COMPARISON OF 2D AND 3D FINITE ELEMENT MODELS OF TUNNEL ADVANCE IN SOFT GROUND: A CASE STUDY ON BOLU TUNNELS

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

BY

SERKAN ÜÇER

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

MAY 2006

Approval of the Graduate School of Natural And Applied Sciences

Prof.Dr. Canan Özgen
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof.Dr. Erdal Çokça
Head of Department

This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

Assoc.Prof. B. Sadık BAKIR
Supervisor

Examining Committee Members

Prof.Dr. M. Ufuk ERGUN	(METU, CE)	
Assoc.Prof. B.Sadık BAKIR	(METU, CE)	
Assoc.Prof. Levent TUTLUOĞLU	(METU, MINE)	
M.Sc.CE Bülent ULUKAN	(Yüksel Domaniç)	
M.Sc.CE Yılmaz DİNÇER	(Yüksel Proje)	

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last name : Serkan, ÜÇER

Signature :

ABSTRACT

COMPARISON OF 2D AND 3D FINITE ELEMENT MODELS OF TUNNEL ADVANCE IN SOFT GROUND
A CASE STUDY: BOLU TUNNELS

ÜÇER, Serkan
M.Sc., Department of Civil Engineering
Supervisor: Assoc.Prof.Dr. B. Sadık BAKIR

May 2006, 101 pages

The Bolu Tunnels lie along Trans European Motorway (TEM) which is connecting Eastern Europe with the Middle East. The tunnels are approximately 3.0 km long, 40 m apart and have excavated cross sections more than 200 m^2. In construction, New Austrian Tunneling Method (NATM) was used in soft ground. Due to the challenging ground conditions, many problems have been encountered during tunnelling. To solve these problems special construction techniques were adapted. To simulate and demonstrate the effectiveness of these construction techniques, 2D and 3D Finite Element Methods are utilized in this study. Through comparison between 2D and 3D modelling of advance of Bolu Tunnels, respective merits of these two approaches are investigated and the conditions under which shortcomings of the 2D approach become serious are identified.

Keywords: Tunnel, NATM.

ÖZ

YUMUŞAK ZEMİNDE TÜNEL AÇILMASININ
2-BOYUTLU VE 3-BOYUTLU SONLU ELEMANLAR
MODELLERİYLE KARŞILAŞTIRILMASI
BOLU TÜNELLERİ ÜZERİNE BİR ÇALIŞMA

ÜÇER, Serkan
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Doç.Dr. B. Sadık BAKIR

Mayıs 2006, 101 sayfa

Bolu Tünelleri, Doğu Avrupa ile Batı Avrupa'yı birbirine bağlayan TEM Otoyolu üzerinde bulunurlar. Uzunlukları yaklaşık 3.0 kilometreyi, aralarındaki mesafe 40 metreyi bulan tüneller 200 m^2'den daha fazla kazılmış kesit alanına sahiptirler. Yumuşak bir zeminde gerçekleşen inşaatta Yeni Avusturya Tünel Metodu (YATM) kullanılmıştır. Tüneller açılırken, zor zemin şartlarından dolayı pek çok problemle karşılaşılmıştır. Bu güç problemleri çözebilmek için özel yöntemler geliştirilmiştir. Bu yöntemlerin etkinliğini gösterebilmek için, çalışmada 2-Boyutlu ve 3-Boyutlu Sonlu Elemanlar Metodu kullanılmıştır. Bolu Tünelleri'nin 2-Boyutlu ve 3-Boyutlu modellerinin karşılaştırılmaları sırasında bu iki yaklaşımın özellikleri tahkik edilmiş ve 2-Boyutlu yaklaşımın dezavantajlarının ciddi olma durumları tanımlanmıştır.

Anahtar Kelimeler: Tünel, YATM.

To those who stood by me during hard times
especially my friends & my family

ACKNOWLEDGMENTS

The author wishes to express his deepest gratitude to his supervisor Assoc. Prof. Dr. B. Sadık Bakır for his guidance, advice, criticism, encouragements and insight throughout the research. Also I would like to thank for his great patience and endless support, since the thesis has been completed in a rather long period of time.

I would like to extend my sincere thanks to the Members of the Examining Committee for their valuable suggestions, comments and help.

I would also like to thank to Bolu Tunnel Project Manager Faik Tokgözoğlu and to Mr. Selami Işık for their suggestions, comments and valuable help through site visits and meetings held in Bolu.

I would also like to thank Burak Güler for helping in hardware and software related problems and to Mr. Savaş Özkuzukıran, one of my best friends, for his endless support.

TABLE OF CONTENTS

LIST OF TABLES

TABLES

LIST OF FIGURES

FIGURES

CHAPTER 1

INTRODUCTION

Tunnels form an important section of subterranean structures, as they can be defined as underground passages constructed for the purpose of direct traffic, or transportation between two points. Simply defined, tunnels are ''underground passages made without removing the overlaying rock or soil''.

The oldest tunnel, built for the expressed purpose of communication was constructed, according to present knowledge, 4000 years ago under the reign of the famous Queen Semiramis (or later under Nebuchadnezzar) in ancient Babylon to underpass the bed of the River Euphrates and to establish an underground connection between the royal palace and the Temple of Jove. The length of this tunnel was 1 km and it was built with the considerable cross-section dimensions of 3.6 m by 4.5 m. The River Euphrates was diverted from its original bed for the construction period and the tunnel, which would be a major project even according to modern standards was built in an open cut. The wall of the tunnel consists of brickwork laid into bituminous mortar and the section is covered from above by a vaulted arch. The vast scope and extent of the undertaking point to the fact that this tunnel was not the first of its kind built by the Babylonians and that they must have acquired skill and practice with several tunnels built earlier. To appreciate the grandeur of the undertaking it should be remembered that the next subaqueous tunnel was opened about 4000 years later, in 1843. This was the tunnel under the River Thames in London (Szechy, 1973).

The purpose of tunnels is to ensure the direct transportation of passengers or goods through certain obstacles. Depending on the obstacle to be overcome and on the traffic, or transportation objective to be achieved, tunnels can be classified into various groups.

A tunnel is much more than just a tunnel. It serves any of myriad functions - highway, railroad, or rapid transit artery; pedestrian passageway; fresh water conveyance, cooling water supply, wastewater collector or transport; hydropower generator; or utility corridor. Tunnels are constructed by cut-and-cover methods; in long prefabricated sections sunk in place as in immersed tubes; in short prefabricated sections pushed into place from jacking pits; by drilling and blasting; by mechanized means such as tunnel boring machines or continuous miners (roadheaders), with the aid of a protective shield in free or compressed air; and they will eventually be constructed in ways now existing only in our imaginations. In cross-section it takes one of several shapes; circular, multicurve, horseshoe, cathedral arch, arched or flat-roofed, and with clear spans of from a few meters to more than 15 m and, in cavern form, much wider. Its length can vary from less than 30 m to more than 50 kilometers. A tunnel can be located in any of a variety of places; under mountains, cities, rivers, lakes, sea estuaries, straits or bays. Finally, a tunnel is constructed in one innumerable media; soft ground, mixed face, rock, uniform, jumbled, layered, dry, wet, stable, flowing, squeezing.

Most of all, a tunnel exists because there is demonstrated need-to move people or material where no other means is practical or adequate, or to accomplish the required movement more directly, more quickly, or less obtrusively. The need maybe for storage, either short term as for storage of stormwater flows to reduce the otherwise high peak capacities required of wastewater treatment plants, or longer term as for storage of vital raw materials or products.

The obstacle may be a mountain, a body of water, dense urban, or industrial areas (traffic, etc.). Tunnels may pass accordingly under mountains, rivers, sea channels, dense urban, or industrial areas, buildings and traffic routes. Their purpose may be to carry railway, road, pedestrian, or water trafic, to convey water, electric power, gas, sewage, etc. or to provide indoor transportation for industrial plants. Tunnels may thus be classified according to their purpose, location and geological situation. Depending on their purpose the following two main groups of tunnels may be distinguished:

A. Traffic tunnels:

1. railway tunnels,
2. highway tunnels,
3. pedestrian tunnels,
4. navigation tunnels,
5. subway tunnels.

B. Conveyance tunnels:

1. hydroelectric power station tunnels,
2. water supply tunnels,
3. tunnels for the intake and conduit of public utilities,
4. sewer tunnels,
5. transportatian tunnels in industrial plants.

In addition to purpose, important classification criteria are location, position relative to the terrain and alignment as well, these having a decisive influence on the tunnel section, the method of construction, the design and the acting forces. Tunnels will hereafter be understood as being underground structures, which apart from serving the above purposes, are constructed by special underground tunnelling methods generally without disturbing the surface.

Tunnels are analysed in several ways according to their construction techniques, shapes, prevailing ground conditions, etc. There are non-numerical ways of obtaining good predictions of the likely ground response to tunnelling and the likely loads in a tunnel lining. These conventional design tools are arguably cheaper and quicker to use. But they are characteristically uncoupled, i.e. the loads are determined by one technique (usually an elastic solution), and movements by another (usually empirical) the two being not linked together. Furthermore, the information gained from conventional analysis is often limited. In a real tunnel, however, the different facets are clearly coupled and the problem is complex, involving pore pressure changes, plasticity, lining deformations and existing structures. Numerical procedures, such as the finite element technique, lend themselves to the analysis of such complex problems (Potts and Zdravkovic, 2001). The finite element method can:

- Simulate construction sequence.
- Deal with complex ground conditions.
- Model realistic soil behaviour.
- Handle complex hydraulic conditions.
- Deal with ground treatment (e.g. compensation grouting).
- Account for adjacent services and structures.
- Simulate intermediate and long term conditions.
- Deal with multiple tunnels.

There are several methods, which are also mentioned in the preceding chapters for analysing tunnel structures. However, amongst them, the most popular one is Finite Element Method. While the finite element method has been used in many fields of engineering practice for over thirty years, it is only relatively recently that it has begun to be widely used for analysing geotechnical problems. This is probably because there are many complex issues which are specific to geotechnical engineering and which have only been resolved recently. When properly used, this method can produce realistic results which are of value to practical engineering problems.

While using numerical methods like Finite Element Method for the solution of that kind of real three dimensional problems, some approximations and simplifications are made to get the solution easier. Although there are many geotechnical problems that can be approximated to either plane strain or axi-symmetric conditions, some remain which are very three dimensional. Such problems will therefore require full three dimensional numerical analysis. In the next chapters, some methods and solutions are presented how to account for such behaviour.

In the light of summarized information above, construction of Bolu Tunnels is investigated as a case here. Bolu Tunnels are transpotation purposed highway tunnels. The tunnels are part of Trans European Motorway (TEM) connecting Eastern Europe with the Middle East. They are approximately 3.0 km long, 40 m apart and have excavated cross sections more than 200 m^2. Original design was based on New Austrian Tunnelling Method principles. However, some modifications and adaptations are made on the design for considering difficult and challenging geotechnical conditions. In this thesis, tunnels are discretized according to available data by 2D and 3D models to study the effects lost while modelling 2-dimensional, instead of modelling as 3-dimensional.

In the following chapters, first theory, preliminary information and earlier works are presented from the literature. Then, the case of Bolu Tunnels is presented with illustrative figures and photographs. Earlier studies, on the issue are also rewieved. Finally, finite element analyses results are discussed.

CHAPTER 2

LITERATURE STUDY AND PRELIMINARY CONCEPTS

2.1. General Approach in Geo-Engineering Processes

The modeling of geo-engineering processes involves special considerations and a design philosophy different from that followed for design with fabricated materials. Analyses and designs for structures and excavations in or on rocks and soils must be achieved with relatively little site-specific data and an awareness that deformability and strength properties may vary considerably. It is impossible to obtain complete field data at a rock or soil site; for example, information on stresses, properties and discontinuities can only be partially known, at best.

Since the input data necessary for design predictions are limited, a numerical model in geomechanics should be used primarily to understand the dominant mechanisms affecting the behavior of the system. Once the behavior of the system is understood, it is then appropriate to develop simple calculations for a design process.

This approach is oriented toward geotechnical engineering, in which there is invariably a lack of good data, but in other applications it may be possible to use geotechnical engineering software directly in design if sufficient data, as well as an understanding of material behavior, are available. The results produced in a software analysis will be accurate when the program is supplied with appropriate data. Modelers should recognize that there is a continuous spectrum of situations, as illustrated in Figure 2.1.

Typical situation	Complicated geology; inaccessible; no testing budget		Simple geology; $$$ spent on site investigation
Data	NONE		COMPLETE
Approach	Investigation of mechanisms	Bracket field behavior by parameter studies	Predictive (direct use in design)

Figure 2.1 Spectrum of modelling situations (taken from FLAC^{3D} Manual (1997), Vol.1, Chapt.3, pg.2)

Software may be used either in a fully predictive mode (right-hand side of Figure 2.1) or as a “numerical laboratory” to test ideas (left-hand side). It is the field situation (and budget), rather than the program, that determine the types of use. If enough data of a high quality are available, softwares can give good predictions.

Since most software applications will be for situations in which little data are available, it is discussed that the recommended approach for treating a numerical model as if it were a laboratory test. The model should never be considered as a “black box” that accepts data input at one end and produces a prediction of behavior at the other. The numerical “sample” must be prepared carefully and several samples tested to gain an understanding of the problem. The steps recommended to perform a successful numerical experiment in geomechanics are listed below. Each step is discussed separately (FLAC^{3D} Manual,1997).

- **Step 1**: Define the objectives for the model analysis
- **Step 2**: Create a conceptual picture of the physical system
- **Step 3**: Construct and run simple idealized models
- **Step 4**: Assemble problem-specific data

- **Step 5**: Prepare a series of detailed model runs
- **Step 6**: Perform the model calculations
- **Step 7**: Present results for interpretation

Step 1: Define the Objectives for the Model Analysis

The level of detail to be included in a model often depends on the purpose of the analysis. For example, if the objective is to decide between two conflicting mechanisms that are proposed to explain the behavior of a system, then a crude model may be constructed, provided that it allows the mechanisms to occur. It is tempting to include complexity in a model just because it exists in reality. However, complicating features should be omitted if they are likely to have little influence on the response of the model, or if they are irrelevant to the model's purpose. It is started with a global view and if necessary refinement is made.

Step 2: Create a Conceptual Picture of the Physical System

It is important to have a conceptual picture of the problem to provide an initial estimate of the expected behavior under the imposed conditions. Several questions should be asked when preparing this picture. For example, is it anticipated that the system could become unstable? Is the predominant mechanical response linear or non-linear? Are movements expected to be large or small in comparison with the sizes of objects within the problem region? Are there well-defined discontinuities that may affect the behavior, or does the material behave essentially as a continuum? Is there an influence from groundwater interaction? Is the system bounded by physical structures, or do its boundaries extend to infinity? Is there any geometric symmetry in the physical structure of the system?

These considerations will dictate the gross characteristics of the numerical model, such as the design of the grid, the types of material models, the boundary conditions, and the initial equilibrium state for the analysis. They will determine whether a three-dimensional model is required or if a two dimensional model can be used to take advantage of geometric conditions in the physical system.

Step 3: Construct and Run Simple Idealized Models

When idealizing a physical system for numerical analysis, it is more efficient to construct and run simple test models first, before building the detailed model. Simple models should be created at the earliest possible stage in a project to generate both data and understanding. The results can provide further insight into the conceptual picture of the system; Step 2 may need to be repeated after simple models are run.

Simple models can reveal shortcomings that can be remedied before any significant effort is invested in the analysis. For example, do the selected material models sufficiently represent the expected behavior? Are the boundary conditions influencing the model response? The results from the simple models can also help guide the plan for data collection by identifying which parameters have the most influence on the analysis.

Step 4: Assemble Problem-Specific Data

The types of data required for a model analysis include:

- details of the geometry (e.g., profile of underground openings, surface topography, dam profile, rock/soil structure),

- locations of geologic structure (e.g., faults, bedding planes, joint sets),
- material behavior (e.g., elastic/plastic properties, post-failure behavior),
- initial conditions (e.g., in-situ state of stress, pore pressures, saturation), and
- external loading (e.g., explosive loading, pressurized cavern).

Since typically, there are large uncertainties associated with specific conditions (in particular, state of stress, deformability and strength properties), a reasonable range of parameters must be selected for the investigation. The results from the simple model runs (in Step 3) can often prove helpful in determining this range and in providing insight for the design of laboratory and field experiments to collect the needed data.

Step 5: Prepare a Series of Detailed Model Runs

Most often, the numerical analysis will involve a series of computer simulations that include the different mechanisms under investigation and span the range of parameters derived from the assembled data base. When preparing a set of model runs for calculation, several aspects, such as those listed below, should be considered.

1. How much time is required to perform each model calculation? It can be difficult to obtain sufficient information to arrive at a useful conclusion if model run times are excessive. Consideration should be given to performing parameter variations on multiple computers to shorten the total computation time.

2. The state of the model should be saved at several intermediate stages so that the entire run does not have to be repeated for each parameter variation. For example, if the analysis involves several loading/unloading stages, the user should be able to return to any stage, change a parameter and continue the analysis from that stage. Consideration should be given to the amount of disk space required for save files.

3. Is there a sufficient number of monitoring locations in the model to provide for a clear interpretation of model results and for comparison with physical data? It is helpful to locate several points in the model at which a record of the change of a parameter (such as displacement, velocity or stress) can be monitored during the calculation. Also, the maximum unbalanced force in the model should always be monitored to check the equilibrium or plastic flow state at each stage of an analysis.

Step 6: Perform the Model Calculations

It is best to first make one or two detailed model runs separately before launching a series of runs. These runs should be stopped and checked intermittently to ensure that the response is as expected. Once there is assurance that the models are performing correctly, several model data files can be linked together to run a number of calculations in sequence. At any time during a sequence of runs, it should be possible to interrupt the calculation, view the results, and then continue or modify the model as appropriate.

Step 7: Present Results for Interpretation

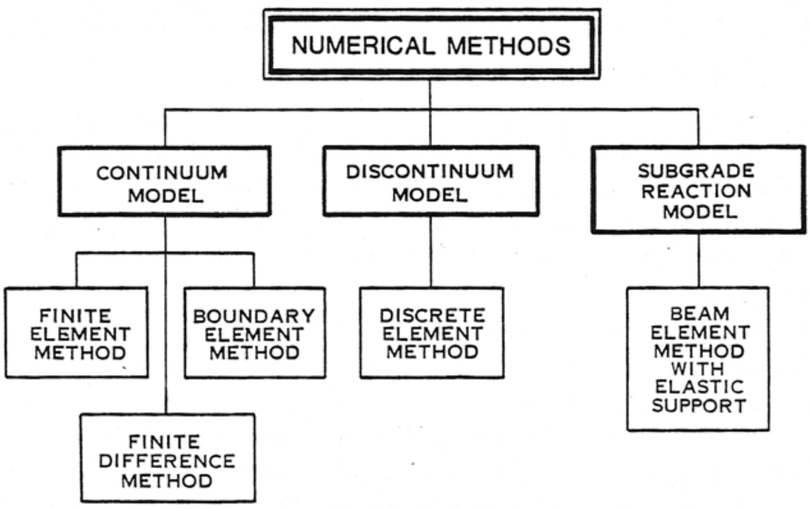
The final stage of problem solving is the presentation of the results for a clear interpretation of the analysis. This is best accomplished by displaying the results graphically, either directly on the computer screen or as output from a hard-copy plotting device. The graphical output should be presented in a format that can be directly compared to field measurements and observations. Plots should clearly identify regions of interest from the analysis, such as locations of calculated stress concentrations or areas of stable movement versus unstable movement in the model. The numeric values of any variable in the model should also be readily available for more detailed interpretation by the modeler.

It is recommended that these seven steps be followed to solve geo-engineering problems efficiently.

2.2 Description and Comparison of Numerical Methods

Numerical methods used for tunnel engineering are listed in Table 2.1. Each method listed involves a discretization of the problem domain, which is facilitated by a computer-assisted analysis. Three different models are identified in the Table as the basis for the numerical methods discussed below. These models are: Continuum Model, Discontinuum Model, and Subgrade Reaction Model.

Table 2.1 Numerical methods and models for tunnel engineering (Gnilsen, 1989).



The numerical methods associated with these models are: Beam Element Method with Elastic Support, Finite Element Method (FEM), Finite Difference Method (FDM) , Boundary Element Method (BEM), and Discrete Element Method (DEM). In addition, hybrid methods have evolved by combining two or more of these individual methods. The methods are discussed individually below by Gnilsen (1989).

2.2.1 Beam Element Method with Elastic Support

The Beam Element Method is also referred to as "Coefficient of Subgrade Reaction Method", and is illustrated in Figure 3-1a. The tunnel lining is simulated by beam elements. The surrounding ground, that provides the embedment of the lining, is simulated by spring elements. Spring elements are typically oriented perpendicular to the lining, simulating the normal stresses induced to the ground from outward lining deflection. In addition, tangential spring elements can simulate shear stresses induced between the lining and the ground. The stiffness of the spring elements is determined from the stiffness, i.e. the modulus, of the ground and the curvature of the lining. To simulate actual conditions, spring elements under tension must be eliminated from the calculation. This is done through an iterative process. The strengths and weaknesses of the method are:

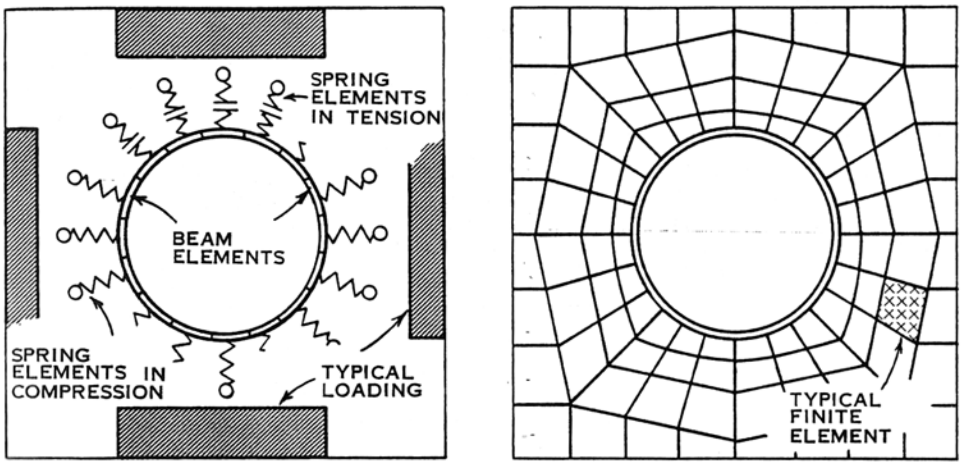
Strengths:

- A large number of structural computer programs can be used to analyze a tunnel lining by means of the Beam Element Method with Elastic Support. The required computer processing and storage capacity is typically small compared with that required for other numerical methods.

Weaknesses:

- The model used for the Beam Element Method with Elastic Support can only simulate simple or very simplified ground and tunnel conditions.

- Each spring element simulates the embedment that is provided by the ground area it represents. Unlike in real conditions, the spring elements, i.e. supporting ground areas, are not connected with each other.



a. b.

Figure 2.2 a. Beam element model with elastic support. **b.** Finite element model (Gnilsen, 1989)

2.2.2 Finite Element Method (FEM)

In the Finite Element Method (Figure 2.2b), the subsurface is predominantly modeled as a continuum. Discontinuities can be modeled individually. The problem domain, i.e. host ground, is discretized into a limited number of elements that are connected at nodal points. Each element is finite, i.e. geometrically defined and limited in size. This characteristic makes for the name of the method, Finite Element Method. The stress-strain relationship of the ground is described by an appropriate constitutive (material) law. The stress, strain and deformation to be analyzed are caused by changing the original (primary) subsurface condition. Such change is, for instance, induced by the tunnelling process. Stress, strain and deformation induced in one element impacts the behavior, of its neighbouring elements, and so forth.

The complex interrelation between the interconnected elements makes for a highly complex mathematical problem. The analysis is performed by solving the equation matrix that models, the mesh made up of the limited number of elements. That is, a system of equations is set up which relates unknown quantities to known quantities via a global stiffness matrix. For instance, the relationship of nodal forces to displacements is analyzed this way throughout the finite element mesh. The concept to solve for unknown values at all points at one time is referred to as implicit approach. For additional selected references on mathematical concepts of the finite element method see Zienkiewicz (1971) and Bathe (1982). The strengths and weaknesses of the method are:

Strengths:

- Highly complex underground conditions and tunnel characteristics can be analyzed. The capability of the Finite Element Method includes the simulation of complex constitutive laws, non-homogeneities, and the impact of advance and time dependent characteristics of the construction methods.

Weaknesses:

- Solving of the complex mathematical problem requires a large computer processing and storage capacity.

- Most Finite Element programs require more program and computer knowledge from the user than other methods do. Also, extensive output is typically generated that makes comprehension of the results more difficult. As a minimum, some graphical display capability should be included with the program. For very complex problems, for instance three-dimensional computations, a pre- and post-processing program is indispensable to facilitate data handling.

- Unless a hybrid model is formed, arbitrary external boundary conditions of the Finite Element Model must be defined. In order to avert any impact from these boundaries on the analysis of stress, strain and deformation close-by and along the tunnel circumference, the boundaries are set at a sufficient distance away from the tunnel. Consequently, a large mesh is required that relates to a large required computer capacity.

