lining with an infinite flexural stiffness, i.e. rigid, would not deform at all.

A third factor which until recently was not a general consideration in lining design is the amount of stress relief and redistribution and consequent ground movement that occurs prior to support placement, and is known as the 'head-end effect'. The major implication of the head-end effect is that a support installed at a distance behind the face will not be subjected to the full loading and deformation as a result of the full overburden pressure.

Kersten (2008) studied about integration of geotechnical and structural design in tunneling. This paper described the methods that could be used to optimize the design of tunnels using a combination of reinforcement and support methods. Two case histories were presented to illustrate the integration of geotechnical and structural design methods. The first was a 12 m span two lane highway tunnel, excavated by top heading and benching in a very weak rock mass and the second was involved a 25 km long, 5.5 m diameter water supply tunnel through the Andes in Venezuela.

The methodologies described in this paper can be used with any modern numerical package provided the input and verification of results are done according to equations and procedures presented. The three-dimensional analyses were carried out using FLAC3D.

Design of tunnel linings has never been described comprehensively in a single document that allows the reader to follow all the derivations and the step by step calculations. To make this process as easy as possible to follow, the authors had included two case history based examples, one for a very shallow tunnel and the other for a very deep tunnel. These examples have been chosen to highlight the complex loading conditions that can occur under different geological and topographic conditions and how these complexities can be incorporated into a rational lining design. The support capacity diagrams were based on elastic analysis of the support elements and this implies that no tensile cracking or compressive crushing of the shotcrete or concrete elements was acceptable.

These simplified calculations allow the user to optimize the design of the lining components relatively quickly and efficiently. It had been demonstrated that, where tensile cracking became an important consideration, more sophisticated non-linear structural design approaches, which allow for crack development, could be used (Kersten, 2008).

2.1.3 Lining Analysis

The calculation of stresses is sensitive to assumptions of governing ground parameters and ground support distribution that are subject to wide uncertainty. Several years ago, Professor Duddeck of Germany undertook (Duddeck, 1982) a comparison of the German, Swiss, Norwegian, British, U.S., and Japanese methods for analyzing stresses in a circular tunnel liner ring. One conclusion from this comparison was that the stress in the lining depends on the country in which the calculations are made. However, the same stress can be obtained by any of the proposed formulas. A thoughtful appraisal of closed form tunnel lining ring analyses is given (Schmidt, 1984).

Early models of tunnel linings represented the lining as a string of interconnected pin-ended structural beams, and the ground as a series of radial (and somewhat tangential) springs. Beam-spring models are discussed extensively by Paul et al (Paul & Hendron, 1981) (Paul S. H., 1993).

Some models used elasto-plastic stiffness's and permitted varying ground properties in different layers. Such models are discussed in Gnilsen (Gnilsen, 1989).

Bakker and Blom (2009) studied about ultimate limit state design for linings of bored tunnels. They evaluated the situation with respect to lining design, some observations from engineering practice were discussed: the first, from the construction of the 2nd Heinenoord tunnel, where the damage to the lining during construction was above average; the second from the construction of the Green Hart tunnel where measurements show, that flexibility of the tube and the influence of interaction between structure and underground could also lead to critical loading conditions for the lining.

The paper neglects the other loading system like water and self-weight of the lining. The design was done in only ultimate state here the serviceability state was not considered.

They found out from the engineering observation of the two tunnel construction Heinenoord tunnel & Green Hart tunnel that the cost for a tunnel was for a large part, sometimes up to 40 % taken by the cost of the lining. Any cost saving in the lining would lead to a cost saving for the user of the infrastructure; therefore it was attractive to look at cost savings in the lining.

For the Ultimate Limit state under overburden pressure only, the preliminary conclusion was drawn that for overburden loading only, there was no necessity to account for lining bending moment; accounting for the hoop forces would suffice.

The construction loading was often dominant over the soil loading, and therefore the indicative design rules that relate overburden pressures with lining thickness couldn't be applied. What remains was that these equations explain that in general the thickness ratio should increase with the depth ratio, and with the diameter.

For the ULS verification at least three structural mechanisms should be analyzed: concrete compression due to jack forces, tangential bending due to incomplete grouting and tangential bending due to soil loads at the serviceability stage. It had been shown that for tunnels with a depth ratio h/D less than 3, that the current design practice to apply a linear elastic approach and the design rule of $d/D = 0.05$ was conservative (Bakker and Blom, 2009).

Lundberg (2009) studied on parametric study of irregularly shaped shotcrete shells. After the tunnel wall is sprayed with shotcrete, the resulting shotcrete surface will also be irregular and have a big variation in thickness due to uneven shotcrete coverage. Today, most design models assume that the shotcrete surface behaves as a concrete slab.

The weakness of the study was the assumption that shotcrete thickness would show a regular variation. In reality, the variation would be far more chaotic, which can be difficult to simulate.

The results show that the influence of the irregularity is highly dependent on the shotcrete thickness and shape and that the placement of the rock bolts is very important and the shotcrete quality is uniform, which is usually not the case in-situ (Lundberg, 2009).

2.2 Design of Bolting Systems

2.2.1 Introduction

As in the design of other support systems, the design of a rock bolting system depends on geotechnical discontinuities in the intact rock, material properties, the magnitude and distribution of the in-situ and induced stresses, support requirements such as acceptable roof deformation and life time of the opening, and the size and shape of the openings. For a complete and appropriate rock bolting system design, the following parameters must be determined properly:

- Bolt type and bolt length;
- Pattern and spacing of bolts & bolt diameter and anchor capacity; and
- Whether pretension should be applied or not. If pretension is desirable, what is the appropriate magnitude of the pretension?

2.2.2 Selection of Bolt Type

The selection of an appropriate bolt type to meet specific requirements at a lowest cost and maximum effectiveness could be a very confused task. Carefully examining the geological conditions and thoroughly

understanding the performance of different types of bolts under different settings are essential for a successful bolt type selection besides the guidelines for choosing bolt type (Smith, 1993).

The major factors that affect mechanical bolt performance are bleeding off of anchorage, deterioration of strata under the bearing plate through weathering, wear and damage of threads on bolts and damaged bearing plates, and the effect of blasting vibration on the anchorage. Once mechanical bolts are chosen, the following guidelines should be followed to ensure best performance (Peng, 1984):

- ✓ Bolt the roof as soon after excavation as feasible to prevent strata separation;
- ✓ Determine the optimum expansion shell and the optimum anchorage horizon through on-site pullout tests;
- ✓ Employ substantial structural components, i.e. high strength bolts and plates with low deformation characteristics, to reinforce the strata effectively;
- ✓ Ensure that all bolts work together by installing them with equal tension.

2.2.3 Bolt Length

Generally, bolt length is based on the total thickness of unstable strata, especially when bolts are used to control slope stability. But in underground situations, the maximum length of a rigid bolt is limited by the roof-to-floor height of the opening. However, some bolts such as 5/8 in. bolts can be bent before inserting into the boreholes and then easily straightened immediately afterward. Rebar's for full-length grouted bolts are notched such that they can be bent and straightened, but the bolt strength is reduced. Bolt length should be long enough to create beam building effects or keying effects (Lang, 1982).

The minimum bolt length should be the greatest of the following three:

- ✓ Twice the bolt spacing
- ✓ Three times the width of critical and potentially unstable rock blocks defined by the average discontinuity spacing in the rock mass; and
- ✓ For span less than 20 feet, bolt length of one half the span. For spans between 20 to 60 feet, linearly interpolate between 10 to 16 feet lengths, respectively. For excavations higher than 60 feet, sidewall bolts are one-fifth of wall height.

2.2.4 Bolting Spacing

Because bolt pattern and strata interact, factors affecting bolt spacing include strata thickness, location of weakness plane, roof condition, bolt tension, and bolt characteristics such as yield strength, length, and diameter.

2.3 Design Methods

There are three distinct approaches to arriving at solutions to rock and soil mechanics problems and these have been utilized in the design of tunnel linings, namely:

1. Analytical methods

2. Computational or numerical methods
3. Empirical methods.

2.3.1 Analytical Methods

In engineering mechanics, analysis consists of resolving a problem into its simplest elements, representing the problem by tractable equations and then solving them (Gibson, 1974). Analytical methods are generally regarded as those which produce closed-form or pseudo closed-form solutions. (Brown, 1987).

In respect of tunneling, the following list draws attention to those problems which have been successfully solved using analytical methods:

The distribution of elastic stresses and displacements in soil or rock around excavations of simple geometry, combined with thrusts and moments in the tunnel lining. Determination of the extent and distribution of stresses and displacements within failed or plastic zones around circular openings are subject to axisymmetric conditions.

This shows the restriction of analytical methods especially in the area of rock mechanics in relation to complex shaped openings combined with rock failure, non-axisymmetric of the problem and non-hydrostatic in situ stress conditions. As a result, other techniques have been developed for such problems.

E.g. In Bedded Beam-Spring Models, the tunnel will be modeled as elastically bedded circular structure. The elastic bedding (soil-structure contact) will be modeled using radially (and in special cases tangentially) arranged springs. For the case of tunnels at shallow depth ($h < 2D$), the top part (crown) will be free of bedding whereas for tunnels at great depth ($h > 3D$) the resistance of the crown will be considered.

2.3.1.1 Continuum Method

The tunnel in deep depth of ground, in which the tunnel is constructed, is idealized as a continuum, i.e. there are no discontinuities in the material. The method assumes that the ground is an infinitely large thin section with a hole at the center.

This calculation method allows the interpretation of the deformation and strains in the ground. It is used in shallow tunnels in weak rock as a partial continuum method and for deep tunnels in weak rock as a continuum method with a bedded crown area.

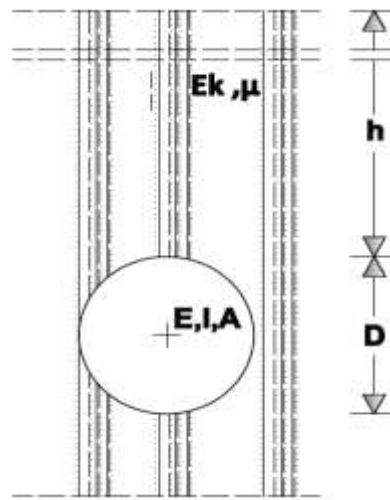


Figure 2.3-1: Continuum model

Jing-Zhou Wen (2014) studied about an analytical model for fully grouted rock bolts in soft rock tunnel. The bolt's axial displacement differential equation is established according to the stress equilibrium in a small section of a rock bolt and the shear stress transfer mechanism of anchorage body interfacial layer between rock bolt and soft rock mass.

The weakness of the paper was that it was limited to only soft ground condition and took the assumption the reaction of the shear stress on interface of the fully grouted bolt on the surrounding rock is assumed to be symmetric radial action.

In order to analyze the mechanical characteristics of the fully grouted rock bolt in soft rock tunnel, this paper develops a simplified analytical model of rock bolt which can consider the factor of interface layer between rock bolt and rock mass. The analytical model can be developed according to the stress transfer mechanism and deformation coordination of anchorage body interfacial layer between rock bolt and surrounding rock.

The fully grouted rock bolt in situ can be installed in soft rock mass, which has in different regions. The mechanical state of bolt-shotcrete supporting is obviously related with the released displacement of the soft rock mass (Jing-zhou, 2014).

2.3.2 Computational or Numerical Methods

As stated, the assumptions made in formulating an analytical solution are in general, far too simple to allow it to retain a degree of relationship to the problem in question. Thus, some other technique is required whereby these assumptions are removed so that more realistic relationships between the parameters of the problem can be used. This usually means that the solution to the resultant equations cannot be obtained by simple mathematical solution and that a numerical or computational method of solution is required. In general, there are two main classes of numerical solutions to boundary value problems such as that of a support being placed in a tunnel:

Differential methods: These involve assumptions being made throughout the entire problem area to some finite boundary where a normality condition exists.

Hybrid methods: A combination technique involving both differential and integral methods has also been developed and it represents a powerful design tool in certain applications. This is referred to as a hybrid type.

The Finite Element Method (FEM) is an example of a differential method and the Boundary Element Method (BBM) is an integral method whilst the Distinct Element Method (DEM) is of the hybrid type. Numerical design methods have been developed to incorporate non-linear ground and lining behavior, anisotropic ground behavior coupled with stratified formations and non-hydrostatic in situ stresses. Nevertheless, they are expensive, time consuming and highly specialized design methods and their application needs careful consideration not only on technical grounds but also economically in terms of time and cost.

Singh (2009) studied about Assessment of tunnel instability using a numerical approach. This paper outlined the application of numerical modeling to predict deformation and stability of tunnel to be excavated in Bansagar, M.P., India. Numerical analysis for the stress-strain distribution of the tunnel excavation and support systems was also carried out. In order to simulate the excavation of tunnel (NATM) at a depth of 150 m below the ground, a series of finite element analyses using Mohr-coulomb elasto-plastic constitutive model has been carried out using PLAXIS 2D.

The drawbacks of the paper were the assumption Intermediate principal stresses have been ignored, as their influence would be almost negligible. The effect of the other small localized opening has been also ignored. The support system used was only lining and bolt was not included.

The paper concluded that Usage of empirical indexes in numerical modeling provided more realistic results. The strength parameters were estimated from the empirical analysis, and they were compiled for use in the numerical modeling; the numerical results showed that the walls and the roof of the excavation were more critical. The stability problem of these sections can be overcome by installation of light support systems. The plastic failure observed in the weakness zones could be prevented by installing lining of 0.275 m thickness (Singh, 2009).

F. Perman (2007) studied about Numerical analyses of shotcrete reinforcement. This paper was aimed at increasing the understanding of how to model and evaluate the behavior of shotcrete in numerical models. A parametric study also was conducted using two- and three-dimensional continuum models.

This study comprised two- (2D) and three-dimensional (3D) continuum numerical analysis, using the finite difference programs FLAC and FLAC3D. In conclusion, it was judged that a 2D-model was sufficient only for analysis of deformation and stability (Olofsson, 2007).

2.3.3 Empirical Methods

An empirical design method is essentially one that is based on in situ observations or measurements made on models from the prototype rock (Obert, 1967). A simple example of this is the use of a database of measured

tunnel closures in a variety of ground conditions from which the designer can select a lining type on the basis of its past performance.

The most important empirical design method in relation to tunneling is possibly that of rock mass classification which involves assessing the competency of the rock mass by determining joint spacing's, joint numbers, rock quality designation (RQD) and various other parameters and combining them to form some arbitrary rock index. From this index, an appraisal of support requirements or the stand-up time of the excavation can, very often, be obtained from simple design rules or monograms which again may be based on past experience.

Lucian C., Wangwe E. M. (2013) studied about the usefulness of rock quality designation (RQD) in determining strength of the rock. The paper illustrated concepts used in determining the strength rock mass using the RQD technique. The study area was Nyakunguru Tanzania; the method used in this project was Diamond drill method (DD) in which the length of drilled core was recovered and arranged well in core tray for further investigation/analysis. They drilled the rock using easy field equipment tools from depth 40m to 314.5 meter and found a rock class of very poor to excellent.

In this paper the rock quality designation, RQD had proved useful in logging rock core. The results of RQD in this paper had shown that the qualities of rock masses range from very bad to very good quality in a continuous and progressive manner, which gives a better representation of the actual quality of rock masses. The use of RQD lowers operating costs by simply employing simple tools to assess the quality of the rock masses.

However, RQD is useful, the paper still didn't consider the limitation of the RQD like no joint orientation and information about <10cm cores (Lucian & Wangwe, 2013).

