BACK ANALYSIS OF A TUNNELLING CASE STUDY IN WEAK ROCK OF THE ALPINE SYSTEM IN NORTHERN GREECE:
VALIDATION AND OPTIMIZATION OF DESIGN ANALYSIS BASED ON GROUND CHARACTERIZATION AND NUMERICAL SIMULATION

by

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A thesis submitted to the Department of Geology and Geological Engineering
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Abstract

The backdrop for this thesis is the tunnelling that is currently nearing completion in the Epirus region of Northern Greece, as part of the Egnatia Odos Highway construction. Highly deformed and altered sediments and low grade rock masses dominate the near surface environment creating a variety of technical challenges for tunnelling. Accurate equivalent rock mass performance reductions for tunnels in these materials are complicated by geomorphologic peculiarities such as those found in Flysch materials. The mechanisms of rock-support interaction related to face or near-face reinforcement systems are poorly understood at this time. As well, the mechanics of weak rock materials in the complex deformation regime in advance of a tunnel face are not robustly integrated into current 2D design models. Design decisions are currently possible using empirical techniques and simplified models, but a true optimized and mechanics-based design process for the various support technologies is not fully developed. This research addresses elements of such issues, such as: use of the Longitudinal Displacement Profile (LDP) of the Convergence-Confinement method of tunnel design, relating 2D numerical models to their distance from the face using the size of the plastic zone as an indicator, near face tunnel support analysis in weak rock masses, boundary condition assessment for numerical modelling of such weak rock masses, the influence of plasticity zones surrounding tunnel excavations, and modelling optimization techniques for weak rock tunnelling in order to optimize the design of such underground structures and better predict near-face deformation and yield development. This work involved the use of 2D and 3D numerical models of tunnel sequencing for numerical simulation of composite material behaviour and sequential tunnel deformation response.
To my wife, Ευφροσύνη, who taught me the value of life-long learning and who supports and believes in us and our family
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Statement of Originality

I hereby certify that all of the work described within this thesis is the original work of the author. Any published (or unpublished) ideas and/or techniques from the work of others are fully acknowledged in accordance with the standard referencing practices.

Nicholas Vlachopoulos

August, 2009
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<td>2D</td>
<td>Two-dimensional</td>
</tr>
<tr>
<td>3D</td>
<td>Three-dimensional</td>
</tr>
<tr>
<td>AC</td>
<td>Anchor Strain Gages</td>
</tr>
<tr>
<td>BE</td>
<td>Extensometers</td>
</tr>
<tr>
<td>CB</td>
<td>Survey Target Points</td>
</tr>
<tr>
<td>DEM</td>
<td>Discrete Element Method</td>
</tr>
<tr>
<td>DFN</td>
<td>Discrete Fracture Network</td>
</tr>
<tr>
<td>FYROM</td>
<td>Former Yugoslavian Republic of Macedonia</td>
</tr>
<tr>
<td>GRC</td>
<td>Ground Reaction Curve</td>
</tr>
<tr>
<td>GSI</td>
<td>Geological Strength Index</td>
</tr>
<tr>
<td>HC</td>
<td>Hydraulic Pressure Cells</td>
</tr>
<tr>
<td>IGME</td>
<td>The Greek National Institute of Geology</td>
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<tr>
<td>KFZ</td>
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<tr>
<td>LDP</td>
<td>Longitudinal Deformation Profile</td>
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<td>NAF</td>
<td>North Anatolian Fault</td>
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<td>NATM</td>
<td>New Austrian Tunnelling Method</td>
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<td>Guidelines for the Environmental Terms</td>
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<td>Design Guidelines for Conducing Road Works Design</td>
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Variables List

\[ \sigma_1' \] Maximum Effective Stress
\[ \sigma_3' \] Minimum Effective Stress
\[ \sigma_{ci}' \] Uniaxial Compressive Strength of Intact Rock
\[ \sigma_{cm}' \] Uniaxial Compressive Strength of Rock Mass
\[ \sigma_{tm}' \] Tensile Strength
\[ m_b' \] Intact Strength
\[ m_i' \] Hoek-Brown Constant
\[ E \] Deformation (Elastic) Modulus
\[ I_s \] Point Load Index
\[ P \] Load
\[ \Phi' \] Friction Angle
\[ E_m \] Modulus of Elasticity
\[ c' \] Cohesive Strength
\[ p_0 \] External, Hydrostatic Stress
\[ p_i \] Internal Support Pressure
\[ p_{cr} \] Critical Pressure
\[ k \] Lateral Earth Pressure
\[ E \] Modulus of Elasticity
\[ \nu \] Poisson’s ratio
\[ u_i \] Tunnel Sidewall Deformation
\[ r_o \] Outer Radius or Radius of the Plastic Zone
\[ u_r \] Radial Displacement
\[ t_c \] Thickness
\[ p_s \] Uniform Pressure
\[ \Delta u_r \] Change in Radial Displacement
**Definitions of Geological Terms**

**Flysch** – is a series of rocks deposited in a deep sea adjacent to a rapidly rising mountain chain, dominated by sandstone with finer siltstone and clay.

**Isopic Zones** – are groups of widespread rocks that share a common history, both in the ancient environments of deposition of sediments (deep ocean, shallow sea, continent etc.) and their faulting and folding (tectonic environment).

**Massifs** – are blocks of metamorphic and plutonic rocks, formally assumed to be much older and more resistant to folding and faulting than adjacent sediments. Lower levels of continental crust exposed by faulting and folding.
Chapter 1
Introduction

1.1 Purpose of Study

Greece is a country born of intense tectonic processes. Highly deformed and altered sediments and low grade metamorphic rock masses dominate the near surface environment creating a variety of technical challenges for tunnelling related to modern infrastructure.

This thesis analyzed the effects of tunnelling in such complex, anisotropic, matrix-rich, and inherently weak rock mass terrain. The mechanisms of rock-support interaction related to face or near-face reinforcement systems are poorly understood at this time. As well, the mechanics of anisotropic and weak rock materials in the complex deformation regime in advance of a tunnel face are not robustly integrated into current design models. Design decisions are currently possible using empirical techniques and simplified models, but a true optimized and mechanics-based design process for the various support technologies is not fully developed. This research addresses elements of such issues associated with design of near face tunnel support in weak rock masses, the influence of plasticity zones surrounding tunnel excavations, and modelling optimization techniques for weak rock tunnelling in order to optimize the design of such underground structures and better predict near-face deformation and yield development.

Three-dimensional (3D) ground reaction considerations were investigated by numerical simulation of composite material behaviour, rock-support interaction mechanics, and sequential tunnel deformation response. In the first phase, stress-deformation test models of weak rock representative of the rockmass domains encountered in Northern Greece (Epirus Region), were used to provide accurate material property parameters for use in tunnel scale modelling. Secondly, existing structural elements were modified and optimized to represent the 3D
mechanics of face and pre-face support and interaction with the progressively deforming rockmass along an advancing tunnel. Finally, tunnel scale models were utilized to quantify the effectiveness of layered support systems for primary support of advancing tunnels in these rockmasses. Parametric analyses were carried out in order to determine the effect of these parameters with respect to tunnel behaviour. These findings were then compared to tunnel behaviour in the field by way of monitoring data of tunnel displacements. During this process, it was also evident that current 2D as well as 3D modelling of such tunnel excavation processes had their limitations. As such, this research undertook to improve these numerical modelling techniques for excavation in the weak rockmass materials.

Ultimately, this rigorous analysis, calibrated using high quality monitoring data from tunnels in Greece, yielded a more logical and generalized analysis guidelines for tunnelling engineers. Improvements in numerical modelling techniques were achieved and incorporated into Phase2, 2D numerical modelling software whereby the 3D research models were used to obtain accurate but simplified procedures for the simpler and more widely utilized 2D design analysis.

The backdrop for this research project was the tunnelling that is currently underway in Northern Greece, specifically, in the Epirus and Western Macedonia regions as part of the Egnatia Odos Highway construction. The author has conducted multiple site visits to the region for physical mapping and rock mass determination / interpretation purposes. The major components of the research program are outlined in the subsequent sections of this document. These include sections dealing with the Egnatia Odos project itself, the geological and tectonic setting for this research, rock material characteristics and mechanics, tunnelling construction techniques, numerical modelling of tunnel construction and specific issues associated with the research conducted as part of this investigation. Also included are 3 journal papers that highlight selected areas of research and relevant findings / contributions.
1.2 Objective(s)

The objective of this investigation was to determine effective methods of modelling tunnel construction within weak masses with a view of improving these analytical techniques for design purposes (to include accurate tunnel displacement predictions and ultimately optimization of temporary support design measures). As such, numerical analysis and modelling was conducted in 3D and validated with survey data obtained from the field. These results were then used to improve 2D numerical techniques. The analysis was conducted within a framework of a specific rockmass model (Geological Strength Index) for the weak rock masses associated with the Driskos Twin Tunnels (as part of Egnatia Odos, Greece). The specific themes are cited in Section 1.4.

The specific objectives of this research projects as they relate to the journal papers that were written, submitted and approved by experts in the field were:

a. To use the accurate monitoring data that was amassed during the construction of the Driskos Twin Tunnels (Case Study) in order to validate the complex, 3D numerical modelling process;

b. Use the data from the Driskos case study in order to provide insight into parameters required for numerical modelling purposes through back analysis;

c. Determine the best method of creating accurate 2D and 3D numerical models of tunnel excavations within weak rock masses in order to ensure the accuracy of boundary conditions, input parameters, temporary support elements and other important influencing factors;

d. Determine the influence of the zone of plasticity around a tunnel excavation on tunnel displacement (closure) with a view to improving 2D numerical analysis, tunnel design and ultimately optimizing temporary tunnel support;

e. Determine how to properly capture real 3D effects in Pseudo-3D (i.e. using 2D models to simulate 3D) by applying unique simplifications in 2D modelling software in order to account for the very real phenomenon of behaviour when dealing with high or isotropic stresses; and
f. Identify sources of conservatism and implications to tunnel design of tunnels that are excavated in weak rock masses.

1.3 Egnatia Odos Project

The validation data that was obtained for this project was associated with the tunnelling conducted as part of the Egnatia Odos construction project. The Egnatia Odos Highway is a massive construction project that is currently under construction in Northern Greece. The project is an upgrade of the current highway across the North of Greece. The old alignment follows the ancient Egnatia Road, an 800 km route constructed by the Romans for military purposes in the 2nd century B.C. The present Egnatia Odos motorway that connects the Ionian coast with the Black Sea revives this historic alignment which played the same role in ancient times as it does today; ensuring a quick and safe connection from the East to Europe.

The Egnatia motorway is a thoroughly designed, modern transport corridor that will serve strategic European traffic crossing Northern Greece and will upgrade domestic links between Eastern and Western Greece. It will minimize the time and distance covered to travel from Epirus to Macedonia through the natural mountain obstacles (Pindos Mountains) in that region. It will also provide safe passage from Igoumenitsa to Alexandroupoli. This highway is a priority project for the European Union and one of the current 14 projects of the TransEuropean Network. Upon completion, the new 670 km Egnatia motorway (Figure 1.1) will have a total of 74 road tunnels with an overall combined length of 100 km, measured at 50 km of single bore. Sixty (60) of these tunnels are bored or blasted tunnels. The remainder are cut-and-cover (Egnatia Odos A.E., 2007). The length of these tunnels range from 800 m to 4,600 m. Over 7% of the overall highway will be carried through tunnels, incurring 35% of the total estimated construction cost. The original estimated overall budget of the project is 3.2 billion Euros, 60% being funded by the
European union and 40% by the Greek government (Silva et al., 2002). In extremely poor geotechnical conditions, the cost per linear metre of tunnelling is estimated at $8000 (US) in good conditions and $32000 (US) in poor geologic conditions (Egnatia Odos AE, 2001). By the end of 2009, 620 km of the route will have been completed (Egnatia Odos, 2009).

Due to geological framework and topography, most of the tunnels are located in the Pindos Mountain region of North-Western Greece (in Epirus and in Central and Western Macedonia). The standard tunnel design that Egnatia Odos SA has adopted is a two-lane highway consisting of two traffic lanes each, 3.75 m wide with a clearance of 5 m. Examples of tunnelling construction and other works are shown in Figure 1.2. These are investigated further in Chapter 4 of this document.
Within each of the major tunnels, advanced monitoring and sensing stations were and are being utilized. Mechanisms include advanced air quality sensing and ventilation systems that work to monitor and maintain general air quality and extract smoke in case of a fire event. Safe and economic operation of the tunnels is achieved through the use of state of the art highway telematics systems along the entire route. For emergency purposes, each tunnel accommodates underground emergency parking, turn-around points and cross passages between Eastbound and

Figure 1.2
Examples of construction as part of Egnatia Odos taken in July 2003 and 2004
Westbound tunnel bores every 350 m as stipulated by EUROCODE 7 (EUROCODE 7, 2008). Reinforced concrete is used to provide the tunnel with a final lining. A drainage system between the lining and the rock ensure that drainage is also adequately dealt with.

Aside from tunnels, when the project is completed, over 1700 structures will be constructed as part of Egnatia Odos. In total, 1650 bridges (20% of the total project cost) of various construction, with a combined length of 40 km will have been built. In addition to this, there will be 43 river crossing and 11 railway crossings. The route also consists of approximately 720 km of secondary road networks consisting of ramps and service roads. As well, 35 junctions will connect Egnatia Odos to existing national and provincial highways as well as local routes.

The Egnatia Odos Highway is being constructed in order to open up new, modern and safe roads connecting the countries of the European Union, the Balkans and the East. The motorway was designed to the specifications of the Trans-European network. The highway serves 4 ports in three Greek regions: Volos in Thesalia, Thessaloniki and Kavala in Macedonia, and Alexandroupolis in Thrace. There are nine perpendicular (or vertical) axes connecting Greece with its neighbouring countries, namely: Albania, the Former Yugoslav Republic of Macedonia, Bulgaria and Turkey. The route is also connected with the airports of Ioannina, Kastoria, Kozani, Thessaloniki, Kavala, Volos and Alexandroupolis. The entire project is divided into three sectors: Western, Central and Eastern. There is a construction manager and three international consultant companies overseeing the construction within each of these sectors.

The benefits of this project as cited by the Greek General Secretariat of Public Works are (Christaras et al., 1994):

a. The fund savings that result from the shortening of the existing route;

b. The improved communication between the North Western provinces of Epirus,
Thessaly and Macedonia (without the danger of the route being blocked during the winter season);

c. The essential improvement of traffic conditions along the international axes connecting to Europe and the Middle East; and

d. The economic benefits associated with the improvement of the exploitation and transportation of the products of the region.

The motorway traverses the entire width of Greece, crossing almost perpendicularly the main geotectonic units (as will be discussed in Chapter 2). As a result, many geotechnically unfavourable characteristics were encountered when deciding how to align the Egnatia Highway. The variety of geological conditions encountered along the axis of the motorway demand that the design of its structures consider these diverse geological situations. Within the Pindos mountain range system, rock masses that were encountered consist mainly of Flysch formations, carbonate and ophiolitic rocks. The presence of Flysch in this area create conditions that disturb the stability equilibrium of hillsides along the highway and trigger slope and excavated cavity movement that vary from creep to landsliding. Problems arising from such phenomena have affected the construction of the roadway since the beginning of its construction (Christaras et al., 1994). Once the material has been identified, the generation and definition of an accurate geotechnical rockmass model must also be investigated in order to choose the appropriate geotechnical parameters for the design of tunnels. Also affecting the alignment were various environmentally sensitive areas and locations of high archaeological interest. The great variety of geological / geotechnical situations imposes the need for different approaches in designing the various components of the highway.
1.4 Organization of Research Document

As can be seen, there are many topics that must be taken into consideration when investigating aspects associated with rock tunnel construction. As such, this thesis project has incorporated all of the influencing components coupled with tunnel construction within a weak rockmass. In order to analyze tunnelling in weak rock masses, numerical modelling techniques must reflect the true nature of the problem and take into account the very real factors that influence this phenomenon. The components that form the framework of this research (and are incorporated in the organization of this document) are outlined in Figure 1.3.

The topics that are covered in each of the chapters that make up this thesis document are elaborated upon. Being a manuscript thesis, the main portions of this thesis are the three (3) major journal paper contributions that are contained within Chapters 6, 7 and 8. The other chapters provide background information, approach, an overview of relevant concepts and tie the three (3) journal papers together. The original intent was to determine how to optimize the temporary support design for the Driskos twin tunnels as field observations were secured by the author from the National Technical University and Egnatia Odos SA (Chapter 8). This data proved useful for validation of numerical models. During this process, however, it was determined that current 2D modelling techniques have definitive limitations when dealing with weak rock materials. As such, other investigations on determining how to optimize numerical techniques for these scenarios were undertaken (Chapters 7 and 8).