2.2.3 Finite Difference Method (FDM)

The method is similar to the Finite Element Method in that the subsurface is modeled as a continuum that is divided into a number of elements which are interconnected at their nodes. The primary difference lies in the approach used to solve the unknown parameters. In contrast to the implicit approach of the Finite Element Method, the Finite Difference Method is explicit approach discussed in the following.

The explicit method builds on the idea that for a small enough time step, a disturbance at a given mesh point is experienced only by its immediate neighbours. This implies that the time step is smaller than the time that the disturbance takes to propagate between two adjacent points. For most Finite Difference programs this time step is automatically determined such that numerical stability is ensured. Initially conceived as a dynamic, i.e. time related, computation approach the Finite Difference method can be used to solve static problems by damping the dynamic solution. Then, "time step" does not refer to a physical but rather to a problem solution (time) step. Analyzed velocities relate to displacement in length per time step.

The separate solution for individual mesh points implies that no matrices need to be formed. For each time step an individual solution is obtained for each mesh point. The calculation cycle leading to the solution involves Newton's law of motion and the constitutive law of the in situ material. The acceleration solved for a mesh point is integrated to yield the mesh point velocity, which in turn is used to determine the strain change. Subsequently, strains determine the corresponding stress increments which in turn generate forces on the surrounding mesh points. These are summed to determine the resulting out-of-balance force which relates to the acceleration that started the calculation cycle. The method is described in more detail by Cundall and Board (1988). The strengths and weaknesses of the method are:

Strengths:

- The explicit approach facilitates analysing the behavior of the problem domain as it evolves with time. This allows for a step-by-step analysis of possible failure mechanisms.

- Because no matrices are formed the required processing and storage capacity of the computer is relatively small.

- The solution without matrices also allows for the analysis of large displacements without significant additional computer effort.

- Most efficient for dynamic computations.

Weakness:

- If used for static problems the method may require more computation time than most other numerical methods.

2.2.4 Boundary Element Method (BEM)

This method has only recently gained on popularity in the engineering community. Today, the Boundary Element Method is increasingly used for the linear and non-linear static, dynamic and thermal analysis of solids. Likewise, transient heat transfer and transient thermal visco-plasticity is simulated with the method. The use of the Boundary Element Method for tunnel engineering is also growing (Banerjee and Dargush, 1988).

Like the Finite Element Method and Finite Difference Method, the Boundary Element Method models the ground as a continuum. Some of the differences to those methods are:

1. Unless singularities of the ground mass shall be modeled, a discretization of the problem domain is necessary for the excavation boundary only. A numerical calculation is confined to these boundary elements. The medium inside those boundaries is typically described and simulated by partial differential equations. These equations are most often linear and represent approximate formulations of the actual conditions.

2. Contrary to the Finite Element Method and Finite Difference Method, the problem is solved by integration of the partial differential equations. This approach gives the Boundary Element Method the alternatiye name "Integral Method". For more detail on the boundary element method, reference is made to Crouch and Starfield (1983).

The excavation boundaries are also referred to as "external boundaries". If discontinuities between the external boundaries shall be analyzed, "internal boundaries" are introduced. "Internal boundaries" model the interfaces between different material types or discontinuities. The method involving the analysis of internal boundary elements is referred to as "Displacement Discontinuity Method" and represents a specific type of the Boundary Element Method. The strengths and weaknesses of the method are:

Strengths:

- The system of equations to be solved is small compared with that required for the Finite Element Method. Hence, a comparably small computer capacity is sufficient.

- Data input and output are comparably simple and are easily processed.

- The Boundary Element Method is very efficient and economical for two or three-dimensional problems when the defined boundaries are of greatest concern.

Weaknesses:

- Today the capacity of most boundary element programs is, with few exceptions, limited to linear constitutive ground behavior. Even so, much progress is currently under way with program developments.

- Complex construction material characteristics procedures and time dependency of cannot be modeled easily.

2.2.5 Discrete Element Method (DEM)

The Disercte Element Method is also referred to as "'Distinct Element Method" or "Rigid Block Method". In contrast to the methods discussed above, the ground mass is not modeled as a continuum. Rather, the ground mass is modeled by individual blocks that are rigid in themselves. The method is applicable if the joint displacements so overshadow the internal block deformation that the latter can be neglected. In this case, the deformation of the ground mass is governed by the movement along the joints between rigid blocks.

The Discrete Element Analysis begins with the computation of incremental forces acting in the joints. The resulting aecelerations of the rigid blocks are integrated to give new positions and orientations of the block centroids. This in turn yields new increments of joint forces, which continue the calculation cycle. See Cundall (1976) for more details. The strengths and weaknesses of the method are:

Strengths:

- The method is especially useful for kinematic studies of large block systems, e.g., where highly jointed rock masses around the tunnel are modelled.

- The magnitude of block movements that can be analyzed is large compared with that obtained from most continuum models. The required computer capacity is comparably small.

Weaknesses:

- The computation requires the input of joint location and orientation. This information is not normally known prior to construction of the tunnel. Even so, parameter ,studies can be performed by assuming various joint configurations.

2.2.6 Hybrid and Complementary Methods

Each numerical method may be used most efficiently if combined with other numerical methods. The purpose of coupling individual numerical methods is typically twofold. First, the strengths of each method can be preserved while its weaknesses may be eliminated. Secondly, the combination of individual methods and their associated models can create a model that best describes the specific problem. Several forms of model combinations are:

1. The problem domain is divided into two or more areas that are analyzed simultaneously. Different models are used for each area.

2. The analysis of the problem domain is performed in two or more computation steps. Different models are used for each step. The outcome of one step is used as input for the subsequent step.

3. The model is first used that is best suited to validate computation parameters. Subsequently, the validated parameters are used with a different model that best generates the necessary data for design.

2.3 Modelling for Numerical Computations

Numerical computations, as a tool of tunnel engineering, aim to analyze, i.e., reproduce, explain and predict the behavior and response of structures and media subjected to impacts from tunneling. Establishing a model of the "real world" conditions is necessary if physical and mathematical concepts are to be employed for the analysis. The modeling of "real world" conditions is difficult because of the unknowns of the subsurface, the complexity of the subsurface and tunnel behaviour, and the problems associated with formulating proper constitutive laws of the ground. Since it is neither possible nor useful to simulate all conditions and parameters in detail, a simplified model must be described. Without simplification, an accuracy might be pretended that could easily prove false. Also, cost considerations typically call for a simplified computation model. Even so, the results gained from the numerical computation must still be of benefit to the engineer and to the engineered product. The experience of the engineer with the numerical tool used is vital for proper interpretation of the results. This includes the understandig of the impacts that specific program characteristics have on the calculation outcome. Each computation method has its strengths and weaknesses (see also Potts and Zdravkovic, 2001).

Model simplification can be achieved by employing one or several of the following approaches:

- Three-dimensional conditions modeled in two-dimensions.
- Utilization of section symmetries.
- Simplified modelling of the ground and the tunneling process.

2.3.1 Three Dimensions Simulated by 2-D Model

Three dimensionality of the subground and tunneling conditions has been analyzed by Wittke (1977) and other auhors, and can be found in various forms:

- Anisotropy of the rock mass (schistose rock, etc.) and discontinuities extending in three dimensions.

- Three-dimensional spatial geometry of the analyzed problem area, for instance in the proximity of portals, pillars, end walls, and the advancing excavation face. Figure 2.3 depicts a tunnel in face proximity where three-dimensionality is encountered in two ways: First, load transfer due to tunnelling induced stress redistribution in the subground occurs in directions both transverse and longitudinal to the tunnel axis. Second, displacements occur along the tunnel circumference, in the ground ahead of the tunnel, and at the tunnel face. The latter can represent a stability case for which a three-dimensional analysis may be critical.

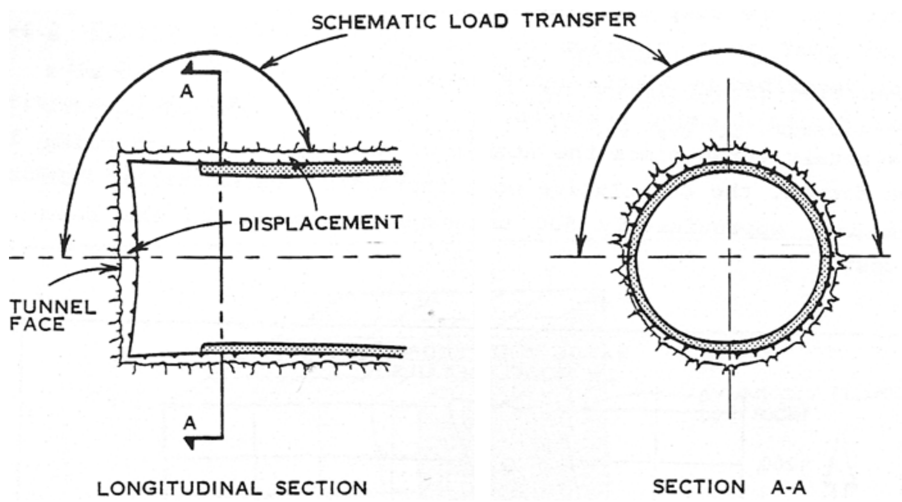


Figure 2.3 Three dimensionality of load and displacement at the tunnel interface (Gnilsen, 1989)

Despite these three-dimensionalities frequently encountered in nature, a three-dimensional analysis is often not necessary.

Instead, a two-dimensional model can be substituted. The decision on whether a two-dimensional or three-dimensional model offers the best solution, should include the following considerations:

- The size and complexity associated with a three-dimensional model, compounded by imperfections inherent to any computer program, may adversely affect the calculation results. Also, the description and processing of complicated models promotes inaccuracies and errors with the calculation input development. Similarly, the large number of calculation output parameters can be difficult to process and interpret. In addition to the engineering computer program, a pre-processing and post-processing program is needed to alleviate these problems.

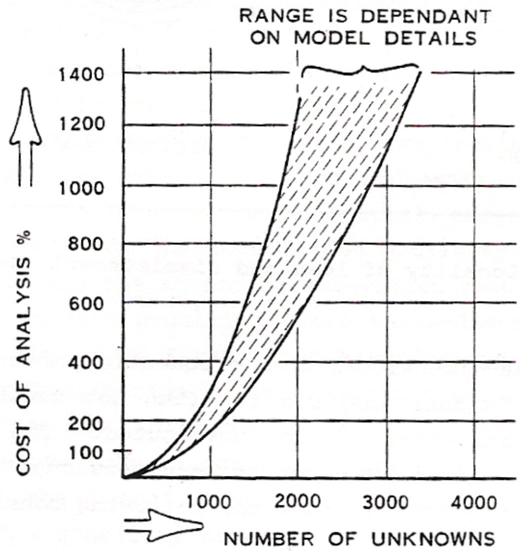


Figure 2.4 Computer cost as function of the number of unknowns - finite element calculation example (Gnilsen, 1989).

- Calculation Costs: The cost to perform a three dimensional analysis obviously exceeds that of the two-dimensional analysis. Costs are incurred due to labor and computer use. For the three-dimensional analysis, additional labor relates to the engineer's efforts to prepare the calculation input data and to evaluate and interpret the calculation results. Costs associated with computer use relate to the type of computer required and the computer time necessary. Figure 2.4 shows the computer cost as a function of the number of unknowns for a problem described by Wittke and Pierau (1976). For the case of a typical Finite Element Analysis, the number of unknowns equals approximately three times the number of nodal points. According to the figure, the calculative cost increases exponentially. In Figure 2.3, approximately 500 unknowns correspond to the base 100% cost.

The simulation of three-dimensional conditions by a two dimensional model requires experience and the understanding of the relationship between these two models. The proper two-dimensional simulation of the three-dimensional load transfer in face proximity (see Figure 2.4) has proven particularly critical to obtaining valid calculation results. This aspect is mentioned in more detail in Section 2.5.

2.3.2 Utilization of Symmetry

If the geometry, the ground mass properties, and in situ stresses are symmetrical to the vertical tunnel axis, only half of the continuum must be analyzed. Figure 2.5 illustrates an example of such simplification.

2.3.3 Simplified Modelling of Subground and Tunnelling Process

The unknowns and complexity of the subground and the tunneling conditions require that simplifications be made for the calculation model. In the following section this is described for the Finite Element Method.

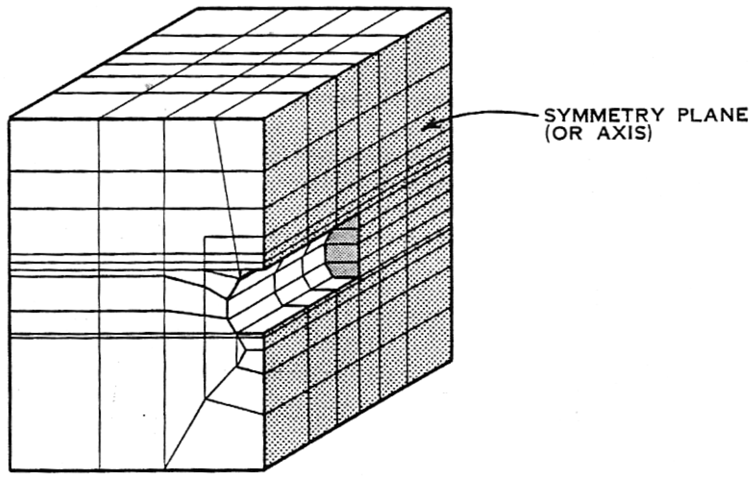


Figure 2.5 Symmetry of the computation model (Gnilsen, 1989)

2.4 Modelling of Subsurface

This section follows up on the discussion of the previous section : Modelling for Numerical Computations. As was stated before, some simplification of the computation model is necessary. This applies also to modelling of the subsurface.

Care is required in the selection of the finite element mesh that models the medium. It is important that the size and type of finite elements be properly selected to ensure accuracy of analysis, convergence of solution and minimization of rounding errors during numerical calculation.

A large number of elements will usually render high accuracy of analysis but will require larger computer capacity and longer computer runs. This may increase the cost of analysis. The finite elements selected should not create spurious energy modes or cause shear locking or membrane locking. Generally, a patch test

(Bathe, 1982) will identify ill-conditioned elements which should be eliminated and modified to obtain a realistic finite element analysis. The aspect ratio (longest/smallest dimension of element) should not exceed three, otherwise considerable calculation errors will be generated.

To some extent, common or similar approaches are used to model the subsurface regardless of actual subsurface conditions. One common approach is that a constitutive law is established that governs the stress/strain relationship of the ground. Constitutive laws used for geotechnical engineering computations are linear elastic, non-linear elastic, linear visco-elastic, elasto-plastic, elasto-visco-plastic, isotropic, anisotropic, thermal-dependent or stochastic. In addition, specific modelling and simulation requirements vary with the ground characteristics in question. Specifics of subsurface modelling relate to the different unknown subsurface parameters and the different parameters that affect ground mass behaviour. For the purpose of discussing subsurface-specific modelling, a distinction is made in the following between rock subsurface and soil subsurface.

2.4.1 Modelling of Soil Subsurface

Modelling considerations for a soil subsurface may differ somewhat from those for a rock mass. For a rock mass, discontinuities pose the prime problem to the model. By comparison, no distinct discontinuities are typically encountered in soil. Instead, the "intact" soil is often difficult to describe. Problems commonly associated with modelling of the soil subsurface are:

- The variability of soil parameters obtained from testing is often too high to determine true values. Substantial efforts have been made in recent years to develop constitutive models for soil. By comparison, the reliability of material constants determined from experimental data has not been addressed adequately (Zaman et al., 1988).

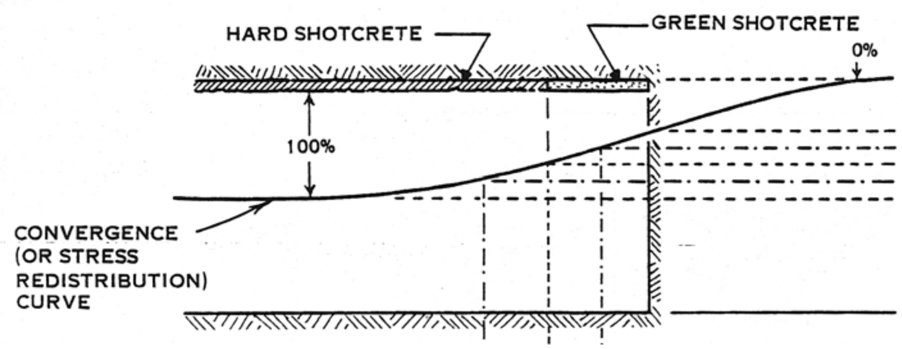


Figure 2.6 Variation of model parameters (Gnilsen, 1989)

- Soil parameters may vary with time due to changing subsurface conditions. Changing conditions may relate to creep effects or to the impact of ground water. For instance, the increasing strength of the shotcrete lining as a function of time, and changing loading conditions as a function of distance from the tunnel face, are taken into account.

- Changing loading and stress condition in the soil a1so relate to the rheologic behaviour where encountered. Changing and complex soil response under complex loading conditions represents another difficulty of modelling the soil subsurface.

2.5 Tunnel Excavation Modelling

Tunnel excavation is a three dimensional engineering process. While recognising that three dimensional analysis is becoming possible in the work place, it is still two dimensional modelling that dominates. This is because there are practical limits on cost and computer resource which, when performing analyses sufficiently sophisticated to handle all the complexities outlined before, restrict us to two dimensional modelling. If multiple shallow tunnels are to be analysed, or if the ground surface response is key to the analysis then a plane strain representation of the transverse section is required (e.g. to study effects on structures, Figure 2.7). If a single deep tunnel is to be investigated, and surface effects are not of prime interest, then an axially symmetric approximation may be appropriate and heading advance can be studied, all be it within a simplified stress regime (Figure 2.7). For the reviewed subsections on Tunnel Excavation Modelling below, see also Potts and Zdravkovic (2001).

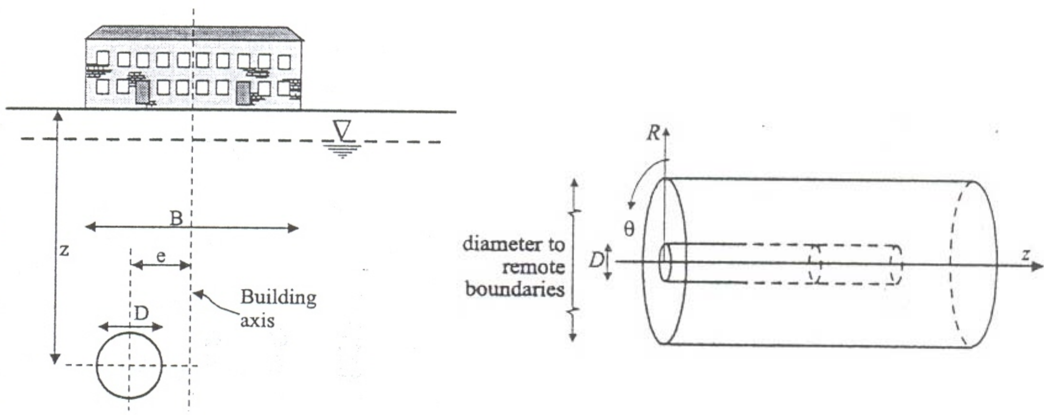


Figure 2.7 Plane strain and axi-symmetric modelling cases (Potts and Zdravkovic, 2001)

2.5.1 The Gap Method

This method was introduced by Rowe et al. (1983). A predefined void is introduced into the finite element mesh which represents the total ground loss expected. In this way the out of plane and in plane ground losses are incorporated together with additional losses to allow for miss-alignment of the shield, the quality of workmanship, and the volume change due to soil remoulding. It is clear therefore how one can account for the different tunnel construction methods outlined above by varying the size of the void.

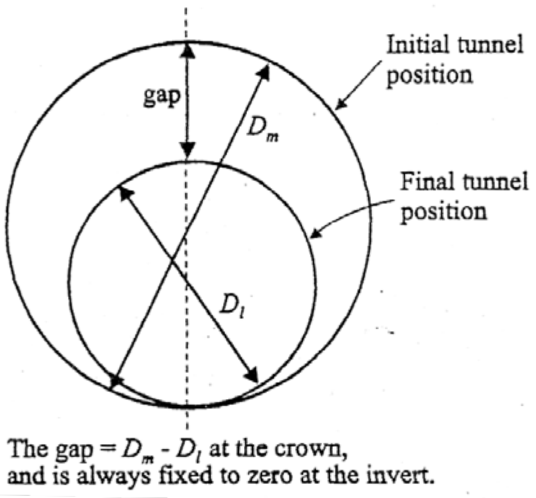


Figure 2.8 The Gap Method for modelling tunnel excavation (Potts et al., 2001)

The void is placed around the final tunnel position and so locates the soil boundary prior to excavation (Figure 2.8). This is achieved by resting the invert of the tunnel on the underlying soil and prescribing the gap parameter at the crown. The gap parameter is the vertical distance between the crown of the tunnel and the initial position before tunnelling. The analysis proceeds by removing boundary tractions at the perimeter of the opening and monitoring the resulting nodal displacements. When the displacement of a node indicates that the void has been closed and the soil is in contact with the predefined lining position, soil/lining interaction is activated at that node. The soil and the lining are actually treated as separate bodies, related only by nodal forces (Rowe et al. (1978)).

2.5.2 The Convergence-Confinement Method

Another approach to modelling excavation is the λ or convergence-confinement method (Panet and Guenot (1982), in which the proportion of unloading before lining construction is prescribed, so volume loss is a predicted value. An internal force vector, $(1-\lambda)\{F_o\}$, is applied at the nodes on the tunnel boundary ($\{F_o\}$ being equivalent to the initial soil stresses $\{\sigma_o\}$). λ is initially equal to 0 and is then progressively increased to 1 to model the excavation process. At a prescribed value λ_d the lining is installed, at which point the stress reduction at the boundary is $\lambda_d\{\sigma_o\}$. The remainder of the stress reduction is applied to create the lining stress. The stress reduction with the lining in place is then $(1-\lambda)\{\sigma_o\}$ (see Figure 2.9).

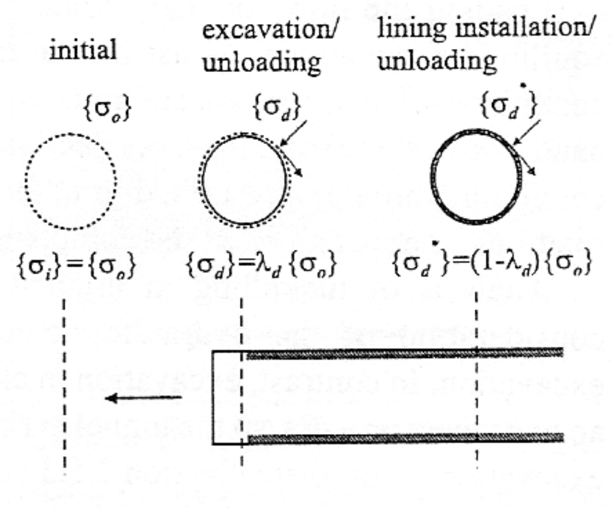


Figure 2.9 Convergence Confinement Method (Potts et al., 2001)

2.5.3 The Progressive Softening Method

A method termed progressive softening was developed for the modelling of NATM (or sprayed concrete) tunnelling, Swoboda (1979). The soil within the heading is softened by multiplying the soil stiffness by a reduction factor β. The effects of the softening are evident when excavation forces are applied to the boundary of the future tunnel. As with the convergence-confinement method, the lining is installed before the modelled excavation is complete, see Figure 2.10. If the tunnel is constructed with a bench and heading, then the above procedure can be applied to each of them sequentially. The same methodology could be applied to an analysis with side drifts.

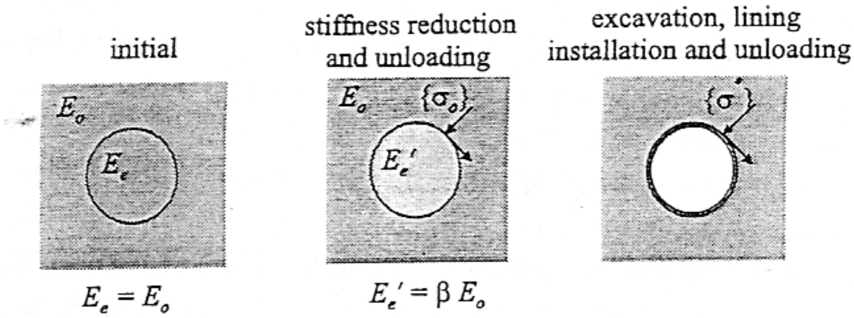


Figure 2.10 Progressive Softening Method (Potts et al., 2001)

2.5.4 The Volume Loss Control Method

This method is similar to the convergence-confinement method, but instead of prescribing the proportion of unloading prior to lining construction, the analyst prescribes the volume loss that will result on completion of excavation, see Figure 2.11. This method is therefore applicable to predictive analyses of excavation in soil types for which the expected volume loss can be confidently (and conservatively) determined for the given tunnelling method. It is also invaluable for worthwhile back analysis of excavations for which measurements of volume loss have been made.