2.4 Optimization

In the theory of optimization, two major classes of optimizations could be identified, based on the number of objective functions used for optimization, as single-objective optimization and multi-objective optimization. In a single-objective optimization, simply searching for the best will be made based on a single objective function, where as in multi-objective optimization the searching is needed to be supported by Pareto dominance. And in the sense of mechanics, structural engineering or civil engineering, there are two main categories of optimizations which are those of structural optimizations and non-structural optimizations. In general however, both the categories involve searching algorithms, in one way or another, operating on a collection of design spaces. Further, structural optimization is classified into shape-optimization and topology-optimization.

Design optimization is an important engineering design activity. In general, design optimization determines values for design variables such that an objective function is optimized while performance and other constraints are satisfied (Papalambros and Wilde, 2000) (Ravindran, 2006) (Arora, 2004) . The use of design optimization in engineering design continues to increase, driven by more powerful software packages and the formulation of new design optimization problems motivated by the decision-based design (DBD) framework (Hazelrigg, 1998) (Renaud, 2006).

A different approach is to classify design optimization models based on the nature of the problem being solved. (Brochtrup, 2006) Propose a three-field classification scheme for product design optimization problems. The three fields are the product type (either single product or product family), variables type (engineering, manufacturing, or price), and objective function type (performance, cost, or profit).

Lijuan (2015) studied about an optimization model of tunnel support parameters. An optimization model was developed to obtain the ideal values of the primary support parameters of tunnels, which are wide-ranging in high-speed railway design codes when the surrounding rocks are at the III, IV, and V levels. First, several sets of experiments were designed and simulated using the FLAC3D software under an orthogonal experimental design. Six factors, namely, level of surrounding rock, buried depth of tunnel, lateral pressure coefficient, anchor spacing, anchor length, and shotcrete thickness, were considered. Second, a regression equation was generated by conducting a multiple linear regression analysis following the analysis of the simulation results. Finally, the optimization model of support parameters was obtained by solving the regression equation using the least squares method.

Six main factors that affect the stability of surrounding rocks were selected for optimization. These factors included surrounding rock level, depth, lateral pressure coefficient, anchor length, anchor spacing, and shotcrete thickness. Five levels of each factor were considered in several orthogonal experiment designs. Satisfactory uniformity results were obtained.

An approximate linear model of support parameters and cost-effectiveness was obtained on the basis of a multiple linear regression analysis following a number of numerical experiments. In addition, the effects of the support parameters on the stability of surrounding rocks were analyzed (Lijuan, 2015).

Chao Su (2014) studied about Optimization design and application of underground reinforced concrete bifurcation pipe. This paper presents an optimization mathematical model of underground reinforced concrete bifurcation pipe structure according to real working status of several common pipe structures from real cases. Then, an optimization design system was developed based on Particle Swarm Optimization algorithm. Furthermore, take the bifurcation pipe of one hydropower station as an example: optimization analysis was conducted, and accuracy and stability of the optimization design system were verified successfully (Chao Su, 2014).

J. Pe´rez-Romero (2007) studied about design and optimization of the lining of a tunnel in the presence of expansive clay levels. This paper described the study made of the Trasvasur tunnels (Canary Islands, Spain), which were excavated in rocky formations of volcanic origin presenting metric levels of expansive clays. The work done has shown that when building tunnels in the presence of expansive clays it is advisable to use circular or similar cross-sections. It has also been found to be advisable to seal off the excavation as quickly as possible to prevent alteration due to the decompression of the expansive clay levels and their absorbing water from the tunnel itself (Pe´rez-Romero, 2006).

2.5 Tunnel Modeling Methods

The prediction of the rock response to the excavation of tunnel is a complex engineering problem. The interest is to assess the stability of tunnel face at intrinsic stage. At the design analysis stage the choice is made between equivalent continuum and discontinuum approach (Shen.B. & N.Barten, 1997).

A. Equivalent continuum: the rock mass is treated as a continuum with equal in all directions input data for strength and deformability property, which define a given constitutive relation for the medium elastic, elasto-plastic etc.

B. Discontinuum approach: the rock mass is represented as a discontinuum and most of the attention at the design stage is devoted to the characterization of the rock elements, the rock joints and discontinuities. The modeling approach consists in considering the blocky nature of the system being analyzed. Each block may interact with the neighboring blocks through the joints (Barla, 2000).

Joints can create loose blocks near the tunnel profile and cause local instability. Joints weaken the rock and enlarge the displacement zone caused by excavation and joints change the water flow system in the vicinity of the excavation. The use of discontinuum modeling has been gaining progressive attention in tunnel engineering mainly through the use of the UDEC and 3DEC codes, for 2D and 3D discontinuum modeling respectively. However discontinuum modeling is not being used as extensively as continuum methods and is considered to be a relatively new and "not-yet-proven" numerical technique to apply for analysis and design in rock engineering projects (Barla, 2000).

2.6 Finite Element Analysis

The finite element method (FEM) is a numerical technique used for solving problems which are described by partial differential equations or can be formulated as functional minimization. A domain of interest is represented as an assembly of finite elements. Approximating functions in finite elements are determined in terms of nodal values of a physical field which is sought. A continuous physical problem is transformed into a discretized finite element problem with unknown nodal values. For a linear problem a system of linear algebraic equations should be solved. Values inside finite elements can be recovered using nodal values (Nikishkov, 2004).

The Finite Element Analysis (FEA) is a numerical method for solving problems of engineering and mathematical physics. FEA are useful for problems with complicated geometries, loadings, and material properties where analytical solutions cannot be obtained. (Anon, 2017)

Analytical Solution do like stress analysis for trusses, beams, and other simple structures. They carried out based on dramatic simplification and idealization. Mass concentrated at the center of gravity beam simplified as a line segment (same cross-section). Design is based on the calculation results of the idealized structure & a large safety factor given by experience.

In Finite element analysis design geometry is a lot more complex and the accuracy requirement is a lot higher. We need to understand the physical behaviors of a complex object (strength, heat transfer capability, fluid flow, etc.) to predict the performance and behavior of the design, to calculate the safety margin, to identify the weakness of the design accurately and to identify the optimal design with confidence (Nikishkov, 2004).

2.7 Design of Data for Optimization

Design of data experiments was invented by Ronald A. Fisher at Rothamsted Experimental Station, an agricultural research station 25 miles north of London. In Fisher's first book on design of experiments he showed how valid conclusions could be drawn efficiently from experiments with natural fluctuations such as temperature, soil conditions, and rain fall, that is, in the presence of nuisance variables. The known nuisance variables usually cause systematic biases in groups of results (e.g., batch-to-batch variation). The unknown nuisance variables usually cause random variability in the results and are called inherent variability or noise.

Although the experimental design method was first used in an agricultural context, the method has been applied successfully in the military and in industr. Besse Day, working at the U.S. Naval Experimentation Laboratory, used experimental design to solve problems such as finding the cause of bad welds at a naval shipyard during World War II. George Box, employed by Imperial Chemical Industries before Coming to the United States, is a leading developer of experimental design procedures for optimizing chemical processes.

W. Edwards Deming taught statistical methods, including experimental design, to Japanese scientists and engineers in the early 1950s at a time when made in Japan meant poor quality. Genichi Taguchi, the most well-known of this group of Japanese scientists, is famous for his quality improvement methods. One of the companies where Taguchi first applied his methods was Toyota. Since the late 1970s, U.S. industry has become interested again in quality improvement initiatives, now known as —Total Quality and —Six Sigma programs. Design of experiments is considered an advanced method in the Six Sigma programs, which were pioneered at Motorola and GE (Telford, 2007).

The fundamental principles in design of experiments are solutions to the problems in experimentation posed by the two types of nuisance factors and serve to improve the efficiency of experiments. Those fundamental principles are, as it is shown and described in (Telford, 2007): The fundamental principles in design of experiments are solutions to the problems in experimentation posed by the two types of nuisance factors and serve to improve the efficiency of experiments. Those fundamental principles are, as it is shown and described in (Telford, 2007) . These are randomization, replication, blocking, orthogonally and factorial experimentation.

There are two common tools available to the practitioner for assisting with the problem statement. These are not actually problem-solving tools but rather are problem identification tools. Although these tools were originally developed for the manufacturing environment, they can easily be adapted to other sectors of industry. These tools are (Christopher, 2004):

✓ Fishbone chart

✓ Pareto chart

The Fishbone chart- the purpose of this chart is to identify the cause of the problem or effect of interest. The Fishbone chart looks similar to the bones of a fish. The head of the fish is labeled with the problem or effect. Each major bone coming out of the spine is a possible source or cause of the problem (Christopher 2004).

The first thing to do in parameter identification is to find a cause-effect relationship which could be realized through the utilization of fish-bone diagrams, in a very smart way. Using the fish bone diagram the causal effect of the tunnel design elements is listed. From those bolt and lining takes the highest effect on tunnel design safety and cost. Consequently, the parameters that affect are identified.

The Pareto chart it may turn out that there are several sources or causes of the problem or problems of interest. The concept of the Pareto chart was developed by the Italian economist Wilfredo Pareto, who postulated that only a few factors are the cause of many problems. This is frequently referred to as the 80–20 rule. The 80–20 rule states 80% of the problem is caused by 20% of the factors. The 80 and the 20 are not magic numbers but seem to be approximately observed in many instances (Christopher, 2004).

CHAPTER 3 NUMERICAL MODELINGS

3.1 Introduction

In order to achieve the objective, the study employs the finite element and polynomial modeling. The data are analyzed using numerical analysis and the software used for the study is Plaxis 2D. In addition to this, the objective function is modeled with polynomial surface model and then optimization is done. The following section describes about the finite element method modeling and the detail optimization process.

3.2 Design of Data for Optimization

Using Fish bone and Pareto dominance of parameters for tunnel support system design, the cumulative effect of the parameters are taken then the parameters with higher effect on tunnel support design are selected for the design of data for optimization. In addition to this, the sensitivity of concrete lining effect on cost and design is of great effect as described by different researchers. Due to this number of rock bolt and lining thickness have been selected as causal parameters. Design of data for optimization involves the generation of the combination of parameters in which the study become easy to understand in a better way. For this particular research the study level is two level of design of data and it is shown below in the table.

Table 3.1: Range of parameters

Parameters	Lower	Higher
Number of Rock Bolt	7	23
Concrete Lining Thickness [cm]	10	40

Table 3.2: Design of optimization

S.No	Number of Bolt (A)	Lining Thickness (B)	Combination Code
1	A ₁	B ₁	A ₁ B ₁
2	A ₁	B ₂	A ₁ B ₂
3	A ₁	B ₃	A ₁ B ₃
4	A ₁	B ₄	A ₁ B ₄
5	A ₁	B ₅	A ₁ B ₅
6	A ₂	B ₁	A ₂ B ₁
7	A ₂	B ₂	A ₂ B ₂
8	A ₂	B ₃	A ₂ B ₃
9	A ₂	B ₄	A ₂ B ₄
10	A ₂	B ₅	A ₂ B ₅
11	A ₃	B ₁	A ₃ B ₁
12	A ₃	B ₂	A ₃ B ₂
13	A ₃	B ₃	A ₃ B ₃
14	A ₃	B ₄	A ₃ B ₄
15	A ₃	B ₅	A ₃ B ₅
16	A ₄	B ₁	A ₄ B ₁
17	A ₄	B ₂	A ₄ B ₂
18	A ₄	B ₃	A ₄ B ₃
19	A ₄	B ₄	A ₄ B ₄
20	A ₄	B ₅	A ₄ B ₅
21	A ₅	B ₁	A ₅ B ₁
22	A ₅	B ₂	A ₅ B ₂
23	A ₅	B ₃	A ₅ B ₃
24	A ₅	B ₄	A ₅ B ₄
25	A ₅	B ₅	A ₅ B ₅

3.3 Numerical Modeling and Analysis

3.3.1 Geometry of the Section

Unlike, the bored tunnel, the geometry of the tunnel adopted for this particular type of ground condition is curvilinear shape of tunnel and the excavation considered here is in full excavation basis. The tunnel section used at this location is enlarged so as to accommodate the train speed and secondary facilities for operation.

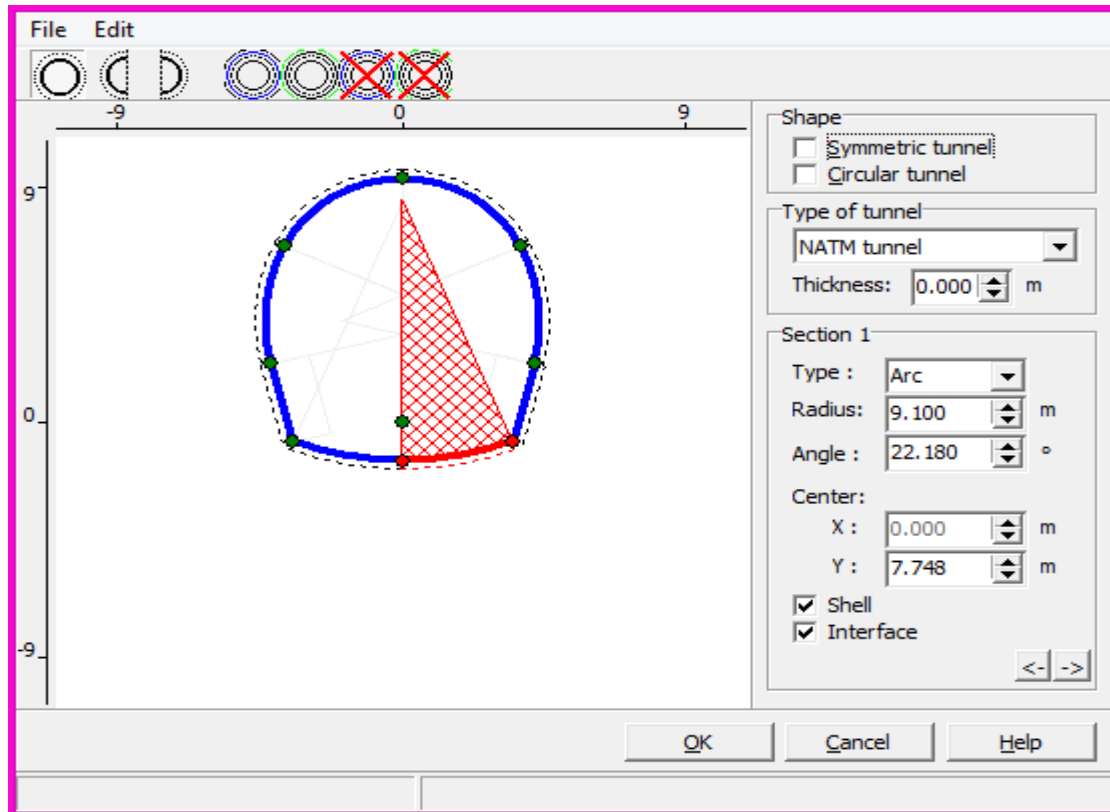


Figure 3-3-1: Tunnel designer window and geometry of the tunnel

3.3.2 Boundary Condition

Apart from total fixities which is a combination of both ($u_x=u_y=0$), standard fixities are selected with the rules for analysis. Vertical geometry lines for which the x-coordinate is equal to the lowest or highest x-coordinate in the model obtain a horizontal fixity ($u_x = 0$). Horizontal geometry lines for which the y-coordinate is equal to the lowest y-coordinate in the model obtain a full fixity ($u_x = u_y = 0$). Standard fixities, being a convenient and fast input option for many practical applications, has been chosen.