The thesis document is broken down into the following chapters:

Chapter #1 – Introduction. This chapter presents an overview of the main organization of the thesis document, relevant concepts, and introduction of the Egnatia Odos project and the specific objectives.
Chapter #2 – Geological Setting. This chapter provides the reader with the geological framework (from the macro to the micro scale) of the research area, citing the main tectonic influences, geologic processes and materials located in this region.

Chapter #3 – Rock Mass Properties and Mechanics. This chapter describes the use of rock mass classification systems, rock mass models and cites the determination of mechanical properties for the weak rock materials of this research project. It also includes the values of the engineering properties that were used as part of this investigation.

Chapter #4 – Tunnelling in Weak Rock. The focus of this chapter is the current design and construction techniques that are employed for tunnelling within weak rockmasses. Specific parameters and design support values are cited that were used for the Driskos Twin Tunnel temporary design and construction.

Chapter #5 – Numerical Modelling. This chapter cites the modelling approaches that one may employ while analyzing the response of a rock mass to the creation of a tunnel cavity. Also cited here are the specifics of the 2D (Phase2) and 3D (FLAC3D) numerical software packages that were used for this research. The chapter ends with considerations associated with interpreting the results of numerical modelling results and the specifics associated with the models used in this investigation. The chapter is closely associated with Appendix A, which contains a detailed summary of the complex 3D numerical model that was created specifically for this investigation.

Chapter #6 – Journal Paper on Tunnel Deformations and Support Considerations using the Driskos Twin Tunnel as a Case Study. This paper cites the issues associated with the excessive deformations that were observed at Driskos. A series of 2D and 3D analyses were tested and conducted in order to accurately model and predict tunnel deformations. Data from the field was also used in order to validate these numerical models.
Figure 1.3 Thesis research rationale, organization and interrelated components.
Chapter #7 – Journal Paper on Longitudinal Displacement Profile (LDP). Using a series of numerical analyses, a new series of functions defining robust longitudinal displacement profiles, as a function of maximum normalized plastic radius, was developed. This approach takes into consideration the effect that a large ultimate plastic radius has on the rate of development of wall displacements with respect to location along the tunnel. Previous LDP functions were inadequate for tunnel analysis in very weak ground at great depth. This approach is valid from the elastic case through to complete plastic closure of the tunnel (as calculated using numerical or analytical solutions).

Chapter #8 – Journal Paper on Comparison of 2D and 3D Analysis Methods for Excavation of Tunnels in Weak, Highly Stressed Ground. This section of the research involved the use of robust 3D numerical models of tunnel sequencing in order to obtain simplified procedures for the more widely utilized Pseudo-3D, 2D numerical design programs. In addition, 3D face effects, support considerations, varying horizontal stress fields and twin tunnel interaction were also studied for circular as well as horseshoe geometries of tunnel cross-sections.

Chapter #9 – Conclusions, Limitations, Contributions and Recommendations. This chapter summarizes the main findings of the research, the main contributions to the academic, scientific and engineering communities and provides recommendations for future topics of potential investigation.

Appendices. The appendices that have been included add value to the overall document by citing specific concepts or data sets that were used within this investigation. Selected examples include: Geological Strength Index (GSI) examples, the FLAC3D numerical modelling code that was used for this investigation, conference papers that were submitted during this research venture, sketches of faces during the excavation process and other relevant information.

Again, being a manuscript thesis, the primary areas of research are captured within the three (3) journal papers that make-up this document as well as the detailed geological framework that was studied as part of this investigation. The other chapters provide the background information and connect the specific themes that are presented in each of the journal papers.
Chapter 2
Geological Setting

2.1 Introduction

The main region of study used as a backdrop for this investigation was the mountainous section of North-Western Greece, in Epirus and in Central and Western Macedonia. As with any rock mechanics and tunnelling analysis, a complete study of the geology (past and current states) of the region is a key component in order to foster an overall understanding of the earth materials located at the site and the processes that have influenced and continue to influence the region. This chapter summarizes the geological events that have shaped this region from the past to the present and from the macro (i.e. Mediterranean Basin) to the micro scale (i.e. Driskos Tunnelling Site).

2.2 Mediterranean Region

The complex Mediterranean geological history including the development of the geotectonic units of Greece have been primarily influenced by the interaction between two substantial tectonic plates of continental crust. These plates were the Euroasian plate to the North and the Godwanaland plate (composed of present day Africa, Arabia and India) to the South (Papanikolaou, 1986). The Alpine structure of the peninsula is a result of the collisions between these plates as well as the closure of the Tethyan Ocean. The geological evolution of the Mediterranean region started approximately at the end of the Paleozoic era (230 million years ago (Ma))(Dermitzakis et al., 1986). The main characteristics of that era were the presence of the Pangea continent - which was composed of the future continents of Europe, Asia, Africa, Australia and Antarctica - as well as the Tethys Ocean (Figure 2.1) which was located between
the EuroAsia in the North and the Gondwanaland (Africa, Australia, Antarctica) to the South. The Tethys Ocean opened to the East joining the other oceans. In the early Mesozoic era, major geotectonic events – which span the last 200 million years - resulted in transforming the region and developing the geology of the Mediterranean as it stands today. The main geotectonic stages from the early Mesozoic to Cenozoic are described below.

### 2.2.1 Opening of Tethys Ocean

At the end of the Paleozoic era, EuroAsia started to separate from Godwanaland as a mid-oceanic ridge at the bottom of the Tethys Ocean began to form. Stretching between these two parts of the Pangea continent caused thinning of the continental crust, faulting and subsequent occurrence of volcanic events during the Triassic period. Following the expansion of the Tethys Ocean, carbonate sedimentation in shallow basins within the continental margins of the two tectonic plates took place. Part of the shallow carbonate formations of these early basins eventually lead to the limestone-rich geotectonic units of Greece. This first stage lasted from early Triassic to the late Jurassic period (Christaras et al., 1997).

### 2.2.2 Beginning of Subduction – Creation of the Orogenetic Arc

In the late Jurassic period, the moving direction between EuroAsia and the Godwanaland changed from outwards to inwards and the gap between the two tectonic plates started to close. The mechanism and reason for this is still a cause of academic debate. The mid-oceanic ridge was still active, creating oceanic crust that was pushed over the continental margins of the two
plates. As a result of the tectonic events during this stage, ophiolites were formed along the mid-oceanic ridge but the subduction processes led to the destruction of the already existing oceanic crust on the Godwanaland margins as the EuroAsia plate moved southwards (Papnikolaou, 1986). New stretching and concurrent subduction events took place leading in activity on the EuroAsia plate where the first orogenetic processes occurred, while the Godwanaland plate remained passive. Finally, the characteristic phenomenon in the Tethys system during this stage was the unconformity observed between folded carbonate sediments and ophiolite formations and overlying clastic sedimentation of the late Cretaceous period (Kenomanian period). This stage lasted from the late Jurassic to the early Cretaceous period (Higgins and Higgins, 1996).

2.2.3 **Subduction Events - Destruction of the Oceanic Crust**

The subduction of the Gondwanaland plate as the EuroAsia plate moved southwards resulted in major orogenetic processes on the continental crust of the EuroAsia plate. The mid-oceanic ridge of Tethys was destructed at this stage and pre-formed ophiolites were thrusted over the continental margin of EuroAsia. This period lasted from early Cretaceous to Eocene-Oligocene (Papanikolaou, 1986).

2.2.4 **Stage of Collision of the Two Continental Plates**

During the Upper Miocene, the orogenesis was a major process within the continental crust of the EuroAsia plate (Higgins and Higgins, 1996). This period is marked by the collision of parts of Godwanaland with EuroAsia. Results of these collisions include the formation of the Alpine region, including the Hellenic mountain ranges, the formation of the Caucasus-Iranian mountain ranges (collision between Arabic tectonic plate and EuroAsia plate) and the creation of
the Himalaya mountain range (collision between India and EuroAsia). This stage extends from the Miocene epoch within the Cenozoic era (today’s geology).

2.3 Geology of Greece

2.3.1 The Geological Environment and Formation

During the continual continental rifting that lasted approximately 200 million years and as the Godwanaland plate rotated in a clockwise direction about an axis in the Atlantic off Gibraltar closing the Tethys Ocean, Greece was located within a shallow, oxygen rich depositional environment during most of the Triassic, Jurassic and Cretaceous periods. The intense crustal deformation that occurred during the Oligocene-Miocene orogenetic processes yielded the Alpine orogenetic system (Papanikolaou, 1986). Greece, as a geological unit, is a characteristic part of this Alpine orogenetic system.

2.3.2 Tectonic Setting

The continuing Hellenic subduction along the south-western border of the Aegean plate has brought about the present tectonic setting of the whole of Greece. The kinematics of this deformation is currently controlled by the relative south-westward movement of the Aegean plate with respect to Europe, where it overrides the oceanic part of the African plate (Figure 2.2). The East deformation of the Aegean region is caused by the Westward motion of Anatolia, and in the West, by continental collision between North-Western Greece and the Apulian platform. A transform fault running in a NE-SW direction, known as the Kefallinia Fault Zone (KFZ), separates the Hellenic subduction zone in the south from continental collision in the North. (Meijninger, 2001).
Figure 2.2  Simplified tectonic map of the Eastern Mediterranean region superimposed on topography and bathymetry. Solid lines are strike-slip faults, dashed lines are possible faults, lines with triangles are thrust faults, and lines with dots are normal faults. The arrows indicate the motions along the fault lines. KFZ = Kefallinia fault zone, NAF = North Anatolian fault, Sc-P = Scoutari-Pec line, M = Macedonia (Greece), E = Epirus, T = Thessaly, c = Corfù, l = Levkas, k = Kephalonia, z = Zakinthos, i = Ioannina, th= Thessaloniki (modified after Walcott 1998, Duermeijer 1999, Kahle et al 2000, Sachpazi et al. 2000 as cited in Meijninger, 2001).

The relative movement between Africa and the Aegean is approximately normal to the Hellenic arc along the Western portion and parallel to the arc along the Eastern segment (Figure 2.3a). The rapid movements in the southeast Aegean are in response to the accelerated sinking of
the subducting plate that allows the arc to override more easily the African plate. Right lateral strike-slip deformation associated with the North Anatolian Fault (NAF) extends into the North Aegean, ending at the Gulf of Corinth or continuing to the KFZ. The orientation of the strike-slip motion along the KFZ indicates an abrupt change in the Hellenic subduction zone. The North Aegean trough and Gulf of Corinth compose the principal northern boundary of the South-Western Aegean plate. The main zone of compression observed along the Hellenic arc and along the Western coast of Northern Greece and Albania (Figure 2.3b), is associated with the subduction of the eastern Mediterranean beneath the Aegean and the continental-continental type collision between the Adriatic (Apulia) microplate and the western Greek-Albanian coasts.

**Figure 2.3** (a) Schematic of the principal motions within the active tectonics of the Aegean and surrounding area. Arrows indicate the direction of the motion relative to Eurasia. The values in mm/yr are shown beside each arrow (b) The compression and extension zones in the Aegean; The main zone of compression with thrust faults observed along the Hellenic arc and western coast of Greece is related to the subduction of the African plate beneath Eurasian plate. The back arc area is dominated by N-S extension. A narrow zone of E-W extension is observed in the forearc area. The volcanic arc associated with subduction is marked with dashed line. NAF = North Anatolian Fault; KFZ = Kefallonia Fault Zone (strike slip deformation); RTF = Rhodes Transform Fault (as cited in Geowissenschaften, 2005).
The largest portion of the Aegean arc is dominated by normal faults with an E-W trend, signifying a N-S extension. Fault plane solutions and GPS measurements indicate a narrow zone of E-W extension in the forearc, which lies between the thrust faults of the outer Hellenic arc and the N-S extension field in the backarc area (Figure 2.3b) (Geowissenschaften, 2005). Naturally, earthquakes and volcanic activities are also associated with these dynamic arcs, thrust and fault zones.

### 2.3.3 Isopic Zones and Massifs

In terms of rock mass formations (Figure 2.4), the overall geology of Greece (as part of the overall orogenesis of the Alpine region) has traditionally been described in terms of isopic zones and massifs (of the Hellenides orogenetic belt); most of which have been overthrust over one another as a result from intense folding, thrusting, faulting and uplifting during the Alpine Mountain building period.

The isopic zones of interest that form the Hellenides orogenetic belt are: the Ionian, Gavrovo, Pindos, Sub-Pelagonian, and Pelagonian geotectonic units (refer to Figure 2.4). Each unit is described in further detail in the following sections and can be co-related to the geologic formation of these units depicted in Figure 2.5. Note that the most recent event in terms of geological time scale begins at the bottom of each figure within Figure 2.5 (a), (b) and (c). The first attempt at determining the evolutionary sequences that led to the formation of the Helenides was by the French, specifically, Aubouin in 1959. (Institut de Géologie et Recherches du Sous-Sol—Athènes and Institut Français du Pétrole, 1966) (Figure 2.5).
2.3.3.1 Ionian Geotectonic Unit

The Ionian geotectonic unit comprises much of Epirus, as well as most parts of Western Greece. From the early Triassic to the end of the Lias period, this zone was characterized by shallow-water carbonate sedimentation. Later, in the mid-Jurassic (Dogger period), the basin of the future Ionian unit deepened and became a trough of deep-water (pelagic) Mesozoic limestone deposition. The Ionian zone is characterized by Upper Eocene-Oligocene Flysch (seen in Figure 2.5). Alternations of various carbonate formations (mainly limestone) with very limited occurrence of schist and local occurrences of anhydrite and gypsum are abundant within the Ionian unit. Large faults, large overthrusts and folded rocks are also present (Papanikolaou, 1986).

2.3.3.2 Gavrovo Geotectonic Unit

The Gavrovo geotectonic unit covers a narrow geographical region of Epirus and extends along Greece in a NW-SE direction. This zone was a continental fragment in its early history (Triassic) and it is characterized by thick, shallow-water limestone formations that extend into Eocene-Oligocene. The Gavrovo unit is characterized by an Eocene-Oligocene Flysch deposition which covers almost completely the underlying carbonate sedimentation (Higgins and Higgins, 1996).
Figure 2.4 Tectonic zones and massifs of the Aegean region. These zones are separated by major NW-SE striking thrusts on the Greek mainland and the Ionian Islands (Higgins and Higgins, 1996).
Figure 2.5(a) The pleogeographical evolution of the Hellenic Arc region (Modified after Papanikolaou, 1986)
Figure 2.5 (b) The pleogeographical evolution of the Hellenic Arc region (Modified after Papanikolaou, 1986)
Figure 2.5 (c) The pleogeographical evolution of the Hellenic Arc region (Modified after Papanikolaou, 1986)
2.3.3.3 Pindos Geotectonic Unit

The Pindos geotectonic unit extends along Central Greece and is one of the most prevalent geotectonic units of Greece. In the Triassic, this zone was an oceanic basin where pelagic carbonate sedimentation occurred. The Pindos unit is mainly composed of deep-oceanic-trough limestone deposition that spanned from the Triassic to the Eocene period (Figure 2.5). During this period, only the Pindos basin existed as a deep (Pelagic) Basin, with carbonate sedimentation and carbon saturation. This zone is characterized also by an Eocene Flysch formation and by intense folding, heavily sheared with numerous overthrusts. The massive degree of tectonic deformation at some locales degrades the quality of the rock mass (Christaras et al. 1997). The Pindos ophiolites nappe is of Cretaceous age. These structures exhibit much heterogeneity of weathering and occurrence of shear zones. There is also a presence of weak Flysch.

2.3.3.4 Sub-Pelagonian Geotectonic Unit

The Sub-Pelagonian zone extends in a NW-SE direction and is characterized by ophiolite formations and shallow-water carbonate sedimentation. It was initially (Triassic period) part of a continental margin between the Pelagonian zone and the Pindos Ocean. The region of the future Sub-Pelagonian zone emerged during the Late Jurassic and Early Cretaceous due to the active geotectonism on the continental plate of EuroAsia leading to the creation of a “melange” of schists-sandstones-cherts that is present in various locations along this geotectonic unit. Further deepening of the area in the Late Cretaceous resulted in reef limestone deposition. During the Oligocene and Miocene, a deep continental trough was developed and a thick clastic sedimentation occurred, namely “molasse” formation that covered the pre-existing geological formations. Flysch deposition was also included in the molassa sedimentation (Bailey et al., 1993).
2.3.3.5 Pelagonian Geotectonic Unit

The Pelagonian geotectonic unit extends in a NW-SE direction and is mainly characterized by the molassa formation (as discussed in the previous section), which is supported by Upper Cretaceous sediments. The presence of thrusted ophiolite formations is also a feature of this zone. Intense metamorphism in the region during the Jurassic period led to the formation of marbles and schists while the volcanism that occurred during the Early-Middle Triassic is depicted in meta-volcanic formations at the base of this zone (Papanikolaou, 1986).