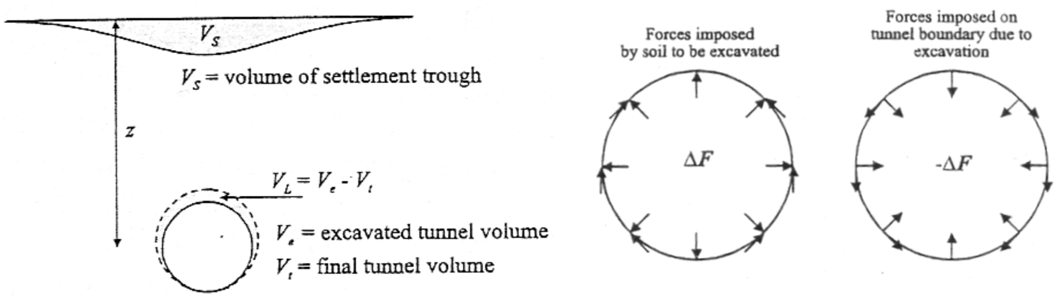


Figure 2.11 Volume Loss Method and modelling excavation of solid elements (Potts et al., 2001)

With the λ method, the outward support pressure on what is to be the tunnel boundary is progressively reduced. An alternative is for the program to calculate the value of $\{F_o\}$, the equivalent nodal forces which represent the pressure exerted, on what is to be the tunnel boundary, by the soil to be excavated. This is linearly apportioned to the number of increments, n, over which the excavation is to take place, to give $\{\Delta F\} = \{F_o\}/n$. The equal and opposite force vector $\{\Delta F\}$ is then applied at the excavation boundary for each of the n increments of excavation (Figure 2.11). The volume loss induced by each inerement of boundary loading can be monitored and the tunnel lining constructed on the increment at which the desired volume loss is achieved. After lining construction, the loading boundary condition, $\{-\Delta F\}$, is still applied to the excavation perimeter for the remainder of the n increments, thus introducing an initial stresses into the lining.

Depending on the stiffness of the lining further volume loss can occur during the latter process. It may therefore be necessary to install the lining at an increment which has a smaller volume loss than that desired, so that after full excavation the desired volume loss is achieved.

2.6 Loading Mechanism for Underground Structures

For sound engineering, the participation of the host media must be accounted for in the design which results in better engineering and reduced cost for the underground structure.

The loading mechanism of an underground structure is different from that of a surface or an aerial structure. For underground structures, the most important loading comes from the host ground itself. In competent host ground the ground loading on the underground structure is quite insignificant and may be equal to zero where as in incompetent ground, it may be quite significant.

The host ground pressures on the underground structure is quite complex. It is dependent on several factors such as the relative stiffness of the structure and the host ground, the elapsed time between the excavation and installation of support, the characteristics of the host ground, the in situ pressures, the size of the opening, the location of water table, and the adopted methods of construction.

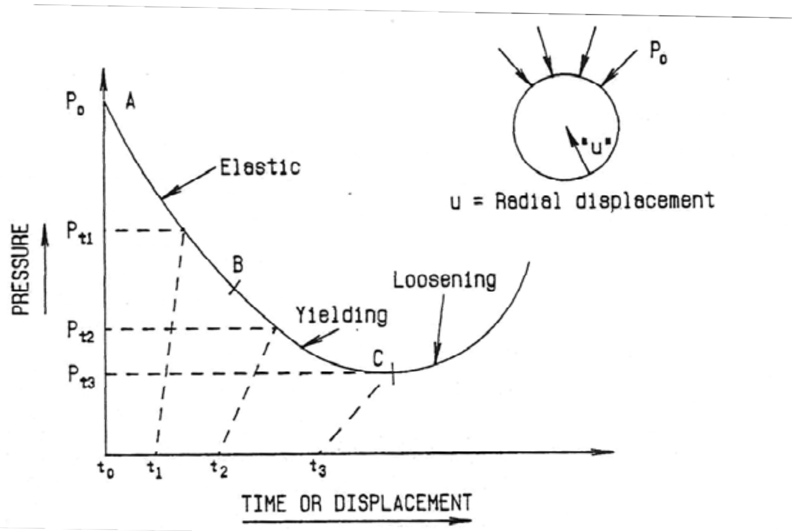


Figure 2.12 Characterictic Curve (Sinha, 1989)

If the support structures used to ensure the stablity of the opening is relatively stiffer than the host ground, the support structure will attract more loading. In the same situation, a support system that is more flexible than the host ground will take lesser load than a stiffer support. In case of a flexible support, the ground by arching will take the major portion of the load and the support system will take a smaller share of load. A stiffer support attracts more load and a flexible support attracts more displacement. A steel support is more flexible than a concrete lining.

Figure 2.12 indicates a ground characteristic curve in which the ground pressure is plotted as an ordinate and radial displacement as an abscissa.

At time "t_o" the theoretical pressure on support is "P_o" the in situ pressure and the radial deformation "u" is zero. Theoretical, because it is impossible to place a support without relaxing the ground and without reducing "P_o". When an opening is created, the excavation moves toward the opening and the value of "P_o" starts to diminish. The portion "AB" of ground characteristic curve is purely elastic. From "B" to "C" the ground starts to yield, but by "arching", it can still take some load. From "C" onward, the ground starts to "loosen" and it can no longer sustain any load. At time "t_1" when a support is placed to arrest the radial movement, it will have to sustain a pressure equal to "P_{t1}." . If the same support is placed at time "t_2," it will have to sustain a load equal to "P_{t2}." As can be seen, "P_{t2}" is smaller than "P_{t1}". A prudent design will be to place the support at time "t_3" or just before the ground starts to loosen itself. At that time, the support will be required to sustain the least pressure "P_{t3}" to keep the opening stable. It is, however, very difficult to assess the exact time "t_3" after which the ground starts to loosen up.

Loosening load is a generic term and indicates the load that comes on the support structure immediately after the ground is excavated. In some cases, the final load coming on the support structure may ultimately exceed the loosening load with time due to the existence of "genuine ground pressure." The genuine ground pressure may be less than or equal to or be several times the in situ ground pressure that existed before the excavation.

2.7 NATM

Tunnelling design is based on engineering judgement and back analysis. In constructing a tunnel the following functions are to be achieved: (1) maintain stability during during construction, (2) prevent undesirable or excessive impact on surroundings, (3) function adequately over the life of the Project (Peck, 1969). To achieve all these, several construction techniques were established. Among these only the New Austrian Tunneling Method (NATM), which is a selected construction method in Bolu Tunnels, will be discussed here.

The New Austrian Tunneling Method (NATM), introduced by Rabcewicz (1964) was slow in getting acceptability throuhout the world. But the real breakthrough came when an Austrian contractor, using NATM, successfully drove a twin sing1e track railway tunnel at Mt. Lebanon in Pittsburgh, USA, in 1984 (Martin, 1987). Then followed the value engineering change proposal to construct, by using NATM methods, the Wheaton subway station and the associated tunnels. At this project an estimated cost saving of $36 million was demonstrated by using NATM. The proposal was accepted and the project completed at substantial savings. This second successful completion of the project by NATM and great cost savings caught the attention of engineers and, now, several other projeets using NATM are being contemplated.

The NATM is a method by which the host ground surrounding an excavation for an underground structure is made into an integral part of the support structure. The host ground and the external support structure together take the full load. The host ground takes a major share of the load and the support takes a much smaller share of the ground load. This results in saving costs of external support systems.

Recalling Kirsch's solution of stress around a circular cavity, one will notice that the tangential stresses are always higher than radial stresses when an opening is created. Thus, if a support system can provide tangential resistance in the form of increased frictional resistance at the support and host interface, then the further relaxation of stresses due to excavation can be adequately resisted. Shotcrete provides strong frictional resistance. The ideal resistance will be provided by a closed ring of a very thin shotcrete membrane. But many times it is not practical to close the invert of the opening by shotcreting. Thus, the shotcrete in the roof and the sidewalls have to provide the tangential resistance. In order to help the resisting capability of this open shotcrete ring thus formed, use of rockbolts become necessary.

Rabcewicz (1964) found that a 150 mm thick shotcrete layer applied to a 10 m diameter tunnel could sustain a loosening load of 23 m of rock. Use of steel or timber support system for the same situation had to be much more expensive.

The NATM is an observational method and requires (1) application of a thin layer of shotcrete with or without rock bolts, wire mesh fabric, and lattice girder; and (2) monitoring and observing the convergence of the opening.

If the observed convergence exceeds the acceptable limits, then subsequent applications of next layers of shotcrete are required until the convergence has stopped or is within the acceptable range. The shotcrete thickness is, thereby, optimized according to the admissible deformations.

The geometry of the opening is very crucial so that adequate ground arching action can develop. Straight reaches are carefully substituted by curved configurations. (see also Cording, 1989)

2.7.1 The Sprayed Concrete Lining (SCL) Method

The SCL method is a soft ground application of the New Austrian Tunnelling Method (NATM), see Figure 2.13. As well as standard circular section tunnelling, SCL can be used in competent ground to create large non-circular openings. The method of excavation is usually by independent track or wheel mounted hydraulic excavators. Support is provided as soon as possible by the application of sprayed concrete (shotcrete). This is often reinforced by a steel mesh or a series of steel hoops or arches installed before concreting. A permanent reinforced lining is usually created at a later date, either by the application of further shotcrete, or in-situ concreting. The current trend is, however, towards using a single shotcrete lining, adequately reinforced.

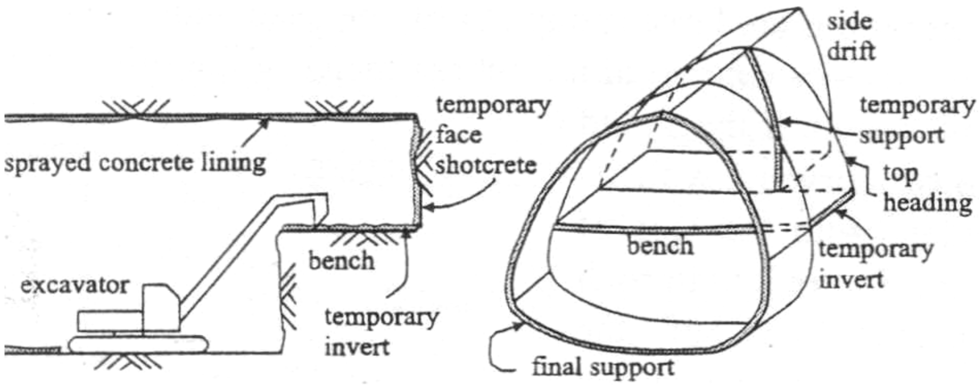


Figure 2.13 Schematic view of SCL method (Potts et al., 2001)

For large openings using SCL it is always the case that the tunnel is created by the method of advanced headings. This can involve excavation of the crown first, leaving a temporary invert, or the use of left and right side drifts, or a combination depending on the ground quality and the size of opening. In all cases the advanced heading is fully lined by shotcrete before the following drift commences.

2.8 Shotcrete

Shotcrete is "concrete shot from a fire hose." As used in underground work, experience and empirical rules dictate a total thickness of only two to six inches (usually four) (50 to 150 mm, usually 100 mm) of shotcrete, to support even very large openings. This is in sharp contrast to earlier North American designs of steel ribs for initial "temporary" support, followed by "final" cast-in-place concrete linings, usually at least 12 inches (300 mm) thick. Shotcrete obviously works, but the use of a thin and relatively fragile layer of shotcrete means that caution must be used by designers, and workmanship in the field is all-important (Rose,1989).

Shotcrete originated with drill+blast work, but the use of modern Tunnel Boring Machines (TBM's) makes the task of good shotcrete design more complex. Steel-fiber-reinforced shotcrete (SFRS) with microsilica is recommended as sturdy, economical, and conservative.

Shotcrete is concrete placed by shooting the cement, aggregates water and various additives (accelerator, retarder, plasticizers, steel fibers, microsilica, etc.) through a hose using compressed air. Shotcrete was first developed in the early 1900's by Carl Akeley of the Smithsonian Institution, to spray on molds of animals in the museum. In 1915, the Allentown Cement Gun company bought Akeley's patent. In the 1930's, Rabcewicz began to use shotcrete for tunnel support in Iran and later in Europe, leading eventually to his concept and theories (with co-workers) of the New Austrian Tunnel Method (NATM). In North America, E.E. Mason used gunite (shotcrete) to seal a tunnel roof in 1957. Use of steel-fiber-reinforced shotcrete (SFRS) in tunnels was pioneered in North

America, following research by Parker and others in the 1970's at the University of Illinois. The use of microsilica in shotcrete was pioneered in Scandinavia in the 1970's, and SFRS with microsilica has been used in a number of underground openings in North America in the 1980's (Rose, 1985).

The term ‘’gunite" was used by Allentown. More recently, the term "gunite" has come to refer to mixes with small-sized aggregate (less than 0.625 mm) and the term "shotcrete" refers to larger-sized aggregate mixes. The distinction is unimportant.

All early shotcrete performed used the "Dry Mix” method, where dry materials were BLOWN by compressed air through the delivery hose, with water added to the mix at the last possible moment, at the shotcrete nozzle. In recent decades, reliable "Wet Mix" equipment has become available where a conventional wet concrete mix is PUMPED through the hose to the nozzle, where compressed air and accelerator is added. Many contractors tend to prefer the Wet Mix method because of less dust, less rebound, and higher production capability. However, the Wet Mix method requires a reliable source of Wet Mix concrete to be delivered to the site, whereas Dry Mix shotcrete can be made up in small batches and applied promptly to the area of distress, whenever and wherever needed. Both methods satisfy designers' requirements.

European tunnel shotcrete practice, developed from drill+blast excavation, has strongly influenced all shotcrete designers. European practice typically includes a wire mesh installed after the first 50 mm layer of shotcrete is placed. Following installation of the mesh, or Welded Wire Fabric (WWF), the second 50 mm layer of shotcrete is placed. North American designers consider the installation of this wire mesh, or WWF, to be awkward, time consuming and expensive.

Further, the WWF will vibrate when the shotcrete second layer is applied and a weak lamination is typically found within the shotcrete at the WWF location (King, 1980).

The use of steel fibers mixed throughout the shotcrete to produce a steel-fiber-reinforced shotcrete (SFRS) was pioneered in the USA. Work by Parker (1975), Henager (1981), Rose et al. (1981), Morgan (1984) and others, led to the use of SFRS in several tunnels in the USA and Canada. At present, the world's longest tunnel supported solely by SFRS is the 3120 meters long foot Stanford Linear Collider (SLC) tunnel in California (Rose, 1986).

2.9. Material Modelling

Material models summarized and described in the following few pages are originally presented by Potts and Zdravkovic (1999). In this thesis, only important parts are mentioned.

Real soils do not behave in an ideal and simple manner like the case in linear elastic material behaviour. If they did, failure would never occur and geotechnical engineers would probably not need. Real soil behaviour is highly nonlinear, with both strength and stiffness depending on strain stress levels. For realistic predictions to be made of practical geotechnical engineering problems, a more complex constitutive model is therefore required. As this involves nonlinear behaviour, further developments are required to the finite element theory. To formulate an elasto-plastic constitutive model requires the following four essential ingredients:

1. Coincidence of Axes: The principal directions of accumulated stress and incremental plastic strain are assumed to coincide. This differs from elastic behaviour where the principal directions of incremental stress and incremental strain coincide.

2. A Yield Function : In the uniaxial situations, the yield stress, σ_y, indicates the onset of plastic straining. In the multi-axial situation it is not sensible to talk about a yield stress, as there are now several non-zero components of stress. Instead, a yield function, F, is defined, which is a scalar function of stress (expressed in terms of either the stress components or stress invariants) and state parameters, $\{k\}$:

$$F(\{\sigma\}, \{k\}) = 0 \quad (2.1)$$

This function separates purely elastic from elasto-plastic behaviour. In general, the surface is a function of the stress state $\{\sigma\}$ and its size also changes as a function of the state parameters $\{k\}$, which can be related to hardening/softening parameters. For perfect plasticity $\{k\}$ is constant and represents the magnitude of the stresses at yield. It is analogous to σ_y in Figure 2.14a. For hardening and softening plasticity $\{k\}$ varies with plastic straining to represent how the magnitude of the stress state at yield changes. It is analogous to the curves BCF in Figures 2.14b (hardening) and 2.14c (softening). If the hardening or softening is related to the magnitude of the plastic strains, the model is known as *strain hardening/softening*. Alternatively, if it is related to the magnitude of plastic work, the model is known as *work hardening/softening*.

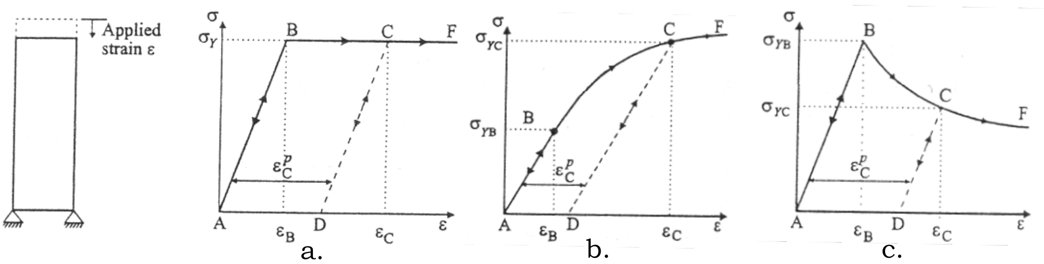


Figure 2.14 Uniaxial loading of - **a**. linear elastic perfectly plastic material **b.** linear elastic strain hardening plastic material **c.** linear elastic strain softening material (Potts and Zdravkovic, 1999)

The value of the yield function F is used to identify the type of material behaviour. Purely elastic behaviour occurs if F({σ},{k})<0, and plastic (or elasto-plastic) behaviour occurs if F({σ},{k})=0. F({σ},{k}) > 0 signifies an impossible situation. Equation 2.1 plots as a surface in a stress space. For example, if Equation 2.1 is expressed in terms of the principal stresses and σ_2 = 0, the yield function can be plotted as shown in Figure 2.15. Such a plot of the yield function is called a *yield curve*. If σ_2 is not set to zero but is allowed to vary, the yield function has to be plotted in three dimensional σ_1-σ_2-σ_3 space where it forms a *yield surface*, see Figure 2.15b. The space enclosed by this surface is called the *elastic domain*. The advantage of assuming isotropic behaviour and therefore expressing the yield function in terms of stress invariants should now be apparent. If such an assumption is not made, the yield function has to be expressed in terms of six stress components and it therefore forms a surface in six dimensional space. Clearly, it is not possible to draw such a space and therefore visualisation of such a surface is difficult.

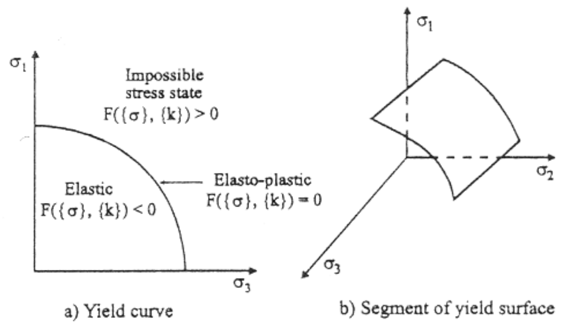


Figure 2.15 Yield function presentation (Potts et al., 1999)

3. A Plastic Potential Function: In the uniaxial loading situations, shown in Figure 2.14, it is implicitly assumed that the plastic strains take place in the same direction as the imposed stress. For the uniaxial case this is self evident. However, in the multi-axial case the situation is more complex as there are potentially six components of both stress and strain. It is therefore necessary to have some means of specifying the direction of plastic straining at every stress state. This is done by means of a *flow rule* which can be expressed as follows:

$$\Delta\varepsilon_p^i = \Lambda \frac{\partial\left(P\{\sigma\},\{m\}\right)}{\partial\sigma_i} \tag{2.2}$$

where $\Delta\varepsilon_i^p$ represents the six components of incremental plastic strain, P is the plastic potential function and A is a scalar multiplier. The *plastic potential function* is of the form:

$$F(\{\sigma\}\ ,\ \{m\}) = 0 \tag{2.3}$$

where $\{m\}$ is essentially a vector of state parameters the values of which are immaterial, because only the differentials of P with respect to the stress components are needed in the flow rule, see Equation (2.2).

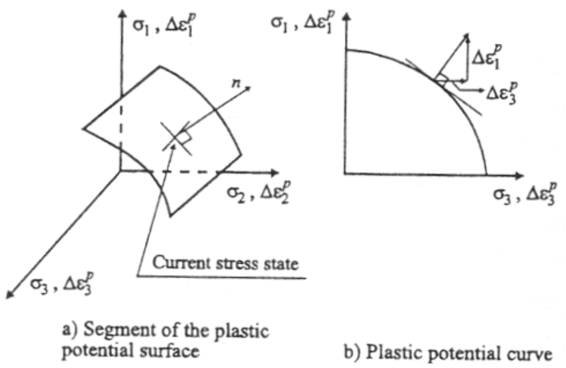


Figure 2.16 Plastic potential presentation (Potts et al., 2001)

Equation (2.2) is shown graphically in Figure 2.16. Here a segment of a plastic potential surface is plotted in principal stress space. Because of the assumption of coincidence of principal directions of accumulated stress and incremental plastic strain, it is possible to plot incremental principal strains and accumulated principal stresses on the same axes. The outward vector normal to the plastic potential surface at the current stress state has components which provide the relative magnitudes of the plastic strain increment components. This is more easily shown in Figure 2.16b, where it is assumed that $\sigma_2 = 0$ and the plastic potential function is plotted in two dimensional σ_1-σ_3 space. It should be noted that the normal vector only provides an indication of the relative sizes of the strain components. The value of the scalar parameter Λ in Equation (2.2) controls their magnitude. Λ is dependent on the hardening/softening rule which is discussed in the next section. In general, the plastic potential can be a function of the six independent stress components and has a corresponding surface in six dimensional stress space, to which the components of a vector normal to the surface at the current stress state represent the relative magnitudes of the incremental strain components.

Sometimes a further simplification is introduced by assuming the plastic potential function to be the same as the yield function (i.e. $P(\{\sigma\},\{m\}) = F(\{\sigma\},\{k\})$). In this case the flow rule is said to be *associated*. The incremental plastic strain vector is then normal to the yield surface and the *normality condition* is said to apply. In the general case in which the yield and plastic potential functions differ (i.e. $P(\{\sigma\},\{m\}) \neq F(\{\sigma\},\{k\})$), the flow rule is said to be *non-associated*.

Flow rules are of great importance in constitutive modelling because they govern dilatancy effects which in turn have a significant influence on volume changes and on strength. Whether or not the flow rule is associated or non-associated also has a cost implication in finite element analysis. As shown before in this section, if the flow rule is associated, the constitutive matrix is symmetric and so is the global stiffness matrix. On the other hand, if the flow rule is non-associated both the constitutive matrix and the global stiffness matrix become non-symmetric. The inversion of non-symmetric matrices is much more costly, both in terms of storage and computer time.

4. The Hardening/Softening Rules: The hardening/softening rules prescribe how the state parameters $\{k\}$ vary with plastic straining. This enables the scalar parameter, Λ, in Equation (2.2) to be quantified. If the material is perfectly plastic, no hardening or softening occurs and the state parameters $\{k\}$ are constant. Consequently, no hardening or softening rules are required. In such materials Λ is undefined. This follows from the fact that once the stress state reaches, and is maintained at, yield the material strains indefinitely. However, for materials which harden and/or soften during plastic straining, rules are required to specify how the yield function changes.

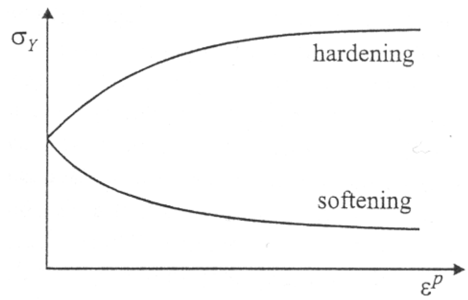


Figure 2.17 Examples of hardening/softening rules (Potts et al., 1999)

For example, in the uniaxial compression of a strain hardening material on Figure 2.14b, the yield stress, σ_y, increases with plastic straining along the path BCF. At any point along this path the strains can be separated into elastic and plastic components. It is then possible to plot how the yield stress, σ_y, varies with plastic strain, ε^p, as shown in Figure 2.17. A relationship of this type is called a *hardening rule*. For the strain softening uniaxial example shown in Figure 2.14c, the yield stress reduces and again it is possible to plot how σ_y varies with plastic strain ε^p. This is also shown in Figure 2.14c and such a relationship is called a *softening rule*.

In multi-axial situations it is common to relate changes in size of the yield surface to the components (or invariants) of the accumulated plastic strain. Such hardening/softening rules are then called *strain hardening/softening*. Alternatively, but less commonly, the change in size of the yield surface can be related to the increase in plastic work, $W^p=\int\{\sigma\}^T.\{\Delta\varepsilon^p\}$ Such hardening and softening rules are called *work hardening/softening*.

So in general, having accepted coincidence of principal directions of accumulated stress and incremental plastic strain, three further pieces of information are required to formulate an elasto-plastic model. A yield function which signals when the material becomes plastic, and a plastic potential function which determines the direction of plastic straining, are compulsory ingredients. If the material hardens or softens, a hardening/softening rule is required.

CHAPTER 3

A CASE STUDY ON BOLU TUNNELS

3.1 Introduction

The Anatolian Motorway from İstanbul to Ankara is a part of Trans European Motorway (TEM) project, which integrates a system of interconnections of roads and harbours of East Europe with various Middle East Countries. Bolu Mountain Crossing is the midway between Ankara and İstanbul, and represents the most challenging section of the motorway construction. Along this 20 km long stretch, four major viaducts and a long tunnel are under construction (see Figure 3.1).