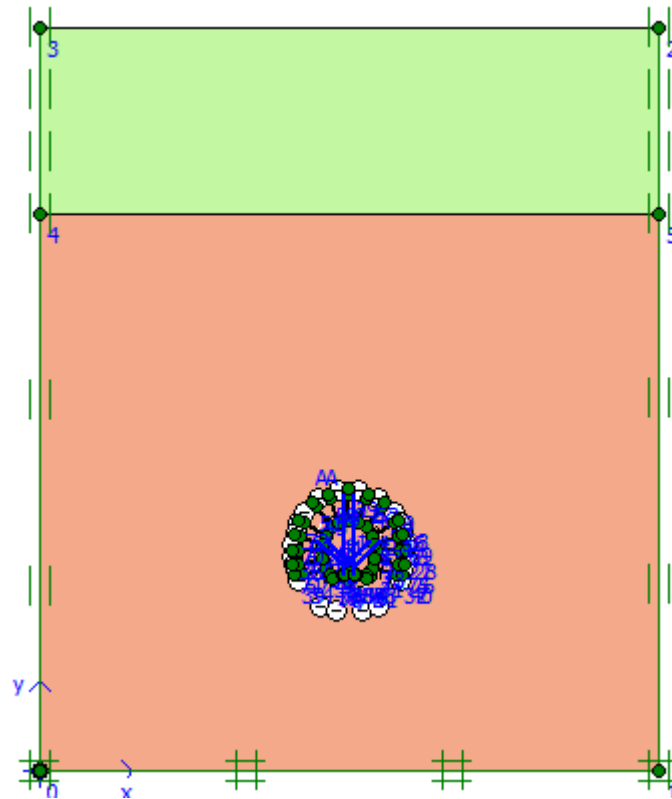


Figure 3-3-2: Boundary condition

3.3.3 Parameter Material Property

By using the design of data for optimization, tunnel support parameters that affect the design of tunnel support are identified. And then the most influential parameters are selected. The interest of this thesis work is on the tunnel supports' number of bolt and lining thickness best combination determination. The parameters used for the development of design of data for optimization are number of rock bolt and thickness of concrete lining.

The study considered curvilinear shape of tunnel. In addition to being specific in its scope, there are some set out/ fixed parameters in order to reduce the number of parameter combinations and possible simulation runs required to deduce on the optimization of tunnel support design. Due to reasons of limited resource and the available research duration, the following parameter properties are considered in this research:

3.3.3.1 Ground Condition Data

The ground condition data is taken from the AKH project during preliminary soil investigation. Along the T04 the different support classes are provided based on the ground condition. From those support classes provided along the tunnel, the critical section (A4) is taken for the study. The critical section is located at chainage of 178 + 300. The soil data used for the numerical computation is in drained condition. Here the long term effect is considered for parameter correlation as well as the optimized combination modeling verification.

Table 3.3 TO4 Support types ((Yepi.Merkezi, 2014)

Support Class	A1	A2	A3	A4
RMR	81-100	61-80	41-60	21-40
Shotcrete	50mm	75mm	100mm	200 mm
Rock bolts	-	-	7	11
Lining	50mm	75mm	100mm	300 mm

Table 3.4 : Geological data for drained condition (Yepi.Merkezi, 2014)

Rock Type	Rock Formation	Chainage [m]	Overburden (m)	γ (KN/m ³)	C [Kpa]	ϕ	E [Mpa]	V	Support Class Type
A	Ashangai "Tas"	178+262 to 178+362	80	24	400	37	250	0.25	A4 Enlarged
B-C	Ashangai "Tas"	@178+300		22	100	36	160	0.28	

3.3.3.2 Structural Support Data

Table 3.5 : Shotcrete & lining characteristics (Yepi.Merkezi, 2014)

Support Type	Shotcrete	Final Lining
Shotcrete layer thickness (m)	0.2	0.3
Shotcrete modulus of elasticity E_c (GPa)	15	31
Shotcrete Poisson's coefficient ν (-)	0.2	0.2
Axial stiffness (MN/m)	3000	9300
Flexional stiffness (MNm ² /m)	10	69.75
Self-weight (kN/m/m)	5	7.5

Table 3.6: Rock bolt characteristics

Chainage [m]	Length [m]	Bar diameter[mm]	Longitudinal Spacing(m)	Ultimate capacity(KN)	EA[KN/m]
178+262 to 178+362	4	26	1	230	$7.08 * 10^4$
Grout, EA(KN/m)	100,000				

Table 3.7 : Support for A4 enlarged section

Rock Support Class	RMR	Excavation Type	Advance per round	Shotcrete	Rock bolts	Final lining
A4	21-40	Full Excavation	@1.0-1.5 m	C25/30 200mm	12/11 \varnothing 26 L=4m @1.0m	300mm inner concrete lining C25/30

Table 3.8: Ballast property

Parameter	Sub Ballast	Ballast
Thickness(m)	0.4	0.4
Unit Weight(KN/m ³)	19	20
Cohesion, C(Kpa)	1	1
Friction Angle,	45	48
Modulus of Elasticity, E(Mpa)	150	200
Poisson Ratio, ν	0.3	0.25

The railway line pass through the tunnel is a single track type with the sleeper dimension $2.4*0.21*0.22$ m and the sleeper is a precast concrete with C35/45 grade and has modulus of elasticity 33Gpa. The rail is UIC54 type has weight 54 kg/m.

3.3.4 Loading Condition Considered

The loads considered in numerical simulations are the overburden weight of the soil, self-weight of rail and sleeper in addition to the ballast and sub ballast load. The pre stress force applied on the rock bolt is 80% its ultimate capacity.

Table 3.9 : Unit cost of bolt and lining (Yepi.Merkezi, 2014)

Item of work	Unit	Unit cost (\$)	Description
Rock Bolt	No	290	The cost includes all cost of installation of the pre stress bolt
Concrete Lining	m ³	3100	The cost at crown and invert is different so the average cost is taken and includes all cost up to concrete casting

3.3.5 Mesh Generation

A local refinement has been achieved by selecting clusters, lines or points and selecting a local refinement option. Hence, by selecting clusters and geometry lines, to get precision of the numerical analysis result, clusters and lines have been refine.

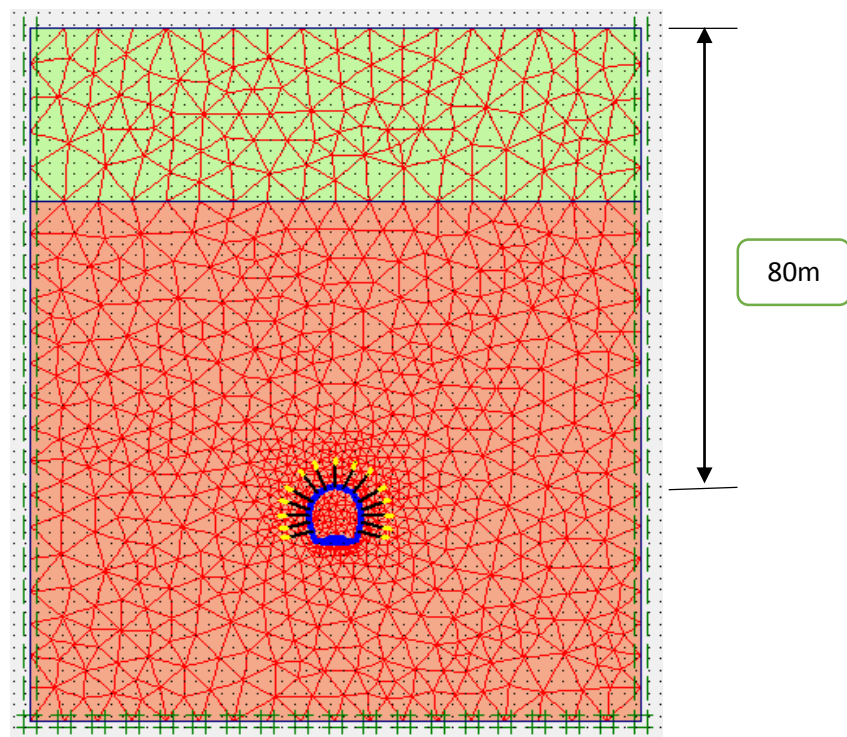


Figure 3-3-3: Mesh generation

3.3.6 Initial Condition

The initial conditions consist of two different modes: One mode for the generation of initial water pressures (water conditions mode) and one mode for the specification of the initial geometry configuration and the generation of the initial effective stress field (geometry configuration mode). Since there is no ground water table

at tunnel location, the ground water table remains at the base of the model geometry. But the initial effective stress generated is 2820Kpa and the numerical modeling analysis output in a figure is found in appendix.

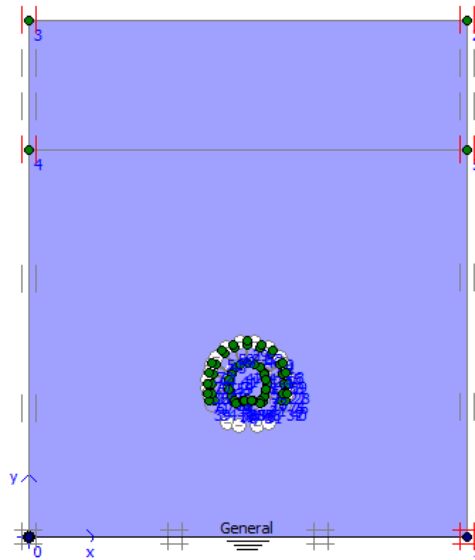


Figure 3-3-4: Ground water condition

Table 3.10: Parameter combination

S. No	Combination Code	Number of Bolt	Lining Thickness [cm]
1	A ₁ B ₁	7	10
2	A ₁ B ₂	7	20
3	A ₁ B ₃	7	25
4	A ₁ B ₄	7	30
5	A ₁ B ₅	7	40
6	A ₂ B ₁	11	10
7	A ₂ B ₂	11	20
8	A ₂ B ₃	11	25
9	A ₂ B ₄	11	30
10	A ₂ B ₅	11	40
11	A ₃ B ₁	15	10
12	A ₃ B ₂	15	20
13	A ₃ B ₃	15	25
14	A ₃ B ₄	15	30
15	A ₃ B ₅	15	40
16	A ₄ B ₁	19	10
17	A ₄ B ₂	19	20
18	A ₄ B ₃	19	25
19	A ₄ B ₄	19	30
20	A ₄ B ₅	19	40
21	A ₅ B ₁	23	10
22	A ₅ B ₂	23	20
23	A ₅ B ₃	23	25
24	A ₅ B ₄	23	30
25	A ₅ B ₅	23	40

3.3.7 Calculation

Staged construction is selected since it the most important type of loading input. In addition, staged construction enables an accurate and realistic simulation of various loading, construction and excavation processes. The loading input are overburden, shotcrete, bolting, and lining. In staged construction zero value time is provided for loading input. The shotcrete thickness of 20 cm is modeled together with all 25 data combinations.

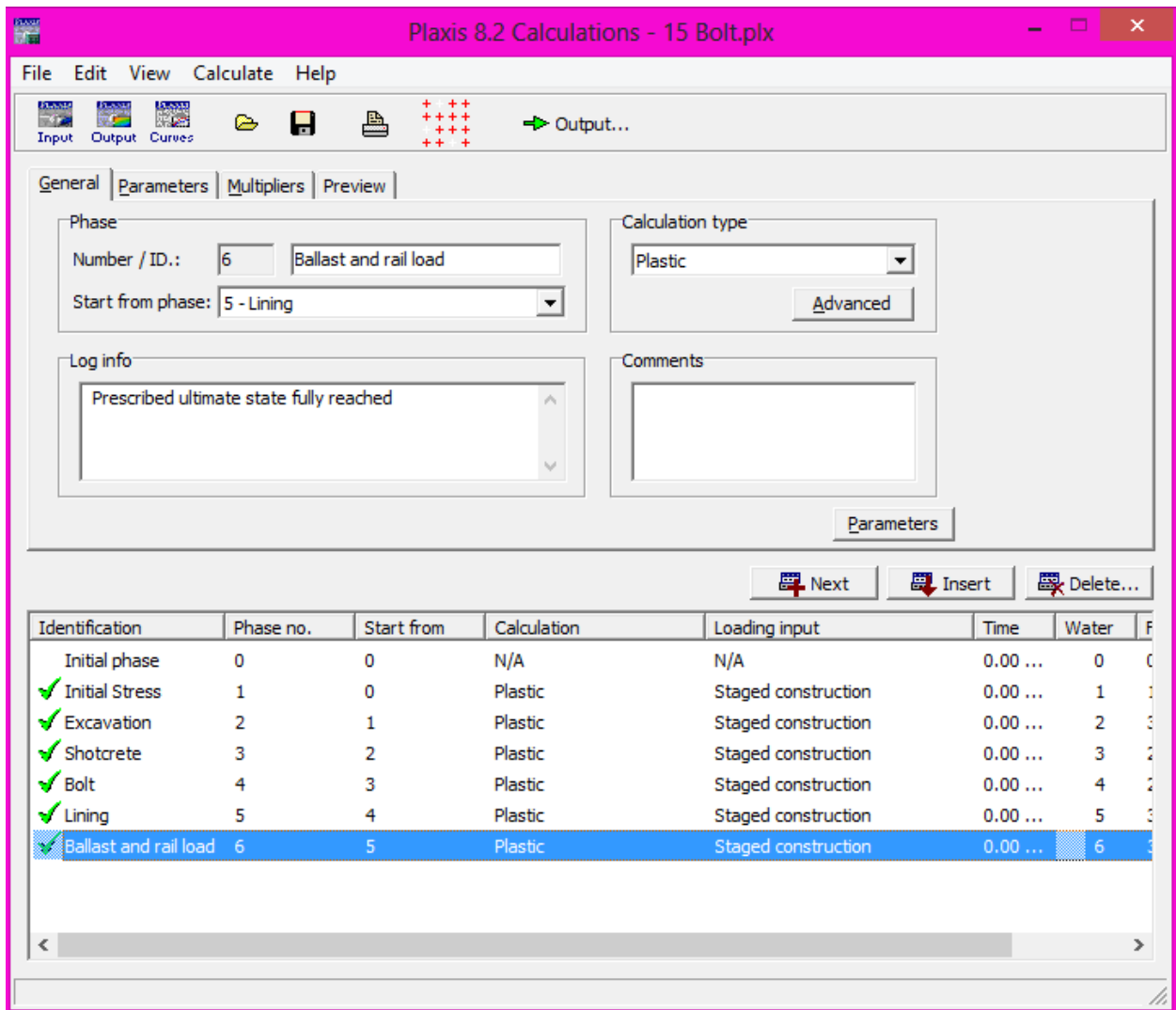


Figure 3-3-5: Calculation

3.4 Numerical Out Put Validation

3.4.1 Stress Around Openings

✓ In situ or pre construction stress

The validation is performed for the section at chainage of 179 +300 m.

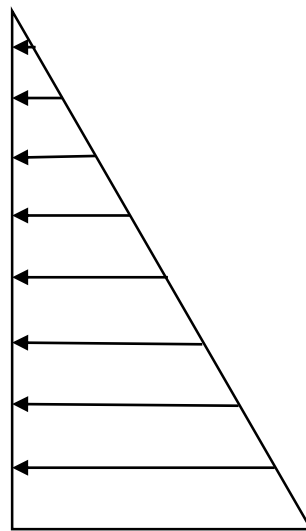
The vertical normal stress component at appoint is given by:

$$P_v = \gamma * h$$

3.1

Where, γ = unit weight of soil (KN/m³)

h = depth of the overburden, (m)



$$P_v = P_{zz} = \gamma * h$$

$$\text{Vertical Stress} = P_{zz} = \gamma * h = 22 \text{ KN/m}^3 * 30\text{m} + 24 \text{ KN/m}^3 * 90\text{m} = 2820\text{KPa} \approx 2820 \text{ Kpa}$$

This is equivalent to the numerical result.