As previously stated, the main Hellenide orogenesis of Greece occurred in the late Mesozoic Era, when numerous microplate collisions occurred in front of the northward moving African Plate (VanAndel et al., 1993). In the Jurassic Period (208 Ma), the direction of movement between Gondwana and EuroAsia changed from outwards to inwards. As such, Gondwana subducted under the Eurasia plate causing the EuroAsian plate to fold. This period also denotes the start of orogeny and the deepening of depositional basins (Figure 2.5). In the Tertiary period (144 Ma) orogenesis continues, as the convergent boundaries are still present. There is the creation of the Pre-Apalian Zone with its pelagic sedimentation. Flysch formations begin in the Eocene epoch. This seals the deposition history and is the final stage of terrestrial erosion into neritic and pelagic basins. Its final stage ended in the Oligocene, characterized by uplift and peneplanation and by extension and subsidence of many intramontane basins (Figure 2.5). The physiography of these basins acted as a funnel concentrating sedimentary deposition or erosion to these areas prior to drying up (Papnikolaou, 1986).
2.3.4 Relevance to Area of Research

The Egnatia Odos motorway passes through almost all of the geologic features of Greece by crossing all of the major geotectonic units, especially in the mountainous Ionian and Pindos regions. As a result, tunnelling works have been and will be executed in a variety of rock mass conditions ranging from very good quality rock masses such as massive limestones or marble to very poor rock masses such as clay shales and Flysch. Each geologic unit exhibits different particularities in terms of weak rock masses and the potential for unstable arrangements of rocks.

The backdrop for this research topic is the tunnelling associated with the Driskos Tunnel. This tunnel is located in the Ionian geotectonic unit in the Epirus region. Figure 2.6 depicts the current orientation of these geotectonic structures for this region. The interlayering of competent and incompetent rock units controls the geometry of folding (Mariolakos, 2004). As such, the next section will concentrate on the geology, geomorphology and tectonics of North-Western Greece.

Figure 2.6 The main geotectonic units traversed by the Egnatia Odos motorway. Overthrusts of isopic zones are evident. Driskos Tunnel is located within the Ionian geotectonic unit (modified after Papanikolaou, 1986).
2.4 Geology of North-Western Greece

2.4.1 Epirus Region

Epirus is the North-Western most province of Greece. It is bounded to the East by the Pindos mountains, to the North by Albania, to the West by the Ionian Sea and to the South by the Ambracian Gulf (Figure 2.7). A series of compressional alpine tectonic events beginning in Jurassic times and continuing through to the present provides the enveloping NW-SE structure of the Hellenides as well as the rest of the Balkan Peninsula. The Hellenides are connected to the mountain systems of the Albanides and Dinarides to the NW (i.e. within present day Albania and the former Yugoslavia (FYROM)). The Hellenides in this region are sub-divided into a number of sedimentary facies or isopic zones as described in Section 2.3.3. The Hellenides are also sub-divided into (a) the internal portion of the Hellenides to the East and (b) the external portion to the West. The external portion consists of the following zones: Vardar, Pelaginian, sub-Pelagonian, Pindos, Gavrovo, Ionian and pre-Apulian zones. Each unit contains Mesozoic carbonates at its base (except the sub-Pelagonian zone) and the general trend from east to west is the reduction in the degree of deformation, metamorphism and tectonism. These zones are separated by major NW-SE striking thrusts on the mainland and the Ionian islands as the main fault systems also follow this trend (Bensonen, 1997).
Lithologies within the internal Hellenides are predominantly gneisses with minor marbles and post-Maastrictian sedimentary and volcanic cover (i.e. Pelagonian Zone). The external, western Hellenides developed on the Apulian margin. This Apulian microcontinent consisted of a carbonate basin during the early Mesozoic period that was sub-divided by rifting in the late Jurassic into deep basins and shallow ridges forming the isopic zones present in Greece today. Carbonate accumulation is prevalent in these isopic zones (ranging from shallow ridges to deep water basins)(Meijninger, 2001).

Considerable tectonic activity began in Epirus within the Gavrovo isopic zone in the late Oligocene. In the Ionian zone, tectonic activity started in the Middle Miocene, generating a NNW to NNE trending fold and thrust belt in western Epirus, due to convergence between the Apulian plate and the Aegean plate. Consequently, vertical movements in Western Greece produced NNW-SSE and WSW-ENE directed normal faults. Since the late Pliocene, long fault bounded intra-montane basins have been formed in Eastern Epirus during general uplift and extension in three directions: NNW-SSE, WSW-ENE and WNW-ESE. The pre-Apulian zone to the west of the Ionian islands is presently actively over thrusting the Ionian basin, since the Early Pliocene.(Meijninger, 2001).

The geological structure of the Epirus region is represented by regional thrust sheets and thrust faults. The normal sequence of rocks in a thrust sheet of this is Eocene Flysch, Cretaceous limestone, Jurassic limestone and rocks of the Radiolarite series and Triassic limestone (listed from top to bottom). The thrust faults have developed commonly in the rocks of the Jurassic formations, particularly in the Radiolarite series, due to the shale-chert constituents, or in the Triassic limestone. In many occurrences, thrust sheets of limestone have overridden the Eocene Flysch of the Pindos zone (Figure 2.6). Thrust sheets are intensely folded with principal fold axes parallel to the NNW strike, and gentle secondary fold axes normal to the regional strike (Figure 2.8)(Bensonen, 1997). In a broad context, this orientation of faults can be rationalized based on the compressional environment that the Epirus region is situated in. More recent works
(Bailey et al. 1993) indicate that strike-slip motion is also a key component that must be understood in order to have a more complete understanding of recent evolution and topography. This deformation is concentrated in a series of structures that are oriented roughly 20° East as shown in Figure 2.8. The structures shown extend into Albania to the North and recent seismic activity indicates that similar parallel features are located offshore in the Ionian Sea. The features South of Arta have similar strike, however, net compression yields to net extension that is characteristic of the Aegean. This compression and uplift restricts the mouth of the Gulf of Arta and extends down the coast of Levkas where it becomes associated with the extremely active Hellenic Arc Subduction system described earlier.

The resulting folds and rock types that now exist in the Epirus region can be seen in Figure 2.9. With the alpine orogeny, limestone beds and marine deposited sediments were lifted within Epirus (and all over Greece) and folded. Therefore, approximately two thirds of the area of Epirus is covered with limestone as well as many other karst phenomena. Many heterogeneous rock masses, such as Flysch, are also abundant. Epirus’ geology is still very active as it is influenced by the NAF and KFZ as a result of the converging plate system between the European and African plate (Higgins et al., 1996).
Figure 2.8 Active structures of the Epirus Region (Bailey et al., 1993)
Figure 2.9  Simplified geological map of Epirus based on the geological maps of Greece (Bornovas 1983) and Epirus (Institut Français du Pétrole 1966). The map depicts the geology of the Ionian zone (stratigraphy, faults and fold axes) and topography of the area near the Epirus capital of Ioannina (modified as cited in Meijninger, 2001).
2.4.2 Ionian Geotectonic Unit within Epirus Region

Of particular interest to this research study is the Ionian Geotectonic Unit as it is the primary unit through which the Driskos Tunnel is situated. As witnessed on multiple site visits to this region by the author, typical structures in this region consist of large-scale overthrusts, large faults and brecciated zones of rocks that are folded. In terms of rockmasses at the surface, the prevalent structures are Flysch and alternations of various carbonate formations. The main characteristic of the unit is the presence of shallow marine biochemical sediments of limestone and dolostone nature, with deep marine biochemical sediments.

The stratigraphic column of the sequence of formation for the Ionian unit is shown in Figure 2.10. As can be seen, the Ionian Unit is built up of mainly evaporites and various limestones. Initially, the sequence is characterised by shallow-marine biochemical sediments (i.e. limestones, dolomites and evaporites). These sediments were deposited in the Triassic Period. Next, thin-bedded limestones were deposited in deep-marine environments due to the orogenic period. Lastly, a period of clastic

![Figure 2.10](image) Representative column of the Ionian Geotectonic unit (modified after Karakitsios, 1995)
Deposition of Flysch occurred between the Upper Eocene and Lower Oligocene Epochs (Figure 2.11). The Flysch formation is characterized by alternations of sandstone, siltstone and at times, conglomerate beds.

Figure 2.11. Depiction of the palaeotectonic positions of Flysch deposits in the External Hellenides and the assumed source terrains in (a) the Late Cretaceous, (b) the Palaeocene-Eocene and (c) the Oligocene. The Parnassos-Ghiona zone is interpreted as continental block separated from the passive Apulian margin. The Pindos Ocean with the Jurassic subduction zone was situated to the east of this zone (modified after Faupl et al. 1998).
2.4.3 Flysch Formation

The Western Hellenic Flysch formation created an externally asymmetric basin that was supplied by a major internal (isolated) source as indicated by palaeocurrent data and general facies trends provided by Richter (1976), Piper (1978), and Alexander (1990)(Faupl et al., 1998)(Figure 2.11).

This cordillera also partially fed by the Mesohellenic Trough, a “piggy-back” basin along its Eastern (internal) flank with terrigeneous material. From a plate tectonic viewpoint, the development of the Pindos Cordillera is believed to be comparable with the formation of an accretionary wedge of stacked tectonic units linked with the final closing stage of the Tethys ocean. In this way, the Pindos cordillera acted as a source terrain from Late Eocene onward, when turbiditic sedimentation began in the transitional beds of the Western Hellenic Flysch.

Within this tectonic context, the plateau-ridge system is associated with a series of fault bounded limestone formations separated by sediment-filled marine basins (Figure 2.12). The basins are uplifted by continual tectonic movements to form Flysch and Flysch-like rocks alongside the limestone formations. These Flysch formations are younger than the limestone and are made from coarser, shallower sediments that create softer, sandstone rocks with varying proportions of silt, clay and other siliceous material. These mechanisms can be seen in Figure 2.12. Continual sedimentation of this nature combined with the tectonic events that continually altered the depositional environment defined the stratigraphic range of Flysch deposits in this region.
Figure 2.12 The formation of Flysch structures; in deep seas adjacent to rapid orogenesis (modified after Bailey et al. 1993 and Mountrakis, 2000).

The stratigraphic column and range of Flysch deposits of the External Hellenides is shown in Figure 2.13. The Flysch sediments of the Ionian and Gavrovo zone together form the Western Hellenic Flysch. These deposits range stratigraphically from Upper Eocene to Lower Miocene (Faupl et al, 1998). The pre-Flysch sediments of the Ionian zone are represented by basinal facies, while a shallow-marine carbonate platform existed in the adjacent Gavrovo zone. The Pindos zone represents the western margin of the basin. The terminal Flysch deposition
commenced in the Palaeocene and lasted until the Oligocene. The first turbidic intervals occurred in the Mid-Cretaceous. Flysch deposition started in the Eocene.

Figure 2.13  Stratigraphic range of the Flysch deposits in the External Hellenides of mainland Greece (modified after Faupl et al. 1997).

In terms of geology, the formation that prevails in the area of the Driskos Tunnel is the Ionian Unit’s Flysch. As seen in Figure 2.12, Flysch is formed as a series of rocks are deposited in a deep sea (pelagic) environment adjacent to a rapidly rising mountain chain. The formation is dominated by sandstone, siltstone and clay. It is classified as a clastic sedimentary rock. Observations by the author in Epirus demonstrate that in some regions there is also an increase of sandstone layers (in frequency and in thickness). The siltstones are of good quality with a subgranular structure with no clayey appearance and they are not sheared. The rock mass is not very fractured. A limited number of geological thrusts and shears were observed at outcrops but an un-sheared rock mass predominates.
The main locations of limestone and Flysch formations within the Epirus region are shown in **Figure 2.14**. As can be seen, the limestone regions are separated from regions of Flysch by narrow limestone ridges that are the topographic expression of the most active fault zones. This is in line with the plateau-ridge folding and faulting interpretation introduced earlier in this chapter. As such, different areas of the topography have unique susceptibilities to tectonically induced surface disturbances; the limestone plateau regions are relatively undisturbed while the Flysch zones are tectonically extremely active.

**Figure 2.14** Schematic geology of the Epirus Region depicting the location of Flysch and Flysch-like deposits (after Bailey et al. 1993).
Tunnelling within Flysch poses a variety of engineering challenges. The Driskos Tunnel was excavated in a varying Flysch formation. Flysch consists of alternations of clastic sediments that are associated with orogenesis. It is characterized by repeating alterations of sandstone and fine grained (siltstones, silty shales and clayey shales) layers (Figure 2.15). The sandstone may contain conglomerate beds. The overall thickness of the Flysch unit is very large. The rock mass characteristics are determined by frequent bedding and shear discontinuities (often resulting in soil-like material), tectonic fatigue, low permeability (presence of clay minerals) and is affected by reverse (and consequent normal) faults and thrusts. These factors combine to degrade the geotechnical quality of the heterogeneous Flysch rock mass. Therefore, large (hundred to a few thousand meters) sheared or even chaotic rock masses can be found throughout this area and within all of Northern Greece.

Figure 2.15 Flysch consisting of alternating Sandstones (thick layers) and Siltstones (thin layers)
The following is a list of geotechnical characteristics associated with Flysch and Flysch rock masses and formations (Marinos, 2001):

a. Presence of clay minerals which also leads to low permeability,
b. Heterogeneity which comes from the alternations of competent and incompetent beds, and,
c. Tectonic fatigue and sheared discontinuities.

2.4.4 Epirus Regional Topography and Seismicity

As can be expected with its geotectonic setting, the topography of Epirus is mountainous and rugged. Epirus is the most mountainous region in Greece, with very little flat land, most of it being at the mouths of the rivers. A series of jagged ridges, with elevations of greater than 1000 m elevation, separate plateau regions that are deeply divided by river systems (Figure 2.16). The high, relatively inaccessible Pindos mountain chain (reaching elevations of 2600 m) located to the North-East bounds Epirus to the East, while elevated Albanian mountains bound the region to the North. Most of Epirus lies windward of these Pindos mountains. Coastal and plain regions are located to the East and to the South of the Epirus region respectively. The largest plains are those of Arta and Ioannina. Of the total area of Epirus only 10% is lowland, 13% is semi-mountainous and 77% is mountainous. Centrally, there are Flysch formations and mountain ridges that separate the coastal region with the inland region.
Figure 2.16 Topography of Epirus Region – Satellite Photo (modified after NASA, 2006).

Figure 2.17 Topography and Relief of Epirus Region (Google Earth, 2007).
Figures 2.17 and 2.18 depict the topography of the Epirus region with emphasis on the elevation of the area. Note that Figure 2.17 has a 3:1 vertical to horizontal ratio in order to exaggerate the vertical mountainous features within Epirus.

As seen in Figure 2.19, Northwest Greece is a seismically active portion of Eurasia. This highlights the underlying effects of tectonically active structures within the region and the dynamic nature of the present compressional tectonic forces (studies of earthquake mechanisms and geology in Epirus coincide with this notion). Epirus is subject to earthquake events at a rate of activity comparable to that of Japan. Uplift rates are between meters and tens of meters per millennium (Bailey et al., 1993).
2.4.5 Rock Masses within Epirus Region Traversed by Egnatia Odos

Most of the tunnelling works for the Egnatia Odos motorway have been and are being conducted in Epirus. The geology of this research area (Figure 2.8) shares the same geologic and tectonic history as described above. A summary of the geological conditions along the motorway route is shown in Table 2.1 and correlate to the units depicted in Figure 2.4.
<table>
<thead>
<tr>
<th>Geological Unit</th>
<th>Extent along Egnatia Motorway Route</th>
<th>Rock Types</th>
<th>Tunnel Relevant Engineering Geological Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ionian Unit (Driskos Tunnel)</td>
<td>Igoumenitsa to Arachtos River</td>
<td>Carbonate rocks with Flysch, some shales and diapiric evaporites. Faulted and folded</td>
<td>Fractured rock, infilled karstic features in carbonate rocks create face and crown instabilities. Landslide areas</td>
</tr>
<tr>
<td>Pindos Unit (Pindos Mountains)</td>
<td>Arachtos River to Metsovo</td>
<td>Sandstone, siltstone and Flysch with some schist and carbonate rocks. All intensely folded, thrusted and faulted</td>
<td>Fractured rock in fault/thrust zones. Weaker argillaceous Flysch. Slope instability at portals. Rapid change in rock quality. Mixed face conditions. Landslide areas</td>
</tr>
<tr>
<td>Ophiolite Rocks</td>
<td>Metsovo to Grevena</td>
<td>Faulted ultra-basic and basic igneous rocks comprising peridotites, basalts, gabbro, dolerite, amphibolite, pyroxenite. Very heterogeneous wrt weathering and shear strength. Some weak Flysch horizons.</td>
<td>Rapid change in rock type and quality</td>
</tr>
<tr>
<td>Molassic Formations</td>
<td>Grevena to Kalamia</td>
<td>Molasse: conglomerate, marl, siltstone and sandstone</td>
<td>Less disturbed by tectonic movements. Mixed face conditions. Local groundwater ingress.</td>
</tr>
<tr>
<td>Pelagonian Unit</td>
<td>Kalamia to Veria</td>
<td>Marble, limestone, gneiss and granite with localized faulting and overthrusting. Sheared phyllite, limestone and ophiolite.</td>
<td>Good quality rock in some areas. Shear zones – fragmented rock – poor arching capability creating crown instabilities, poor founding strata and local water ingress. Slope instability at portals and landslide areas.</td>
</tr>
</tbody>
</table>
2.4.6 Weak Rock Masses

The dominant weak rock masses along the Egnatia Motorway, often in chaotic and heterogeneous forms are (examples shown in Figure 2.20):

a. Various forms of Flysch (depending on the proportion of weak siltstone-clayey members and the tectonic deformation);

b. Ophiolites (depending on the weathering and tectonic deformation) These rocks comprise basaltic pillow-lavas, dolerite dykes, gabbros and peridotites with a varying amount of alteration and weathering;

c. Phyllitic schists (depending on tectonic pre-shearing and weathering);

d. Sheared zones in hard rocks; and

e. Weathered zones in gneisses, granites and schists.