When completed, the Bolu Tunnels will constitute a part of the motorway connection between İstanbul and Ankara. They are located between the Asarsuyu Valley and Elmalık Village in Stretch-II of the Gümüşova-Gerede section of the Anatolian Motorway (Figure 3.1). The twin tunnels, a part of 1.5 billion dollar project that aims at improving transportation in mountaneous terrain to the west of Bolu, are approximately 3.0 km in length. These tunnels are designed to accommodate ultimately three-lane directional traffic. The vehicular clearance is 5.0 m and the width of the tunnel is determined through the requirements of three lanes, each 3.75 m wide and the safety walks, each 0.75 m wide for each tube. These high standards necessitate an excavation of cross-section in excess of 200 m^2 for each tube (Figure 3.2). The tunnel has an excavated arch section 15 m tall by 16 m wide. Construction has been unusually challenging because the alignment crosses several minor faults parallel to the North Anatolian Fault. A 40 m wide rock pillar separates both tubes. Vehicular and pedestrian cross adits at regular distances connects the tubes.

3.2. Project History

Excavation started in 1993 from the Asarsuyu (İstanbul) side and in 1994 from the Elmalik (Ankara) side. The design was based on the NATM principles according to Austrian Standard ÖNORM 82203 with some modifications to account for the local conditions (Unterberger, W. and Brandl, H.). For rock, five support classes were foreseen (A1, B1, B2, C1 and C2), for the portal stretches two (L1 and L2). Bolu Tunnels are excavated using conventional backhoes and other earth moving equipments.

The excavation was characterized by large, constant movements, which could only be stopped, or at least reduced after ring closure. Deformations of primary shotcrete lining of more than 1 m led to extensive reprofiling. Repeated invert heave necessitated the replacement of the originally installed shotcrete invert by a deep monolithic concrete invert. During 1996, the first major low angle fault gouge was encountered at the excavation from the Elmalık side, after approximately 300 m of advance. This zone could be crossed with the right tube, although with considerable difficulties. Excavation of the left tube in this fault gouge zone caused massive movements and damaged the already excavated first tube. Accordingly, the excavation in the fault gouge zone was decided to be led by a short pilot tunnel. During excavation of the main tube top heading, severe cracking of the shotcrete lining was observed in the pilot tunnel lining. Similar problems were encountered when following the top heading excavation with bench and invert. Cracking of the top heading shotcrete was followed by a failure of the top heading temporary invert. The top heading had to be backfilled to avoid a collapse. The area was then remined using two

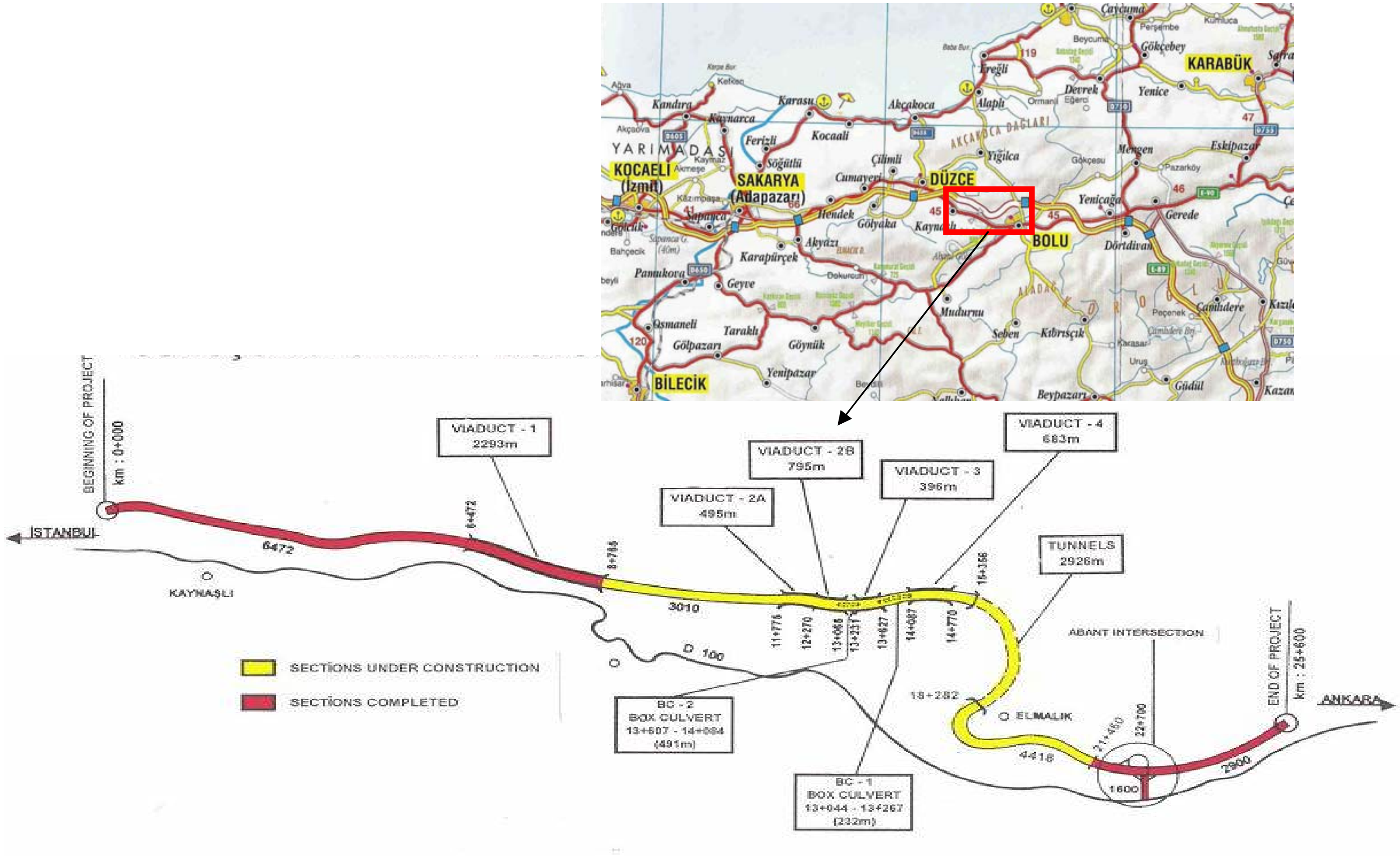


Figure 3.1 Project location (Yüksel Proje, 2003)

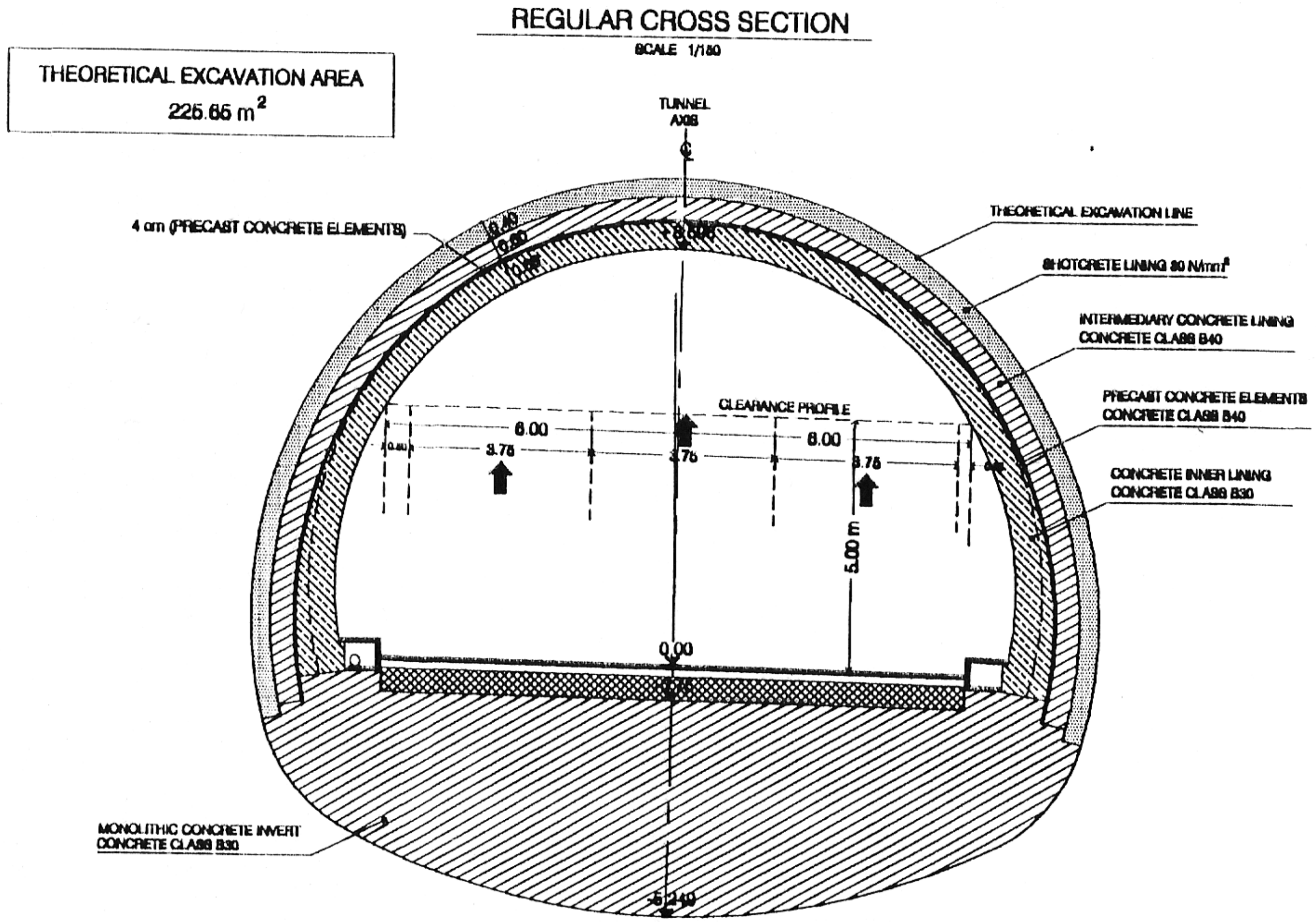


Figure 3.2 Typical tunnel section (Çakan, 2000)

pilot tunnels per tube at bench level, which were backfilled with concrete to provide abutments for the top heading. A 70 cm shotcrete top heading lining was then excavated and the ring closed by following 15 m with a massive monolithic invert. In this way, the fault gouge zone could be crossed successfully until early 1998.

In the late 1997 severe invert heave was encountered in the first tube for a stretch extending up to 200 m backwards from the face. At the same time, radial deformations at the face exceeded 1.2 m. As a consequence, further advance of the two Elmalık drives was stopped and an extensive review of excavation and support methods was performed.

3.3. Investigation Program

Before the new design solutions could be developed, it was considered as essantial to gain a definitive geological picture of the sections to be excavated. For this purpose, a 4.6 m inner diameter pilot tunnel was advanced both from Elmalık and Asarsuyu sides. Additionally, several surface investigation boreholes were drilled. An extensive laboratory testing program was carried out, including soil classification, shear box tests (CU and CD) including residual strength measurement, triaxial tests (UU and CU) plus pore water pressure measurement, consolidation tests as well as swelling potential and swelling pressure measurements. For the determination of stiffness parameters, pressuremeter and dilatometer tests were performed both inside of the pilot tunnel and in the already excavated sections of the main drives. Several monitoring stations consisting of pressure cells, shotcrete strain meters, piezometers and extensometers were installed in the pilot tunnel. The results of this extensive investigation program allowed for a detailed classification of rock mass into several lithological units and identification of the key parameters associated with each unit.

3.4. Current State of the Project

Following the large-scale investigation program and the determination of design solutions for the difficult ground conditions the advance of Elmalık drives started again. Before further advance of the tunnels, the two major earthquakes of 17 August 1999 Marmara and 12 November 1999 Düzce occured.

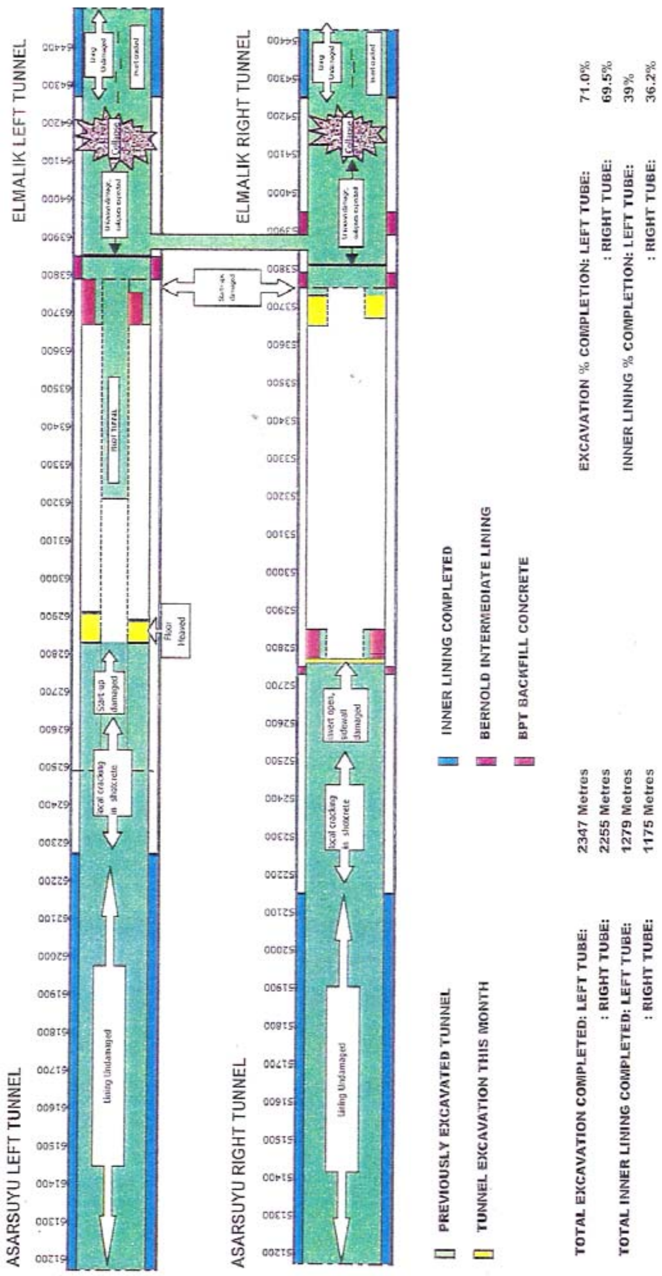


Figure 3.3 Bolu Tunnels after Earthquake Collapse (Çakan, 2000)

The August 17, 1999 earthquake was reported to have had minimal impact on the Bolu Tunnels. The closure rate of one monitoring station was reported to have temporarily increased to an accelerated rate for a period of approximately 1-week, then become stable again. Additionally, several hairline cracks, which had previously been observed in the final lining, were continuously monitored, however, no additional movements due to earthquake were observed.

The November 12, 1999 Earthquake caused the collapse of both tunnels starting at 300 m from their eastern portal (Figure 3.3). At the time of the earthquake a 800 m section had been excavated, and a 300 m section of unreinforced concrete lining had been completed. The collapse took place, in clay gouge material in the unfinished section of the tunnel. The section was supported with shotcrete and bolt anchors.

Several mechanisms have been proposed for explaining the collapse of the tunnel. These mechanisms include strong motion, displacement across the gouge material, and landslide. However, further and detailed studies are required to determine the actual reason of collapses.

After the long breaks in the project, excavations started again in late 2001 with the realigned route (Figure 3.4). Excavation was completed in both tunnels in the middle of 2005.

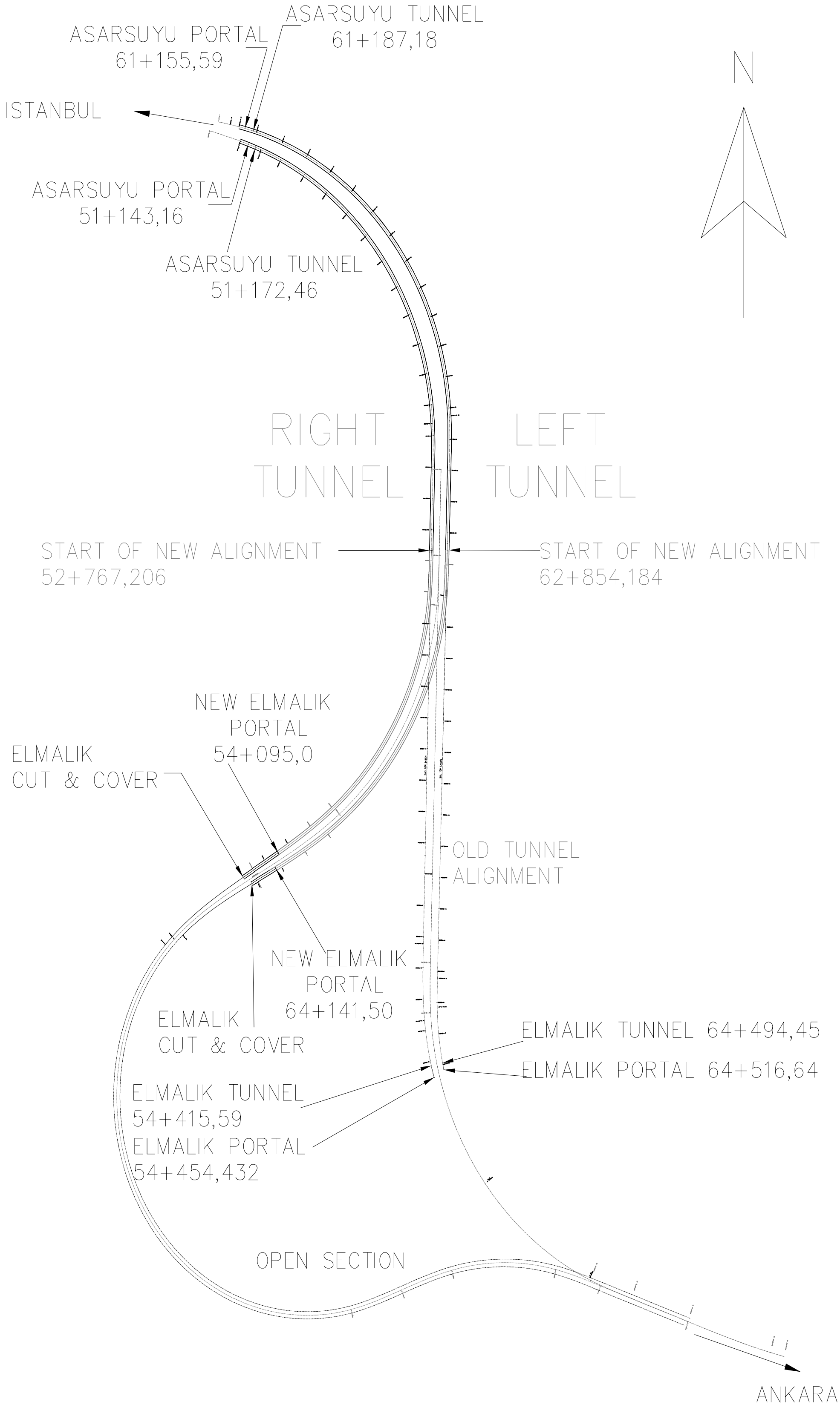


Figure 3.4 Old and new tunnel alignments (Yüksel Proje, 2001)

3.5 Geology of the Area

The project lies about 10 km north of the main branch of North Anatolian Fault Zone (NAFZ), which is the plate tectonic boundary between the Eurasian plate on the north and the Anatolian block on the south (Figure 3.5). The fault is characterized by steep, E-W striking strike-slip faults intersecting the tunnel alignment. The fault is active with movements of approximately 15 mm per year in the Bolu region (Unterberger, W. and Brandl, H.). The tectonic environment was characterized by thrust faulting. This lead to the formation of low-angle fault gouge zones, some of them up to 300 m wide. The rock mass consists of conglomerates, arkoses, sand-stones and marly shales, limestones and dolomitic limestones. Tectonic movement have sheared and displaced the various rock types, such that one unit rarely can be found continuously over a stretch exceeding a few hundred meters in length.

In the course of geological and geotechnical studies following steps have been performed:

i) Stereoscopic determination and evaluatian of fault zones and landslides using a scale of 1:10000

ii) Geological mapping using a scale of 1:5000 for the whole corridor. All natural rock outcrops have been inspected and evaluated on lithology, weathering, discontinuities etc.

iii) A subsurface investigation program has been set up following the results and the interpretation of the previous two steps. In the course of this program 33 investigation drillholes with a total length of 2200 m performed with continuous coring has been executed between summer 1990 and winter of 1991. Many of the boreholes were in very difficult access conditions. Also, difficult ground conditions exist in the field due to the heavy tectonism in the vicinity of the NAFZ. In the portal locations some of the drillholes have been equipped with inclinometer tubes to allow the monitoring of possible movements and their change in time. Also, many of the drillholes were equipped with open standpipe type piezometers to allow long term water level monitoring.

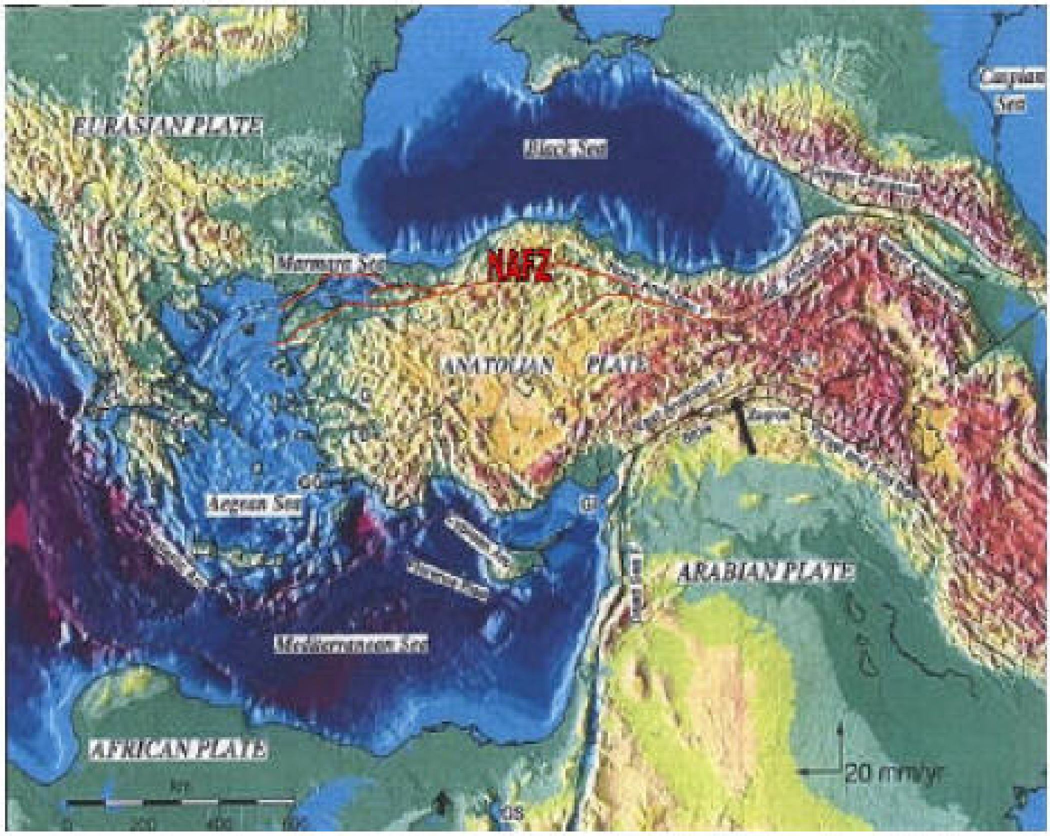


Figure 3.5 Tectonic setting of Turkey (Boğaziçi University, 2000)

Over the NAFZ, the Anatolian block moves westward relative to the Eurasian plate. The general geologcical situation (after Niehof,1976) is as follows:

The basis is built up by the Northern Anatolian polymetamorphic crystalline basement. Its age is considered to be most probably Precambrian.