Horizontal stress = $P_{xx} = k * P_v$, Where k = at rest earth pressure coefficient

$$P_{xx} = k * P_v = 0.43 * 2820\text{KPa} = 1212\text{Kpa} \approx 1210\text{Kpa}$$

This is similar to the numerical analysis result.

3.4.2 Displacement

For the most part, ground displacements around a rock tunnel can be estimated from elastic theory or calculated using any of a number of computer programs. Elastic theory allows an approximate calculation of the ground displacements around a round tunnel in rock. The approximate radial displacement at a point directly around a tunnel in elastic rock is given by:

$$U = \frac{Pz(1+\nu)a^2}{E r} \tag{3.2}$$

Where:

u = Radial movement, in.

a = Radius of opening

r = Radius to point of interest

E = Rock mass modulus

$$U = \frac{Pz(1+\nu)a^2}{E r} = \frac{2820(1+0.265)*5.4}{205000} = 9.13 \text{ cm} \approx 6.35 \text{ cm}$$

The result of displacement using numerical analysis is lower than the elastic theory analytical calculation. This is because the numerical analysis result is found from plastic analysis and even the shape is not circular. In addition to this, the rock condition is very poor and it is weathered rock.

3.5 Parameter Optimization Process

For the optimization process, the numerical analysis result has been taken in order to formulate polynomial model for each parameter combination. The pre stress bolt reduces the convergence of the tunnel by increasing contact between the rocks. This is the primary concern of the optimization that accomplishes this condition. From numerical computation using Plaxis 2D, the following result has been found.

Table 3.11: Displacement from numerical analysis results

S. No	Combination Code	Number of Rock Bolt	Final Concrete Lining Thickness[cm]	Total Displacement[mm]
1	A ₁ B ₁	7	10	65.14
2	A ₁ B ₂	7	20	64.62
3	A ₁ B ₃	7	25	64.4
4	A ₁ B ₄	7	30	64.21
5	A ₁ B ₅	7	40	63.96
6	A ₂ B ₁	11	10	64.75
7	A ₂ B ₂	11	20	64.23
8	A ₂ B ₃	11	25	64.01
9	A ₂ B ₄	11	30	63.82
10	A ₂ B ₅	11	40	63.57
11	A ₃ B ₁	15	10	64.13
12	A ₃ B ₂	15	20	63.61
13	A ₃ B ₃	15	25	63.4
14	A ₃ B ₄	15	30	63.2
15	A ₃ B ₅	15	40	62.95
16	A ₄ B ₁	19	10	63.34
17	A ₄ B ₂	19	20	62.82
18	A ₄ B ₃	19	25	62.63
19	A ₄ B ₄	19	30	62.41
20	A ₄ B ₅	19	40	62.16
21	A ₅ B ₁	23	10	62.39
22	A ₅ B ₂	23	20	61.87
23	A ₅ B ₃	23	25	61.66
24	A ₅ B ₄	23	30	61.46
25	A ₅ B ₅	23	40	61.21

The above table is represented using polynomial linear model of degree two of the form:

$$Q(x, y) = 66.09 + 0.000135x - 0.07211y - 0.005763x^2 - 1.239 * 10^{-9} xy + 0.0006522y^2 \quad 3.3$$

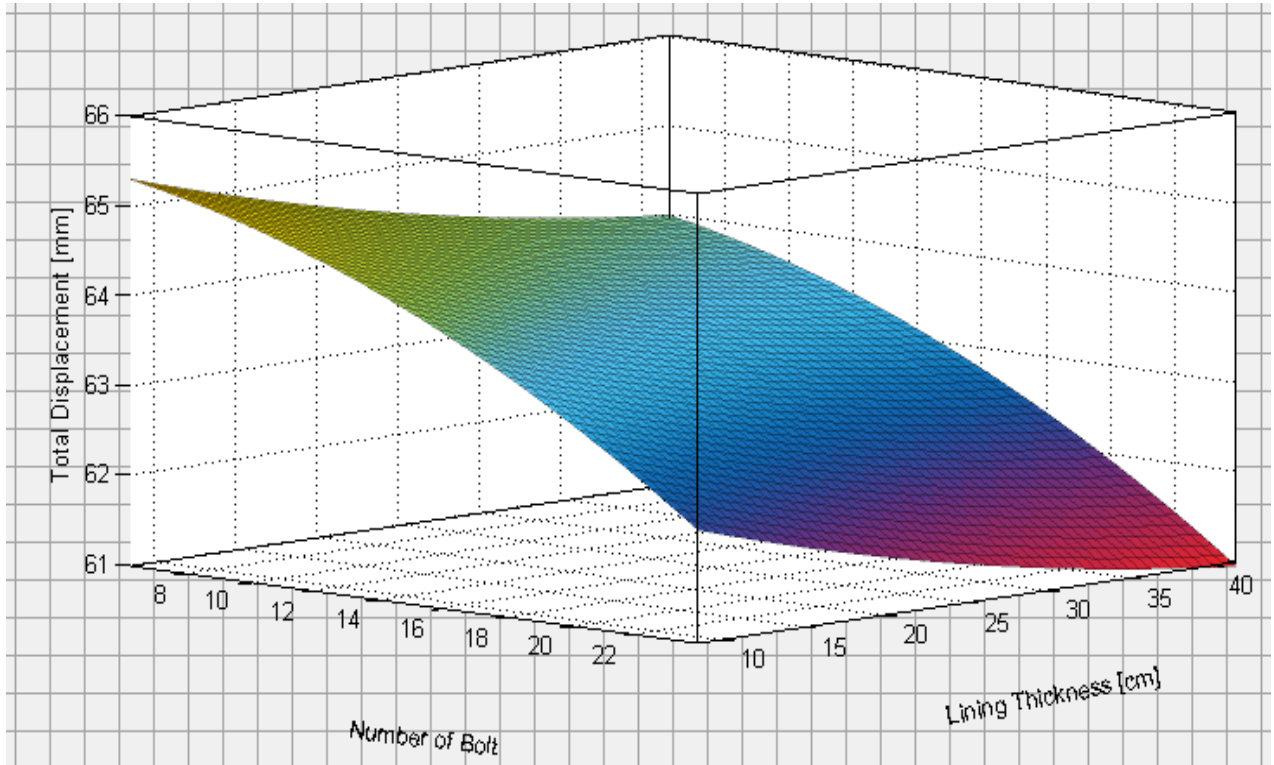


Figure 3.5-1: Tunnel settlement graphical presentation from numerical result

Table 3.12: Design of experiment data of cost for Polynomial Model

S. No	Number of Rock Bolt	Concrete Lining Thickness [cm]	Total Cost Rock Bolt & Concrete Lining
1	7	10	12,570
2	7	20	23,110
3	7	25	28,380
4	7	30	33,650
5	7	40	44,190
6	11	10	13,730
7	11	20	24,270
8	11	25	29,540
9	11	30	34,810
10	11	40	45,350
11	15	10	14,890
12	15	20	25,430
13	15	25	30,700
14	15	30	35,970
15	15	40	46,510
16	19	10	16,050
17	19	20	26,590
18	19	25	31,860
19	19	30	37,130
20	19	40	47,670
21	23	10	17,210
22	23	20	27,750
23	23	25	33,020
24	23	30	38,290
25	23	40	48,830

Polynomial model of degree four for the cost parameter correlation using the above table becomes:

$$\begin{aligned}
 F(x,y) = & -2.292 * 10^{-10} + 290 * x + 1054 * y - 1.567 * 10^{-12}x^2 + 4.729 * 10^{-13}x * y - 2.873 * 10^{-12} y^2 \\
 & + 1.061 * 10^{-13} x^3 - 6.301 * 10^{-14} x^2 * y + 1.275 * 10^{-14} x * y^2 + 7.692 * 10^{-14} y^3 \\
 & - 2.216 * 10^{-15} x^4 + 1.34 * 10^{-15} x^3 * y - 4.769 * 10^{-29} x^2 * y^2 - 1.072 * 10^{-16} x \\
 & * y^3 - 7.409 * 10^{-16} y^4
 \end{aligned}
 \tag{3.4}$$