The engineering properties of these weak rock masses, primarily that of Flysch, will be investigated in the Rock Mechanics and Modelling Chapters following this chapter. It is the unique nature of these rock masses that pose such a challenge to tunnelling construction within this region. The primary focus and backdrop for the tunnelling work that has been investigated as part of this study is the excavation associated with the construction of the Driskos twin tunnels. The next section will concentrate on the geology associated with the Driskos site.
Figure 2.20 Typical geology of Epirus Region. (a) Lightly Metamorphosed limestone within the Ionian Zone. (b) Ophiolite nappe overthrust on Pindos Unit - bench excavation; (c) Transition from carbonate sedimentation to Flysch: Gavrovo Unit. Flysch of thick bedded sandstones and thin-bedded siltstones; angular folds and fracturing (faults) along the axial planes; (d) Upper Cretaceous clastic sediments overlaying older carbonate formations – Pelagonian Unit. Outcrop of very brecciated Limestone (disintegrated, ravelling conditions); (e) Intense Tectonism within light coloured deep water limestones of Pindos Unit; (f) Molasse formation – Sub Pelagonian Unit; (g) Intense tectonism in shallow water limestone formations within Gavrovo Unit; (h) Intense tectonism during Jurassic period resulting in metamorphosis of limestones into marbles – Pelagonian Unit, and (i) Flysch consisting of alternating Sandstones (thick layers) and Siltstones (thin layers) within Sub Pelagonian Unit.
2.5 Driskos Tunnel Region

2.5.1 Location of Driskos Twin Tunnels

The Driskos twin tunnelling site is located to the North-East of Ioannina City within Epirus. It is situated along the Alps Orogenic Zone between the Ionic and Pindos Massifs (as described as part of Figure 2.4 presented previously in this chapter). It constitutes the mountain of lowest elevation within the Mitsikeli mountain range. Figure 2.21 shows the location of the Driskos Tunnel using various reference materials (Egnatia Odos, 2001, Michelin, 2004 and Google Earth 2007). The alignment follows underground the general direction of the current National Highway route through the region. The geology of the Driskos region was determined by a thorough desk study of the region and multiple site visits to the region by the author.

Figure 2.21 Location of the Driskos Twin Tunnels within Epirus, East of the city of Ioannina (modified after Egnatia Odos, Michelin Road Map)
2.5.2 Geology of Driskos Region – Desk Study

In terms of the tunnelling project for the region of Driskos, geology has been established to be the most important factor governing the feasibility of a tunnel project. As with any geological engineering project, the first task was to amass as much information on the region of interest in the form of a desk study prior to deployment to the field. This serves many purposes. Not only does one gain a thorough understanding of the geology and terrain to be encountered, but it also allows one to prepare themselves properly (i.e. planning of their site visit) for the field in order to fully exploit the valuable and costly time on the ground.

Limited resources were available for the Driskos region of interest. The geology of the region was initially investigated by Egnatia Odos (Geological Study R2A001, 1998) in order to determine the specific geology and feasibility of tunnelling within Mt. Driskos. Other major sources of information concerning the geology of the Driskos area come from the Greek National Institute of Geology (IGME), Supplementary studies by Subcontractors of Egnatia Odos, Geodata and Odotechniki Ltd (2003), as well as a Master’s thesis by Vassilis Marinos (2001).

The geological structure of the Driskos tunnel region is shown in Figure 2.22. The official Greek geological map (scale 1:50 000) has been obtained from IGME. Unfortunately, no other maps of the region were available with a smaller scale factor. As can be seen, the tunnel was to be excavated through Flysch formations (Upper Eocene) and was not expected to meet any limestone formations. It is the variations within the Flysch formations that will be investigated in order to determine the specific engineering properties and ensuing behaviour of these geomaterials.

Superimposed upon Figure 2.22 are the orientation, length and sense of the Driskos Twin Tunnels. The chainage values correspond to traverse measurements conducted by Egnatia Odos and will be discussed later within this section. The tunnel itself is approximately 4570 m
in length with an overburden (at most) of 225 m. The tunnel entrance is located at the South Western portion of the noted tunnel outline while the tunnel exit portions are located to the
North East. Given this orientation, the tunnelling sequence begins within Flysch alternations of silty sandstones and siltstones that are proceeded by a transitional zone of a Flysch sequence of sandstones and siltstones. Finally, Flysch alternations of great thickness are located at the exit of the tunnels.

The main lithological formations that were encountered during the excavation of the Driskos Tunnels are as follows (Egantia Odos, 2003):

a. Siltstones with thinly bedded Sandstones (< 10 cm) (Si)
b. Thin to medium bedded alternations of Siltstones and Sandstones (SiSa)
c. Medium to thick-bedded Sandstones with interbeded Siltstones (SaSi)
d. Thick-bedded Sandstones with alternations of thin bedded Siltstones (Sa)
e. Conglomerates (Fc/SiSa)

The tunnel alignment can also be seen in a 3D, Google Earth projection in Figure 2.23. The Driskos Tunnel is situated in a series of varying lithological features of the Ionian tectonic unit adjacent to the Pindos isopic unit. The material is less tectonically disturbed than the Pindos Flysch and therefore, there is an absence of extensive chaotic zones within the Ionian Flysch.
Figure 2.23 Driskos Tunnel alignment with respect to topography and isopic units.
2.5.3 Geology of Driskos Region – Field Studies

Multiple site visits were conducted by the author as part of the overall field study for the region (accompanied at times by Dr. Diederich and Dr. Marinos). The visits were conducted in September 2003, December 2003, and October 2007. Site visits and geological mapping of the region by the author enhanced the overall geological understanding of the area; it also allowed the author to develop the framework of geology as presented within this section.

Walking the ground, one gains a sense of the complexities associated with tunnelling through such rockmasses. Outcrops reveal alternating layers of siltstones and sandstones of various thicknesses. Figure 2.24 shows by percentage the various rock formations that are present along the tunnel alignment. As can be seen, the largest percentage of rock mass material consists of thin to medium bedded alternations of Siltstones and Sandstones (SiSa) as well as medium to thick-bedded Sandstones with interbeded Siltstones (SaSi).

![Figure 2.24](image)

**Figure 2.24** Rock masses (by percentage) that the Driskos Tunnel encounters

With respect to the tunnel alignment, the Flysch formation of altering Siltstone and Sandstone formations are ideally represented in Figure 2.25. The figure was drawn based on fieldwork; information obtained though the analysis of outcrops as well as an evaluation of the tunnel faces during the excavation stages. Note that the left hand side of the figure denotes the SW tunnel portal entrance and that the right hand side denotes the NE portal entrance.
Figure 2.25. Idealized cross-section of Driskos Tunnel alignment depicting the rock formations that have been traversed.

As can be seen, travelling SW to NE, the tunnel enters a Flysch rockmass that has bedded alternations of sandstones and siltstones whereby the siltstones are the predominant formation. Next, one encounters a vertically oriented rockmass structure of bedded alternations of sandstones and siltstones where the sandstones are more dominant in size and structure. Within the centre region of the tunnel alignment, one can see outcrops of very thin bedded sandstones and thinly bedded siltstone layers. There is also evidence of tectonism in this region as there is evidence of faulting and folding of this rockmass. The rock masses that are evident closest to the NE portal entrances are characterized by thick bedded sandstones with rare siltstone layering as well as conglomerates of silty-sand matrix with alterations of siltstones and sandstones. The overall orientation of the rockmasses suggests that the rockmasses form a syncline while also deteriorating in structure due to tectonism.

The rockmasses that are evident within the Driskos Tunnel alignment will be discussed in the following sub-sections.
2.5.3.1 Siltstones (Si)

The siltstones in this region (Figure 2.26) are extremely weak structures as they have been tectonically disturbed. They are mainly located in the central part of the tunnel and compose 7% of the overall rock mass that was encountered. The silt percentage in these formations exceeds 80% and the sandstone material is limited to thin, sporadic layers. Internal thrust and differential movements are ever present in the Flysch formation, downgrading the strength of the siltstones by a significant amount. This is particularly evident for the central part of the tunnel excavation where siltstones are prevalent.

2.5.3.2 Thin bedded alternations of Siltstones and Sandstones (SiSa)

This rock mass structure is denoted by thin-bedded to medium-bedded alternations of Siltstones and Sandstones (Figure 2.27). The percentage of Siltstone material varies in the range from 50 to 80%. The surfaces along the bedding planes are potential sheared zones. Stability problems have been observed in areas with similar characteristics. As such, an area of concern is the central portion of the Driskos alignment (in particular, chainage 8+400 to 8+700) in order to deal with the inherent weakness(es) of this material. Many folds are evident in this structure at various geological scales. These rock masses are primarily located to the southern part of the tunnel zone and consist of approximately 44% of the entire tunnel length.
2.5.3.3 Medium to thick-bedded Sandstones and thin-bedded Siltstones (SaSi)

This formation is denoted by medium to thick-bedded Sandstones and thin-bedded Siltstones (Figure 2.28) with a percentage of sandstone material ranging from 50 to 80%. The bedding planes are well developed and well defined. The formation is brittle in nature. As a result, angular fractures and faults can be seen along axial planes. This formation is primarily located at the mid-span of the tunnel zone. Smaller outcrops are also visible in the Northern portion closer to the tunnel portal exit. This rockmass consists of 42% of the entire formation that span the Driskos twin tunnels.
2.5.3.4 Thick bedded Sandstones with alternations of thin bedded Siltstones (Sa)

This rock mass is denoted by its dark gray, silty Sandstones (over 80% by volume) and rare, thinly interbedded Siltstones (Figure 2.29). The Sandstones are weathered and are not too tectonically disturbed. The fractures are mainly open and allow transport of water with ease. The contact between the Sandstones and the Siltstones often consists of a spring line. This formation does not have great thickness and is located in limited areas within the Northern half of the Driskos zone. One can also see the alternation between a thick sandstone layer and smaller alternating sandstone and siltstone layers. This rock type comprises only 3% of the overall formations of the tunnelling area.
2.5.3.5 Conglomerates (Fc/SiSa)

The strength and nature of the conglomerates within the region (Figure 2.30) vary from low to high strength (i.e. there is great variability amongst their formation and thus, their strength). The matrix of the conglomerates is composed of a silty clayey material to a silty sandy to a sandy calcitic material. The formation is a lithos without bedding. Light tectonism is also evident in these structures. The only area where this formation was evident was at the northern part of the tunnel alignment and it consists of only 4% of the rocks within the tunnel zone.

Figure 2.30 Conglomerate formation
2.5.4 Geological Cross Section of Driskos Tunnel

Based on a geological evaluation of the study area, a more detailed cross-section of the Driskos Tunnel was produced. As previously mentioned, the cross-section was produced (Figure 2.31) based on a desk study, mapping within the region, statistical analysis of the geometric data of the discontinuities, borehole interpretation, interpolation from tunnelling logs and the author’s interpretation of all relevant data.

The cross-section produced spans the 4.5 km length of the carriageway tunnel and shows the overburden (at most 220 m) above the tunnel alignment. As can be seen in Figure 2.31, the main Ionian Unit components that are traversed are those that have been identified previously in Section 2.4.3. Also inherent in this cross-section are the geological and tectonic structures of the existing rock masses. Many bedding planes, joints, fractures, faults and folds are evident in the overall structure of the rock investigated at this geological scale. The following sub-sections will comment on these structures that were observed and noted.

2.5.4.1 Bedding Planes

The bedding planes that have been measured have a wide range of dip directions due to their structural setting and the ensuing folding of the formation. In general, there are four major orientations of bedding planes for the Driskos cross section. The first two series of bedding planes are oriented in the SW or NE direction (in cross-sectional view) and dip between 20°-40°. The third series of bedding planes are oriented SSE (167°) at a shallow dip angle of 20°. The last series of bedding planes is oriented E to ENE (94°) and dip at an angle of 45° to 20°. For the first three categories, the angle of attitude of the bedding planes and the alignment of the tunnel is 56°-62° (in 85% of the cases). For the last series of bedding planes, the attitude of these planes and the alignment of the tunnel have an angle of merely 09°.
Figure 2.31 Geologic Cross Section Interpretation of Driskos Tunnels (modified after Egnatia Odos, 1998)
The bedding planes make up the main surface of discontinuity of the rock formation along the entire length of the tunnel. They are well developed in most of the structures encountered with the exception of the siltstone and conglomerate rock masses. These bedding planes may constitute planes of weakness and often form shear surfaces especially on the thin beds of siltstone. It is then paramount to investigate the tunnel alignment and the attitude, dip angle and orientation of the bedding plane formations in a geotechnical sense in order to predict possible areas of concern or problematic sections of tunnelling construction.

These bedding plane orientations are consistent with the borehole investigation results that are summarized in Egnatia Odos’ Geological Study of the Driskos Tunnelling site (Egnatia Odos, 1998). In total, 15 deep borehole samples were taken, recovered and used to determine the orientation of the geologic bedding planes as well as the strength of the rock masses that were present.

2.5.4.2 Joints

Joints and fractures are prevalent along the entire length of the Driskos Tunnel. Within the thick-bedded sandstones and conglomerate formations, joints exhibit significant extension and length. These joint systems are fully developed in only the sandstone beds as well as the conglomerate formations. In general, the joints are open and their surface conditions vary from smooth to rough. As also denoted by Egnatia Odos (Egnatia Odos, 1998), there are four main systems of joints within the Driskos tunnelling region.

a. Joint System #1 has a NW (325°) or SE (131°) strike direction and dip angle 75° to 78°. This system of joints accounts for 25% of the joints present.

b. Joint System #2 has a N (001°) or SSE (166°) strike direction and dip angle 72° to 78°. This system of joints accounts for 24% of the joints present.

c. Joint System #3 has a NE (48°) or SW (224°) strike direction with dip angle 74° to 75°. This system of joints accounts for 16% of the joints present.
d. Joint System #4 has an E (89°) or W (270°) strike direction with dip angle 75° to 77°. This system of joints accounts for 22% of the joints present. These systems of joints constitute another surface of discontinuity (the bedding planes being another) of the rock mass. These joint systems have been plotted on stereographic projections and are included in the geologic study of the Driskos Tunnels (Egnatia Odos, 1998).

The angle between the orientation of the tunnel and the attitude of the joints is 38°, 76°, 57°, and 16° for joint systems 1 to 4 respectively.

2.5.4.3 Faults

Major fault systems were discovered in and around the Driskos Tunnel site. This was to be expected, given the tectonic setting of the area. Not all of the fault systems that were noted in the area were expected to intersect with the tunnel alignment. Seven major fault systems were revealed through geologic mapping of the area. Faults F3, F4, F5 and F7 are denoted in Figure 2.31. Faults F1 and F2 are located to the SW of the tunnel entrance within the Eocene limestones many kilometres from the tunnel site. The overburden cover of the Flysch did not allow an accurate assessment of these faults and of their possible extension into the tunnel zone. Faults F3 and F4 are large enough and were expected to extend to the zone of tunnelling. These SW oriented faults are predicted to meet the tunnel within Sections 6+940 and 7+030 respectively. Other fault systems were discovered by the tunnel.

Figure 2.31 Fault Feature F3, closest to SW Portal Entrance.
excavation phase of the project. Fault F5 occurs near the centre of the tunnel alignment (9+040) and also has a SW orientation. Fault F6 is located at the NE portion of the Driskos site and dips significantly to the East. As such, it did not meet the tunnel alignment. Fault F7 is located at the northern portion of the Driskos site and dips to the SE. In Figure 2.31, it was estimated to intersect section 10+400. Limited fault features are shown in Figure 2.32. In terms of overall strength of the Flysch formation, this is reduced locally by the presence of a higher than normal concentration of sub-horizontal bedding crossed by frequent faults (Hoek and Marinos, 1998).