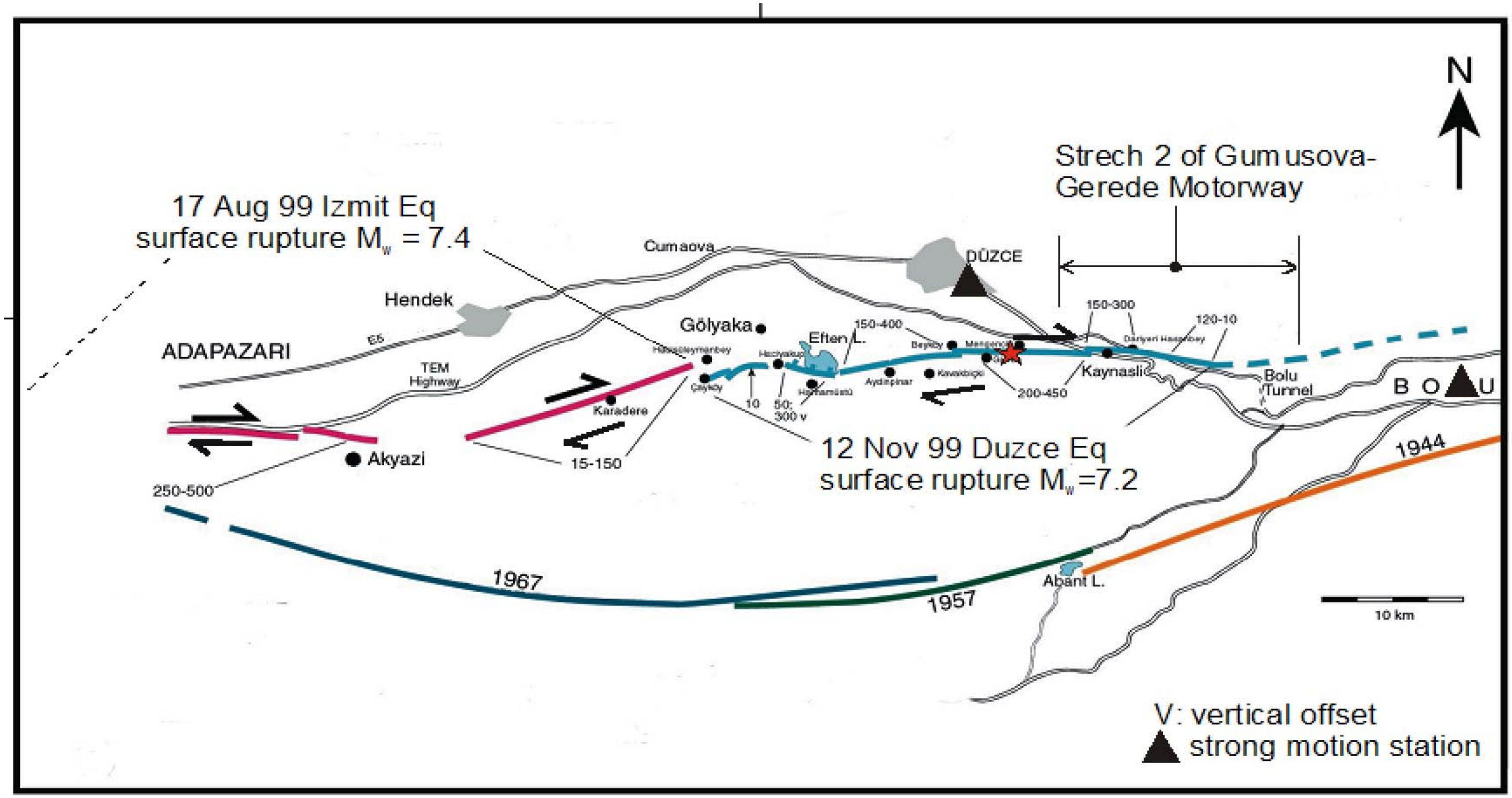


Figure 3.6 NAFZ and recent major earthquakes in the area of concern (Yüksel Proje, 2005)

In the Silurian, Devonian and Carboniferous ages conglomerates, arcoses, sandstones, greywackes and marly shales, limestones and dolomitic limestones have developed.

The crystalline basement rocks consist predominantly of granites, granodiorites, quartzdiorites and diorites and metamorphic rocks of the amphibolite fazies as migmalites, gneisses and amphibolites. This ridge of crystalline basement rocks has been uncovered in the older Palezoic, in the younger Palezoic it divided a northern continental basin from a southern marine basin.

This development has been affected by a variscian low grade metamorphism (greenshist fazies) so that the former sediment cover has been changed to marbles, phyllites, schists etc.

In the Tertiary age further conglomerates, breccias, sandstones, marls, limey marls, siltstones and nummulithic limestones have been deposited, as well as evaporates as gypsum have been generated. Miocene dykes and local tuffites have developed.

All these rocks have been heavily affected by the North Anatolian Fault Zone, which in the section of Yeniçağa-Gerede shows a post Pliocene rigth lateral total strain of about 35 km, being an average of 3.5 to 7.0 mm each year. On the Elmalık side of the tunnel alignment as a result of heavy faulting, the more competent rock mass blocks (crystalline basement, meta-sediment rock series and the competent parts of the flyschoid sequence) do rarely exceed a few hundreds of meters in length, being "embedded” in fault gouges as a kind of large scale matrix. Geological profile for the tunnel is given in Figure 3.7.

3.5.1 Engineering Geology

The whole area of the tunnel alignment is heavily affected by the North Anatolian Fault Zone as mentioned before. Discontinuity data (orientaion of bedding planes, schistosity, joints and slikensides) have been collected during the geological field mapping campaign from natural rock outcrops along the tunnel alignment. Five different homogeneous areas concerning structural features have been distinguished by statistical evaluation of these discontinuity data (Geoconsult, Elmalık Tunnel Final Design Geological Report). Proceeding from North to South these are:

In the first homogeneous area the prevailing schistosity shows a steeply inclination towards north to north-northwest and displays fold structures with occasional overturned limbs. Three major joint sets have been identified. The first trends to WSW, the second trends to WNW and the third trends N-S, all of them dipping very steeply to vertically. Two sets of slickensides occur, one of them trends to NE dipping steeply to almost vertically, the second one trends E-W with almost vertical dipping.

The second homogeneous area is located in the metasediment series. The bedding displays a mean strike direction from WNW to ESE with almost vertical dip angles. The joint distribution shows irregular trends however with steep dip angles in general. Slickensides show almost vertical dip and trend WNW to NW.

The third homogeneous area is located in the northern part of the ‘’flyschoid sequence” (sedimentary rock series). The bedding shows various minor maxima with medium steep to quite gentle dip angles striking in different directions. The jointing varies between gentle to almost vertical dip angles with irregular trends.

Slickensides usually approximately trend in NE-SW directions with steep to vertical dip angles.

The fourth homogeneous area is situated in the southern area of the "flyschoid sequence". The bedding shows a strict trend in E-W directions and has steep to very steep dip angles towards north and south since being folded. Three joint sets are distinguished. One of them strikes SW, the second towards NW, the third in N-S direction. All three sets have very steep to vertical dip angles. Two sets of slickensides have been identified, one of them striking WNW, the second trends NE-SW. Dip angles vary from steep to vertical.

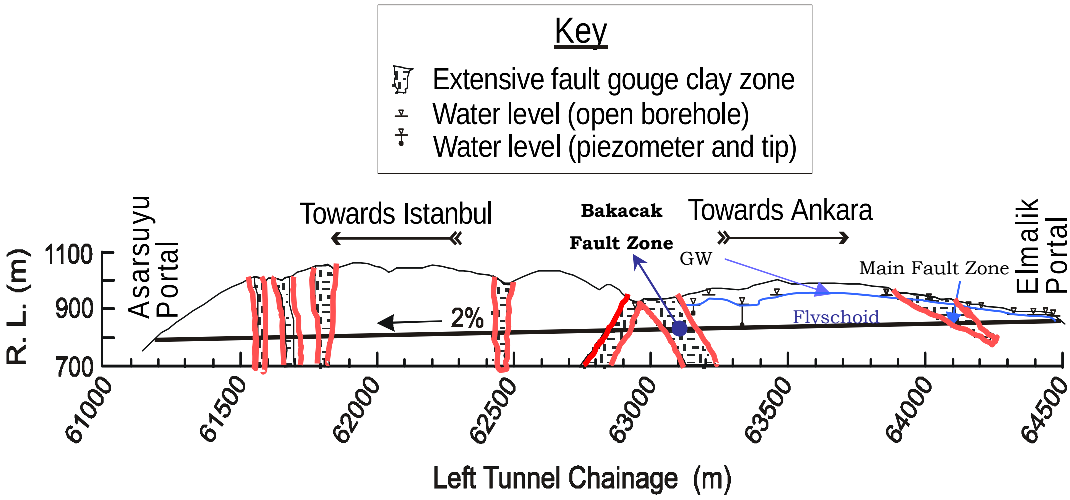


Figure 3.7 Geological profile along the tunnels (Yüksel Proje, 2004)

The fifth homogeneous area is in the more competent rocks of the "flyschoid sequence" which are frequently surrounded by fault gouge material. The bedding plane mean dips gently to medium steeply towards NE. Two joint sets have been monitored, one of them dips medium steeply towards NW, the second dips medium steeply towards SW. The evalution of the slickenside data did not lead to a significant maximum.

3.5.2 Fault Gouge

Faults are shear planes and commonly contain the debris from the frictional contact of the two surfaces. In strong rocks, material is fragmented to create a zone of crushed rock or fault breccia. In weaker rocks, the material in the fault plane can be reduced to a very fine clay-size infill known as fault gouge. Over time, crushed rock can react with subsurface fluids to produce a variety of other secondary minerals, many of them in the "clay" family. Often, fault gouge is a mixture of crushed rock and several of these fine-grained alteration minerals. However, some fault gouge may be composed of finely-ground particles of just one principle type of mineral. The "gouge zone", where the grinding and shearing takes place, may be up to a kilometer wide in large faults.

Gouge is very significant in engineering terms, since the shear strength of the discontinuity is that of the weak gouge rather than the wall rock. From the engineering point of view, the properties of fault gouge is similar to soft soil in soil mechanics.

CHAPTER 4

ANALYSES FOR OPTION 4

4.1 Finite Element Program Plaxis 3D Tunnel V2.0

The PLAXIS 3D Tunnel program, which is an implicit code, is a geotechnical finite element package specifically intended for the three-dimensional analysis of deformation and stability in tunnel projects. Geotechnical applications require advanced constitutive models for the simulation of the non-linear, time-dependent and anisotropic behaviour of soils and rock. In addition, since soil is a multi-phase material, special procedures are required to deal with hydrostatic and non-hydrostatic pore pressures in the soil. Although the modelling of the soil itself is an important issue, many tunnel projects involve the modelling of structures and the interaction between the structures and the soil. The PLAXIS 3D Tunnel is equipped with special features to deal with the numerous aspects of complex geotechnical structures.

4.2 Analysed Cases and Conditions

The part of the tunnels which go through the Bakacak Fault Zone was considered. In this zone the material is fault gouge clay. The construction methodology, so called Option 4 (see Figure 4.1), was applied for passing through fault clay gouge zone. Option 4 is a rather complicated and difficult construction technique (see Figure 4.2). It has been clearly demonstrated that the smaller the cross-sectional dimensions of an underground cavity, the less critical is its excavation, and the longer the natural arching action period of the rock. Thus, it is an obvious and a long established procedure

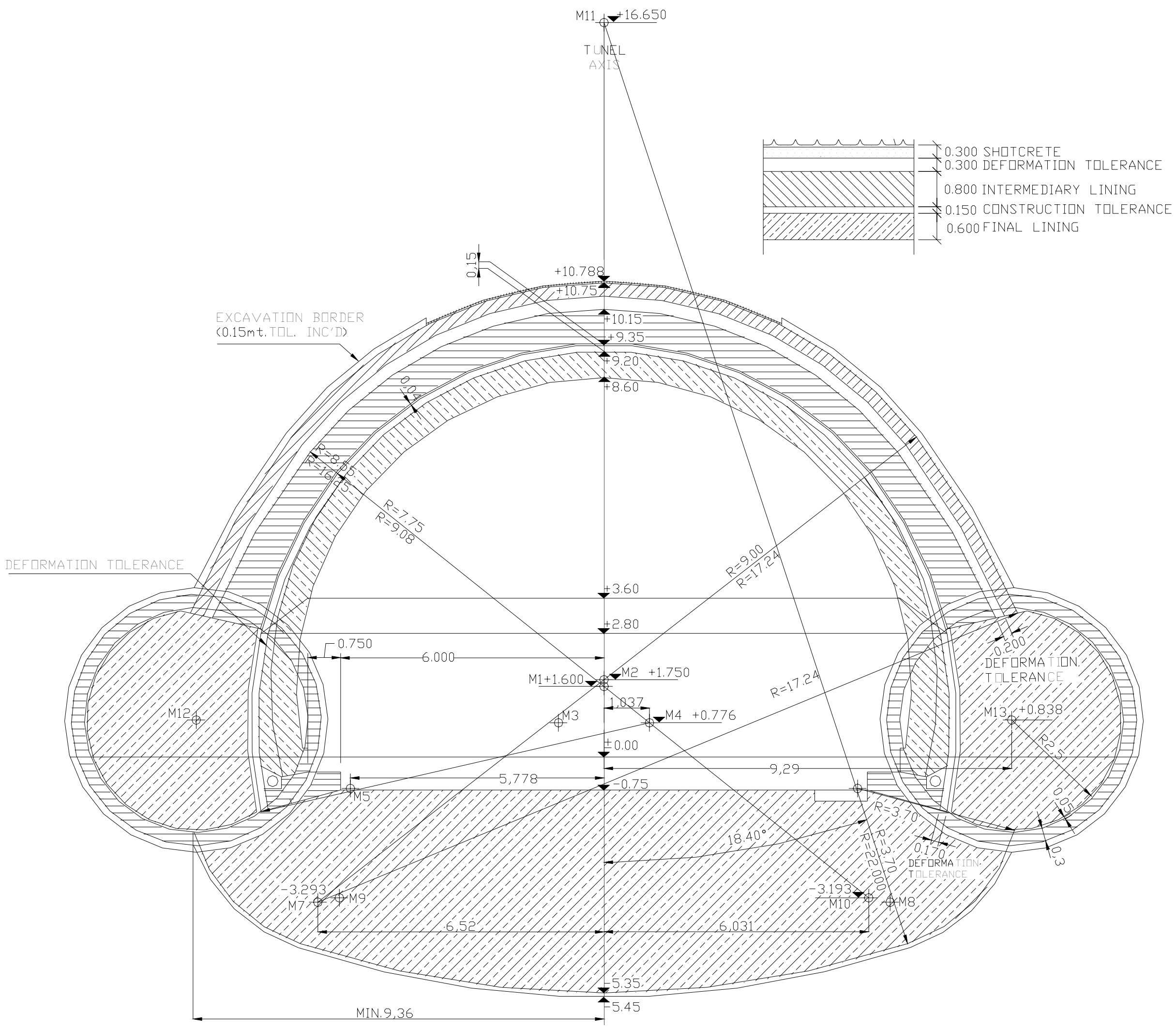


Figure 4.1 Cross-secion of Option 4 (Yüksel Proje, 2000)

Figure 4.2 Construction of Option 4 (Yüksel Proje, 1999)

Figure 4.3 Staged excavation for Option 4 (Yüksel Proje, 2003)

to excavate the tunnel's cross-section not in full face at once, but in smaller parts by driving of smaller specially arranged individual headings (see Figure 4.2 and 4.3). The arrangement and sequence of these headings should always be adapted to the necessary operations to be carried out in them (excavation, installation and construction of temporary and permanent lining etc.) and to the nature of the rock, and also to the rock pressure conditions encountered.

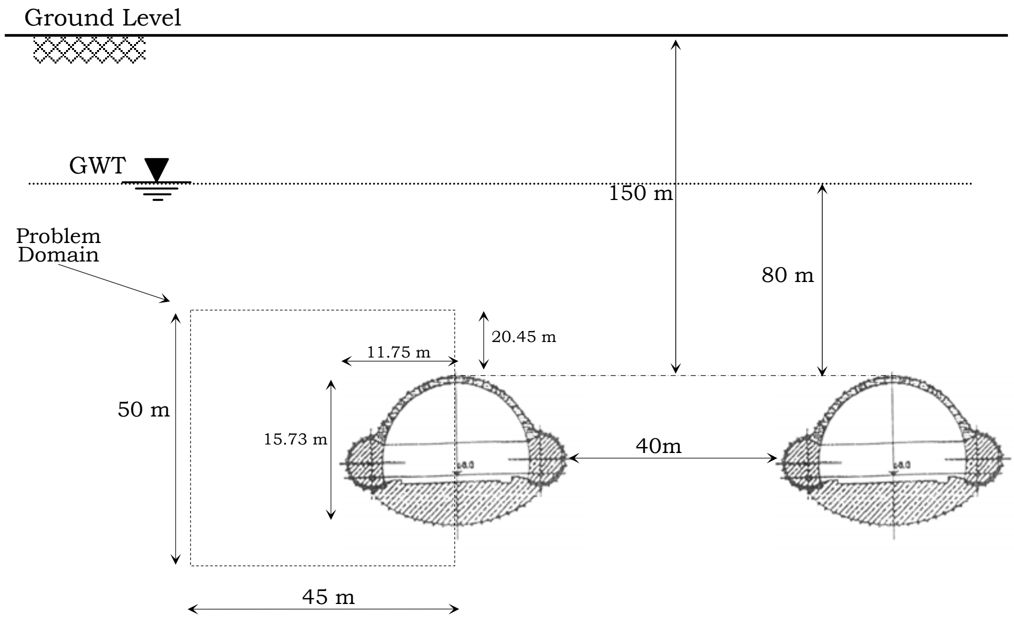


Figure 4.4 Problem discretization (not to scale)

For the modelled case the, thickness of cover over the crown is 150 m. According to the piezometer and surface borehole readings, maximum groundwater level over the crown is 80 m.

Excavation is made in fully saturated fault clay gouge material. So, undrained behavior of the ground is important in this fault clay gouge zone. As a result, total stress analyses with total stress parameters (γ_{total}, E_u, υ_u, c_u and ϕ_u=0) was performed without considering the pore pressures separetely. Also, consolidation and the effect of one tunnel on the other are not modelled in the analyses as can be understood from the selected problem domain.

4.3 Geotechnical and Material Parameters for Input

Geotechnical parameters used in the analyses regarding the undrained behaviour together with Mohr Coulomb material models are listed below:

γ = 20 kN/m^3

γ_{sat} = 22 kN/m^3

ϕ_u = 0, ψ=0

c_u = 600 kN/m^2

E_u = 500,000 kN/m^2

ν_u = 0.45

Material and sectional properties of the supporting elements used in Elastic material models are listed below:

Shotcrete:

thickness = 30 cm

modulus of elasticity = 4,000,000 kN/m^2 (green)
15,000,000 kN/m^2 (hard)
7,500,000 kN/m^2 (damaged)

γ = 24 kN/m^3

Intermediary Lining:

thickness = 80 cm

modulus of elasticity = 30,000,000 kN/m^2

γ = 24 kN/m^3

Invert:

modulus of elasticity = 27,500,000 kN/m^2

γ = 24 kN/m^3

Bench Pilot Tunnel (BPT):

shotcrete thickness = 30 cm

modulus of elasticity = 27,500,000 kN/m^2

γ = 24 kN/m^3

4.4 Model Definition and Geometry

The geometry of the tunnel and its discretization is seen in Figure 4.1 and Figure 4.4. As it is understood from the figures, the model is symmetric, so only one half of the problem needs to be modelled. Additionally, all the host ground is not completely modelled, but the overburden is taken into consideration as a surcharge load of 2690 kPa. This will reduce the model dimensions and decrease the solution time as it was discussed in Section 2.3. In the analyses, only one of the tunnels is modelled, since the basic aim of the study is to investigate the three-dimensional effects.

Corner points of the structural elements may cause large displacement gradients. Hence, it is preferable to use a finer mesh in those areas. Additionally, it is good to refine the mesh where stress concentrations occurs. The stress concentrations are expected to occur on the immediate periphery of the tunnel. As it is seen from Figure 4.5 and Figure 4.6, the mesh is divided into 3 zones, which get finer and finer from outside to inside.

At the left and right boundaries of the model, displacements in x-direction, in the front and back of the model, displacements in z-direction and in the bottom boundary of the model displacements in y direction are fixed. Top boundary of the model is free and the constant surcharge load of 2690 kPa is applied to this boundary. In addition to the standard displacement fixities mentioned, fixed rotations are introduced to the upper most point of the tunnel lining.

Plaxis 3D Tunnel software is utilized in both 2D and 3D analyses. The analyses of the 2D model in 3D software are achieved utilizing a model having a unit thickness and boundary conditions of the plane strain assumption. The generated mesh of the models consists of 6-node triangular elements and it provides a second order interpolation for displacements. The element stiffness matrix is evaluated by numerical integration using a total of 3 Gauss Points (stress points). For the 15-node triangle the order of interpolation is 4 and the integration involves 12 stress points. The accuracy of the 15-node wedge for a 3D analysis is comparable to the 6-node triangle in 2D analysis. Higher order element types are not considered for 3D analysis, because this will lead to large memory requirement and unacceptably large calculation times. Earth materials, shotcrete, monolithic concrete invert and bench pilot tunnel (including its 30 cm thick shotcrete lining) are modelled with these 6-node triangular elements.

Mohr Coulomb Model is used for the fault clay gouge material in the analyses. Soil and rock tend to behave in a highly non-linear way under load. This non-linear stress-strain behavior can be modelled at various levels of sophistication. The well-known Mohr-Coulomb model can be considered as a first order approximation of real soil behavior. This elastic-perfecly plastic model requires five basic input parameters, namely Young modulus, E; Poisson's Ratio, υ; a cohesion intercept, c; friction angle ϕ and dilatancy angle ,ψ. For the analyses of shotcrete, monolithic concrete invert and bench pilot tunnel (including its 30 cm thick shotcrete lining) linear elastic material behaviour is assumed.

In usual NATM applications, the final lining (inner lining) is not the main load carrying element, however it provides additional structural safety. Since, in this study, the main focus is to investigate the structural stability during driving of the tunnel, the final lining (inner lining) is not taken into consideration in the analyses (see Figure 4.1). Intermediary lining is modelled with beam elements. Beams are composed of beam elements with 3 degrees of freedom per node. Two translational degrees of freedom (ux and uy) and one rotational degrees of freedom (rotation in the x-y plane; phi z). When 6 node soil elements are employed, each beam element is defined by 3 nodes. The beam elements are based on Mindlin's beam theory. This theory allows for beam deflections due to shearing as well as bending. In addition, the element can change length, when an axial force is applied. Bending moments and axial forces are evaluated from the stresses at stress points.

Initial effective stresses are generated by means of K_o procedure. The initial stress state in a soil body is influenced by the weight of the material and the history of its formation. This stress state is usually characterized by an initial vertical stress $\sigma_{v,o}$ which is related by the coefficient of lateral earth pressure K_o, $\sigma_{v,o}$=K_o.$\sigma_{h,o}$. The default K_o value is based on the Jaky's formula (K_o=1-sinϕ). By the ϕ_u=0 assumption, K_o is calculated as 1.

4.5 2D Analyses and Definition of Excavation Stages

The 2D model used in the analyses is shown in Figure 4.5. This 2D model is approximately same with the 3D model shown in Figure 4.6. The mesh is 1 m long, 50 m high and 45 m wide. It consists of 711 triangular wedge elements and 3315 nodes. The only difference is that, 2D model consists of only 1 m thick slice. The original undrained analyses of this 2D model were performed by Çakan (2000) with Phase^2 finite element software. The relaxation factors are taken from his study which are determined through pilot tunnel back analyses utilizing the aid of real pilot tunnel convergence measurements. Through his study, he determined the radial deformation at the face from axi-symmetric analyses. He reduced the stiffness of the ground material incrementally and recorded deformations corresponding to each value. He determined the appropriate relaxation factor when the radial movement in the plane strain model matched the face radial movement in the axi-symmetric model.

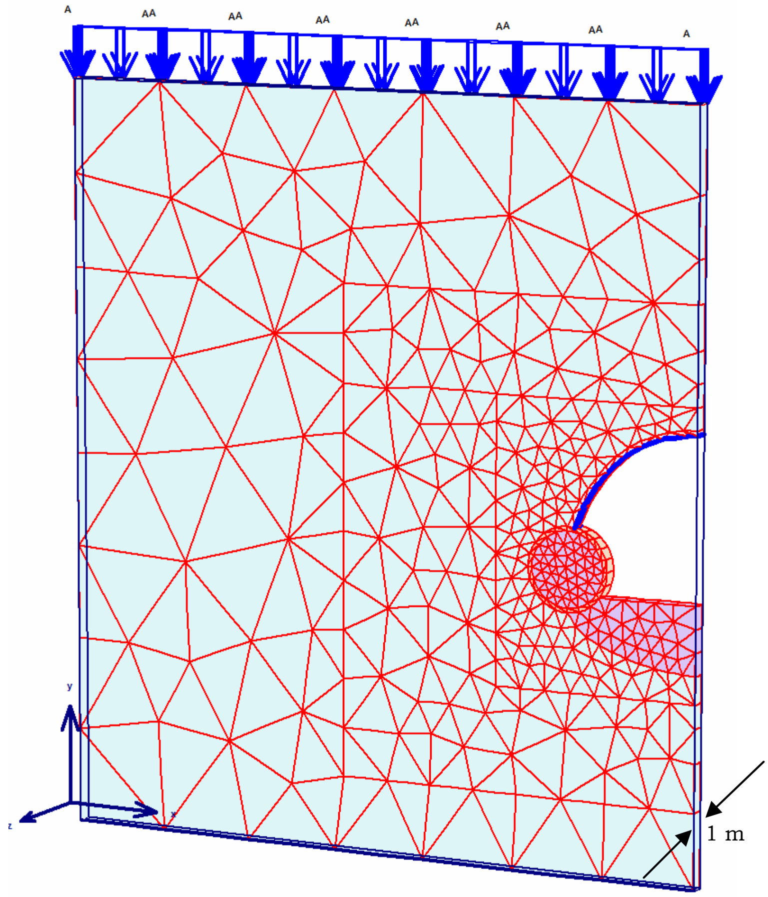


Figure 4.5 2D analysis model

In Plaxis, the model is solved in 12 consecutive construction stages. The details of these construction stages are as follows:

1. initial stress state generation also self weight of the geomaterials (application of 2690 kPa overburden stress)
2. 60% relaxation of bench pilot tunnel (BPT)
3. 100% relaxation (complete excavation) of BPT and 30 cm thick shotcrete lining installation
4. BPT backfilled
5. 60% relaxation of top heading

6. 65% relaxation of top heading + shotcrete application (fresh)
7. 75% relaxataion of top heading + shotcrete hardening
8. 20% relaxation of bench and invert + intermediary lininig application
9. 90% relaxation of top heading, bench and invert
10. installation of monolithic concrete invert
11. 100% relaxation of top heading and bench
12. shotcrete damaged

The results are presented in Chapter 5.