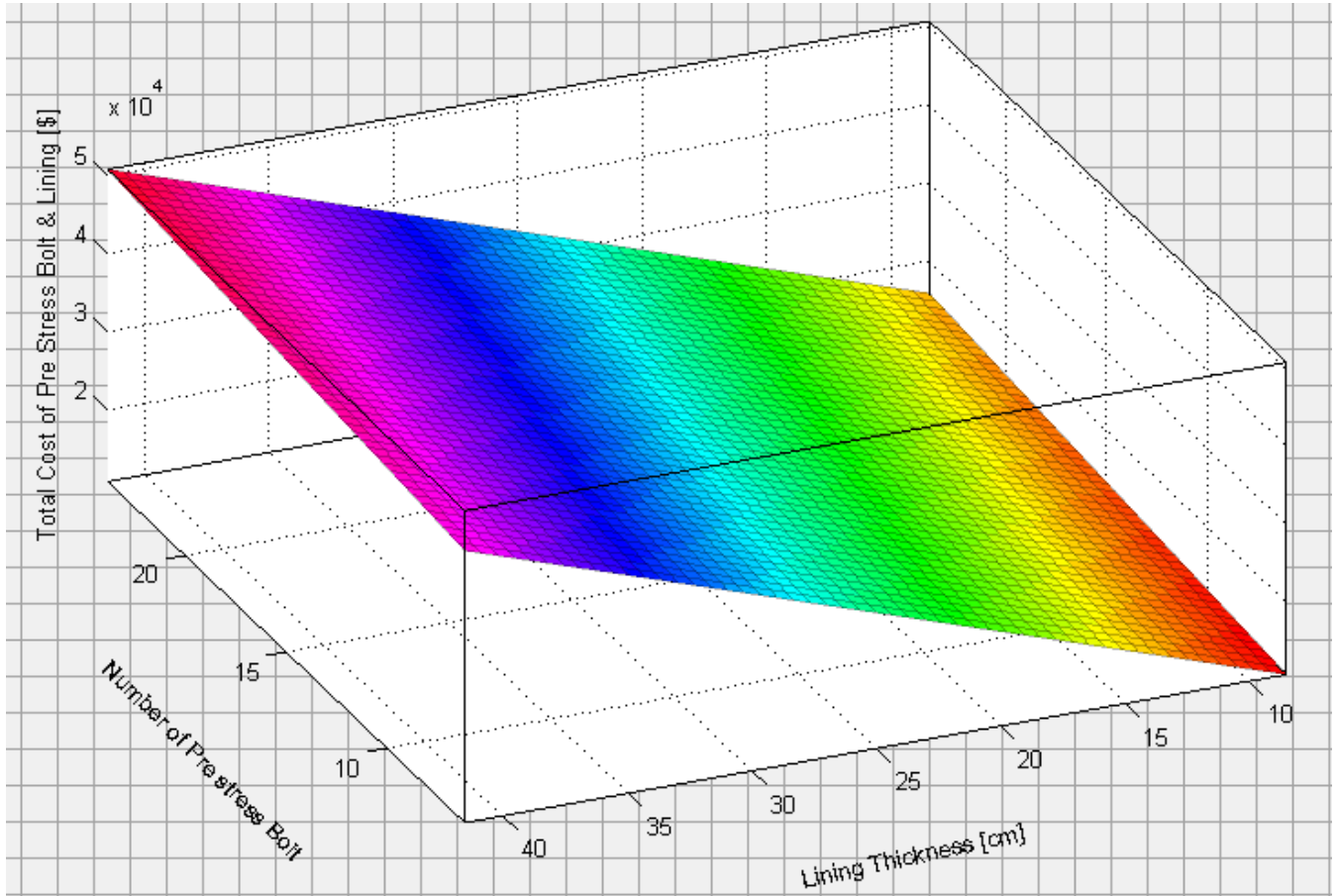


Figure 3.5-2: Cost function graphical representation

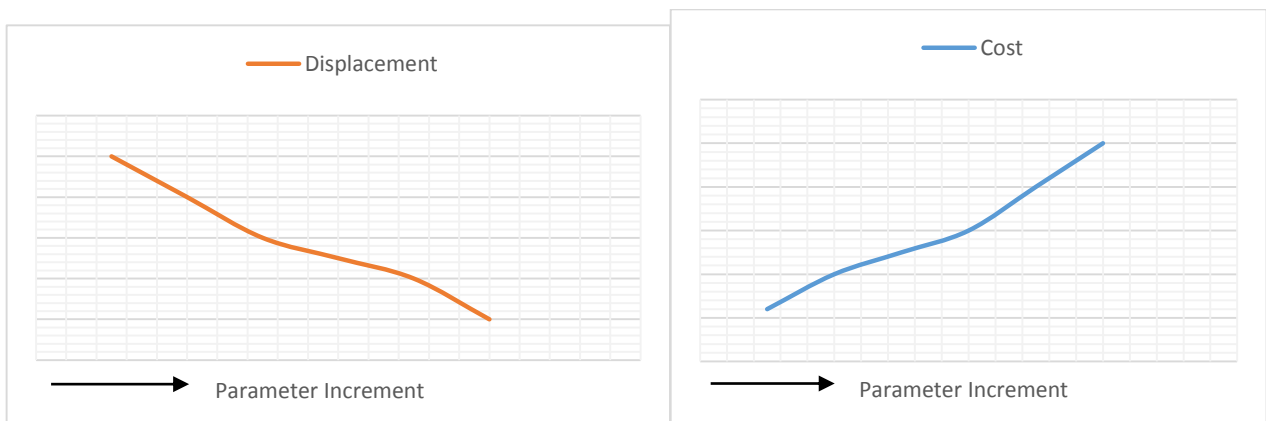


Figure 3.5-3: Displacement & cost relationship with parameters increment

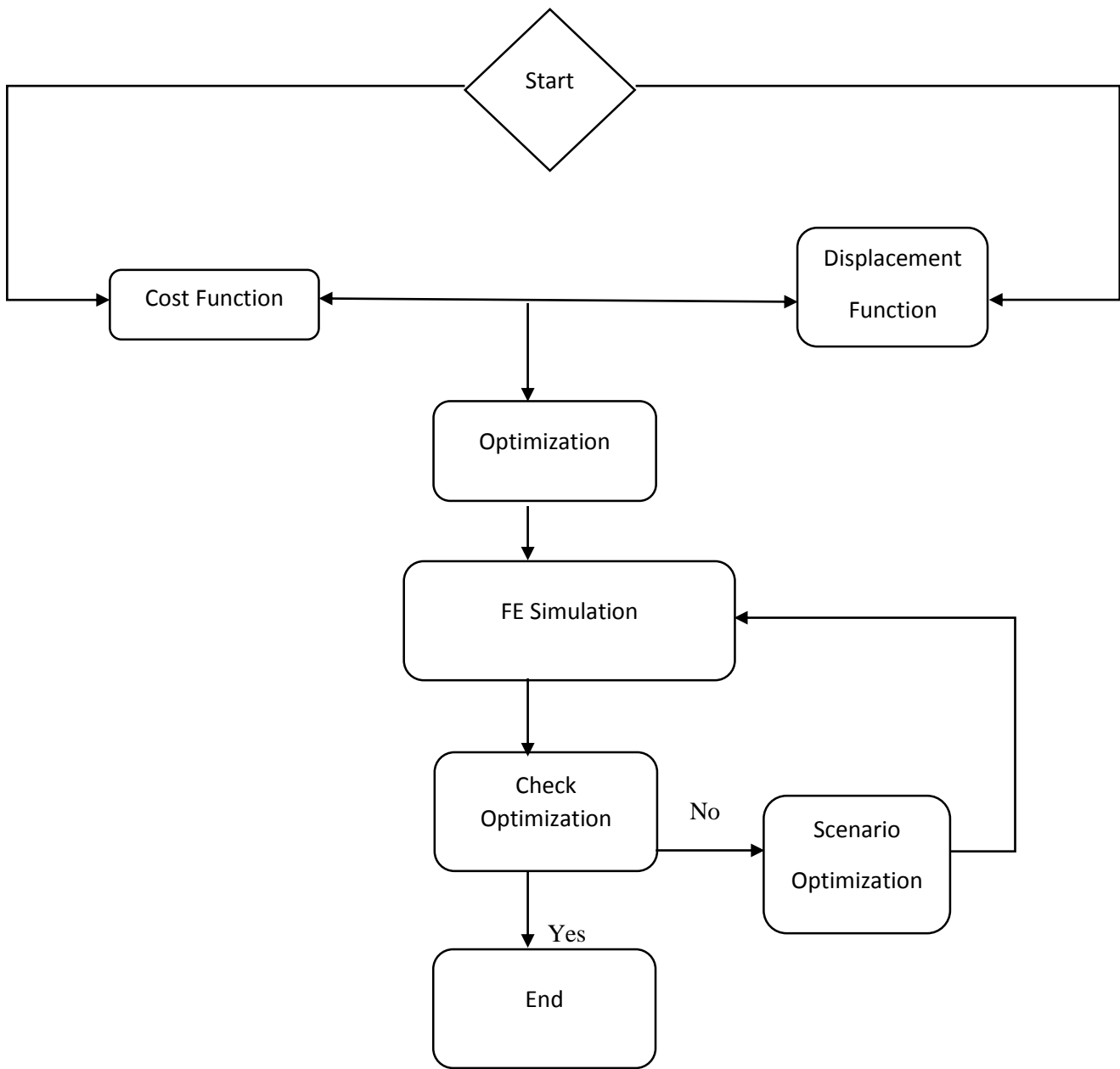


Figure 3.5-4: Optimization decision based algorithm

For the optimization, the two parameter correlations equations include equation 3.3 and 3.4 have been formulated. The polynomial model equations of cost and displacement analysis have great concern. But the settlement is of primary concern and the cost correlation is secondary.

As shown on the equation 3.3 and 3.4, as number of bolt and thickness of lining increase the function $Q(x, y)$ decreases. Conversely, function $F(x, y)$ goes up. Therefore, the optimum parameter combination is found starting from less displacement combination and continues until the combination is better than the Yepi Merkezi. The relationship between the two polynomial model equations is shown in the appendix in figure A-5.

Considering objective functions lonely, the cost and the displacement function, the optimization result using Mat lab solver give 7 bolts and 10 cm and 23 bolts and 40cm respectively as optimized combination. But these values are at the boundary. Case scenario 1 (7 bolts with 10 cm lining) settles more than the other 25th scenarios but cost effective. The other is case scenario 25, which has lower settlement than the other but it is so costly. So considering the two extremities (case 1 and case 25), the optimum combination could be a combination that meets the two objective functions.

The case scenario based optimization, which is selecting the best from those tried cases, applies now by considering the two equations. The optimization solution with the constraints given is case scenario 13 which is 15 bolts with 25cm lining thickness. Therefore, the optimized parameter combination of 15 bolts with 25 cm lining thickness is taken as optimum solution for this condition.

CHAPTER 4 RESULTS AND DISCUSSION

4.1 Introduction

In Section 3 of the paper, in which the modeling and the analysis of the problem were conducted in order to find correlation between parameters and the determining the best combination, has been presented well. Consequently, the optimized design combination has been modeled and verified in this section.

Using the data used for the parametric combinations, the modeling and analysis is done by Plaxis 2D for the optimized design parameter combination so as to check and verify. The numerical simulation output has discussed in the following section:

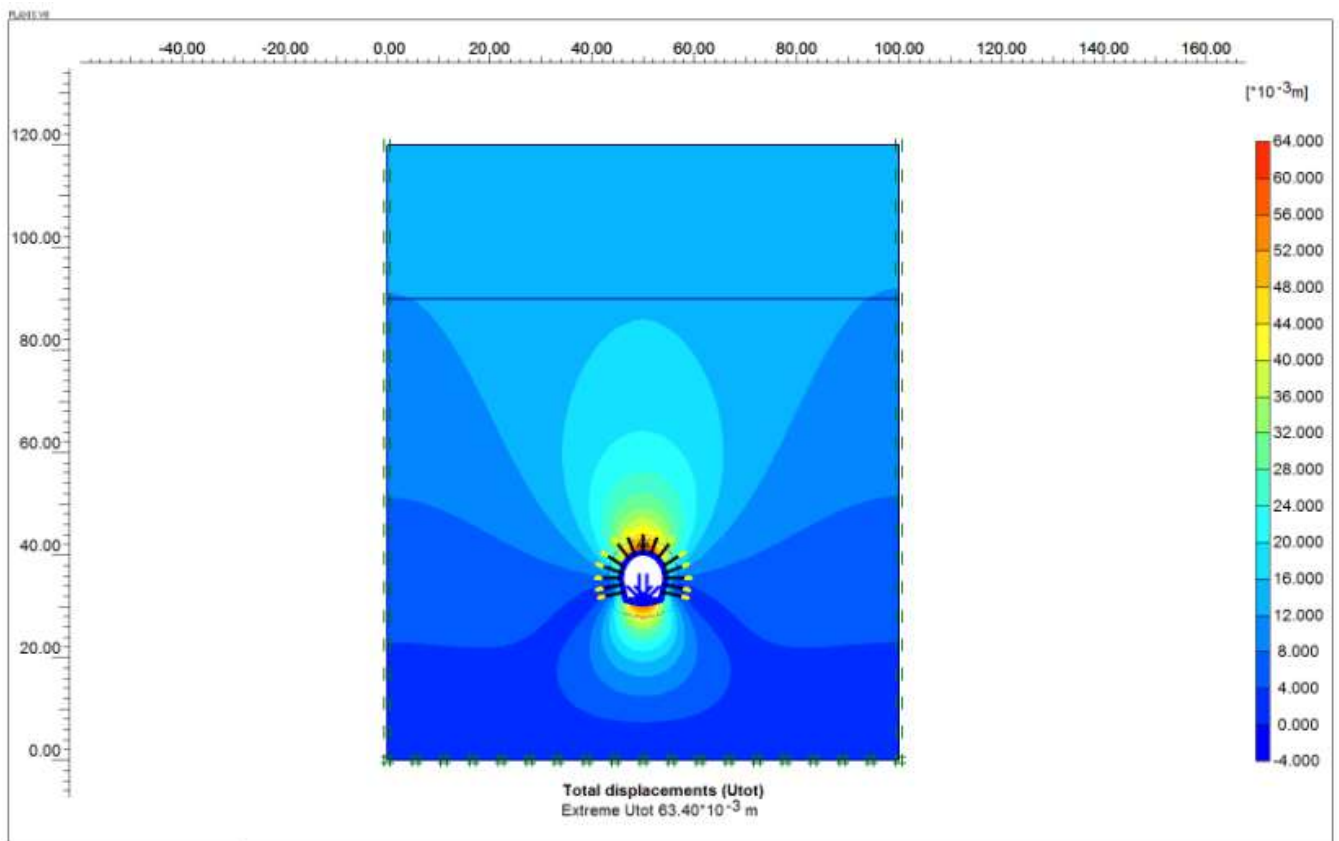


Figure 4-1 : Total displacement of the optimized combination after all phases

The maximum total displacement induced on the tunnel is 6.34 cm as shown in the above figure.

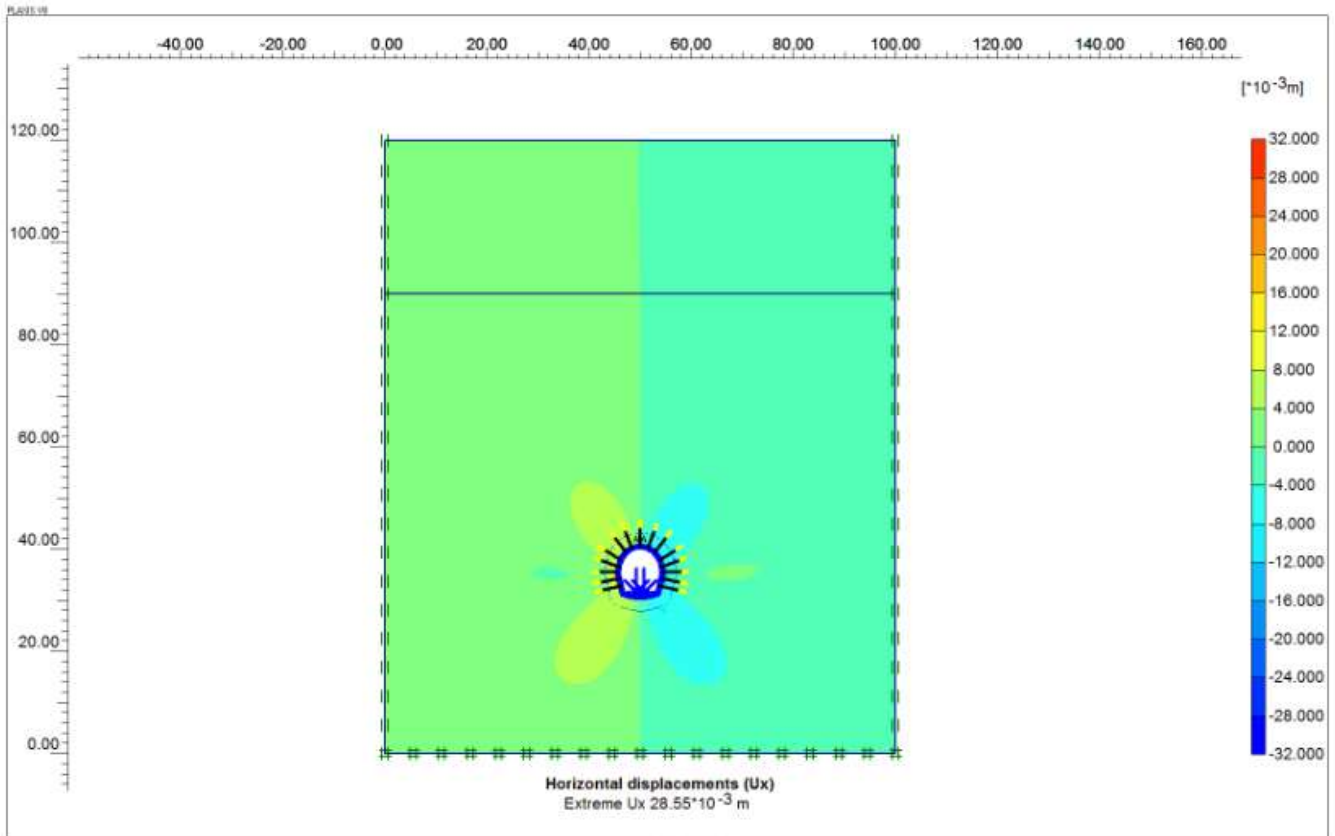


Figure 4-2: Horizontal displacement of the optimized combination after all phases

The maximum horizontal displacement of the tunnel with the optimized parameter combination is about 2.86cm.

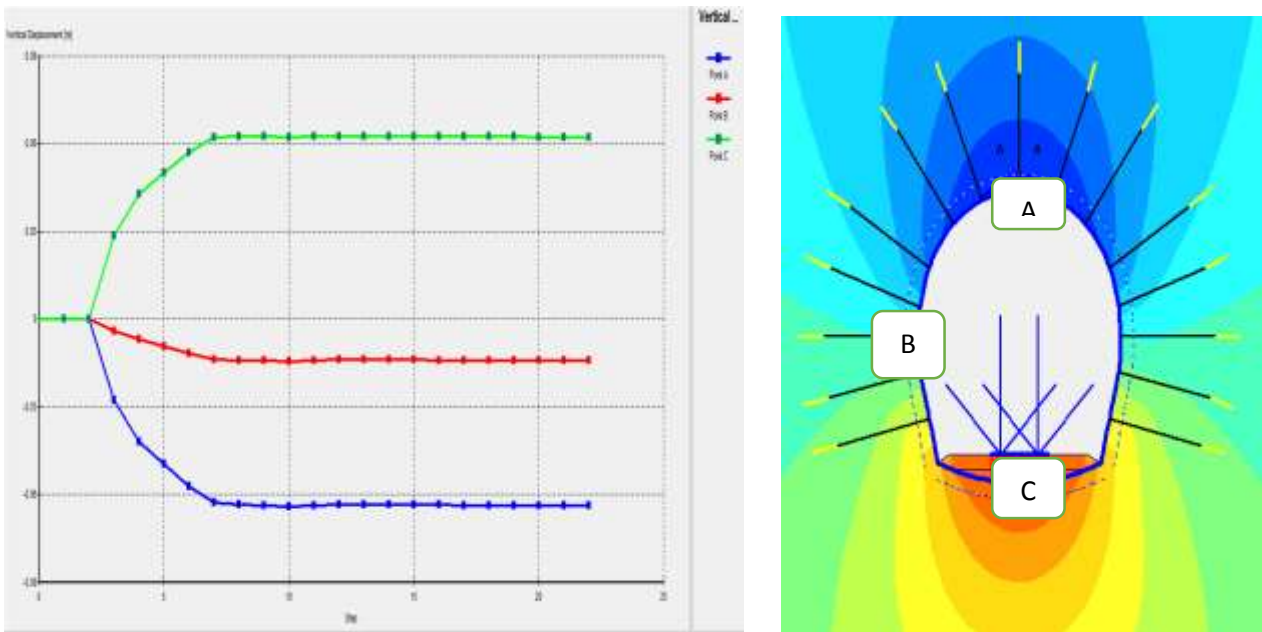


Figure 4-3: Vertical displacement at point A, B, C after all phases

As shown on the above figure 4-3 the crown displaced 6.34 cm downward and invert is 6.2 cm upward and side displaces vertically 1.4 cm. The displacement of the crown is higher than that of the invert and the side.