2.5.4.4 Folding

Folding of rock masses can be seen at various geological scales within this cross section. Within the SW portion of the tunnel (at chainage 7+100), it can be seen that the limestone base material is arranged as a wide anticline. The Flysch material above this base at chainage 7+450 is also an anticline. All of the southern features suggest a general trend of an anticline given the very shallow angles of dip (5°-20°) to the S-SW and the low to medium degree of localized tectonism in the area. From chainage 8+450 to 9+100, there is a high degree of tectonism (i.e. tectonic distress) as the formations within these locations demonstrate very intense folding, very sheared rock masses without structure. Especially within zones 9+100 to 9+500, the formations are upturned, recumbent and follow a “zigzag” pattern. At the northern end of the tunnel, the beddings are upturned and slightly folded without any angular folds. The degree of shortening to which a rockmass has been subjected can be directly correlated to the amount of folding that is present in a region.

Figure 2.33 depicts various geological points of interest as amassed through multiple site visits. Outcrops provided a good geological representation of the rock masses evident in the region and gave an indication of the geological challenges associated with tunnelling through such rock masses. Viewing Figure 2.33, one gains a sense of the terrain and rock masses that are evident at various locations along the alignment. Also cited in Figure 2.33 are
Figure 2.33. Geology within Driskos Tunnel Region as determined by desk study and multiple site visits.
elevations and Global Positioning System (GPS) co-ordinates of the locations where photos were taken. Location and elevation data were obtained using a Garmin 12XL GPS system. An estimated positional error (EPE) is displayed on the GPS unit based on the satellite geometry at the time. At best, the error is +/- 3 m, but is typically between 4-6 m. The data was then plotted on Garmin Map Source Trip & Waypoint Manager V4 (2006) software and superimposed on a map of Google Earth. Embedded within Figure 2.33 are previously seen Figures 2.22 and 2.25. These figures have been cross-referenced with geological features on the ground.

2.5.4.5 Three Dimensional (3D) Geologic Model of Driskos Twin Tunnels

Marinos V., Fortsakis P. & Prountzopoulos G. (2006) also constructed a geological model (Figure 2.34) of the area within the Driskos Tunnel construction. It is this geological model that is a key component on which the design of a tunnel is based as it allows for the identification and an overall assessment of the potential hazards within a proposed tunnel excavation site.

![3D Geological model of a section of the tunnel for Driskos Tunnels](image)

**Figure 2.34** 3D Geological model of a section of the tunnel for Driskos Tunnels (Marinos V., et. al. 2006).
The 3D conceptual geological model coincides with the geological study that was undertaken by the author and Egnatia Odos (Figure 2.20). As seen in Figure 2.34, the Driskos twin tunnels are located in an area of major tectonic thrusts whereby the tunnel alignment crosses the major tectonic features perpendicularly. The Flysch formations are disturbed in an exceedingly wide zone often extended through satellite shears zones. As mentioned previously, the twin tunnel system tunnel was driven through the Ionian Flysch formation. The formation comprises alternations of sandstones and siltstones with a predominance of either sandstone or siltstone. The tunnel has a perpendicular alignment (N10°) to the axis of large synclines and anticlines (N80°-N260°). Excessive folding is evident in the weak rock mass of Flysch that is composed of siltstones with thin layers of sandstone. Folding and faulting often lead to a clearly visible chaotic structure of isolated blocks of hard rock embedded within a soft clayey-silty matrix. The tunnels vertically crossed the well-defined shear zones that are often oriented parallel to foliation planes that constitute the widespread structural feature of the Flysch rock mass. These weak zones were accurately cited and predicted at the design stage (Marinos V. et al., 2006).

2.5.5 Idealized Cross Section and Sub-Sections of Driskos Tunnel

In order to more effectively study the various formations and rockmasses that were encountered by the Driskos Tunnel alignment, an idealized Driskos Cross-Section was fabricated by the author and can be seen in Figure 2.35. In all, the Driskos cross-section was sub divided into 14 distinct sub-sections. The division of these sections was primarily due to the variations in the rockmass material or any geologic discontinuities (upright bedding planes, faults, major joints, degree of tectonism etc.) that differed in adjacent sections along the Driskos cross section. The strength values that are part of this figure will be elaborated upon within the engineering properties portion of this document. As well, clearly labeled on this diagram are the chainage values that have been referenced in the previous sections. These values of
Driskos Tunnel – Egnatia Odos

(Egnatia Odos, Modified by Vlachopoulos, 2005)

Figure 2.35 Idealized Cross Section of Driskos Tunnel
chainage correspond with the original geodetic investigation by Egnatia Odos (1998) and have been used here for ease of reference and cross-referencing purposes. The Driskos Tunnel alignment begins at a chainage of 6+124 (i.e. at kilometer 6 within this section of Egnatia Odos construction works; at 124 m in, the SW tunnel portal begins) and makes its way to 10+727, the mark for the NW tunnel portal entrance. Table 2.2 below summarizes the 14 distinct subsections of the Driskos Tunnel. The labeling of the sections commences from Section 1 to the SW through to Section 14 to the NE. Section 4 was subdivided into sub-subsections due to the complexity of this unit and the abundance of monitoring data in this weaker rock mass. This will be elaborated upon in Chapter 3 as there are consequences to the geologic and ensuing engineering / mechanical characterization of this material. This geologic section (4) of the Driskos Cross section was deemed the most problematic during tunnel construction. The chainages correspond to those within Figure 2.35.

**Table 2.2** Driskos Geological Sections and Sub-Sections

<table>
<thead>
<tr>
<th>Section Number</th>
<th>Start Point Chainage</th>
<th>End Point Chainage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6+124</td>
<td>7+625</td>
</tr>
<tr>
<td>2</td>
<td>7+625</td>
<td>7+840</td>
</tr>
<tr>
<td>3</td>
<td>7+840</td>
<td>8+385</td>
</tr>
<tr>
<td>4</td>
<td>8+385</td>
<td>9+035</td>
</tr>
<tr>
<td>4.1</td>
<td>8+385</td>
<td>8+500</td>
</tr>
<tr>
<td>4.2</td>
<td>8+500</td>
<td>8+650</td>
</tr>
<tr>
<td>4.3</td>
<td>8+650</td>
<td>8+750</td>
</tr>
<tr>
<td>4.4</td>
<td>8+750</td>
<td>9+000</td>
</tr>
<tr>
<td>4.5</td>
<td>9+000</td>
<td>9+035</td>
</tr>
<tr>
<td>5</td>
<td>9+035</td>
<td>9+085</td>
</tr>
<tr>
<td>6</td>
<td>9+085</td>
<td>9+425</td>
</tr>
<tr>
<td>7</td>
<td>9+425</td>
<td>9+580</td>
</tr>
<tr>
<td>8</td>
<td>9+590</td>
<td>9+825</td>
</tr>
<tr>
<td>9</td>
<td>9+825</td>
<td>9+975</td>
</tr>
<tr>
<td>Section Number</td>
<td>Start Point Chainage</td>
<td>End Point Chainage</td>
</tr>
<tr>
<td>----------------</td>
<td>----------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>10</td>
<td>9+975</td>
<td>10+105</td>
</tr>
<tr>
<td>11</td>
<td>10+105</td>
<td>10+260</td>
</tr>
<tr>
<td>12</td>
<td>10+260</td>
<td>10+420</td>
</tr>
<tr>
<td>13</td>
<td>10+420</td>
<td>10+610</td>
</tr>
<tr>
<td>14</td>
<td>10+610</td>
<td>10+727</td>
</tr>
</tbody>
</table>

More photos and geologic and material property information concerning each of the subsections can be seen in **Appendix B and Appendix C**. Most of this information was gathered during the construction of the Driskos tunnels and includes photos and sketches of tunnel face conditions and information amassed and interpreted by the author from borehole analysis and tunnel boring logs obtained from Egnatia Odos.

### 2.6 Relevance to Research Project

An accurate assessment of the geology of the area is paramount in order to mitigate the risk of encountering unexpected geological conditions. A thorough and comprehensive site investigation will accurately identify hazards associated with the rock masses and other geological structures within the area of study. Failure to anticipate site ground conditions generally results from an inadequate geological understanding (Fookes et al., 1997). An attempt was made, then to provide the reader with an in depth account of the geologic setting for the research area, in order to accurately anticipate, observe and understand the site conditions.

The geologic/geotectonic settings that have been identified and discussed for the Driskos site in this chapter result in a variety of difficult engineering conditions associated with weak rock masses that require the use of sophisticated tunnel design methods. The design of underground
excavations in these weak materials requires knowledge of the mechanical properties of the rock masses in which these excavations are carried out. The next Chapter will deal with estimating the mechanical properties of such weak heterogeneous rock masses (Flysch) that are common throughout Northwestern Greece.
Chapter 3
Weak Rock Mass Properties and Mechanics

3.1 Rock Mass Classification Systems

In order to begin the process of dealing with a rock mass as an engineering material, one must analyze and categorize the rock mass based on its composition, material properties or both. A classification scheme can be used in this regard. A classification system allows for a systematic analysis and characterization of a rock mass. It can be used as a checklist in order to ensure that all relevant information concerning the rock formation has been included and considered. Once a rock mass has been classified (and if enough relevant data is available within the database associated with the specific rock) other material properties such as strength, deformation and ultimately support requirements can be inferred by the classification. Note that many formations are site-specific and have unique characteristics and properties.

There are many rock mass classification systems that are used within geomechanics. They stem from empirical techniques to multi-parameter classification systems. The most widely used rock mass classification systems are:

a. *Terzaghi (1946).* This is a descriptive classification system that is used to estimate tunnel support. The governing factor in rock mass behaviour is based on gravity. The method takes into consideration such factors as: intact, stratified, jointed, blocky, crushed, squeezing and swelling rock.

b. *RQD – Rock Quality Designation Index* (Deere et Al., 1967). This classification system is based on quantitative results obtained from drill cores. It is defined by the percentage of intact core segments that are longer than 4 inches (100 mm) within the entire length of the core obtained.
c. **RSR – Rock Structure Rating** (Wickham et al., 1972). Based on qualitative results from the excavation of small tunnels. This system takes into consideration rock type origin, hardness and geological structure combined with joint condition spacing and orientation. It also considers direction of tunnel drive (with or against joint dip) and amount of water inflow. Some credit RSR as the basis of the next two classification systems (Hoek, 2004).

d. **RMR – Rock Mass Rating** (Bieniawski, 1976). The RMR system is also named the geomechanics Classification system. It takes into consideration the following rock mass parameters: uniaxial compressive strength (UCS), RQD, spacing, condition and orientation of discontinuities and groundwater conditions. The RMR value corresponds to a set of guidelines associated with tunnel support.

e. **Q – Rock Tunnelling Index** (Barton, 1974). Is a classification system that was developed by the Norwegian Geotechnical Institute used for the determination of rock mass qualities with a view to establishing associated tunnel support requirements. The parameters that are used for the purpose of this system are: RQD, joint set number, joint roughness number, joint alteration number, joint water reduction factor and a stress reduction factor.

f. **GSI – Geological Strength Index** (Hoek, 1995; Hoek, Kaiser and Bawden, 1995). This classification system is based on an estimation of the rock mass strength in varying geological conditions. It is based on the rock mass properties proposed by Hoek and Brown (1997). The main criteria associated with the GSI classification system is a detailed engineering geology description of the rock mass and is qualitative in nature. This grew out of the notion that numbers on joints were largely meaningless for the weak and complex rock masses (Marinos et al., 2006). It should be noted that the GSI classification system was used for this research investigation as intact core samples could not be obtained from the weak rock masses that are part of the Driskos study area. This method combined with the Hoek-Brown criteria are elaborated upon within the next section this chapter.

For the most part, the above-mentioned classification systems take into consideration geological, geometric and design/engineering parameters in order to determine the rock mass quality. One must ensure that they have a fundamental understanding of each system prior to its
use and one must also know the limitations associated with these systems based on their assumptions and unique development.

3.2 Estimation of Rock Mass Properties

In tunnelling through weak heterogeneous rock masses such as those found in Greece, it is important to obtain reliable strength estimates of these materials in order to predict potential tunnelling problems as early as possible in the design process. These parameters must be incorporated into an overall rock mass criteria framework (i.e. Hoek-Brown rock mass characterization tool) which are also part of a well-defined rock mass characterization system (i.e GSI for weak heterogeneous rock masses).

Currently, the most widely used criteria for estimating rock mass properties is that presented by Hoek and Brown (1997), and updated by Hoek, Carranza-Torres & Corkum (2002). This generalized Hoek-Brown criterion (Figure 3.1) for intact rock samples approximates the non-linear relationship between maximum axial stress $\sigma_1$ that can be sustained by the sample and the applied confining stress $\sigma_3$. In its generalized form, the following parabolic law defines this relationship:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left( m_b \frac{\sigma_3'}{\sigma_{ci}} + s \right)^a$$

Eq’n 3.1

where, $\sigma_1'$ and $\sigma_3'$ are the maximum and minimum effective stresses at failure respectively, $\sigma_{ci}$ is the uniaxial compressive strength of the intact rock pieces, $m_b$ (or $m_i$ for intact strength) is a Hoek-Brown constant and a parameter deduced from $\sigma_1'$ and $\sigma_3'$ test results of a particular rock type. Constants $s$ and $a$ are unique to the rock mass and are based upon the specific rock mass characteristics.
Figure 3.1 Summary of Equations Associated with Hoek-Brown Failure Criteria  
(Hoek and Marinos, 2007)
A new set of relationships between GSI, $m_{\nu}$, $\nu$, and $\sigma$ is introduced to give a smoother transition between very poor quality rock masses (GSI < 25) and stronger rocks. A disturbance factor D to account for stress relaxation and blast damage is also introduced. Equations for the calculation of Mohr Coulomb parameters $c$ and $\phi$ are introduced for specific ranges of the confining stress $\sigma_{c\text{max}}$ for tunnels and slopes.

All of these equations are incorporated into the Windows program RocLab that can be downloaded from the Internet site www.rocescience.com. A copy of the full paper is included with the download.

\[
\sigma_{1} = \sigma'_{1} + \sigma_{c} \left( \frac{m_{\nu} \sigma_{c}'}{\sigma_{c}} \right)^{\frac{1}{d}}
\]

\[
m_{\nu} = m_{\nu} \exp \left( GSI - 10(28 - 14D) \right)
\]

\[
s = \exp \left( GSI - 10(99 - 3D) \right)
\]

\[
a = \frac{1}{2} + \frac{1}{6} \left( e^{-GSI/15} - e^{-20/3} \right)
\]

\[
E_{n} (GPa) = \left( 1 - \frac{D}{2} \right) \left[ \sigma_{c} \frac{1}{100} \right. \left( GSI - 10(14 - 4D) \right)
\]

\[
\phi' = \sin^{-1} \left( \frac{6m_{\nu}(s + m_{\nu}\sigma_{c})^{\frac{a-1}{2}}}{(1 + a)(2 + a) + 6m_{\nu}(s + m_{\nu}\sigma_{c})^{\frac{a-1}{2}}} \right)
\]

\[
c' = \frac{\sigma_{c}^{(1 + 2a)a^{2} - (1 - a)m_{\nu}\sigma_{c}^{2}^{2} + m_{\nu}(s + m_{\nu}\sigma_{c})^{2}}}{(1 + a)(2 + a)^{2} + (6m_{\nu}(s + m_{\nu}\sigma_{c})^{2})^{2} / (1 + a)(2 + a)}
\]

where, for tunnels

\[
\frac{\sigma_{3\text{max}}}{\sigma_{c\text{min}}} = 0.47 \left( \frac{\sigma_{c\text{min}}}{\gamma H} \right)^{-0.34}
\]

- $H$ is the depth below surface

for slopes

\[
\frac{\sigma_{3\text{max}}}{\sigma_{c\text{min}}} = 0.72 \left( \frac{\sigma_{c\text{min}}}{\gamma H} \right)^{-0.91}
\]

- $H$ is the slope height

$\gamma$ is the unit weight of the rock mass

---

**Figure 3.1** Summary of Equations Associated with Hoek-Brown Failure Criteria (Hoek and Marinos, 2007)(con't).
The Hoek-Brown criterion was the primary characterization tool used for this investigation. As cited above, three parameters are required in order to estimate the strength and deformation properties: the uniaxial compressive strength ($\sigma_{ci}$) of the “intact” rock elements; a constant $m$ that defines the frictional characteristics of the rock, and the Geological Strength Index (GSI). GSI was introduced by Hoek et al. (1995), Hoek & Brown (1997) and extended by Hoek et al. (1998) and Hoek et al. (2002). A history of the empirical relationships used within the Hoek-Brown and expanded model are located in Figure 3.1. These equations summarize the development of the Hoek-Brown Failure Criteria (Hoek and Marinos, 2006).