4.6 3D Analyses and Definition of Excavation Stages

The 3D model used in the analyses is shown in Figure 4.5. The differences from the 2D model is the thickness (and hence the number of slices). 3D model consists of 13 slices. The slices numbered from 1 to 10 have a thickness of 4 m (which is the round length of each construction stage at the site), slice 11 is 5 m and the last two slices are 7.5 m thick. As a result the 3D model is 60 m long, 50 m high and 45 m wide. It consists of 9243 triangular wedge elements and 25479 nodes. As it is easily recognised the number of elements are 13 times greater than that of the 2D model which results in a considerable increase in the solution time.

A face stabilizing pressure of 200 kPa is applied at each step to the excavation face of the tunnel to account for the supporting system at the tunnel top heading.

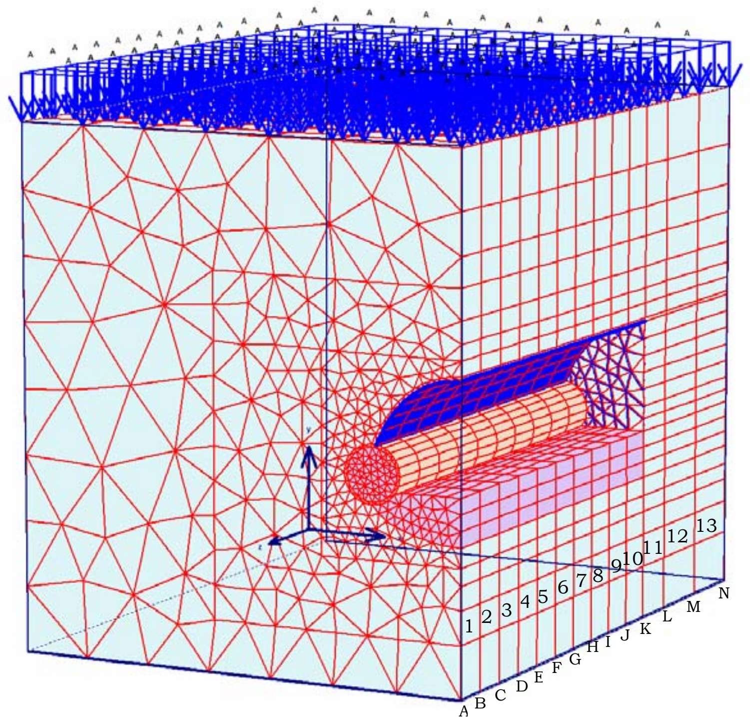


Figure 4.6 3D analysis model

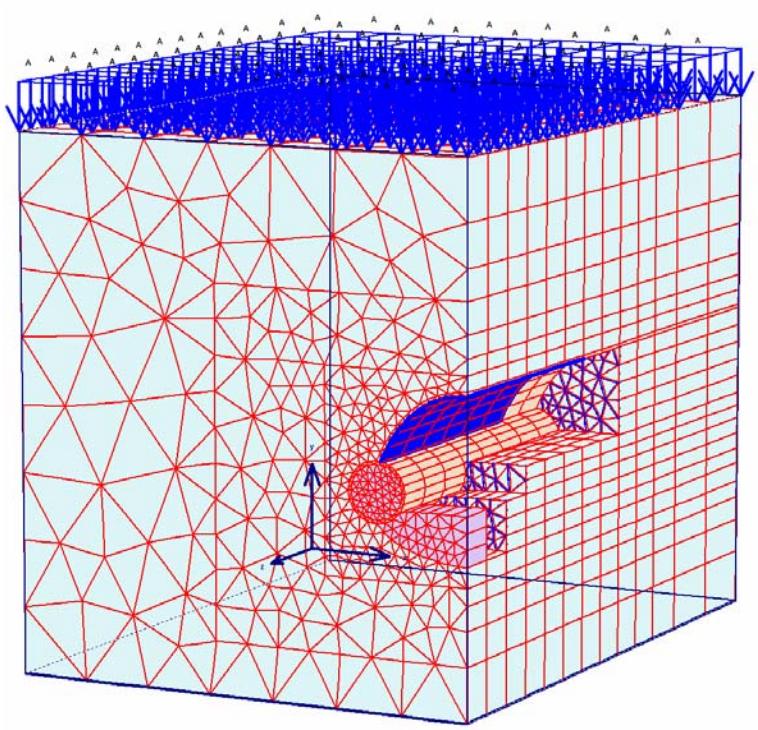


Figure 4.7 3D analysis, sample construction stage (#29)

In Plaxis, the model is solved in 38 consecutive construction stages. These stages are as follows:

1. generation of initial stress state also self weight of the geomaterials (application of 2690 kPa overburden stress)
2. excavation of bench pilot tunnel (BPT) + installation of 30 cm thick shotcrete lining (slice #1)
3. excavation of BPT + installation of 30 cm thick shotcrete lining (slice #2)
4. excavation of BPT + installation of 30 cm thick shotcrete lining (slice #3)
5. excavation of BPT + installation of 30 cm thick shotcrete lining (slice #4)
6. excavation of BPT + installation of 30 cm thick shotcrete lining (slice #5)
7. excavation of BPT + installation of 30 cm thick shotcrete lining (slice #6)
8. excavation of BPT + installation of 30 cm thick shotcrete lining (slice #7)
9. excavation of BPT + installation of 30 cm thick shotcrete lining (slice #8)
10. excavation of BPT + installation of 30 cm thick shotcrete lining (slice #9)
11. excavation of BPT + installation of 30 cm thick shotcrete lining (slice #10)
12. backfilling of BPT (slice #10)
13. backfilling of BPT (slice #9)
14. backfilling of BPT (slice #8)
15. backfilling of BPT (slice #7)
16. backfilling of BPT (slice #6)
17. backfilling of BPT (slice #5)
18. backfilling of BPT (slice #4)

19. backfilling of BPT (slice #3)

20. backfilling of BPT (slice #2)

21. backfilling of BPT (slice #1)

22. top heading excavation + shotcrete (green) application (slice#1)

23. top heading excavation + shotcrete (green) application (slice#2)

24. top heading excavation + shotcrete (green) application (slice#3), intermediary lining installation + hardened shotcrete (slice#1)

25. top heading excavation + shotcrete (green) application (slice#4), intermediary lining installation + hardened shotcrete (slice#2)

26. top heading excavation + shotcrete (green) application (slice#5), intermediary lining installation + hardened shotcrete (slice#3)

27. top heading excavation + shotcrete (green) application (slice#6), intermediary lining installation + hardened shotcrete (slice#4), bench excavation + damaged shotcrete (slice#1)

28. top heading excavation + shotcrete (green) application (slice#7), intermediary lining installation + hardened shotcrete (slice#5), bench excavation + damaged shotcrete (slice#2), invert excavation (slice#1)

29. top heading excavation + shotcrete (green) application (slice#8), intermediary lining installation + hardened shotcrete (slice#6), bench excavation + damaged shotcrete (slice#3), invert excavation (slice#2), concreting of invert (slice#1)

30. top heading excavation + shotcrete (green) application (slice#9), intermediary lining installation + hardened shotcrete (slice#7), bench excavation + damaged shotcrete (slice#4), invert excavation (slice#3), concreting of invert (slice#2)

31. top heading excavation + shotcrete (green) application (slice#10), intermediary lining installation + hardened shotcrete (slice#8), bench excavation + damaged shotcrete (slice#5), invert excavation (slice#4), concreting of invert (slice#3)
32. intermediary lining installation + hardened shotcrete (slice#9), bench excavation + damaged shotcrete (slice#6), invert excavation (slice#5), concreting of invert (slice#4)
33. intermediary lining installation + hardened shotcrete (slice#10), bench excavation + damaged shotcrete (slice#7), invert excavation (slice#6), concreting of invert (slice#5)
34. bench excavation + damaged shotcrete (slice#8), invert excavation (slice#7), concreting of invert (slice#6)
35. bench excavation + damaged shotcrete (slice#9), invert excavation (slice#8), concreting of invert (slice#7)
36. bench excavation + damaged shotcrete (slice#10), invert excavation (slice#9), concreting of invert (slice#8)
37. invert excavation (slice#10), concreting of invert (slice#9)
38. concreting of invert (slice#10)

The results are presented in chapter 5.

CHAPTER 5

RESULTS AND DISCUSSION

In this part of the study, the results obtained from the 2D and 3D undrained analyses of Option 4 are presented and discussed. The analyses results with original site data are presented through Figures 5.1 through 5.21 are also summarized in Table 5.1 to give a perspective idea.

Axial forces, shear forces, bending moments and deformations in the intermediary (Bernold) lining along the excavation boundary are presented in Figures 5.5 through 5.14 from analyses results of the last excavation stages.

Excavation in the ground induces stress relief which causes soil movements towards the opening as it is seen from Figures 5.1 and 5.2. However, there is an important difference between movement mechanisms of 2D and 3D models. Although the vertical displacement plots of intermediary lining (see Figure 5.5 and 5.6) display a similar trend, horizontal displacements at the bench level occur in opposite directions. This difference can be recognized from a contrast of Figures 5.7 and 5.8, and also from the different results of internal force diagrams of the intermediary (Bernold) lining. The maximum crown deformations (see pt. A in Figures 5.1 and 5.2) of 2D and 3D analysis cases are 32 mm and 165 mm downwards, respectively. At the bench level, maximum horizontal displacements in the intermediary lining for 2D and 3D models are -20 mm and 62 mm, respectively (see pt. B in Figures 5.1 and 5.2). Minus sign indicates that displacement is outward. This is an interesting finding and an unexpected result of the 2D analyses. Because the expected displacement pattern of the tunnel is inwards. Recorded maximum crown deformation and maximum horizontal displacement at the bench level are around 110 mm and 50 mm, respectively. As it is seen the results of 3D analyses are much more closer to site data. Additionally, the movement behaviour of the 2D and 3D models are also verified from the plot of plastic points around the tunnels in Figures 5.3 and 5.4, respectively.

Table 5.1 2D & 3D analyses results compared with site recorded data

analysis	slice	axial force x 10^3 kN/m	shear force x 10^3 kN/m	moment x 10^3 kN.m/m	hor. displ. at pt. A (mm)	ver. displ. at pt. B (mm)
3D	A	12.48	1.64	1.86	60	157
	B	10.28	1.41	1.70	60	158
	C	9.65	1.44	1.56	62	162
	D	9.13	1.44	1.42	61	164
	E	8.36	1.38	1.23	60	165
	F	6.72	0.92	0.98	59	164
	G	6.17	0.84	0.82	58	162
	H	5.10	0.75	0.71	55	157
	I	2.54	0.41	0.53	51	149
	J	3.71	0.53	0.68	47	136
2D	A	19.30	2.45	-1.88	-20	32
site data	A	10.00	-	1.20	50	110

In 3D model, displacement calculated at tunnel face (see Figure 5.19) is around 260 mm. This value is closer to site data which the recorded values range between 170 mm and 223 mm.

In both analyses models, some amount of ground heave is calculated at points C and D (see Figures 5.1 and 5.2). The approximate value of ground heave at points C and D for 2D and 3D analyses are around 20 mm and 200, respectively. Ground heave is expected in such a soft ground at point C, but the values of heave at point D are unexpectable due to the monolithic invert placement after excavation (see Figure 5.19). The site reported ground heave values at point C ranges between 100 mm and 500 mm in that fault clay gouge zone.

Comparing the internal forces (axial forces, shear forces and bending moments) observed in the intermediary lining, the results of 2D analyses are relatively greater than that of the 3D analyses. This is mainly due to the lost arching action in 2D model. The maximum values of axial forces are 19300 kN/m and 12480kN/m for 2D and 3D analyses, respectively. As it seen from Table 5.1, axial forces decrease as the excavation approaches to the tunnel heading. This is an expected result due to the 'pile effect' of the bench pilot tunnel. The same reasoning is also valid, to explain the shear forces and bending moments. The maximum values of bending moments are 1880 kN.m/m and 1860 kN.m/m for 2D and 3D analyses, respectively. The difference between these two values is not so much as in the case for axial forces. But the main difference in the bending moment is their signs (see Figures 5.11 and 5.12). The change of sign in the moments caused from the opposite way of horizontal displacement behaviours in the intermediary lining at the bench level (see Figures 5.7 and 5.8). The maximum values of shear forces are 2450 kN/m and 1640 kN/m for 2D and 3D analyses, respectively. Again the results of 2D analyses are approximately 50 percent greater than that of 3D analyses as in the case for axial forces.

In the 3D model, displacement in longitudinal direction is permitted (except boundaries), whereas, in the plane strain model, displacements perpendicular to the cross section are assumed to be zero. But this is not a realistic assumption like the Bolu Tunnels. In the 2D model arching actions are cancelled in y-z and x-z planes by which the solution is strongly affected. Rock at depth is subjected to forces due to the weight of overlying material and forces due to tectonic processes. Excavation for an underground structure, such as a tunnel of large cross-section causes a local stress redistribution in the vicinity of the excavation such that the forces previously carried by the excavated rock must now be transmitted or 'arched' around the opening (as it is seen from the Figures 5.16 through 5.18). The purpose of the support is said to be ''to help the rock to support itself'', or to ensure effective ''arch'' action of the forces around the tunnel. At the face of the excavation, where the arching of forces occur onto the rock ahead of the face as well as on the walls, it is sometimes referred to as ''dome action.''

Bolu Tunnel support failure mechanisms must be such that inner support has to behave as a thin walled shell maximizing normal stress and minimising movements and avoiding internal shear. However, in 2D model, axial forces are high, but accompanying shear forces and bending moments are also high. On the other hand, the results of the 3D solution are much closer to what is expected. As it can be easily imagined, bench pilot tunnel must work in the ground like a pile embedded laterally. But this effect is lost due to plane strain assumption in the 2D model. Additionally, together with the monolithic invert, bench pilot tunnels must create a cantilever effect. The results presented in Table 5.1 reveal this effect obviously. Results of the 2D model, in which this pile effect is lost, are higher than that of the 3D model.

The plot of effective stresses shows that arching occurs around the tunnel face in the axial direction as well, and this reduces the stresses acting on the tunnel lining. As a result, the axial forces become lower.

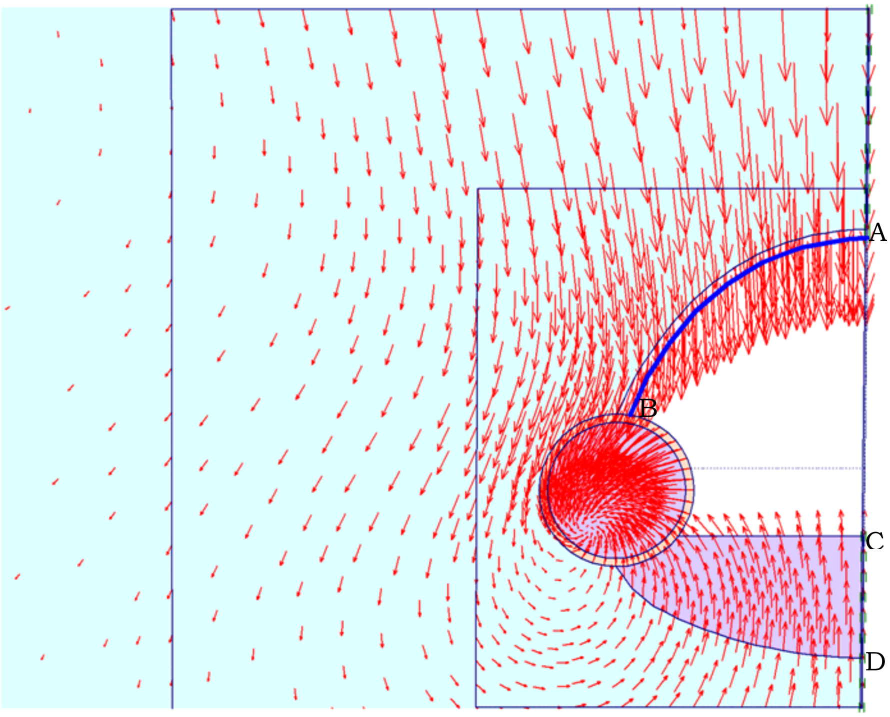


Figure 5.1 Detailed view of induced displacement vectors around the tunnel (2D model)

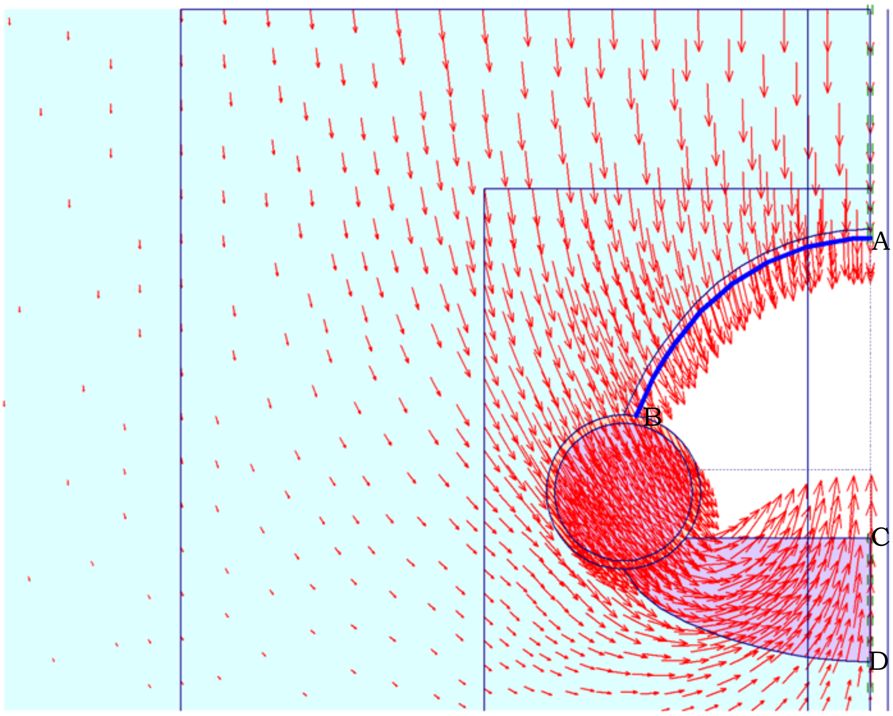


Figure 5.2 Detailed view of induced displacement vectors around the tunnel (typical section from 3D model)

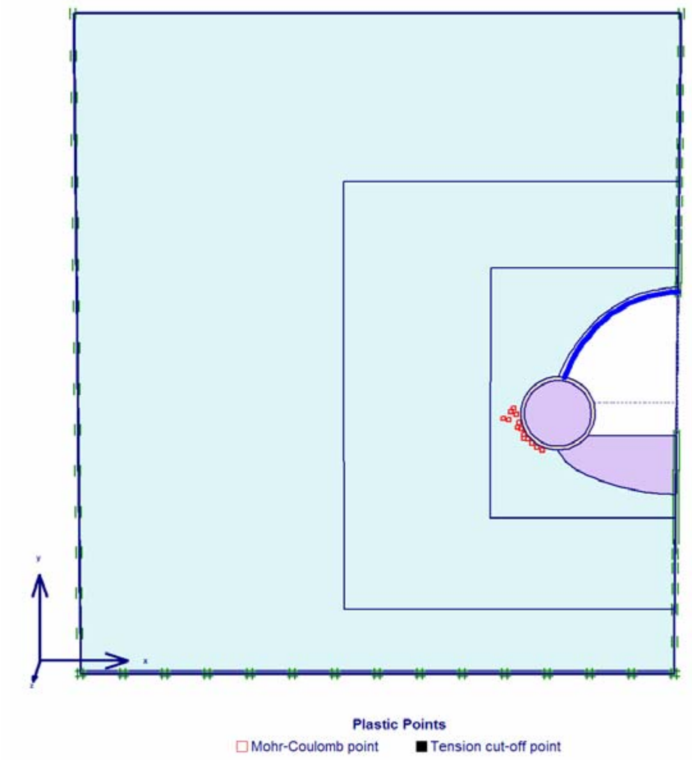


Figure 5.3 Plastic points around the tunnel (2D model)

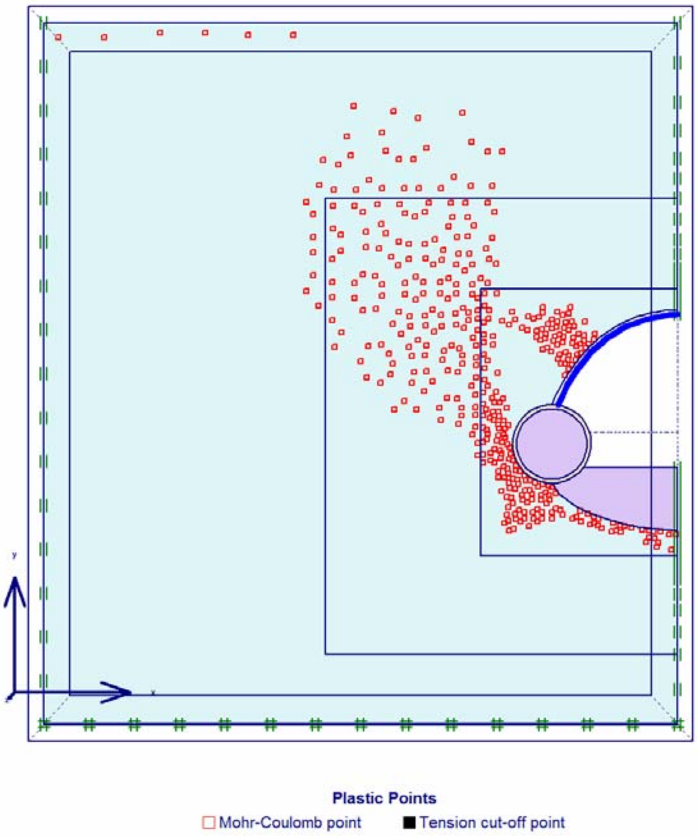


Figure 5.4 Plastic points around the tunnel (3D model)

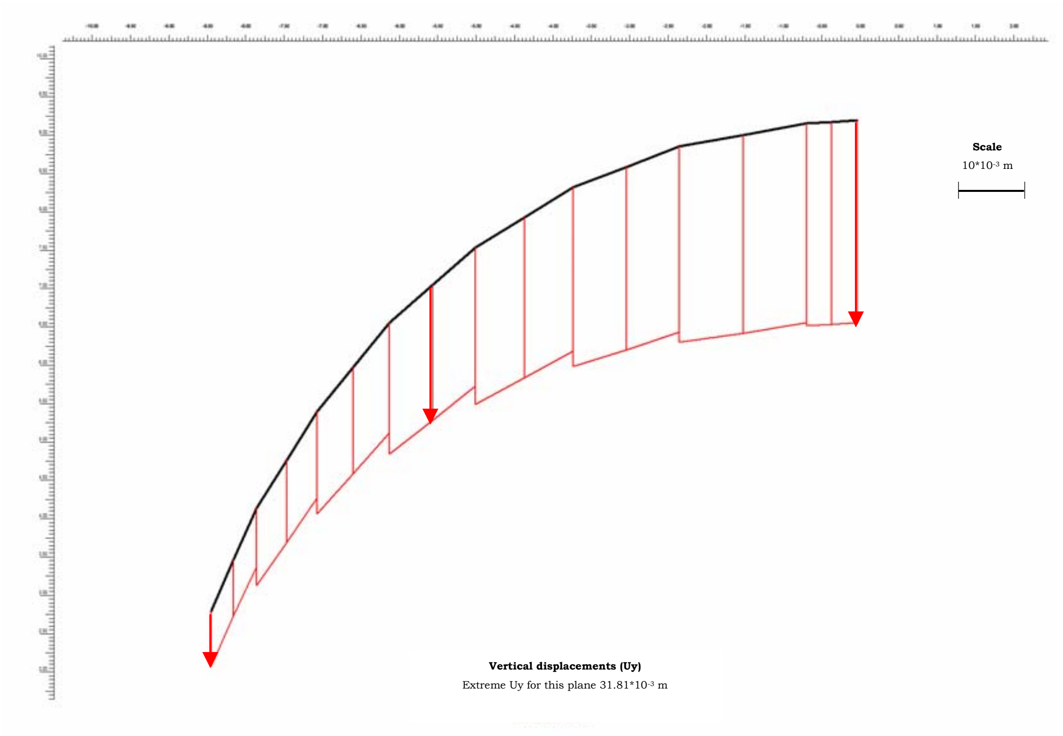


Figure 5.5 Displacement of lining in y-direction (2D model)