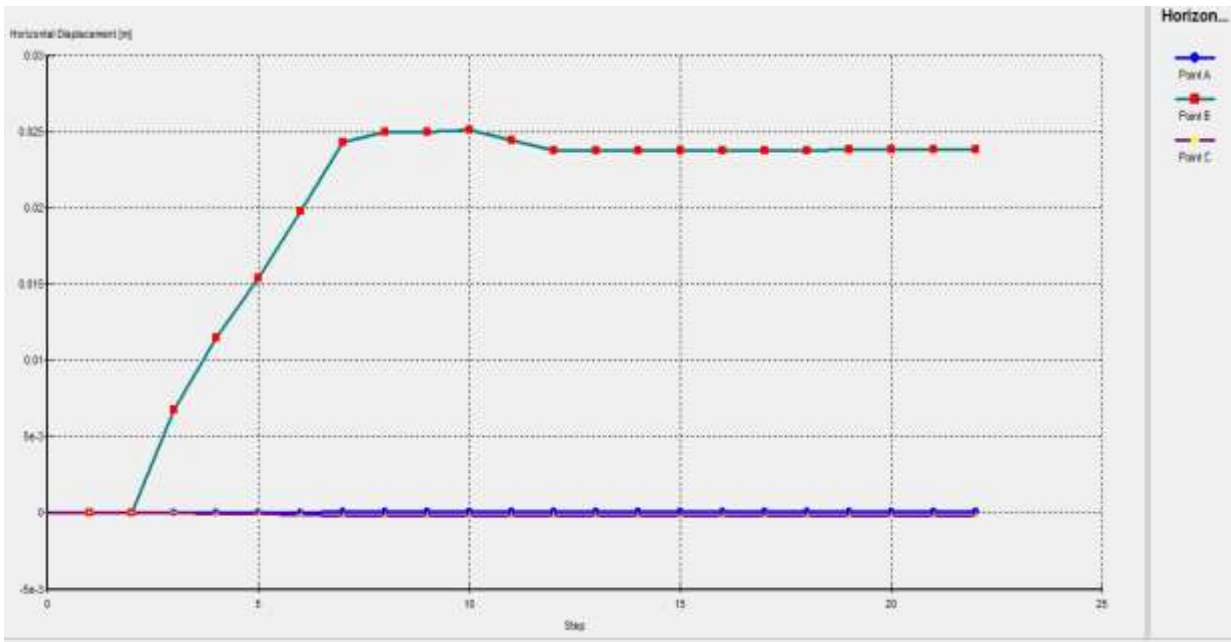


Figure 4-4: Horizontal displacement after all phases

The horizontal displacement of the side (point B) is higher than that of the crown (point A) and invert (point C). As shown on the above figure 4-4 the side displaced 2.4 cm laterally but the crown and invert lateral movement is very small as compared to the side.

Of the three points, Point B where it is located at side of the tunnel, the horizontal displacement is higher. This is due to the arching effect of the load. But point A (crown) and C (invert), their lateral movement is low compared to the vertical settlement.

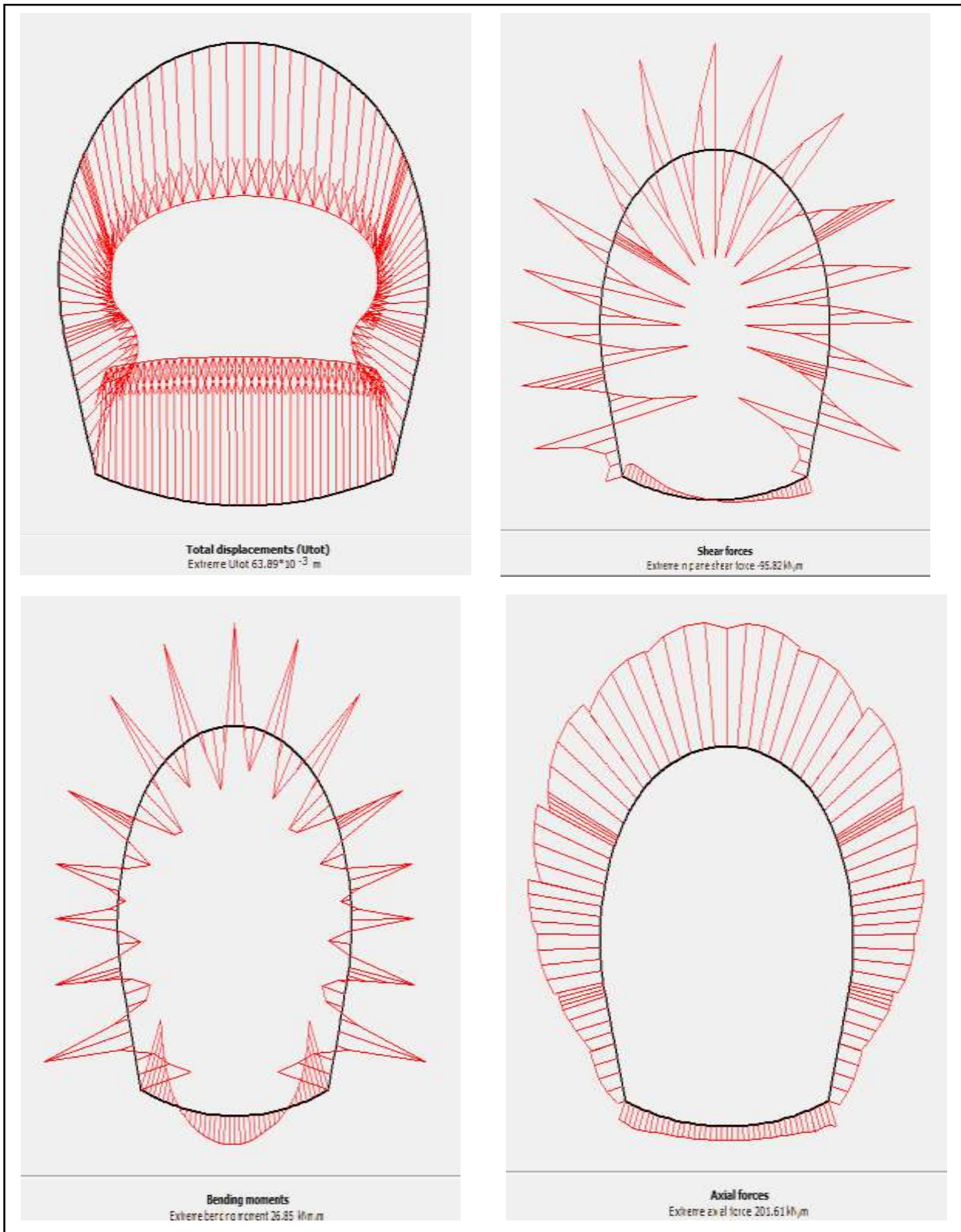


Figure 4-5: Numerical force outputs of the optimized combination

4.2 Optimized Combination Verification

In section 3.5 of the paper the optimized parameter combination was 15 bolts with lining thickness of 25 cm. For verification purpose, 15 bolts with 25 cm lining thickness are taken. The maximum acting forces on plate are the followings. The verification is done with regard to section capacity of the lining.

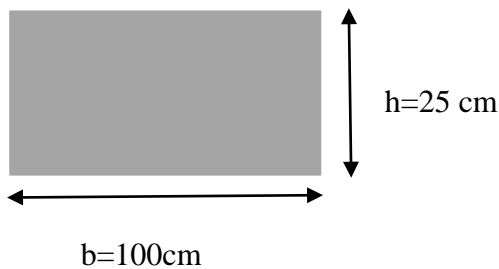
4.2.1 Lining Moment Verification

4.2.1.1 Forces

Table 4.1: Moment force found from the numerical simulation

Axial force, Nd	201.61 KN/m
Shear Force, Vd	95.82 KN/m
Moment, M	26.85KNm/m

Lining x-section considered as beam:



Section modulus, S given by:

$$S = \frac{I}{e} = \frac{bh^3}{12} * \frac{2}{h} = \frac{bh^2}{6} \quad 4.1$$

$$S = \frac{bh^2}{6} = \frac{1 * 0.25^2}{6} = 0.0104 m^3$$

The following design procedures should be followed for structural plain concrete.

i). Calculate the moment capacity on the compression face of the lining as follows:

$$\varphi M_{nc} = \varphi 0.85 * f_c' * S \quad 4.2$$

Where: M_{nc} = the nominal resistance of the compression face of the concrete

$\varphi = 0.55$ for plain concrete

f_c' = 28 day compressive strength of the concrete

S = the section modulus of the lining section based on the gross un cracked section

$$\varphi M_{nc} = 0.55 * 0.85 * 25 * 0.0104 = 121.55 \text{KNm/m}$$

$$M_{nc} = \frac{121.55 \text{KNm/m}}{0.55} = \frac{221 \text{KNm}}{\text{m}} > M_d$$

ii). Calculate the compressive strength of the lining as follows:

$$\phi P_c = \phi 0.6 * f'_c * A \quad 4.3$$

Where: P_c = the nominal resistance of lining in compression

A = the cross sectional area of the lining section

$$\phi P_c = 0.55 * 0.6 * 25 * 0.25 = 2.06 \text{Mpa}$$

$$P_c = 2.06/0.55 = 3.75 \text{Mpa}$$

∴ Check the compression face as follows:

$$\frac{Q_A}{\phi P_c} + \frac{Q_M}{\phi M_{nc}} \leq 1 \quad 4.4$$

Where:

Q_A = The axial load force effect modified by the appropriate factors

Q_M = The moment force effect modified by the appropriate factors

$$\frac{201.61}{2060} + \frac{26.85}{121.55} = 0.32 \leq 1$$

∴ Therefore, the compression face of lining is safe.

iii). Calculate the moment capacity on the tension face of the lining as follows:

$$\phi M_{nT} = \phi 5 (f'_c)^{1/2} S \quad 4.5$$

Where: M_{nT} = the nominal resistance of the tension face of the concrete

$$\phi M_{nT} = 0.55 * 5 * (25)^{1/2} * 0.0104 = \frac{143 \text{ KN}}{\text{m}} > M_d$$

iv) . Calculate the tension strength of the lining as follows:

$$\phi P_T = 5 (f'_c)^{1/2} \quad 4.6$$

Where: P_T = the nominal resistance of lining in tension

$$\phi P_T = 5 * (25)^{1/2} = 13.75 \text{Mpa}$$

∴ Check the tension face as follows:

$$\frac{Q_M}{S} - \frac{Q_A}{A} \leq \varphi_{PT} \quad 4.7$$

$$\frac{26.85}{0.0104} - \frac{201.61}{0.25} = 2 \text{Mpa} \leq \varphi_{PT} = 13.75 \text{Mpa}$$

Therefore, the tension face of lining is safe.

4.2.2 Lining Shear Verification

The design value for the shear resistance $V_{Rd,c}$ is given by Euro code 2:

$$V_{Rd,c} = [C_{Rd,c} k (100 \rho_1 f_{ck})^{1/3} - 0.15 \sigma_{cp}] b_w d \quad 4.8$$

Where: f_{ck} in MPa

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2 \quad \text{Where, } d \text{ is in mm.}$$

$$\rho_1 = \frac{A_{s1}}{b_w d} \leq 0.02$$

Then, $f_{ck} = 25 \text{Mpa}$

$$k = 1 + \sqrt{\frac{200}{200}} = 2$$

$$\rho_1 = \frac{442}{(1000 * 200)} = 0.002 \quad \text{Here the wire mesh area is taken.}$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} \leq 0.02 \quad [\text{Mpa}]$$

$$\sigma_{cp} = \frac{201.61}{0.02} = 1 \text{ N/mm}^2$$

$$C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$$

$$V_{Rd,c} = [0.12 * 2(100 * 0.002 * 25)^{1/3} - 0.15 * -1] 1000 * 200$$

$$V_{Rd,c} = 112.07 \text{KN}$$

The highest shear force acting on the lining is $V_d = 95.82 \text{ KN/m}$. So the section is safe against shear without shear reinforcement.

4.3 Parametric Study on Pre Stress Rock Bolt

In chapter three, the optimized combination result came up with 15 bolts with lining thickness of 25cm. The optimization design combination has been showed that the design as economical and safe by increasing number of pre stress bolt. Therefore, there is a need to explore more about on pre stress bolt. Though there are many pre stress bolt parameters, this part deals and discusses about the effect of the number of pre stress rock bolt installation on tunnel displacement, stress, strain, force and plastic point.

4.3.1 Effect of Pre Stress Rock Bolt on Stress

The rock bolts dramatically increase the minor principal stress which results in significant increase in material strength. Consequently, the major principal stress and vertical stresses that the material can carry will greatly increase. Therefore, the stress increment by the pre stress bolt could be in accordance with the rock and lining strength capacity.

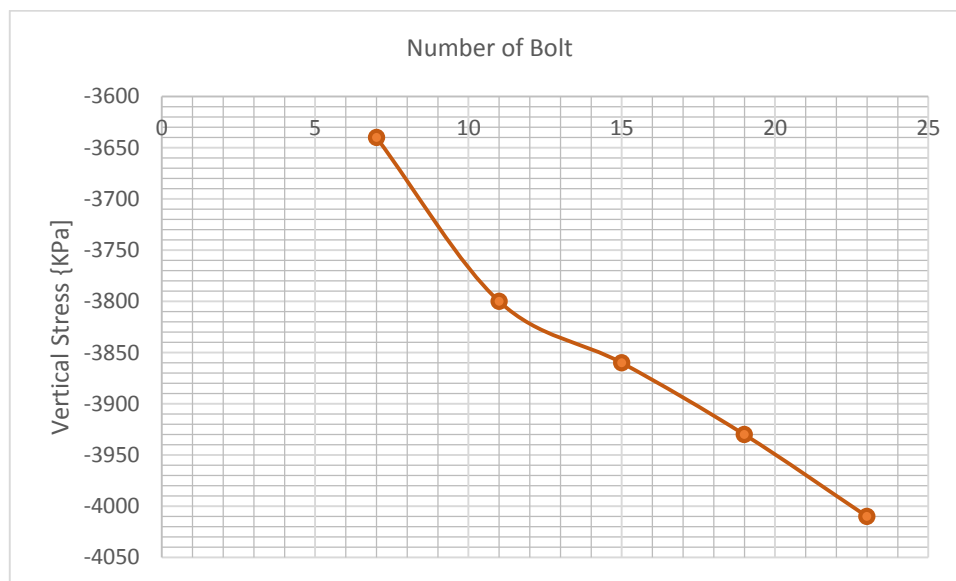


Figure 4.3-1 : Effect of pre stress rock bolt on vertical stress

4.3.2 Effect of Pre Stress Rock Bolt on Displacement

From the figure below, increasing the number pre stress bolts decrease tunnel displacement. Because of reinforcing the rock using the pre stress bolt is good to prevent the convergence of tunnel; the pre stress force applied on bolt stiffens and binds the rock together. The pre stress force applied on bolt increases bolt surface friction property. Therefore the binding effect of bolt becomes better.

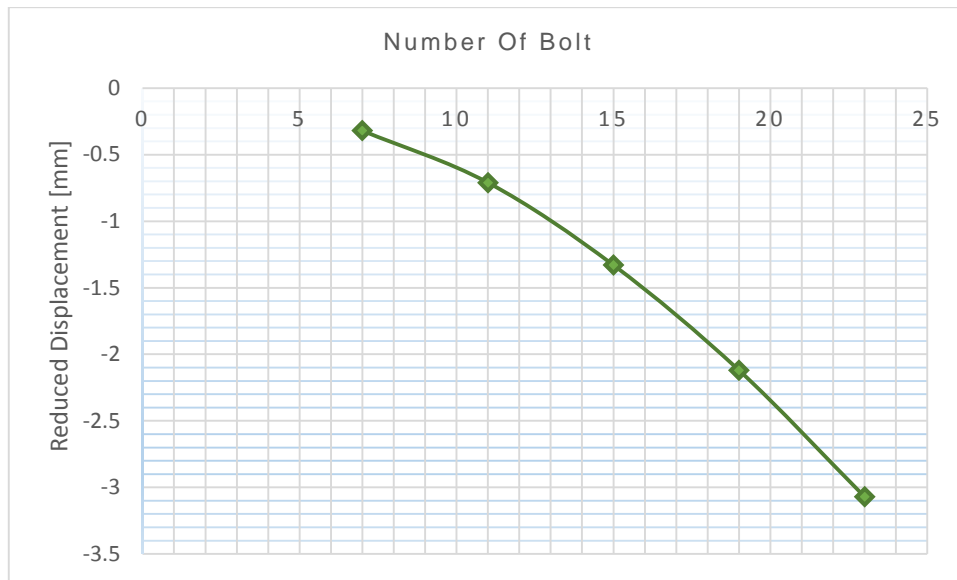


Figure 4.3-2: Number pre stress rock bolt on reducing displacement

4.3.3 Effect of Pre Stress Rock Bolt on Axial Force

Since the pre stress force is applied on the rock bolt, the ground is stiffened more than before and this force has effect on lining. However, the effect on lining is good as it prevent plate buckling. The figure shows that the axial force increase as number of bolt increase.