### Figure 3.2
Steps involved in GSI approach in order to obtain strength and engineering properties of rock mass.

Practical application of the GSI system and the Hoek-Brown failure criterion in a number of engineering projects around the world have shown that this system provides reasonable estimates of the strength for a wide variety of rock masses. These estimates must be tailored to individual conditions and are habitually based upon back analysis of tunnel behaviour. However, these strength estimates also provide a sound basis for design purposes.
The most important component of the Hoek-Brown system for rock mass strength determination is the process of reducing the Hoek-Brown constant \( m_i \) and the uniaxial compressive strength \( \sigma_{ci} \) from their laboratory values to realistic in-situ values. The tool used to accomplish this is the Geological Strength Index (GSI). The GSI relates the properties of intact rock elements to those of the overall rock mass. The steps involved for obtaining selected mechanical properties of a rock mass using the above mentioned parameters are illustrated in Figure 3.2. Each of these parameters is addressed separately in the sub-sections that follow.

### 3.2.1 Uniaxial Compressive Strength of Intact Rock \( (\sigma_{ci}) \)

The difficulty of obtaining an “intact” core sample from heterogeneous rock masses poses a challenge in obtaining samples for laboratory testing. Samples obtained from rock masses will most definitely contain discontinuities in the form of bedding and joints or schistosity planes. Samples will likely also contain several component rock types. As a result, any laboratory tests carried out on core samples will be more representative of the rock mass rather than the intact rock components. Using these results will give unrealistically low values for rock mass strength (Hoek and Marinos, 2000). A more conventional strength reading (with its limitations) is obtained using the Point Load Test (Franklin 1985, Thuro & Plinninger 2001) on irregular samples. The specimens used for testing in this case are hand samples or pieces broken from the core. The point load index \( I_s \) can be calculated using:

\[
I_s = \frac{P}{D^2} \quad \text{Eq’n 3.2}
\]

where, \( P \) is the load on the points, \( D \) is the equivalent distance between points and \( \sigma_{ci} \) is estimated as 24\( I_s \) (although the coefficient 24 varies for different rock types).

If site-specific strengths are not obtainable, estimated values for strength of intact rock can be obtained by using Table 3.1.
Table 3.1  Field estimates of uniaxial compressive strength of intact rock (Hoek, 2006)

<table>
<thead>
<tr>
<th>Grade*</th>
<th>Term</th>
<th>Uniaxial Comp. Strength (Mpa)</th>
<th>Point Load Index (Mpa)</th>
<th>Field estimate of strength</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>R6</td>
<td>Extremely Strong</td>
<td>&gt;250</td>
<td>&gt;10</td>
<td>Specimen can only be chipped with a geological hammer</td>
<td>Fresh basalt, chert, diabase, gneiss, granite, quartzite</td>
</tr>
<tr>
<td>R5</td>
<td>Very strong</td>
<td>100-250</td>
<td>4-10</td>
<td>Specimen requires many blows of a geological hammer to fracture it</td>
<td>Amphibolite, sandstone, basaltan, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff</td>
</tr>
<tr>
<td>R4</td>
<td>Strong</td>
<td>50-100</td>
<td>2-4</td>
<td>Specimen requires more than one blow of a geological hammer to fracture it</td>
<td>Limestone, marble, phyllite, sandstone, schist, shale</td>
</tr>
<tr>
<td>R3</td>
<td>Medium strong</td>
<td>25-50</td>
<td>1-2</td>
<td>Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer</td>
<td>Claystone, coal, concrete, schist, shale, siltstone</td>
</tr>
<tr>
<td>R2</td>
<td>Weak</td>
<td>5-25</td>
<td>**</td>
<td>Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer</td>
<td>Chalk, rocksalt, potash</td>
</tr>
<tr>
<td>R1</td>
<td>Very weak</td>
<td>1-5</td>
<td>**</td>
<td>Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife</td>
<td>Highly weathered of altered rock</td>
</tr>
<tr>
<td>R0</td>
<td>Extremely weak</td>
<td>0.25-1</td>
<td>**</td>
<td>Indented by thumbnail</td>
<td>Stiff fault gouge</td>
</tr>
</tbody>
</table>

* Grade according to Brown (1981).
** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.
3.2.2 Hoek-Brown Constant ($m_i$)

This parameter has a significant influence on the strength characteristics of a rock mass. The constant $m_i$ depends on the frictional characteristics of the component minerals of an intact rock sample (Table 3.2). This parameter can only be determined by triaxial testing of core samples or estimated by Hoek and Brown’s (1997) qualitative description of the rock material.

**Table 3.2** Values of the Hoek-Brown constant $m_i$ for intact rock by rock group (Marinos and Hoek, 2000)

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Group</th>
<th>Class</th>
<th>Texture</th>
<th>Coarse</th>
<th>Medium</th>
<th>Fine</th>
<th>Very fine</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sedimentary</td>
<td>Clastic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Non-Clastic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Organic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Carbonate</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Chemical</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Non Foliated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slightly foliated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Foliated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Metamorphic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Non Foliated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slightly foliated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Foliated</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Igneous</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Light</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dark</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extrusive</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pyroclastic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

These values are for intact rock specimens tested normal to bedding or foliation. The value of $m_i$ will be significantly different if failure occurs along a weakness plane.

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3.2.3 Rock Mass Model: Geological Strength Index (GSI)

This index is based upon an assessment of the structure, lithology and condition of discontinuity surfaces in the rock mass. The GSI is estimated through visual examination of the exposed tunnel face (rock mass). The GSI value of a rock mass is incorporated into calculations in order to determine the reduction in the strength of the rock mass as compared with the strength of the intact rock components. This GSI method was deemed as an appropriate tool for evaluating closely jointed rock masses. Oftentimes, the RQD index and RQD value for evaluating rock mass conditions yielded zero for similar rock masses. However, tightly interlocking angular pieces of strong rock samples under confined conditions exhibit good stability under confined conditions.

Tables developed by Hoek and Marinos (2000) can be used to estimate the GSI value for typical jointed rock masses. Due to the numerous engineering projects under construction in heterogeneous rock masses, attempts have been made to provide better and enhanced engineering geology tools. To this end, Marinos and Hoek (2000) also developed a GSI table specifically for heterogeneous rock masses such as Flysch and is shown in Table 3.3. Marinos, V. (2007) also updated the GSI table for Flysch material but this assessment was not available at the design stage of the Driskos Tunnel and was not used within this research investigation. All of the design tables associated with GSI (past and present) have been included in Appendix D of this document. Also located within the appendix are photos from the Driskos Tunnel site and associated GSI determinations. For completeness, the newest GSI table for Flysch has also been included within Appendix D. These estimations of geotechnical properties of heterogeneous rock masses such as Flysch are well documented within Hoek and Marinos, 2001.
Table 3.3 GSI estimates for heterogeneous rock masses such as Flysch (Marinos and Hoek, 2000).

A further modification to Table 3.3 was proposed by Hoek and Marinos in order to take into consideration the intact rock properties of $\sigma_{ci}$ and $m_i$. Within Flysch, it was noted that rock to rock contact between sandstone was limited due to weaker layers of siltstone or shales. Therefore, a weighted average of strong and weak layers was introduced (Table 3.4). This weighted average was used in order to assess the strengths of the Flysch materials within the author’s research.
Table 3.4  Proportions of parameters $\sigma_i$ and m, for estimating rock mass properties for Flysch as suggested by Marinos and Hoek (2000).

<table>
<thead>
<tr>
<th>Flysch type</th>
<th>Use weighted average of components after adjusting sandstone values:</th>
</tr>
</thead>
<tbody>
<tr>
<td>A and B</td>
<td>Use values for sandstone beds</td>
</tr>
<tr>
<td>C</td>
<td>Reduce sandstone by 20%, full values for siltstone</td>
</tr>
<tr>
<td>D</td>
<td>Reduce sandstone by 40%, full values for siltstone</td>
</tr>
<tr>
<td>E</td>
<td>Reduce sandstone by 40%, full values for siltstone</td>
</tr>
<tr>
<td>F</td>
<td>Reduce sandstone by 60%, full values for siltstone</td>
</tr>
<tr>
<td>G</td>
<td>Use values for siltstone or shale</td>
</tr>
<tr>
<td>H</td>
<td>Use values for siltstone or shale</td>
</tr>
</tbody>
</table>

3.3 Mechanical Properties for Weak Rock Masses

3.3.1 Empirical Relationships and Framework

Once the parameters $m_i$, $\sigma_i$, and GSI have been defined, the next step is to estimate other mechanical properties associated with the rock mass. Empirical relationships exist that relate these values to “instantaneous” friction angle ($\phi'$) and cohesive strength ($c'$) as well as rock mass compressive ($\sigma_{cm}$) and tensile strength ($\sigma_{tm}$) and deformation modulus ($E_m$), among others. These relationships are introduced later on in this chapter. Currently standard elasto-plastic analysis has been used for non-linear analysis of rock mass behaviour based on Hoek-Brown parameters or Mohr-Coulomb values.

In terms of the Hoek-Brown failure criteria, a relaxation and damage parameter D (Hoek et al. 2002) can be used to modify the effect of GSI and is described in detail in Hoek and Diederichs (2006) but is not used here (D=0) for undisturbed or carefully excavated tunnel...
environments. Used in this research investigation are the GSI tables presented in Hoek and Marinos (2000) and Marinos and Hoek (2000). These Flysch-specific tables (also included within Appendix C) can be used to estimate the GSI value for typical rock masses as well as heterogeneous rock masses such as those encountered on Egnatia and more specifically the Driskos Tunnel.

The rock mass strength envelope is given as:

$$
\sigma_1 = \sigma_3 + \sigma_{ci} \left( m_b \frac{\sigma_{ci}}{\sigma_{ci}} + s \right)^a
$$

Eq’n 3.3

where for D=0,

$$
m_b = m_i e^{\frac{GSI-100}{28}}
$$

Eq’n 3.4

$$
s = e^{\frac{GSI-100}{9}}
$$

Eq’n 3.5

$$
a = \frac{1}{2} + \frac{1}{6} \left( e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}} \right)
$$

Eq’n 3.6

Hoek and Diederichs (2006) provided an improved, simplified relationship for rockmass modulus, one that is not dependant on $\sigma_{ci}$ (D=0):

$$
E_{rm} = E_i \left( 0.02 + \frac{1}{1 + e^{(60-GSI)/11}} \right)
$$

Eq’n 3.7

In the case of many of the rocks in this region, intact modulus is not available but can be related to the intact strength as per Hoek & Diederichs (2006). An index value for uniaxial rock mass strength is given respectively by Hoek (1999) and (Hoek and Marinos (2000)):

$$
\sigma_{cm} = 0.019 \sigma_{ci} e^{0.05GSI}
$$

Eq’n 3.8a

$$
\sigma_{cm} = \left( 0.0034 m_i^{0.8} \right) \sigma_{ci} \left( 1.029 + 0.025 e^{-0.2m_i} \right)^{GSI}
$$

Eq’n 3.8b

An alternate formulation is given in Hoek et al. (2002) but was not used on the Egnatia project. An alternative modulus equation was also given by Hoek and Marinos (2000):
These are the key empirical relationships and ensuing material property values that were used in this investigation. All of the GSI values were determined by the author through site visits and interpretation of available reference materials (tunnel design and tunnel boring logs).

3.3.2 Rock Mass Values Used as Input for Numerical Model

The modelling portion of this research project required a realistic and accurate characterization of the rock material to be used as well as relevant input parameters. GSI was the tool and framework within which the material strength was characterized.

The idealized geological cross-section diagram of the Driskos Tunnel depicted in Figure 2.35 and introduced in Chapter 2 contains GSI and rock mass strength parameters (GSI, $E$, $\sigma_{ci}$, and $\sigma_{cm}$ as obtained in the field, through interpretation of initial geological study by Egnatia Odos (Egnatia Odos, 1999) and interpretation of the Driskos tunnelling logs (Egnatia Odos, 2003) and multiple site visits by the author. Selected tunnelling logs and GSI formulations that have been selected, screened and formatted by the author are located in Appendix C and Appendix D respectively. Also contained in Figure 3.3 is an assessment of the GSI values for the rock masses that are located within the Driskos area as determined by the author. The main area of focus for this research study was Section 4, the weakest rock mass material of the entire Driskos cross-section. This section was heavily monitored and required additional support during the tunnel excavation phase (to be elaborated upon in Chapter 4 of this document). The monitoring data was key in providing validation data for the numerical models used in this investigation.
Figure 3.3. Representative rock masses of Driskos Tunnel Area, Egnatia Odos, Greece
Section 4 of the Driskos cross-section was further subdivided into 5 sub-sections (4.1-4.5) based on varying geological features within the larger geological section (Section 4 based on Figure 2.35). The sub-section margins were based on the rock masses orientation, degraded structure based on intense tectonism, in-situ stress conditions based primarily on overburden and an interpretation of the rock mass strength. The variation in overburden is approximately 100 m amongst the highest and lowest elevation points within the overall section. The reason for the sub-sections is that the original design was based on a general characterization of this section over the 650 m that span the section. As is evident in Figure 3.4, rock mass composition is not the only factor that should be considered when characterizing this region.

**Figure 3.4** Sub-Sections of Geological Section 4 of Driskos referencing Figure 2.31 and Figure 2.35.
Table 3.5. Properties associated with Section 4 Rock Mass of Driskos Tunnel

<table>
<thead>
<tr>
<th>Section</th>
<th>Chainage</th>
<th>Flysch Category</th>
<th>Lithology</th>
<th>Rock Mass Category</th>
<th>GSI-Low</th>
<th>GSI High</th>
<th>GSI Avg</th>
<th>Overburden (m)</th>
<th>( \phi ) (degrees)</th>
<th>( \nu )</th>
<th>( \gamma ) (KN/m²)</th>
<th>( E_{rm} ) (MPa)</th>
<th>( E_{m} ) (MPa)</th>
<th>Average ( E_{m} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>8+000 - 8+550</td>
<td>B-C</td>
<td>SiSa</td>
<td>IV-V (Va, IVN, S, S-Vb)</td>
<td>18</td>
<td>41</td>
<td>29.5</td>
<td>145</td>
<td>38</td>
<td>0.25</td>
<td>26</td>
<td>1270.9</td>
<td>1881.5</td>
<td>1576.2</td>
</tr>
<tr>
<td>4.2</td>
<td>8+550 - 8+680</td>
<td>B-C</td>
<td>SiSa</td>
<td>IV (S, C)</td>
<td>21</td>
<td>30</td>
<td>25.5</td>
<td>130</td>
<td>37</td>
<td>0.25</td>
<td>26</td>
<td>993.8</td>
<td>1494.5</td>
<td>1244.1</td>
</tr>
<tr>
<td>4.3</td>
<td>8+680 - 8+850</td>
<td>E-F</td>
<td>SiSa, Si</td>
<td>III / IV (C,S)</td>
<td>22</td>
<td>40</td>
<td>31</td>
<td>100</td>
<td>36</td>
<td>0.25</td>
<td>26</td>
<td>1168.1</td>
<td>1716.1</td>
<td>1442.1</td>
</tr>
<tr>
<td>4.4</td>
<td>8+850 - 9+000</td>
<td>E-F</td>
<td>Si, SiSa</td>
<td>III / IV (S,Va)</td>
<td>31</td>
<td>40</td>
<td>35.5</td>
<td>180</td>
<td>31</td>
<td>0.25</td>
<td>26</td>
<td>1578.4</td>
<td>2223.6</td>
<td>1901.0</td>
</tr>
<tr>
<td>4.5</td>
<td>9+000 - 9+110</td>
<td>B</td>
<td>SaSi</td>
<td>III / IV (IVN)</td>
<td>35</td>
<td>41</td>
<td>38</td>
<td>220</td>
<td>47</td>
<td>0.25</td>
<td>26</td>
<td>3915.0</td>
<td>4340.4</td>
<td>4127.7</td>
</tr>
</tbody>
</table>

**Reference**

Egnatia Odos, 1999

Author / Design Category as per Driskos Design, Egnatia Odos, 2001

Author Egnatia Odos, 2001

Roclab 2007

Egnatia Odos 2003

GeoData, 2003

Hoeck Dieckmanns Eqn, 2007

Eqn 3.7

Hoeck-Brown 2002

Eqn 3.9


<table>
<thead>
<tr>
<th>Section</th>
<th>( m_i )</th>
<th>( \sigma_{cl} ) (MPa)</th>
<th>MR</th>
<th>( E_i ) (MPa)</th>
<th>( \sigma_{cm} ) (MPa)</th>
<th>( \sigma_1 ) (MPa)</th>
<th>( \sigma_3 ) (MPa)</th>
<th>( \sigma_{cm} ) (MPa)</th>
<th>( E_{im} ) (MPa)</th>
<th>Cohesive Strength ( c ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>10.3</td>
<td>37.5</td>
<td>430</td>
<td>16125</td>
<td>2.10</td>
<td>-0.017</td>
<td>0.62</td>
<td>3.1</td>
<td>1270.9</td>
<td>0.44</td>
</tr>
<tr>
<td>4.2</td>
<td>10.3</td>
<td>37.5</td>
<td>430</td>
<td>16125</td>
<td>1.85</td>
<td>-0.013</td>
<td>0.46</td>
<td>2.5</td>
<td>993.8</td>
<td>0.37</td>
</tr>
<tr>
<td>4.3</td>
<td>7.75</td>
<td>26.25</td>
<td>512.5</td>
<td>13433.1</td>
<td>1.31</td>
<td>-0.018</td>
<td>0.48</td>
<td>2.3</td>
<td>1168.1</td>
<td>0.29</td>
</tr>
<tr>
<td>4.4</td>
<td>7.75</td>
<td>26.25</td>
<td>512.5</td>
<td>13433.1</td>
<td>1.52</td>
<td>-0.026</td>
<td>0.65</td>
<td>2.9</td>
<td>1578.4</td>
<td>0.46</td>
</tr>
<tr>
<td>4.5</td>
<td>17</td>
<td>75</td>
<td>375</td>
<td>28125</td>
<td>7.52</td>
<td>-0.041</td>
<td>2.18</td>
<td>9.5</td>
<td>3915.0</td>
<td>1.02</td>
</tr>
</tbody>
</table>

**Reference**

Author Table 3.2

Author Hoek & Marinos 2001

H&D 2007

Author Roclab 2007

Author Eqn 3.8b

Author Roclab 2007

Author Roclab 2007

Author Eqn 3.8a

Author Roclab 2007

Author Roclab 2007

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The rock mass properties were then obtained for each of these sections using GSI and empirical methods as contained in Table 3.5. As seen in this table, the main sub-sections are aligned in rows and the major rock mechanic parameters / values are arranged in columns. The last row in the table is used to reference how that particular value was obtained. All values were obtained or calculated directly by the author citing sources of respected empirical relationships.