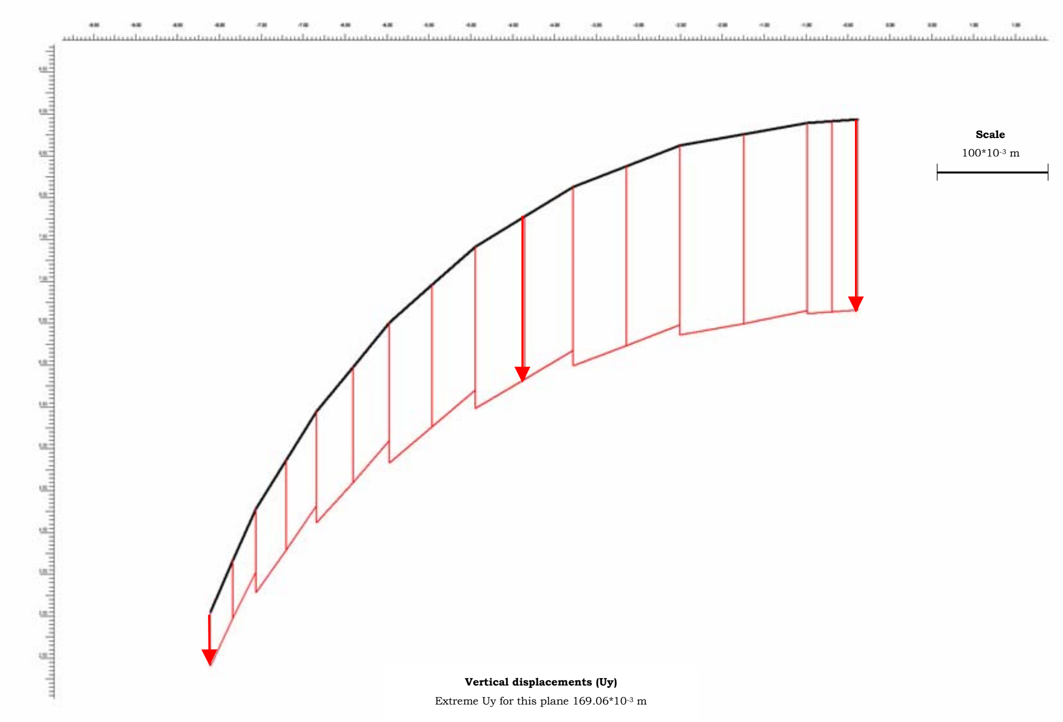


Figure 5.6 Displacement of lining in y-direction (3D model, section-d)

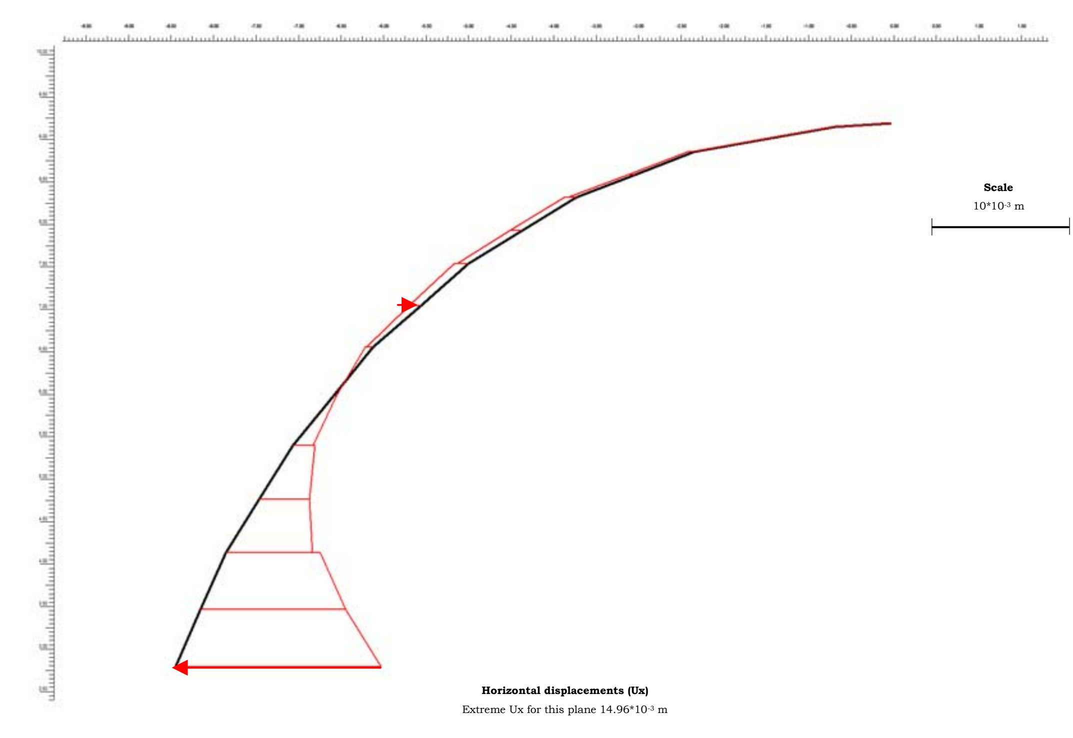


Figure 5.7 Displacement of lining in x-direction (2D model)

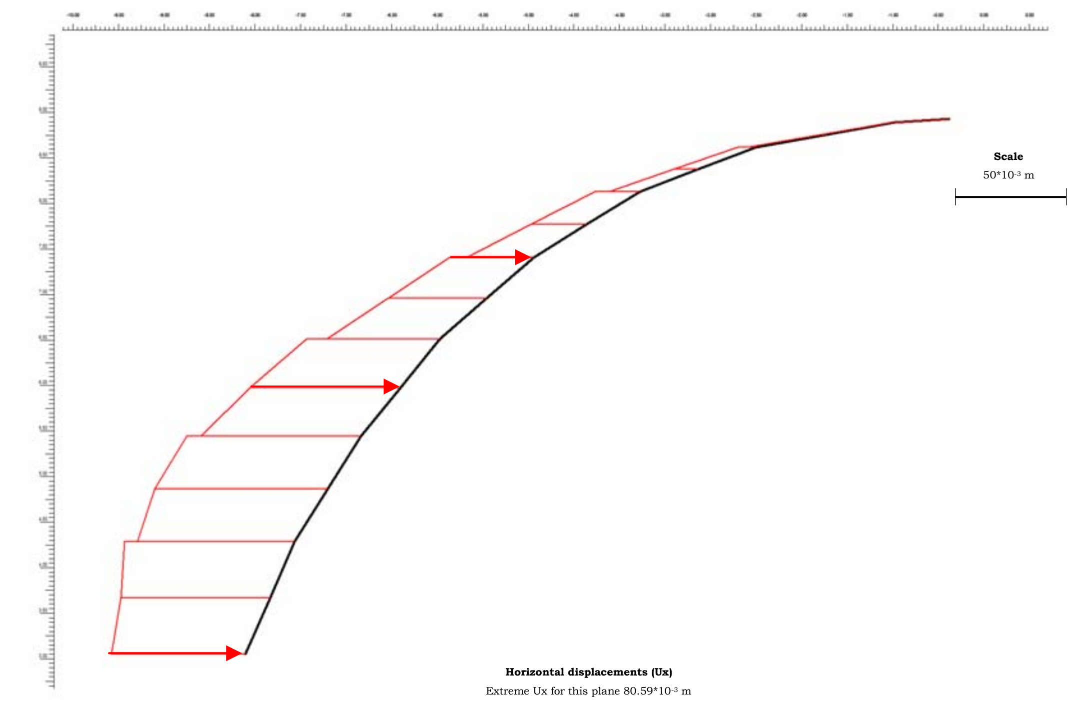


Figure 5.8 Displacement of lining in x-direction (3D model, section-d)

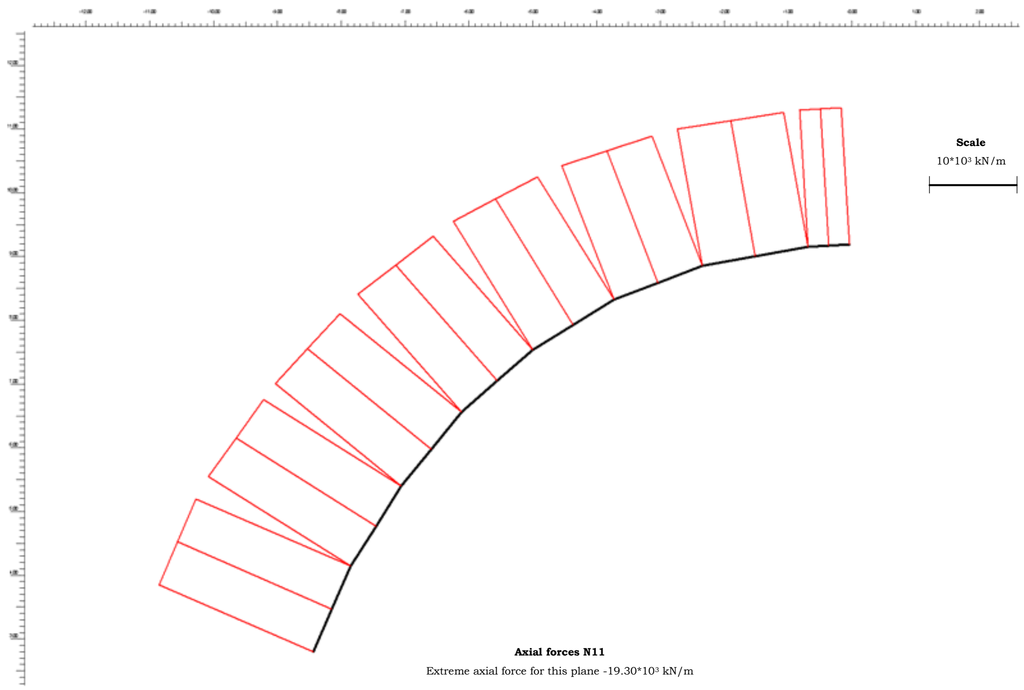


Figure 5.9 Typical axial force diagram (2D model)

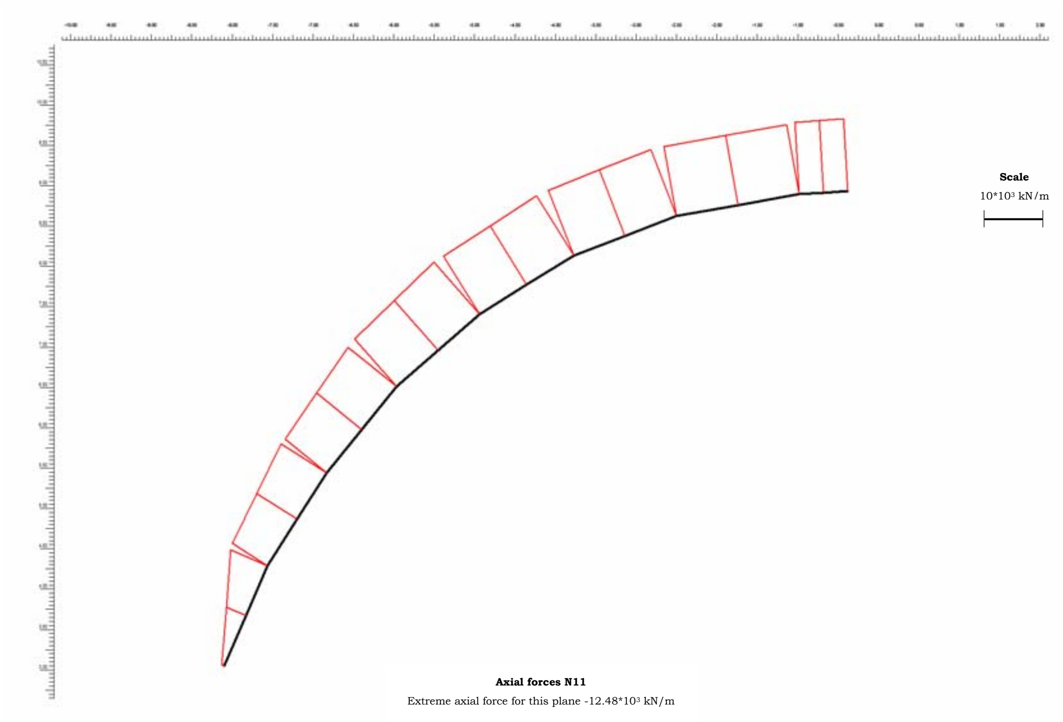


Figure 5.10 Typical axial force diagram (3D model, section-a)

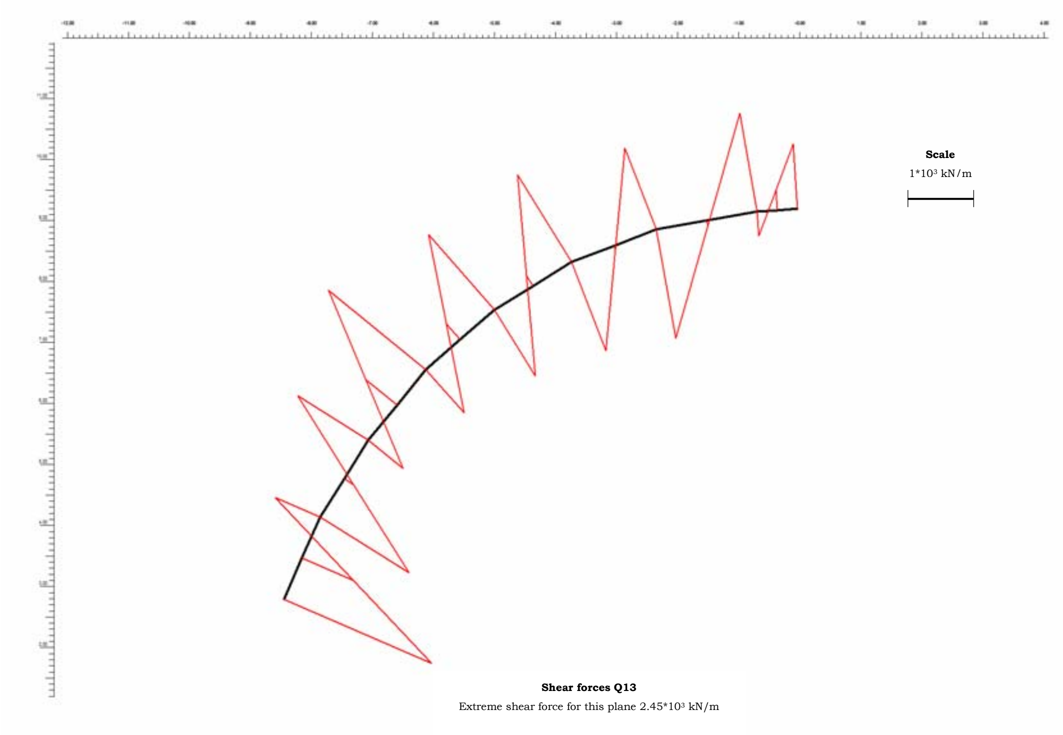


Figure 5.11 Typical shear force diagram (2D model)

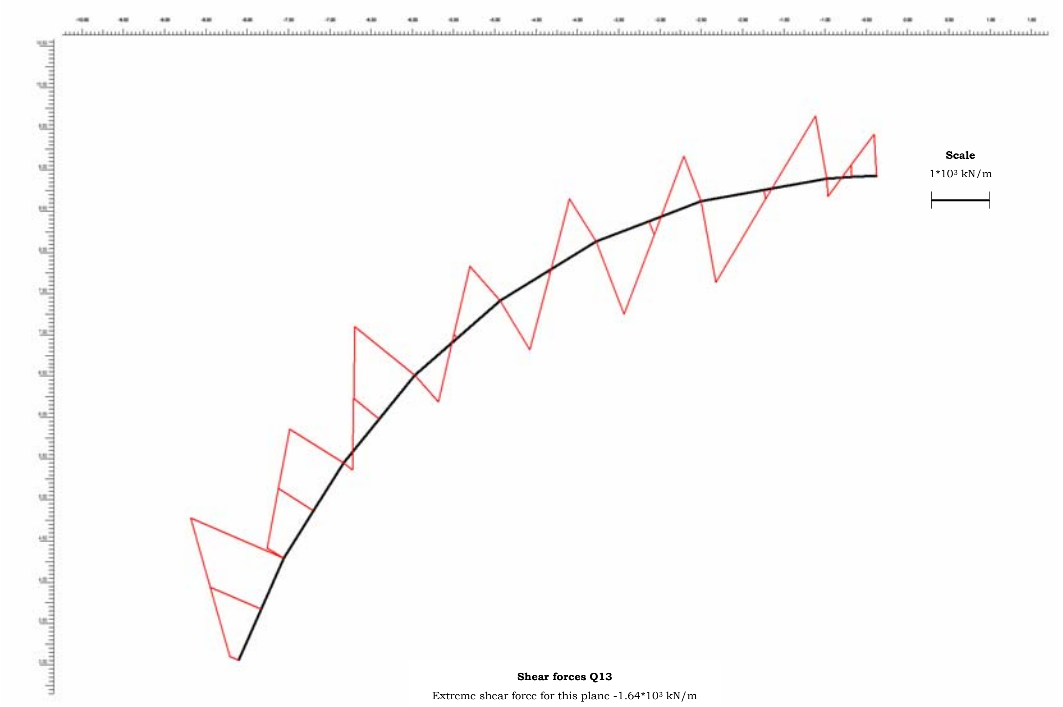


Figure 5.12 Typical shear force diagram (3D model, section-a)

Figure 5.13 Typical bending moment diagram (2D model)

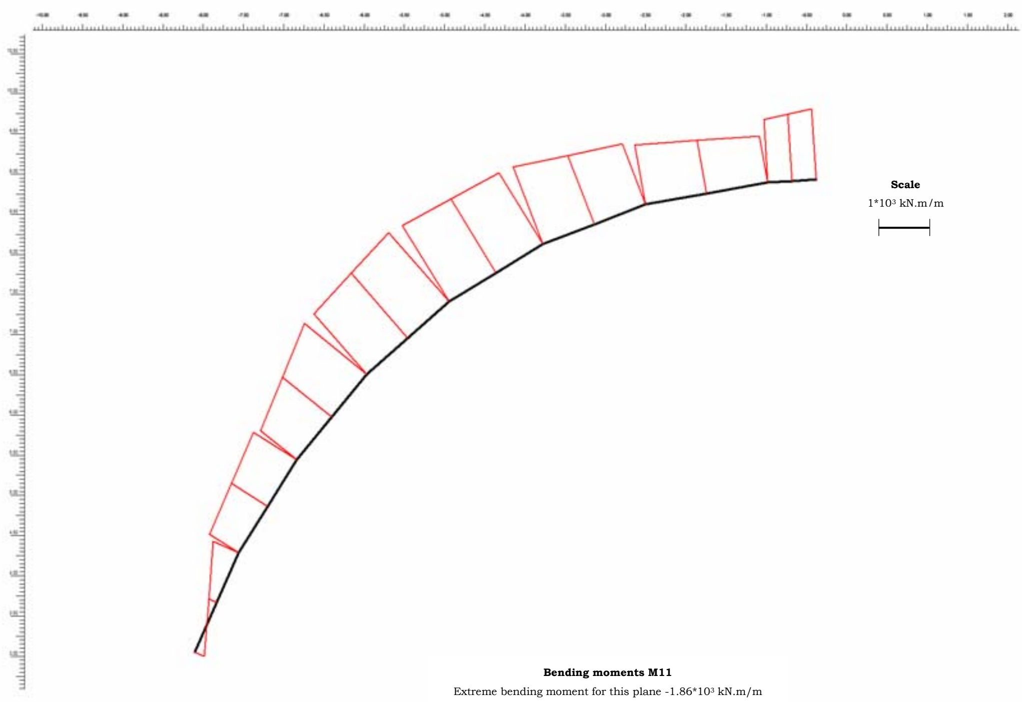


Figure 5.14 Typical bending moment diagram (3D model, section-a)

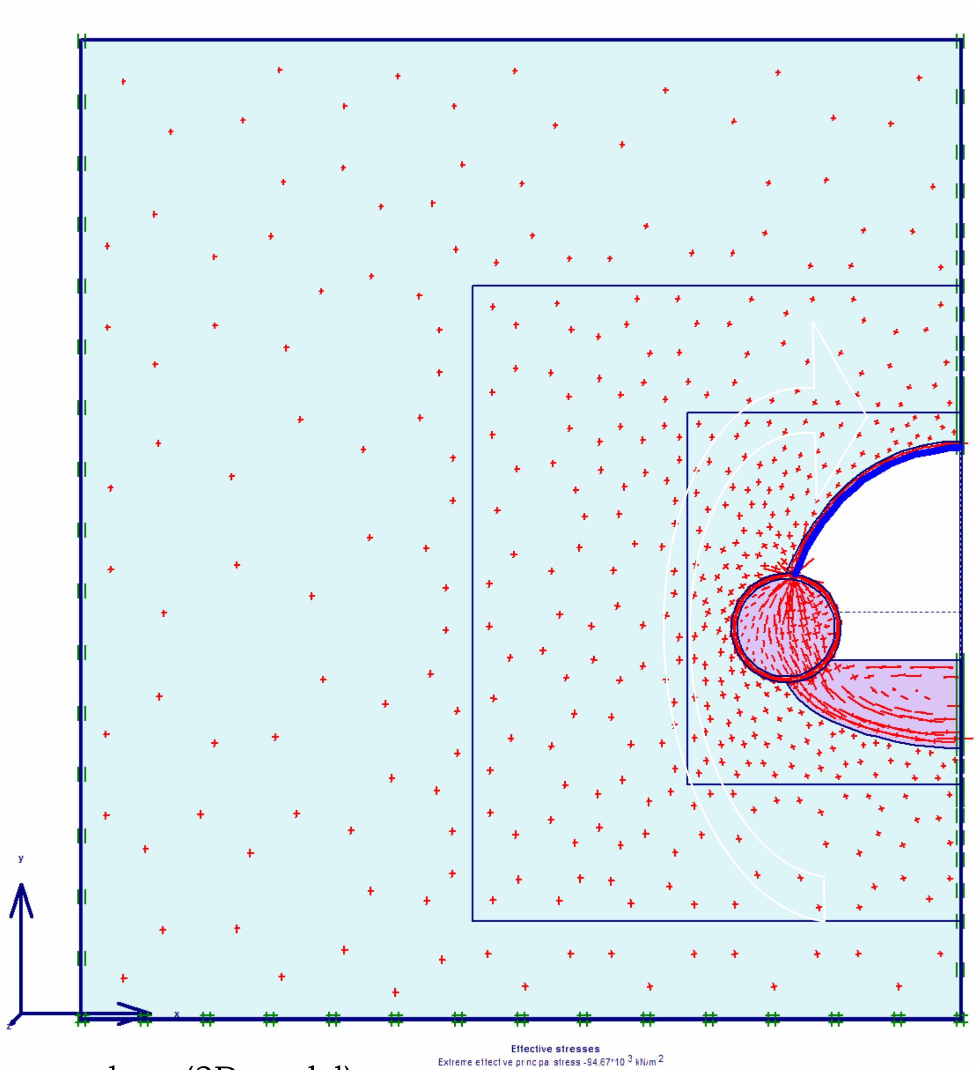


Figure 5.15 Arching effect in x-y plane (2D model)

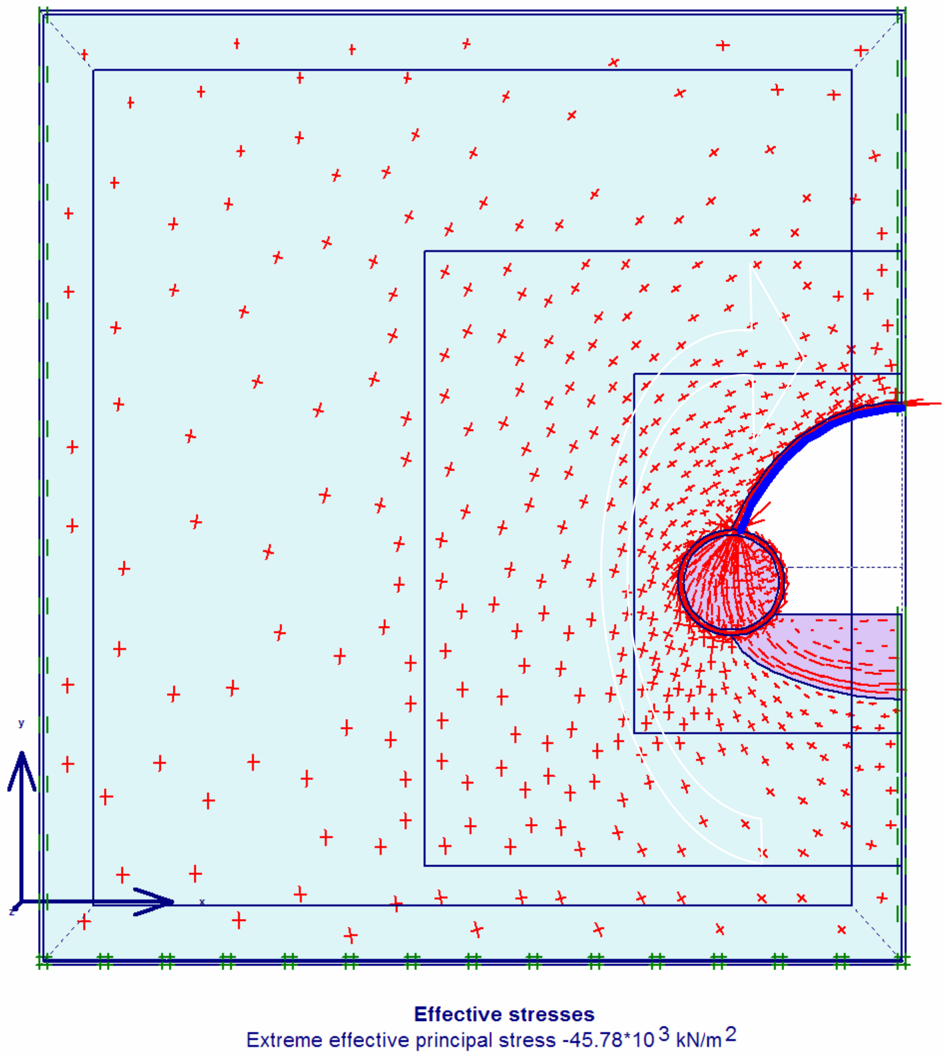


Figure 5.16 Arching effect in x-y plane (3D model)