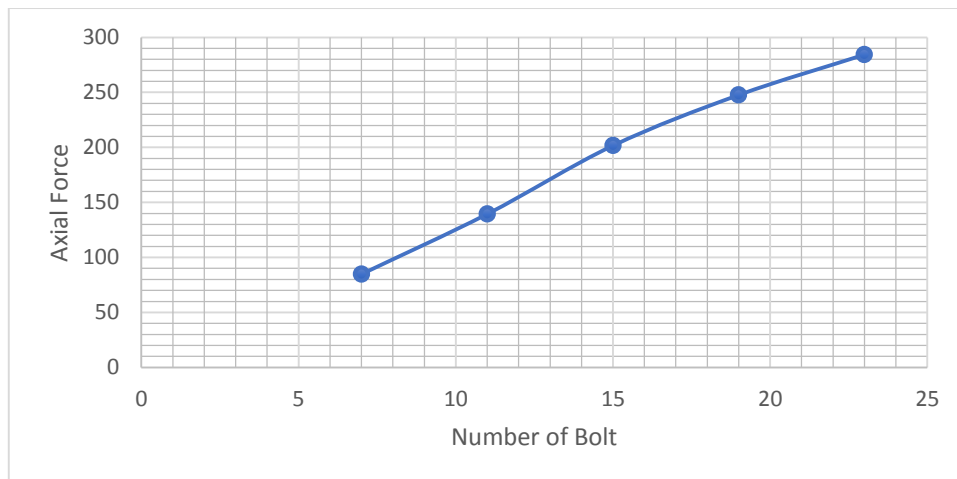


Figure 4.3-3: Number of pre stress rock bolt and pre stress force

4.3.4 Effect of Pre Stress Rock Bolt on Shear Strain

As the rock bolt reinforces the ground, the stress somehow increases and there is movement due to the pre stress force. So, definitely the strain will slightly increase due to the pre stressing force exerted on the soil around the bolt.

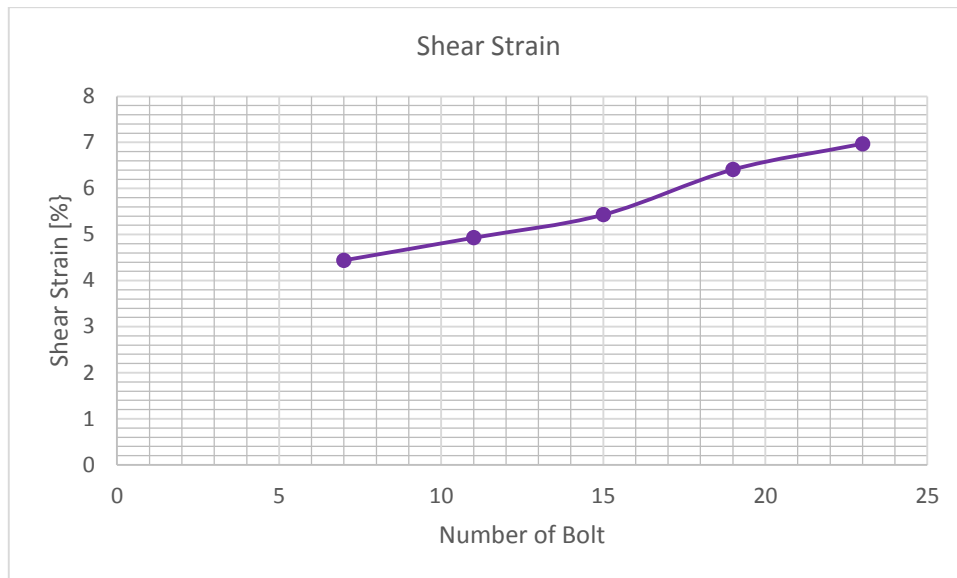


Figure 4.3-4: Effect of number of pre stress rock bolt on strain

4.3.5 Effect of Pre Stress Force on Tunnel Displacement

The pre stress bolt has a significant effect in reducing the tunnel ground displacement. But the amount of the pre stress force applied has also an effect on tunnel displacement. Recalling the Euro code for pre stressing steel, this force could be in between 80% to 95% of its ultimate capacity.

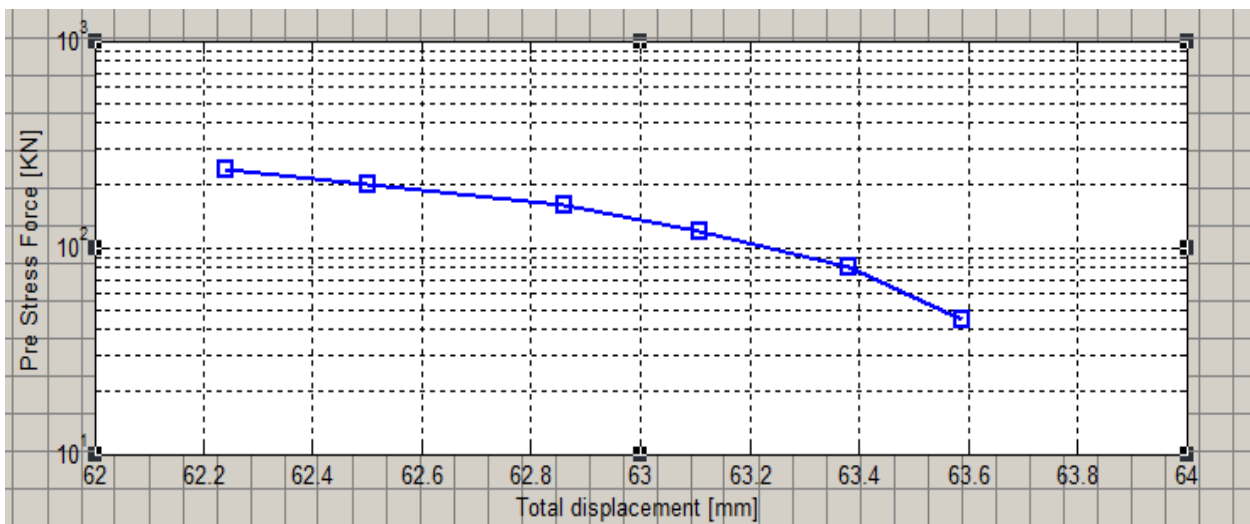


Figure 4.3-5 :Semi-log scale plot pre stress force vs displacement

4.3.6 Effect of Pre Stress Rock Bolt on Plastic Points

Plastic points are important parameters since their effect on the convergence of the tunnel, is critical. Due to the openings the rock around the tunnel may behave plastically through time and these points are much prevented and get strength by bolt. So that installing the rock bolt is the best as it gives strength through binding or reinforcing them.

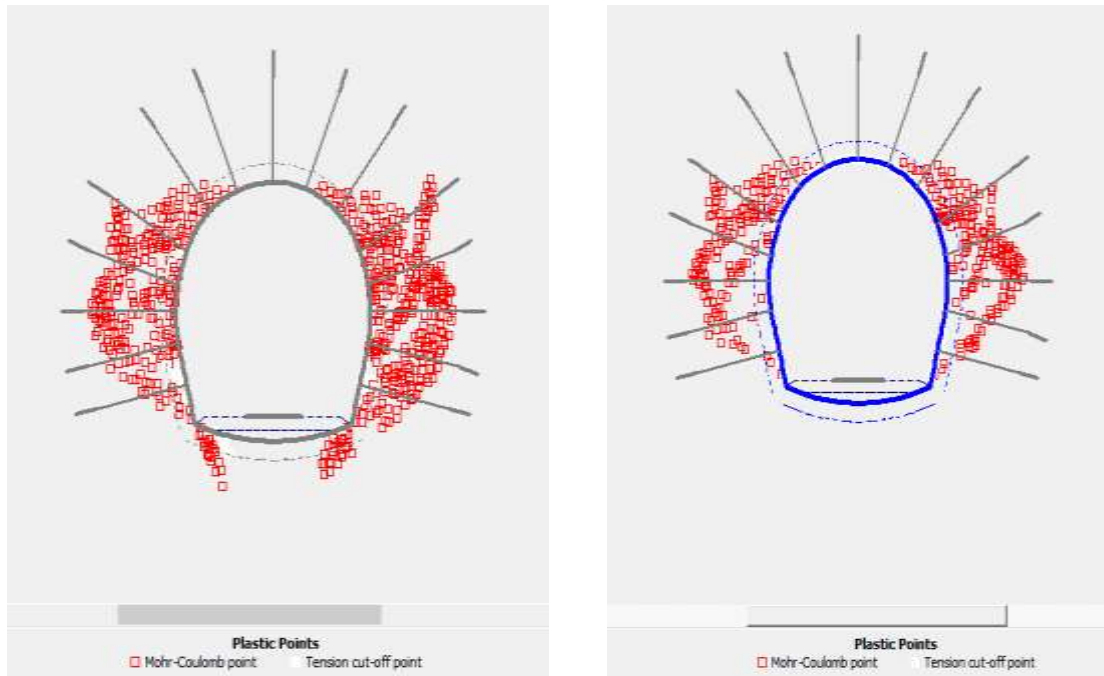


Figure 4.3-6: Plastic Points after excavation & shotcrete respectively

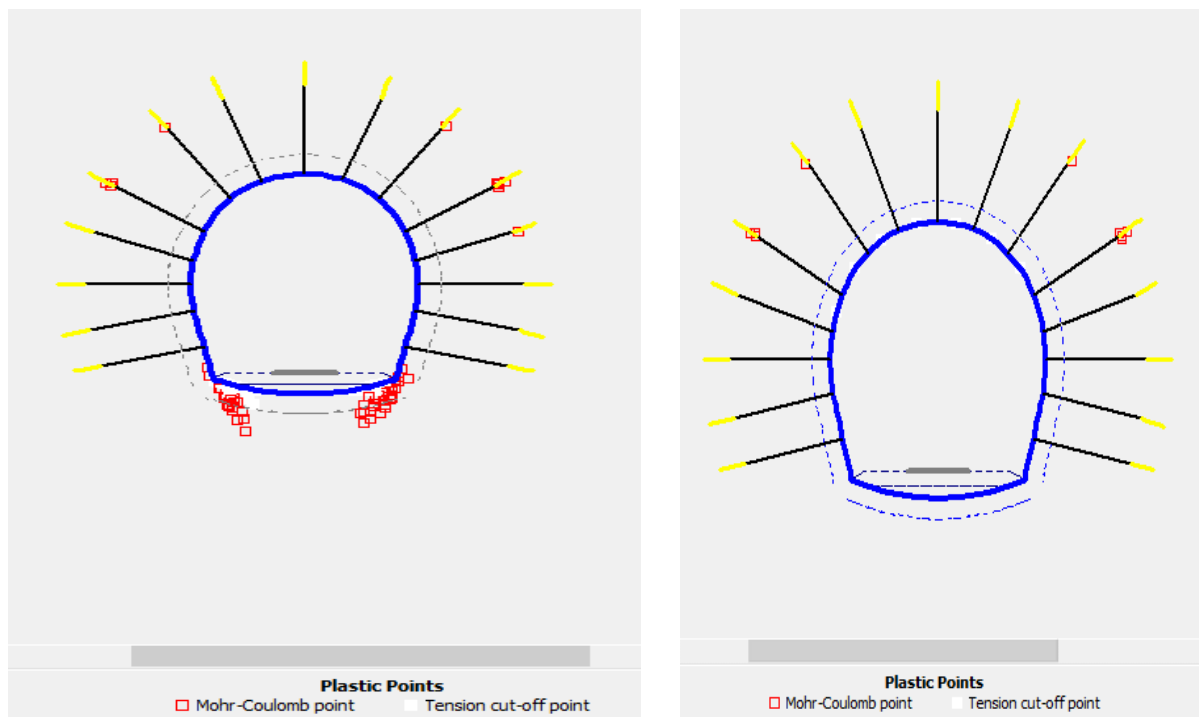


Figure 4.3-7: Plastic Points after pre stress rock bolt installing and concrete lining respectively

Bolts are also very important in enhancing the tensile strength of the shotcrete and reinforcing/binding it to that of the rock. The plastic points are concentrated mainly around the side wall of the tunnel. As shown in the above figure 4.3-6 the elasticity of the rock have been kept significant amount by shotcrete. The significance of bolt in reducing the plasticity of the rock is higher than shotcrete. This is because the pre stress bolt binds itself to that of the rigid ground or elastic rock.

4.4 Comparison of the Optimized and the AKH Design Combination

Table 4.2: Results of optimized and AKH design combination

Design Combination	AKH (Yepi M.)	Optimized	Observed	Comment
Parameter Combination	11 rock bolts 30cm lining	15 rock bolts 25cm lining	11 rock bolts 30cm lining	The displacement of tunnel before the instrument install is unknown. But this value usually is 3 to 4 times of the current observed instrument reading which becomes nearly 60mm.
Displacement	63.82 mm from Numerical Result	63.4 mm from Numerical Result	15 mm observed total displacement after Support installations	
Mean Stress[Kpa]	2080	2090	-	
Cost per Meter (Longitudinal)[\$]	34840	30700	34840	

T04 is designed and constructed by Yepi Merkezi. The number of rock bolt and thickness of final lining provided at final design stage was 11 rock bolt and 30cm thick concrete lining. However, the research come up with an optimization design combination, with limitations posed is 15 rock bolts with 25 cm thick concrete lining.

From the table, the optimized combination stress is slightly higher than the AKH design. This is because of the increased number of rock bolts. But the displacement is decreased from 63.82 mm to 63.4 mm. The pre stress rock bolts are able to reduce the convergence of the tunnel by enhancing the rock stabilization. So that the displacement become lower in the optimized combination.

On the above table, the displacement of tunnel before the instrument install is unknown. But this value usually is 3 to 4 times of the current observed instrument reading. So the total observed displacement inside the tunnel becomes four times of 15 mm which is nearly 60 mm. Since the observed displacement is lower than the predicted displacement using numerical analysis, the optimized combination is better.

The other difference comes from cost. But the cost difference accounts only for one meter of rock bolt and concrete lining. Design provided by Yepi Merkezi, with 11 bolt and 30cm thick lining cost using polynomial function, which was used for polynomial modeling becomes:

Using equation 3.4 the AKH design combination of rock bolt and concrete lining cost becomes:

Cost = \$34,840 dollar

Rock bolt cost includes all costs up to installation (material, grout, equipment) whereas concrete lining cost includes all costs up to casting (concrete material, formwork, and equipment). Similarly, the optimized combination cost can be calculated as follows:

Cost = \$30,700

∴ Therefore, the cost difference between the two design combinations is come to be \$4140. But cost difference accounts are for one meter length of tunnel which equals 34m² of rock bolt and concrete lining construction only.

4.5 Summary

Numerical simulation of the optimized combination has been made using the plaxis 2D. Consequently, the optimized parameter combination has been verified along with the interpretation as well. And lastly, the optimized combination have been compared to that of the final design combination made by the Turkish contractor namely Yepi Merkezi.

CHAPTER 5 CONCLUSION AND RECOMMENDATION

This section is the final part of this research and gives a concluding remark at the beginning and then followed by proposing of some further works to be made that are recommended by the author for the better understanding and advancement of knowledge in tunnel rock bolt and concrete lining combination optimization.