Table 3.6 provides an abbreviated summary of the specific values used in this investigation.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Section 4.1</th>
<th>Section 4.2</th>
<th>Section 4.3</th>
<th>Section 4.4</th>
<th>Section 4.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chainage</td>
<td>8+385-8+500</td>
<td>8+500-8+650</td>
<td>8+650-8+750</td>
<td>8+750-9+000</td>
<td>9+000-9+035</td>
</tr>
<tr>
<td>Flysch Category</td>
<td>B-C</td>
<td>B-C</td>
<td>E-F</td>
<td>E-F</td>
<td>B</td>
</tr>
<tr>
<td>Lithology</td>
<td>SiSa</td>
<td>SiSa</td>
<td>SiSa, Si</td>
<td>Si, SiSa</td>
<td>SaSi</td>
</tr>
<tr>
<td>Rock Mass Category</td>
<td>IV-V</td>
<td>IV</td>
<td>III / IV</td>
<td>III / IV</td>
<td>III / IV</td>
</tr>
<tr>
<td>Overburden (m)</td>
<td>145</td>
<td>130</td>
<td>100</td>
<td>180</td>
<td>220</td>
</tr>
<tr>
<td>v</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>GSI</td>
<td>18 - 41</td>
<td>21 - 30</td>
<td>22 - 40</td>
<td>31 - 40</td>
<td>35 - 41</td>
</tr>
<tr>
<td>GSI avg</td>
<td>29.5</td>
<td>25.5</td>
<td>31</td>
<td>35.5</td>
<td>38</td>
</tr>
<tr>
<td>(\sigma_{ci}) (MPa)</td>
<td>37.5</td>
<td>37.5</td>
<td>26.25</td>
<td>26.25</td>
<td>75</td>
</tr>
<tr>
<td>(m_i)</td>
<td>10.3</td>
<td>10.3</td>
<td>7.75</td>
<td>7.75</td>
<td>17</td>
</tr>
<tr>
<td>E_i (MPa)</td>
<td>16125</td>
<td>16125</td>
<td>13453</td>
<td>13453</td>
<td>28125</td>
</tr>
<tr>
<td>D</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>(m_b)</td>
<td>0.83</td>
<td>0.72</td>
<td>0.66</td>
<td>0.77</td>
<td>1.8</td>
</tr>
<tr>
<td>s</td>
<td>0.000396</td>
<td>0.000254</td>
<td>0.000468</td>
<td>0.000772</td>
<td>0.001019</td>
</tr>
<tr>
<td>a</td>
<td>0.52</td>
<td>0.53</td>
<td>0.52</td>
<td>0.52</td>
<td>0.51</td>
</tr>
<tr>
<td>(c) (MPa)</td>
<td>0.44</td>
<td>0.37</td>
<td>0.29</td>
<td>0.46</td>
<td>1.02</td>
</tr>
<tr>
<td>(\phi) (degrees)</td>
<td>38</td>
<td>37</td>
<td>36</td>
<td>33</td>
<td>47</td>
</tr>
<tr>
<td>(\sigma_i) (MPa)</td>
<td>-0.017</td>
<td>-0.013</td>
<td>-0.018</td>
<td>-0.026</td>
<td>-0.041</td>
</tr>
<tr>
<td>(\sigma_c) (MPa)</td>
<td>0.62</td>
<td>0.46</td>
<td>0.48</td>
<td>0.65</td>
<td>2.18</td>
</tr>
<tr>
<td>(\sigma_{cm}) (MPa) (^*)</td>
<td>3.1</td>
<td>2.5</td>
<td>2.3</td>
<td>2.9</td>
<td>9.5</td>
</tr>
<tr>
<td>(E_{mn}) (MPa) (^*)</td>
<td>1576</td>
<td>1244</td>
<td>1442</td>
<td>1901</td>
<td>4127</td>
</tr>
<tr>
<td>(\sigma_{cm}/P_o)</td>
<td>0.84</td>
<td>0.77</td>
<td>0.92</td>
<td>0.64</td>
<td>1.70</td>
</tr>
</tbody>
</table>

\(^*\) simplified (original) formula from Hoek (1999)
The values located in Table 3.6 were used as the primary input parameters for all of the 2D and 3D numerical models that were used within this investigation. These models are elaborated upon in Chapter 5 of this document. The calculations involved in determining the Mohr-Coulomb equivalent factors from Hoek-Brown parameters are the focus of the next section.

3.3.3 Mohr-Coulomb Equivalent Materials from Hoek-Brown Criteria

Many numerical model software packages require the use of Mohr-Coulomb parameters as input values. As such, values obtained using the Hoek-Brown criteria must be modified in order to produce Mohr-Coulomb equivalent parameters. Hoek et al. (2002) described this method as part of the Hoek-Brown Failure Criterion. Parameters of interest are values for friction angles and cohesion. These are obtained by fitting an average linear relationship to the function provided in Equation 3.3 for a range of minor principal stresses as per Figure 3.5.

![Figure 3.5](image-url)

**Figure 3.5** Relationships between minor principal stresses and a) major principal stresses and b) shear stress (Hoek et al., 2002)
The ensuing friction angle and cohesion values are obtained using the following formulations respectively:

\[
\phi' = \arcsin\left[\frac{6\alpha m_b (s + m_b \sigma'_{3, n})^{\alpha-1}}{2(1 + \alpha)(2 + \alpha) + 6\alpha m_b (s + m_b \sigma'_{3, n})^{\alpha-1}}\right] \quad \text{Eq'n 3.10}
\]

\[
c' = \frac{\sigma_{ci} [((1 + 2\alpha)s + (1 - \alpha)m_b \sigma'_{3, n})(s + m_b \sigma'_{3, n})]^{\alpha-1}}{(1 + \alpha)(1 + 2\alpha)\sqrt{1 + (6\alpha m_b (s + m_b \sigma'_{3, n})^{\alpha-1})}} \quad \text{Eq'n 3.11}
\]

where, \(\sigma'_{3, n} = \frac{\sigma'_{3, \text{max}}}{\sigma_{ci}}\) and \(\sigma'_{3, \text{max}}\) is the upper limit of the confining stress over the relevant stress ranges between Hoek-Brown and Mohr-Coulomb criteria. The equivalent plot (Figure 3.5a) is defined by the relationship:

\[
\sigma_1 = \frac{2c' \cos \phi'}{1 - \sin \phi'} + \frac{1 + \sin \phi'}{1 - \sin \phi'} \sigma_3 \quad \text{Eq'n 3.12}
\]

The Mohr-Coulomb shear stress \(\tau\) can also be determined using the following expression (Figure 3.5b):

\[
\tau = c' + \sigma \tan \phi' \quad \text{Eq'n 3.13}
\]

For deep tunnels, the following relationships are applicable:

\[
\frac{\sigma'_{3, \text{max}}}{\sigma_{cm}} = 0.47\left(\frac{\sigma'_{cm}}{\gamma H}\right)^{-0.94} \quad \text{Eq'n 3.14}
\]

\[
\sigma'_{cm} = \sigma'_{ci} \frac{[m_b + 4s - a(m_b - 8s)(m_b / 4 + s)]^{\alpha-1}}{2(1 + a)(2 + a)} \quad \text{Eq'n 3.15}
\]

where \(\sigma'_{cm}\) is the strength of the rock mass (Equation 3.15), \(\gamma\) is the unit weight of the rock mass and \(H\) is the depth of the tunnel. The value of \(\sigma'_{3, \text{max}}\) is non-trivial. Results of studies for deep tunnels are plotted in Figure 3.6 and coincide with the resulting, empirical determination of Equation 3.14.
Roclab, Rocscience software (2007) was used in order to obtain relevant strength and Modulus of Elasticity parameters for this investigation. The premise of Roclab is to conduct rock mass strength analysis using the Hoek-Brown failure criteria in order to obtain Mohr-Coulomb parameters that can be used as input parameters for numerical models. The empirical formulations used within the software stem from 3 sources: Hoek-Brown failure criteria (Hoek et al., 2002), history of the Hoek-Brown Equation (Hoek et al. 2006) and empirical estimation of the rock mass modulus (Hoek and Diederichs, 2006). Input values that are required are: $\sigma_{ci}$, GSI, $m_i$, D, $E_i$ or MR (modulus ratio from which an intact modulus of elasticity can be obtained), unit weight and tunnel depth. Output parameters include: Hoek-Brown Criteria parameters of $m_b$, s and a, Mohr-Coulomb equivalent parameters of $\phi$ and c, rock mass parameters of $\sigma_t$, $\sigma_c$, $\sigma_{cm}$, and
E_{rm}. The use of the Roclab software allows for a simple and intuitive implementation of the Hoek-Brown failure criterion, allowing the user to easily obtain reliable estimates of rock mass properties, and to visualize the effects of changing rock mass parameters, on the failure envelopes. The use of this program is ideal for input values for numerical analysis / modelling programs such as FLAC3D (the 3D numerical analysis program that was used as part of this investigation).

Material properties of geological sub-sections were initially assessed and obtained using the Hoek-Brown Criterion and were converted into Mohr-Coulomb equivalent material properties using the method described above. The material properties for sub-Sections 4.1, 4.2, 4.3, 4.4 and 4.5 are included in Table 3.6. Plots of Hoek-Brown and Mohr-Coulomb criteria for these materials are seen in Figure 3.7. This figure contains Hoek-Brown plots of (a) major and minor principal stresses, (b) shear and normal stresses as well as Mohr-Coulomb plots of (c) major and minor principal stresses, and (d) shear and normal stresses.

3.4 Relevance to Research Project

Key technical issues associated with the rock masses within the study area first require an accurate characterization of the rock mass and precise strength estimation. These estimates must capture the true likeness of the rock with respect to its physical mature and mechanical behaviour. This chapter provided the rationale and the framework of why GSI was chosen for this particular investigation versus other rock mass classification systems and provided the empirical relationships that were used by the author to determine the unique strength parameters for various sections, and specifically geological Section 4, of the Driskos Tunnel. Finally, the rock mass values that were obtained were used as input parameters for the numerical models that were extensively used in this research project.
Figure 3.7 Comparison of Hoek-Brown and Mohr-Coulomb strength parameters for Subsections 4.1-4.5. Hoek-Brown plots of (a) major and minor principal stresses, (b) shear and normal stresses, Mohr-Coulomb plots of (c) major and minor principal stresses, and (d) shear and normal stresses.
4.1 Introduction

Tunnel design and construction has gone through major advancements in excavation techniques with the use of enhanced numerical analysis methods (hardware and software), more accurate rock mass classification systems, hi-tech machineries, more tunnel construction and improved ground reinforcement techniques. All of these factors have aided in a better understanding of the tunnel mechanics and better stabilization of the tunnel face, limiting the possibilities of complete tunnel collapse.

Classical tunnel designs have been based on the Rock Mass Ratio (RMR) (designing with respect to deformations) and Terzaghi based designs (designing primarily to support all loads including overburden pressure by the final lining). A newer tunnelling method such as the New Austrian Tunnelling Method (NATM) incorporates an observational approach that is deformation based. This method integrates the surrounding rock into the overall support structure (i.e. the supporting formations will themselves be a part of the supporting structure as the rock is able to support itself to a certain degree) (Romeo, 2002). Using the NATM, a controlled deformation of the rock mass is permitted (a limited strain of approximately 1%) and this gives the stresses an opportunity to be partly released and less stiff. Therefore, a less-expensive support system can be used (Kondogianni and Stiros, 2002).
This chapter will investigate the specific design and tunnelling techniques that are used with respect to tunnelling within weak rock masses. The empirical determination of stresses using the convergence-confinement method (Carranza-Torres and Fairhurst, 2000) will be elaborated upon as an initial idealization of the displacement and stress determinations within a rock mass. Tunnel design and support considerations will then be highlighted with emphasis on the NATM. Finally, the support classes and design of the Driskos twin tunnels is included with relevant instrumentation detail.

4.2 Method of Stress Analysis

In order to understand the issues that arise when attempting to design tunnel support, it is necessary to examine some rudimentary concepts of how a rock mass surrounding an excavation behaves and how (and when) the tunnel support can be introduced into this system.

A simplistic closed-form solution model for the analysis of tunnel behaviour is a circular tunnel which has been excavated. The mechanics from such a model will allow one to develop an elementary understanding of the basis of tunnel behaviour. In order to set-up this model, one can assume a tunnel of radius \( r_i \) that is subjected to a hydrostatic stress \( p_o \) as well as an internal (uniform) support pressure \( p_i \) (as seen in Figure 4.1).
Figure 4.1 Simplistic mechanics associated with circular tunnel excavation

The radius to the outer boundary is denoted by $r_o$ while the internal radius of the excavated portion is $r_i$. If one assumes that the rock will fail with no plastic volume change, the critical stress level at failure is denoted by a critical pressure, $p_{cr}$:

$$p_{cr} = \frac{2p_o - \sigma_{cm}}{1 + k}$$  \hspace{1cm} \text{Eqn 4.1}

where,

- $p_{cr}$ - critical pressure
- $p_o$ – external, hydrostatic stress (pressure)
- $\sigma_{cm}$ – the uniaxial compressive strength of the rock mass
- $k$ – lateral earth pressure

(i.e. $k = (1+\sin\phi)/(1-\sin\phi)$; where $\phi$ is the friction angle of the rock mass)
If the support pressure is less than the critical pressure (i.e. \( p_i < p_{cr} \)), the radius of the plastic zone \( r_o \) and the inward deformation of the tunnel wall \( u_{ip} \) are defined by (Hoek, 2003):

\[
\frac{r_p}{r_i} = \left[ \frac{2(p_o(k-1) + \sigma_{cm})}{(1+k)((k-1)p_i + \sigma_{cm})} \right]^{1/(k-1)} \quad \text{Eqn 4.2}
\]

\[
\frac{u_{ip}}{r_i} = \frac{(1+\nu)}{E} \left[ 2(1-\nu)(p_o - p_{cr}) \left( \frac{r_o}{r_i} \right)^2 - (1-2\nu)(p_o - p_i) \right] \quad \text{Eqn 4.3}
\]

where,

\( E \) – Modulus of Elasticity

\( \nu \) – Poisson’s ratio

Within a Mohr-Coulomb failure criterion, the uniaxial compressive strength, \( \sigma_{cm} \), of a rock may be defined as:

\[
\sigma_{cm} = \frac{2c'\cos\phi'}{(1-\sin\phi')}
\quad \text{Eqn 4.4}
\]

where,

\( c' \) – cohesion of the rock mass (effective parameter)

\( \phi' \) - is the friction angle of the rock mass (effective parameter)

Based on these simplistic geometries and mechanics, Hoek undertook a study whereby he tried to determine the strength and deformation characteristics of a rock mass associated with behaviour of the rock mass surrounding the tunnel (Hoek, 2003). He created dimensionless plots as determined by the results of parametric studies; a Monte Carlo analysis in which the input parameters for rock mass strength and tunnel deformation were varied at random using 2000 iterations. These plots (Figure 4.2) are used not only to provide insight into potential tunnel behaviour, but as the basis of indicating what type of tunnel support may be required.
Figure 4.2 Normalized tunnel deformation versus ratio of rock mass strength to in situ stress (Hoek, 2003).

Note that once the rock mass strength ratio falls below 20% (this being the rock mass strength and in situ stress level ratio) deformations increase substantially. Unless control measures are taken, it is predicted that the tunnel will most likely collapse.