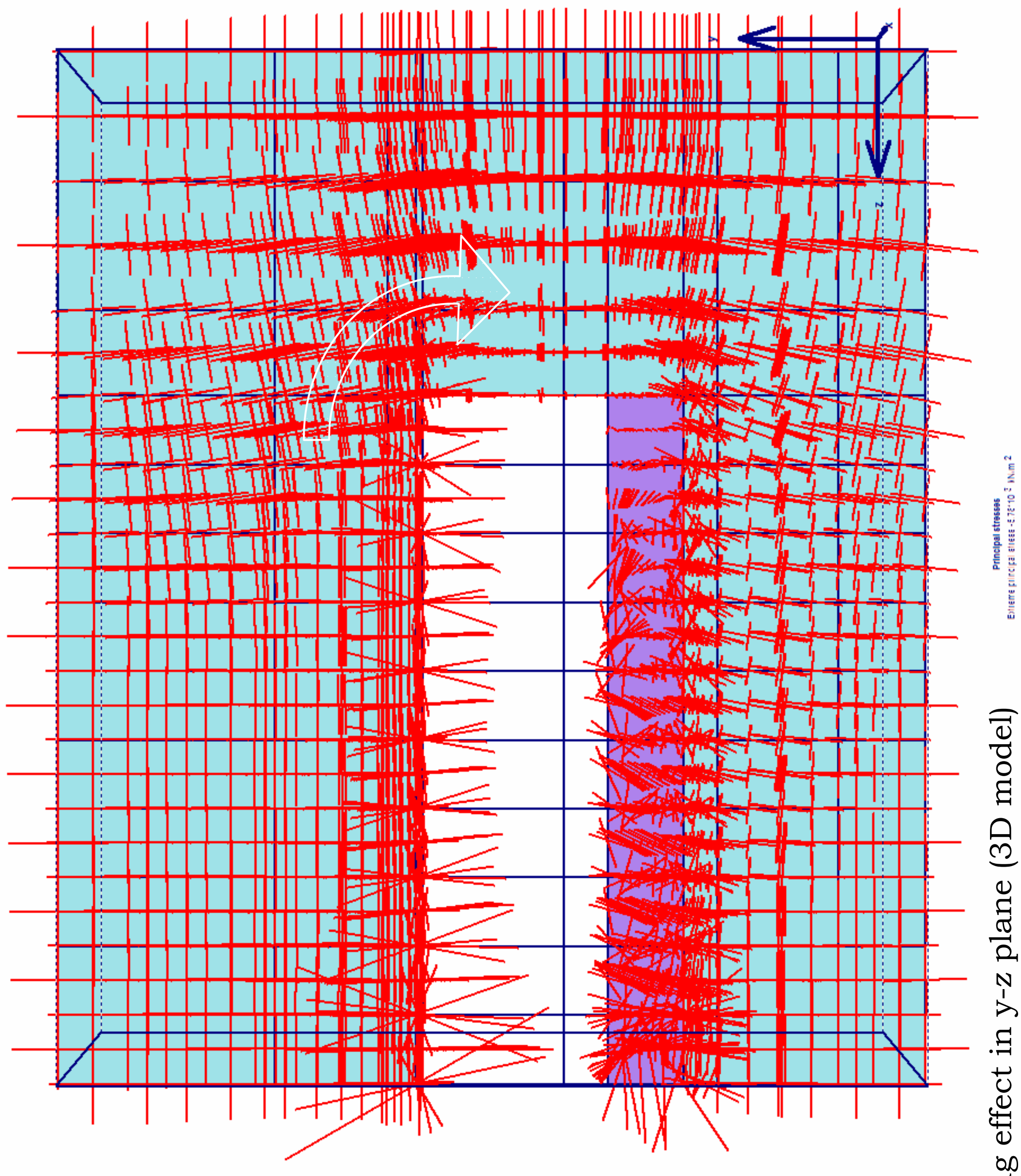


Figure 5.17 Arching effect in y-z plane (3D model)

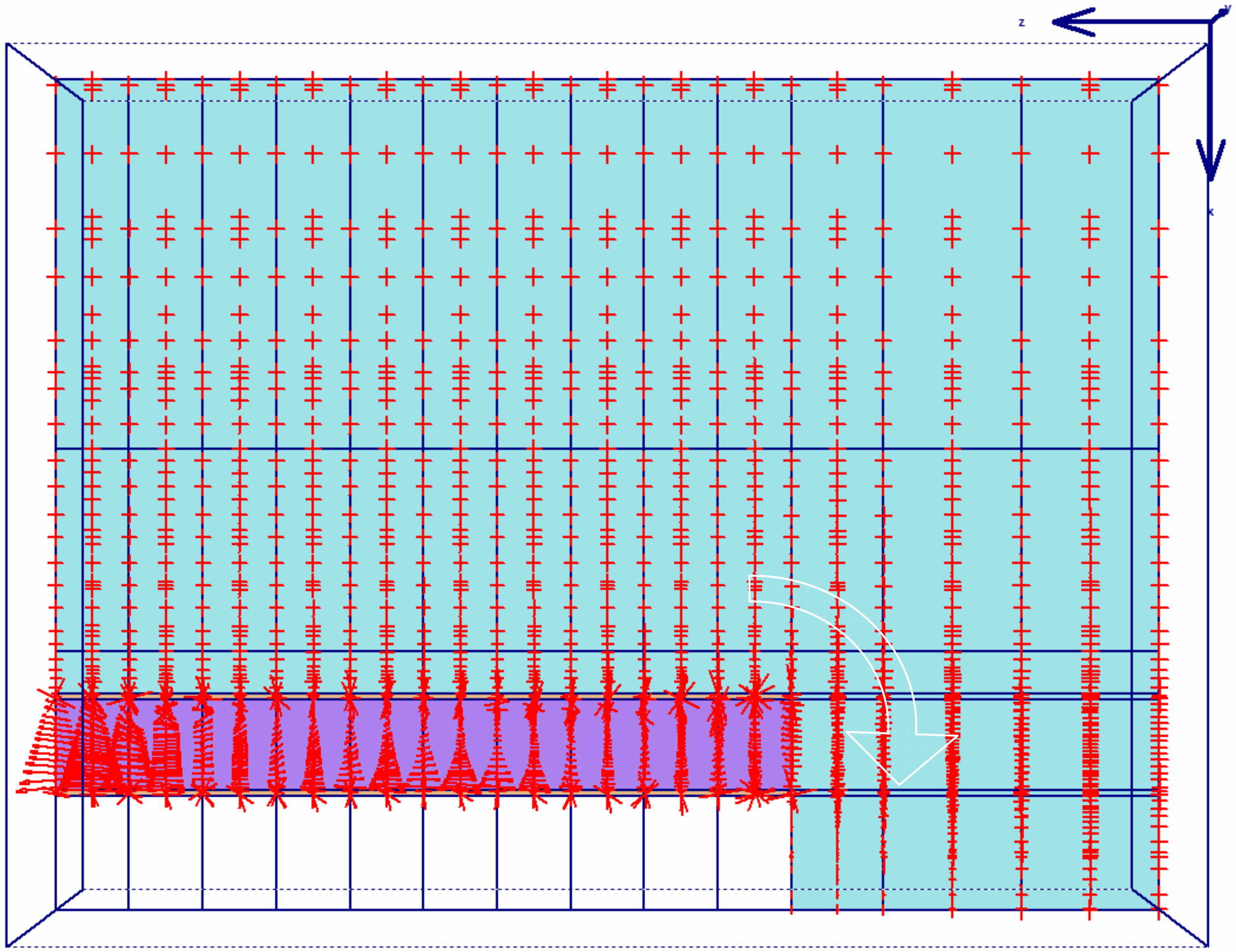


Figure 5.18 Arching effect in x-z plane (3D model)

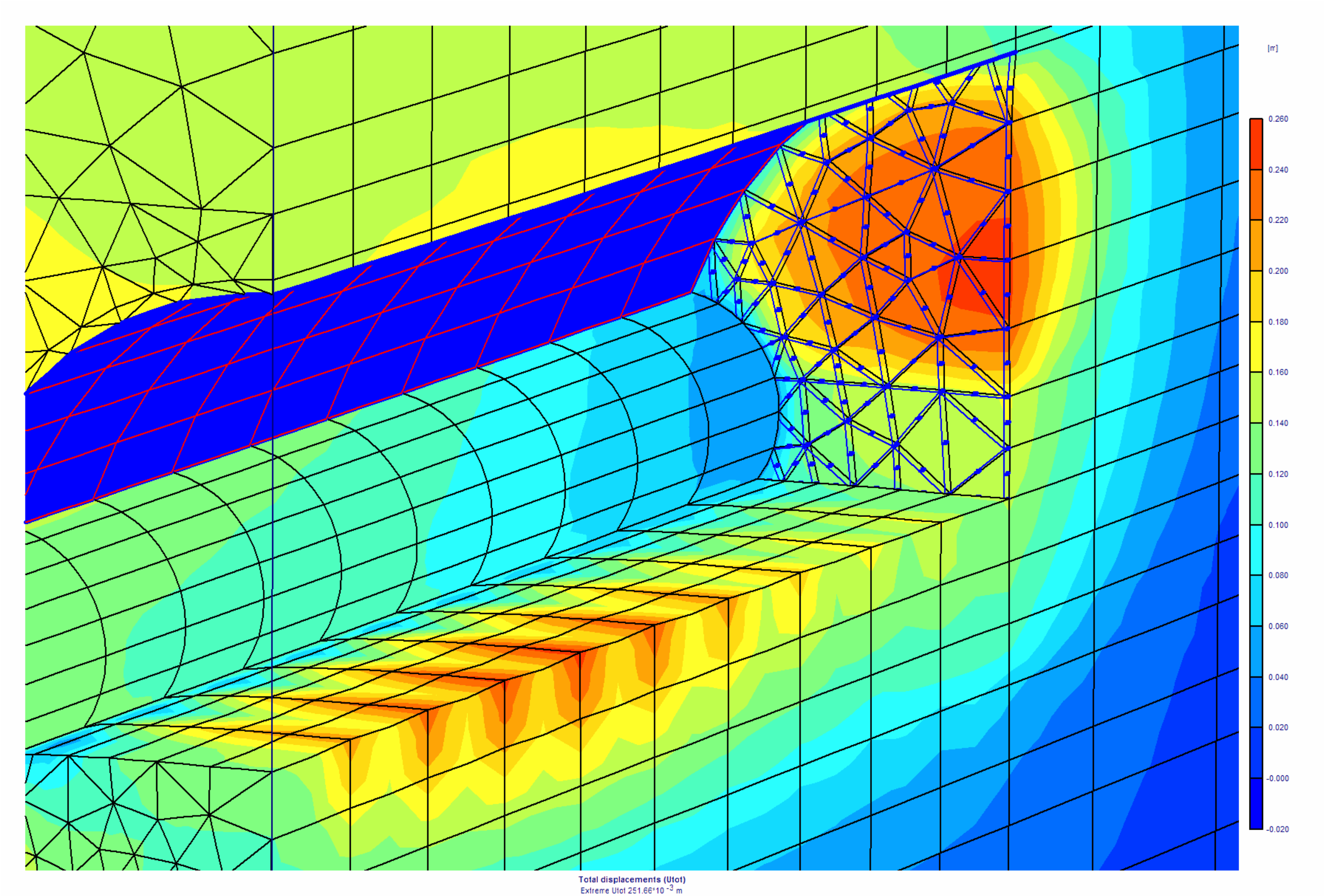


Figure 5.19 Ground heave and and heading displacements (3D model)

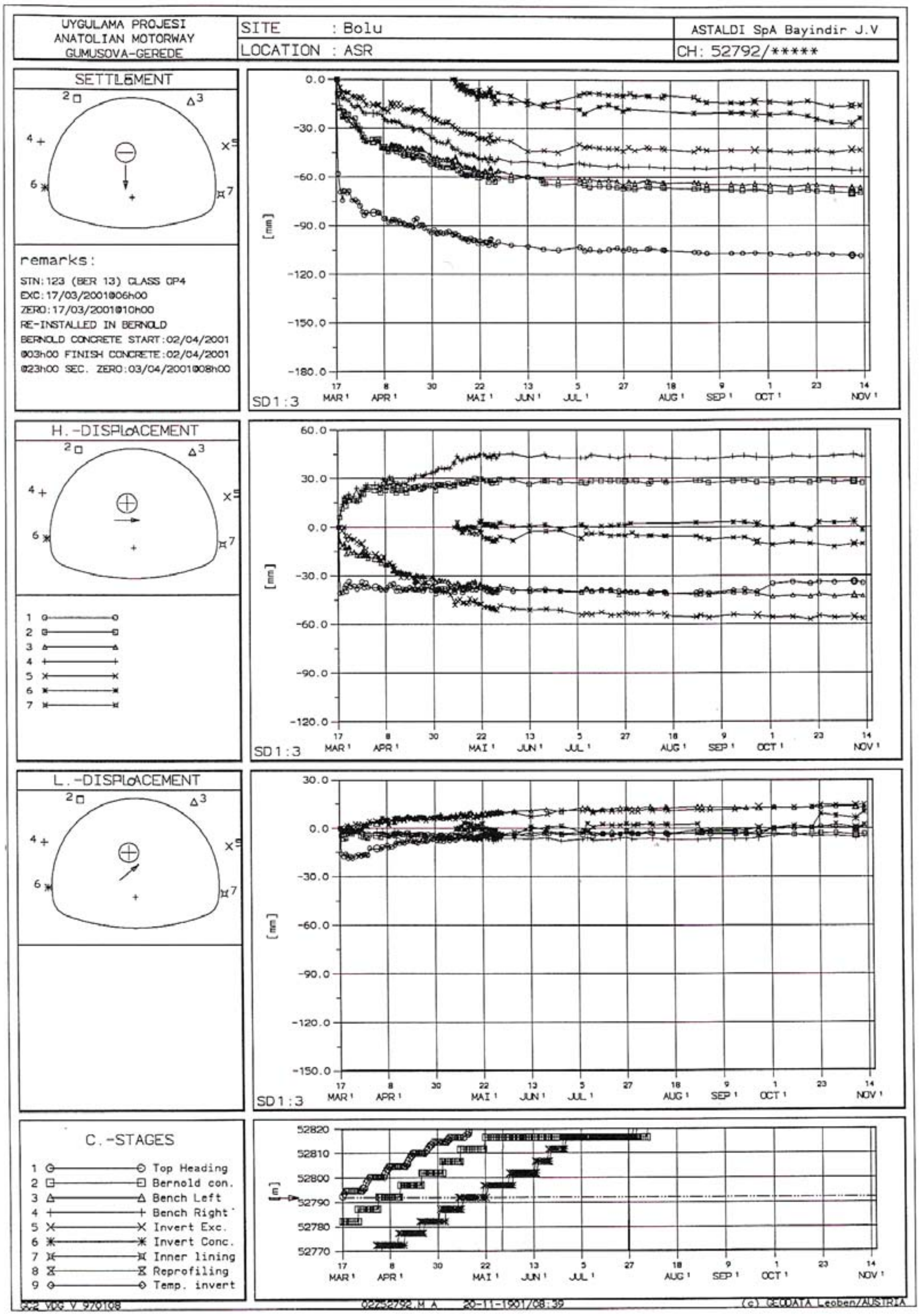
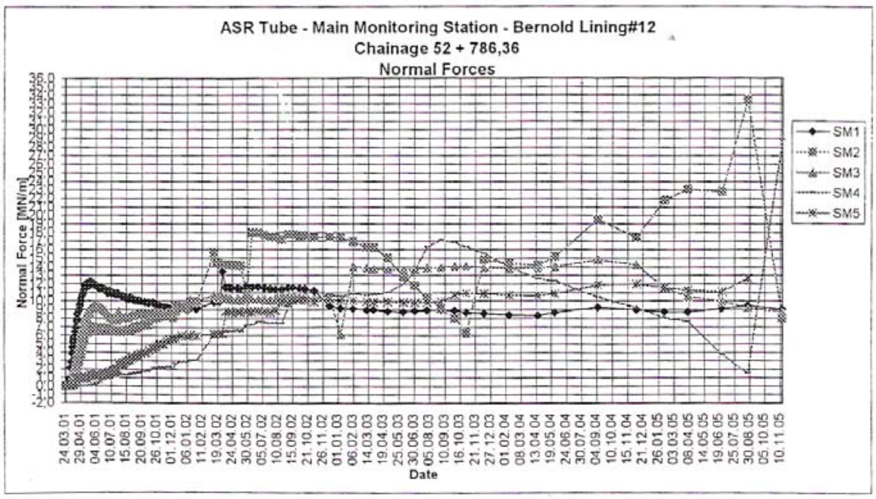


Figure 5.20 In-situ displacement measurements on a typical section of Option-4 (Yüksel Proje, 2005)



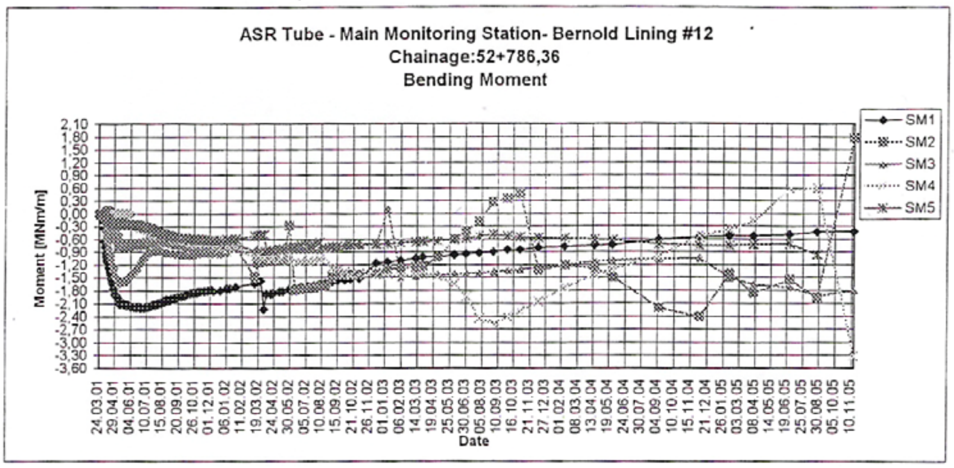


Figure 5.21 In-situ normal force and bending moment measurements on a typical section of Option-4 (Yüksel Proje, 2005)

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

The performances of the 2D and 3D undrained analyses of tunnel advance in high plasticity index flyshoid material encountered in the construction of Bolu Tunnels are investigated by comparing the results obtained from model studies to site data. The comparisons show that 3D analysis is superior to 2D analysis particularly in such difficult tunnelling conditions. Because, certain geotechnical problems, as in the case of the stress-strain state in the vicinity of a tunnel face, are highly three dimensional in reality. Accordingly, although, plain strain or axi-symmetric approximations are not unreasonable in most cases, there exist certain circumstances which must be analyzed through three dimensional models.

The most striking difference between the analyses results of 2D and 3D is the reverse horizontal displacement of the tunnel lining at the bench level. This is mainly due to the lost 3D effects of the tunnel lining and bench pilot tunnels in 2D models. Consequently, the internal reactions in the intermediary (Bernold) lining are higher in the case of 2D model.

The results of 3D analyses are in good agreement, in general, with the site data. However, the predictive capability of the 3D model can be increased by further calibrating the model. To give an example, the ground behaviour can be represented by different material models other than Mohr Coulomb model to improve the accuracy of the results. Besides, in the analysis undrained material behaviour is considered. Analyses with effective stress parameters by also considering the water table may add improvement to the design from the long term point of view. Additionally, use of more realistic material behaviour different than linear elastic to model the shotcrete and concrete may also improve the solutions. In fact, the shotcrete and concrete do not behave linearly.

In usual NATM applications, final (inner) lining is not considered in the analysis, but adds extra structural safety. However, in the case of Bolu Tunnels, site investigation showed that inner lining carries a considerable amount of load. Hence, to improve the results the conribution of the inner lining is to be taken into consideration in the design.

It is surprising to find such a large amount of ground heave at point C in 3D model calculations. The large amount of ground heave may be expected at point D, but it is unexpected at point C. Because, point C is on a monolithic concrete invert which is very stiff. Furthermore, monolithic concrete invert is placed after an excavation stage. So, the displacements must be around 1 mm to 15 mm at point C which are also in agreement with the recorded values at the site. However, calculated values are around 200 mm (see Figure 5.19). This error may be caused from the Plaxis 3D Tunnel software and ignored in the calculations.

Another way of increasing the accuracy of results is to include the effect of one tunnel on the other by using a finer mesh. But this has severe implications regarding modelling effort and computer resources. For example, while a 2D finite element analysis may take a matter of a few minutes on a fast workstation, similar analysis of a 3D analysis may take several hours. Most of this extra time is spent to invert the global stiffness matrix. Therefore, wherever possible, it is preferable to apply simplifications and consequent reduction to 2D as discussed in Chapter 2. In most cases, this 2D modelling may reflect the project conditions easily and economically, however in certain conditions, as in the case of Bolu Tunnels, 2D modelling may not be sufficient to capture the actual model behavior and the critical 3D effects may be lost.

For future further detailed 3D investigations of Bolu Tunnels, the following items can be considered utilizing improved hardware and software:

- more realistic material models,
- finer mesh and large problem boundaries including two tunnels,
- long term behaviour with effecetive stress parameters,
- final (inner) lining in the design.

REFERENCES

Bickel, J. O., Kuesel T. R. and King, E. H. (1996). Tunnel Engineering Handbook. United States of America: Chapman & Hall.

Çakan, A. G. (2000). Analysis of Tunnel Advance in Soft Ground Using Finite Element Method – A Case Study on Bolu Tunnels, M.Sc. Thesis. Ankara: Department of Civil Engineering, METU pp. 68-83.

Esward, T. J. and Wright L. (2003). Guide to Use Finite Element and Finite Difference Software. Teddington, Middlesex: National Physical Laboratory.

FLAC^{3D} User's Manual (1997). Minneapolis, Minnesota: Itasca Consulting Group, Inc.

Geoconsult, (1999). Anatolian Motorway Stretch-2 Bolu Tunnels Option 4 Calculation Report (No: 45.110/R/2161).

Geoconsult, (1998). Anatolian Motorway Stretch-2 Bolu Tunnels Design Methodology, Primary and Inner Lining, (Report No: 45.110/R/2159).

Geoconsult, (1997). Anatolian Motorway Stretch-2 Bolu Tunnels Review of Geotechnical Conditions at Bolu Tunnels, (Report No: 45.110/R/2125).

Geoconsult, (1998). Anatolian Motorway Stretch-2 Bolu Tunnels Geotechnical Parameters, New Test Results (Report No: 45.110/R/2155).

Geoconsult, (1997). Anatolian Motorway Stretch-2 Bolu Tunnels Design Methods for Unexcavated Sections, (Report No: 45.110/R/2149).

Geoconsult, (1998). Anatolian Motorway Stretch-2 Bolu Tunnels, Pilot Tunnel Back Analysis (Report No: 45.110/R/2156).

Geoconsult, (1992). Anatolian Motorway Stretch-2 Bolu Tunnels, Elmalık Tunnel Final Design Geological Report.

Melbye, T. (1999). Sprayed Concrete for Rock Support. Switzerland: MBT International Underground Construction Group.

Ng, C. W. W., Simons, N. And Menzies B. (2004). A Short Course in Soil-Structure Engineering of Deep Foundations, Excavations and Tunnels. Bodmin, Great Britain: Thomas Telford.

PLAXIS 3D Tunnel User's Manual (2004). Delft, Netherlands: Delft University of Technology & PLAXIS B.V.

Potts, D. M. and Zdravkovic, L. (1999). Finite element analysis in geotechnical engineering – theory. London, Great Britain: Thomas Telford, pp. 133-139.

Potts, D. M. and Zdravkovic, L. (2001). Finite element analysis in geotechnical engineering – application. London, Great Britain: Thomas Telford, pp. 38-52.

Szechy, K. (1973). The Art of Tunnelling. Budapest, Hungary: Akademiai Kiado.

Schubert, P., Moggioli, M., Brandl, J. And Golser, J. Extraordinary Difficulties Driving the Motorway through Bolu Mountains, Turkey. Felsbau 15, 1997, Nr5.

Sinha, R.S. (1989). Underground Structures, Design and Instrumentation. Netherlands: Elsevier, pp.84-123.

Sinha, R.S. (1989). Underground Structures, Design and Construction. Netherlands: Elsevier.