5.1 Conclusion

- By considering the displacement and cost polynomial functions, the optimized parameter combination as optimization result has been found and the optimized parameter combination become 15 rock bolts with 25 cm thick concrete lining.
- Concrete lining has influential impact in keeping the tunnel in position. However, its costly nature. So there is a requisite to find way of reducing cost, which is increasing the number of pre stress rock bolt. Therefore, like concrete lining, by increasing the number of pre stress bolt, concurrently the cost and convergence of the tunnel can be reduced.
- The effect of number of rock bolt on tunnel displacement, stress, strain and force has well been presented and the result showed that as number of rock bolt increase the stress and strain will slightly increases. As well, the effect of pre stress force on tunnel displacement has also been performed and the result showed that the displacement is decreased with increasing pre stress force.

5.2 Recommendation

From the analysis results and discussions made, increasing the number of pre stress rock bolt could reduce the tunnel convergence. In addition, the optimum parameter combination shows that the possibility of increasing number of pre stresses rock bolt in order for the design be safe and economical. Therefore, the tunnel designers are highly advised to look at the possibility of increasing number of pre stress rock bolt as optimum solution provided that adequate study on number of pre stress rock bolt is performed.

5.3 Future Works

This work has been made with simplifications and assumptions which had made the research manageable in the time frame of this thesis work as well as due to computational capability limitations posed. However, in order for maximizing the accuracy and precision of the research and gain in an in depth understanding of the phenomenon, further works are in demand and includes the following but not limited to:

- Investigation of more parameters for the optimization work;
- Experimental campaigns to validate and calibrate the models for better precision and accuracy outcomes;
- Robustness analysis in order for accounting variability and uncertainties in input values like, material properties, rock characters, etc.;
- Investigation on pre stress rock bolt and its effect on tunnel displacement, stress, strain by including different parameters of pre stress rock bolt;

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Appendix A

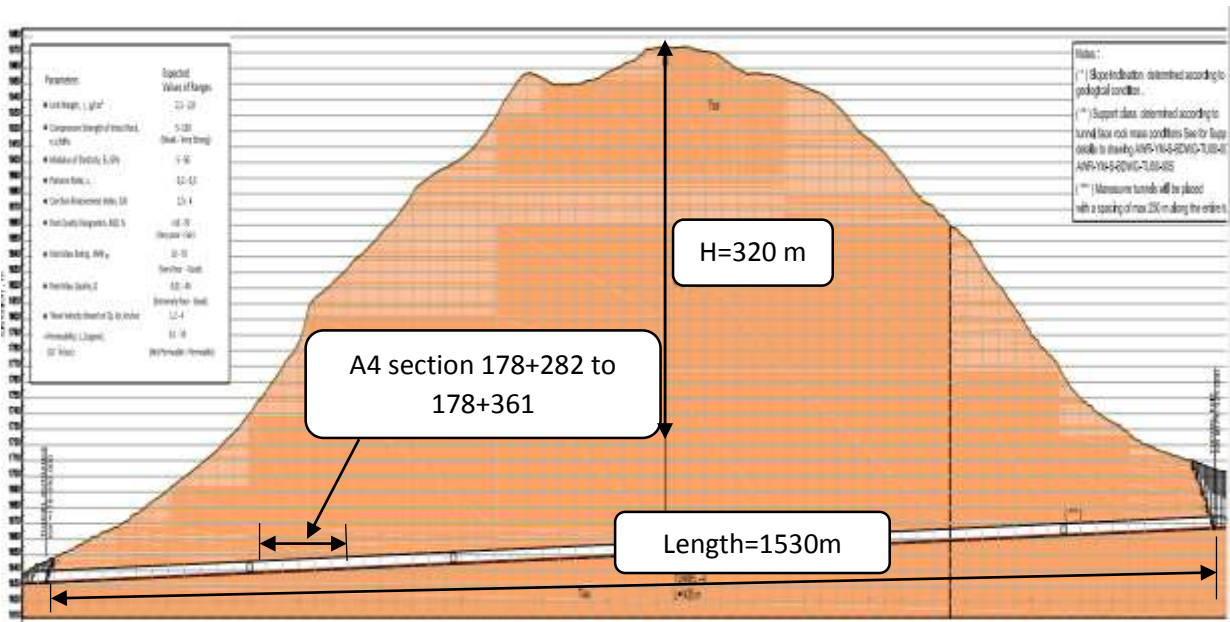


Figure A-1: TO4 overburden and A4 support location

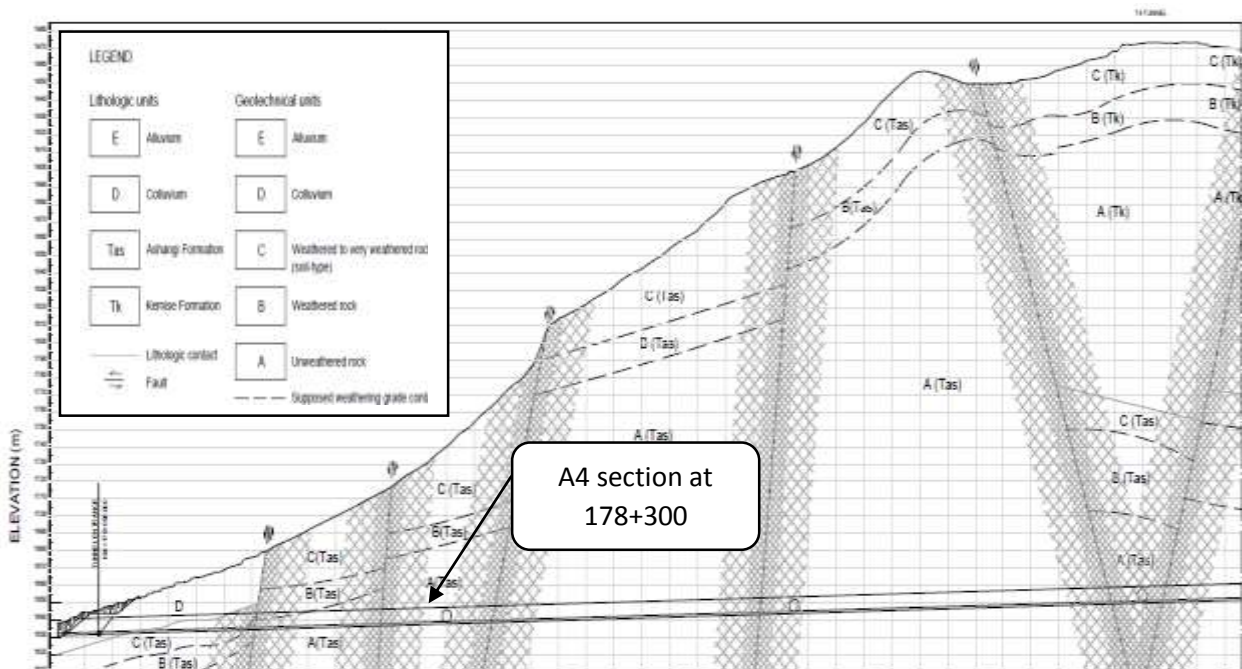


Figure A-2: Ground rock condition of T04

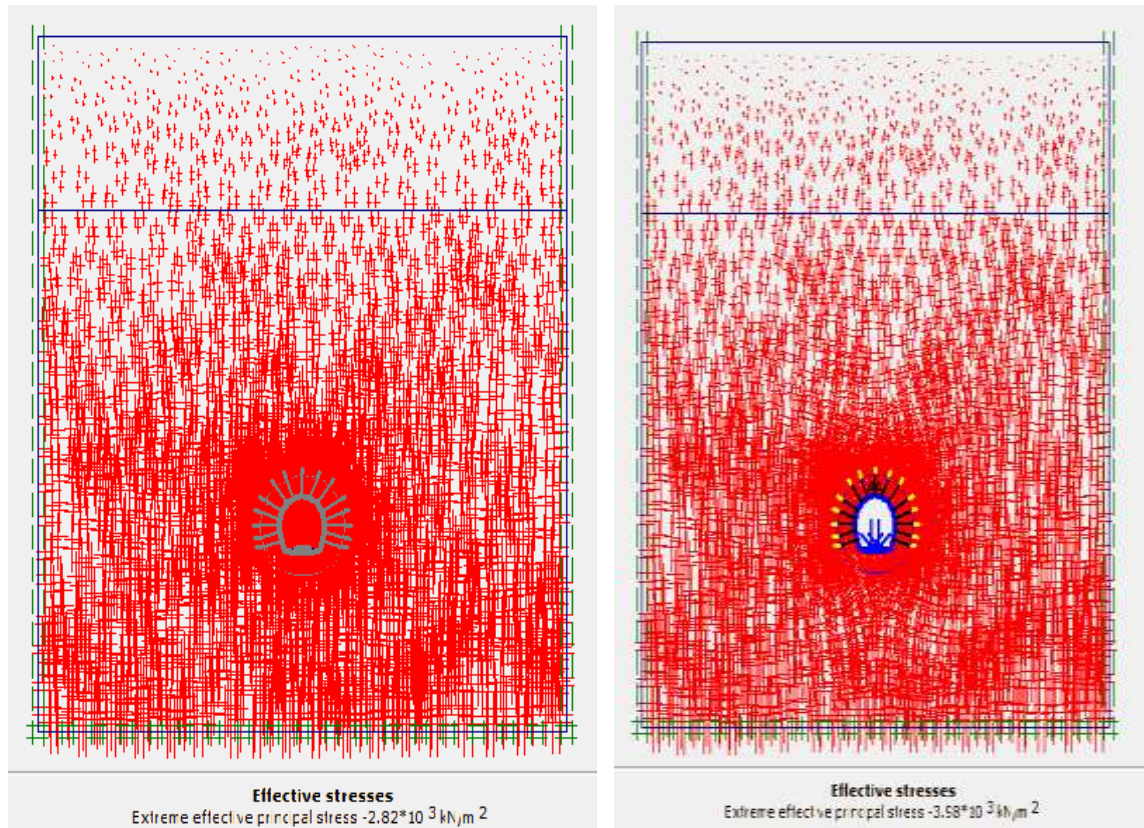


Figure A-3: Initial stress and secondary stress after excavation respectively

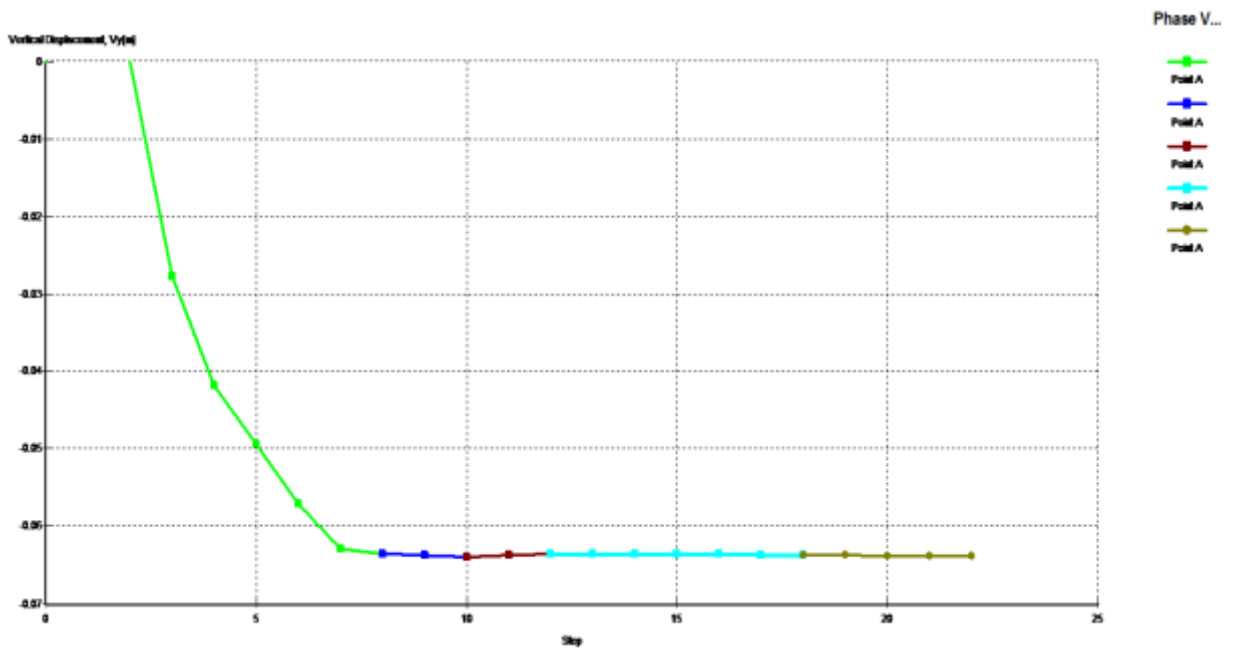


Figure A-4: Phase vertical displacement at crown

From above figure the colors show that lime for excavation, blue line for shotcrete, maroon for bolt, aqua for lining and olive for ballast loading.

Gamultiobj-Multi objective optimization using Genetic Algorithm

The solver used for calling and analyzing to relate the two objectives was gamultiobj-multi objective Optimization using Genetic Algorithm. The displacement and cost objectives relationship are presented in the following figure.

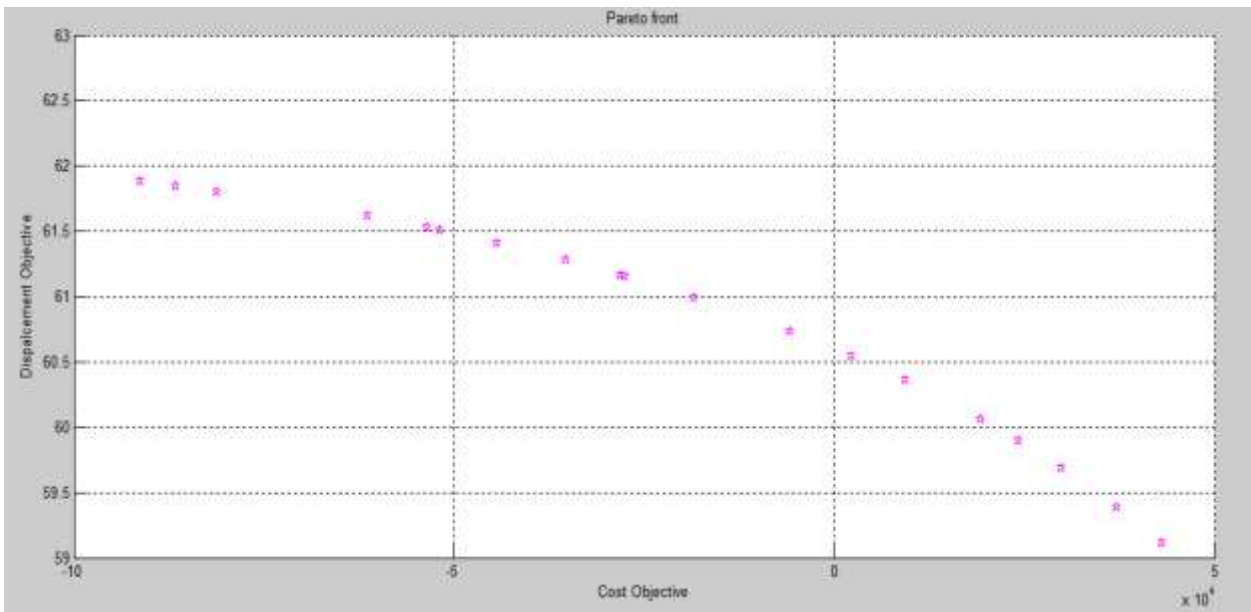


Figure A-5: Pareto displacement and cost relationship