In order to introduce the concept of support, a similar series of analyses was conducted by Hoek, whereby a wide variety of support pressures versus in situ stresses were examined. The resulting equations (Equations 4.5 and 4.6) and curves that were produced are shown in the following:
where,

\( u_i \) – tunnel sidewall deformation

\( r_o \) – the outer radius or external radius of the plastic zone

The ultimate goal of the determination of rock mass behaviour is to establish what type of support is required for a particular behaviour (strain). Where a particular rock mass plots with respect to the strain versus rock mass strength to in situ stress ratio (Figure 4.4(a)) determines potential support requirements (Figure 4.4(b)).
Figure 4.4  (a) Potential tunnelling problems associated with the level of strain and (b) face excavation and support options for large tunnels (Hoek, 2001b).
4.3 Convergence-Confinement Method

This procedure yields an estimate of the load imposed on the support immediately behind the newly excavated face of a tunnel (i.e. upon excavation, the area immediately behind the face is partially supported by the face itself and does not carry the full load of an open cavity at that moment). As the tunnel and face advance with further excavation, the support must carry a greater portion of the load and eventually the entire load when the face has moved well away from the support (Carranza-Torres and Fairhurst, 2000). Idealistically, this problem can be illustrated as shown in Figure 4.5.

![Figure 4.5](image)

Figure 4.5 Idealized tunnel excavation; (a) Cylindrical tunnel of radius $R$ driven in rock mass, (b) Cross-section of the rock mass at section A-A’ and (c) Cross-section of the circular support installed at section A-A’ (modified after Carranza-Torres and Fairhurst, 2000).

Two important assumptions to this problem (and of the convergence-confinement method) are that it assumes a uniform or hydrostatic stress field, $p_o$ or $\sigma_o$ (i.e. deformation occurs in a plane perpendicular to the axis of the tunnel – 2D plane strain conditions) and the tunnel
cross-section is circular of radius R. A support is placed at section A-A’ (Figure 4.5a), located a
distance L from the tunnel face. The problem has been idealized in such a way as to determine
the loads that will be transmitted to the support once the excavation has progressed (i.e. L
increasing) and the support provided by the face does not affect the supported region (A-A’) any
longer. As seen in Figure 4.5b, excavation of the rock mass (assumed to be an elastic medium)
produces a failed rock zone around the cavity. The far-field hydrostatic stress is denoted by \( \sigma_0 \)
radial displacement \( u_r \) and the pressure \( p_i \) (i.e. reaction of support on the walls of the tunnel).
Figure 4.5c shows a cross section at section A-A’. The support is of thickness \( t_c \) and the external
radius is \( R \) or \( r_0 \), the uniform pressure, \( p_s \) is the load transmitted by the rock mass to the support.
The radial displacement \( u_r \) is the displacement due to \( p_s \). The convergence-confinement method
can be described as a series of excavation stages varying \( L \) with time. At time = 0, \( L \) is as
depicted in Figure 4.5a. The ground has converged radially by a certain amount \( (\Delta u_r) \). Again, it
is assumed that at this time the rock mass does not transmit load to the support as long at the face
does not advance. As the tunnel advances, (i.e. \( L \) increases) the support (at A-A’) begins to carry
some of the load that has been previously carried by the face and the ground convergence has also
increased. At a time when the face has moved far enough ahead and is not influencing the
support at A-A’, the support system is at equilibrium and the support carries the entire, final or
design load. At this time, the support and the ground have converged together by the same
amount.

In this way, three deformation curves can be plotted with respect to time: The
Longitudinal Deformation profile (LDP) – the tunnel as it moves forward, the Ground Reaction
Curve (GRC) – the section of excavation perpendicular to the tunnel axis and the Support
Characteristic Curve (SCC) – the support installed at that section. Interpretation between these
curves allows one to define the pressure that the ground transmits to the support as the face
advances. This is consistent with observed convergence within tunnel excavations as seen in the field (Figure 4.6).

Also seen in Figure 4.6 is a simple circular tunnel deformation analysis within an elastic medium and hydrostatic stress conditions conducted by Hoek (1995). This idealized model also corresponds to observations made in the field. The rock mass ahead of the tunnel excavation begins to yield as a plastic zone forms ahead of the excavation.

Figure 4.6 Top: Percentage of the total convergence (GRC) in a tunnel with diameter D as a function of distance from the excavated face, assuming a nearly constant excavation rate. The excavation front is at 0, but deformation extends to the unexcavated area, along a distance of approximately -1.5D. The corresponding closure equals to about 1/3 of total deformation. Elastoplastic deformation extends up to a distance of 1.5D. Beyond this, tunnel deformation is related to creep. Shaded area corresponds to deformation not recorded by instrumentation measurements. Bottom: Vectors of mass displacement in the plastic zone around the excavated area (Kondoyianni and Stiros, 2002).
Convergence-confinement analysis for tunnelling is a standard approach for preliminary analysis of anticipated wall deformation and support design in squeezing ground. Whether this analysis is performed using analytical (closed form) solutions or with plane strain numerical models (to be discussed in Chapter 5), an LDP is required to relate tunnel wall deformations at successive stages in the analysis to the actual physical location along the tunnel axis. As such, this is a fundamental issue that required further investigation. The journal paper included in Chapter 7 presents a new and robust formulation for the LDP calculation that takes into account the significant influence of ultimate (maximum) plastic radius associated with tunnelling in weak rocks at depth (Vlachopoulos and Diederichs, 2009). Even after all parameters are appropriately normalized, the LDP function varies with the size of the ultimate plastic zone. Larger yield zones take a relatively longer normalized distance to develop, requiring an appropriately calculated LDP. Failure to use the appropriate LDP can result in significant errors in the specification of appropriate installation distance (from the face) for tunnel support systems (also investigated in the journal paper included in Chapter 8). Such errors are likely to result in failure of the temporary support. The equations presented within the paper are readily incorporated into analytical solutions and a graphical template is provided for use with numerical modelling. This has had an impact in the field as Rocscience and Rocsupport numerical modelling software packages have readily accepted and incorporated this Vlachopoulos and Diederichs (2009) method within their numerical software packages as seen in the tutorial provided at Rocscience website:


Timing of the installation of the support also becomes an issue. If the support is installed pre-maturely, the stresses may be too large causing the support to fail. If the support is installed too late, the strain will have induced radial closure and the opening will not be sufficient as per
the design (Figure 4.7). This fact is also highlighted in the Vlachopoulos and Diederichs paper (2009) showing the conservatism associated with support design associated with elastic theory (Panet, 1995) and field data (Chern, 1998) as well as Vlachopoulos and Diederichs support curve considerations taking into account the size of the ultimate plastic zone (Figure 4.8).

![Figure 4.7 Installation of support](image)

![Figure 4.8 Example comparison of Factor of Safety (F.S.) obtained through simple analytical convergence-confinement analysis using 3 different LDP’s to guide support installation. Refer to Vlachopoulos and Diederichs, 2009 (Chapter 7).](image)
4.4 Observational Design Method – New Austrian Tunnelling Method (NATM)

The New Austrian Tunnelling Method was first employed in the United Kingdom within the mining field in 1964 (Karakus et al., 2004). NATM is based on a concept whereby the rock surrounding an underground excavation becomes a load bearing structure itself. It consists of the application of temporary support measures in the form of shotcrete and steel arches that are closed at the earliest possible moment by an invert to complete a ring (or an auxiliary arch), the deformation of which is measured as a function of time until equilibrium is obtained (Karakus et al. 2004). As such, NATM is a specific method of excavation and support techniques as can be seen below in Figure 4.9.

![Diagram of NATM excavation stages](image)

**Figure 4.9** Depiction of the excavation stages involved with NATM (Holland, 2006).

The material is excavated in stages using conventional techniques. The top heading is the top portion of the tunnel portal and is dug out first. This is followed by the bench excavation and if required, further followed by a supported invert. As excavation is being conducted, support in the form of rockbolts, forepoles (to be discussed within the next section), steel sets and shotcrete are installed directly behind the advancing face. Throughout all stages of tunnelling, the tunnel is being monitored in order to determine the rock mass behaviour and change (if required) the rock
support class of a particular tunnel section. Successful application of this technique depends upon a high degree of understanding of support mechanics and intelligent and experienced interpretation of geological conditions and monitoring observations. This technique has been used extensively on tunnelling projects around the world and is deeply rooted in sound geomechanical principles. The NATM was chosen as the technique for use with the excavation of the Drikos Tunnel.

### 4.4.1 Support Design and Considerations

There are major design considerations that must be addressed and properly interpreted. The material through which one tunnels cannot be fully defined in terms of well known strength and deformation properties as previously identified in Chapter 3 of this document (i.e. materials are often discontinuous, inhomogeneous and anisotropic in nature). A design must take into consideration the effects of the disturbance caused by tunnel excavation including stages of excavation not completely confined by the long term support and final lining. It is during this stage that the pre-existing stresses in the rock mass (deviated by the opening of the tunnel) are channelled around the cavity in an arch effect, creating zones of increased stress on the walls of the excavation. The most important task of a tunnel design engineer is to determine how and if an arch effect can be triggered when a tunnel is excavated. The engineer must then ensure that the arch effect is formed by calibrating excavation and stabilization operations (Lunardi, 2000). Understanding this rock mass and support interaction becomes a critical issue.

Lunardi (2000) conducted field investigations of Italian tunnels excavated in rocks and categorized support methods into three different groups. Each group exerts a different effect on the tunnel cavity. He found that the rigidity of the core determines the stability of a tunnel since
the deformation of the advanced core causes (a) the extrusion of the face, (b) preconvergence behind the face and (c) the convergence of the cavity. This has also been described in Section 4.2 and is shown in Figure 4.6.

The purpose of tunnel support is to maintain confinement for the rock mass in order to help the rock mass support itself. Under these confined conditions, the interlocking components of the rock pieces produce a strong and stable rock mass. Care must be taken when excavating the face in order to ensure that confined conditions can be maintained. This is achieved through the immediate installation of support technologies such as (not the same in all cases) fibreglass dowels, spiles, shotcrete, rockbolts and grouting. Again, the initial support systems installed at or in advance of the tunnel face serve to retain the rock mass integrity and provide all of the short term support and permit the ultimate installation of the final lining. Excavation in most tunnels within a weak rock mass is carried out in this staged fashion (Examples of support systems employed and various stages of construction are shown in Figure 4.10 and Figure 4.11 below).

As with the NATM technique mentioned previously, a top heading can be excavated and then a bench (or invert sections) may be left in place for further support. The primary support comes from the initial installation of rock bolts and steel arched rib sections supplemented with shotcrete.

A typical approach would involve the development of a number of typical cross sections for support design. Each section would be related to an anticipated magnitude of strain (or radial displacement). When advancing through difficult ground the use of the forepoling umbrella arch method is oftentimes employed (Figure 4.10 and Figure 4.11). For a 10 m span of tunnel, this method would typically involve the installation of 12 m long, 75 mm diameter grouted pipe forepoles at a spacing of 300 to 600 mm. The forepoles are installed every 8 m to provide a minimum overlap of 4 m between successive umbrellas. This method is usually used in combination with other support systems such as steel sets embedded in shotcrete, face
stabilization by grouted fibreglass dowels and the use of a temporary invert (bench) to control floor heave (Hoek, 1999).

**Figure 4.10.** A comprehensive primary support system for weak rock tunnelling (courtesy E. Hoek), including pre-support beyond the face.

**Figure 4.11** depicts various stages of the construction sequence associated with the Egnatia Odos tunnelling works currently underway in Northern Greece.
Figure 4.11 Sequencing of tunnel construction at selected stages; (a) forepoles, (b) installation of forepoles at tunnel face using forepoling machine, (c) view of tunnel face with forepole umbrella inserted, fiberglass dowels, and drainage pipes, (d) shotcrete being applied, (e) controlled excavation of face, (f) excavated material, (g) removal of excavated material, (h) installation of structural support – steel sets, (i) installation of further radial support – grouted rockbolts, (j) successive forepole umbrellas seen as well as bench, (k) installation of geomembrane, (l) installation of steel reinforcement for final lining, (m) detail of reinforced final lining, and (n) completed tunnel.
Another major consideration when dealing with face stability is water pressure and drainage. Water may accumulate behind material within fault zones and these barriers may act as dams. Therefore, high pressure water may potentially be trapped behind the face. Care must be taken in order to ensure that a probe hole be advanced well ahead of the face at all times. This allows for the detection of the presence of high pressure water and to allow for sufficient drainage prior to excavation. Also incorporated in the design is the use of a geosynthetic impermeable membrane and drainage layer placed between the primary support and the final lining. Reduction of water pressures is dealt with through the use of drainage pipes located on either side of the tunnel footing.
4.5 Tunnels of the Egnatia Odos - Design and Construction Considerations

4.5.1 Design of Tunnel and Temporary Support

As introduced in the preceding section, tunnelling in weak rock presents many challenges since misjudgment in the design of support systems can lead to under design and costly failures or over design and high tunnelling costs. In order to achieve technically and economically optimized constructions, the general principles for the tunnel design for Egnatia Odos were (Egnatia Odos AE, 2001):

a. a good general tunnel design layout that does not force tunnel constructions to be sited into the worst conditions within the project area,

b. investigations and exploration that clarify all details of soil and geo-mechanical behaviour before the design phase, and

c. a design that should adapt well to all steps of execution, while the lifetime construction should be guaranteed by taking into consideration all the in-situ conditions and/or influences.

The design and construction methods employed for the tunnels of Egnatia Odos took on the form of a modified version of the NATM, an observational approach utilizing much instrumentation and an extensive monitoring program. Egnatia Odos S.A. created new guidelines for tunnel design based on international best practices. For the most part, the tunnels were/are designed in accordance with German and Greek standards as stipulated by the following documents: OSMEO – Design Guidelines for conducting Road Works Design, TSY – Tunnels Materials and Workmanship Specification, OSAT – Guidelines for the Environmental terms and landscape design and EAK 2000 - Greek Seismic Code (Rawlings et al., 2001).

The standard cross-section of the excavated tunnel portals on the Egnatia Odos are horseshoe shaped and provides for a 5 m structure clearance with 2 traffic lanes 3.75 m wide each (after final lining). All Egnatia tunnels have a design life of 120 years. A typical tunnel cross-
section can be seen in Figure 4.12 below. The tunnels are designed as twin tunnels and pedestrian cross passages are provided every 350 m and emergency vehicle cross passages and parking areas every 1000 m (Figure 4.12). The drainage system of the tunnel pavement includes a continuous fissured (slotted) gully which discharges every 50 m into a 400 mm diameter main collector pipe (Lambropoulos, 2005).

The first step in the tunnel design process was the engineering geological model/map development which was preceded by the geotechnical site investigation. This formed the basis for the tunnel design and provided the essential geotechnical and ground parameters for tunnel primary support and final lining design. In this investigation, the author concentrated on the primary or temporary support design considerations.

In designing the temporary support measures, the anticipated response of the ground to the stresses induced by tunnel excavation at all geological locations was taken into account. The
design was largely influenced by the rock and groundwater conditions, rock mass properties, deformation behaviour in relation to the in situ stresses (e.g. squeezing conditions), overburden and ground discontinuities. Rock mass primary support categories were then developed for ranges of the rock mass classification values for individual tunnels (and site-specific sub-sections of rock masses located at each tunnel location). The rock mass mechanical properties derived were used as input into 2D numerical analyses of the ground/primary support behaviour and input into the final design.

In keeping with the observational method, in progressively worsening rock mass conditions that were encountered in selected complex geological structures common to the tunnels of Egnatia, the initial category of primary support was upgraded to higher support classes (Lambropoulos, 2005). One of the most common additional support measures was the use of an *elephants foot* (Figure 4.13) located at either side of the top heading. This was inserted in order to improve the load distribution (and bearing capacity and stability) of the top heading support arches.

![Figure 4.13 Schematic representation of all possible support measures for Egnatia Odos Tunnels (Egnatia Odos AE, 2001).](image)
4.5.2 Tunnel Instrumentation and Influence on Tunnel Design

Instrumentation and monitoring play a vital role in verifying design assumptions and calibrating numerical models. As well, monitoring serves as an alert if the initial support or lining is not performing as intended or if the tunnel is in danger of collapse. Deformation is a main factor in controlling the failure and cost-effectiveness of underground excavations. As such, in the last two decades deformation monitoring has become a fundamental requirement for assessing the stability of underground openings and for quantifying the acceptable risk of rock response (Kontogianni and Siros, 2003). Monitoring data also provides a wealth of data as to the 3D behaviour of the rock mass, support and the (time) history associated with excavation. This information can be used to improve geotechnical models and optimize the excavation process, as was conducted in this research project.

The monitoring program within the tunnels of Egnatia Odos incorporates the use of inclinometers, extensometers, strain gauges, load cells, instrumented rock bolts and standard convergence and deformation measurements (Hindley et al., 2004). Within the concept of the observational method of tunnel construction, monitoring has also played an important role in making design changes to primary support systems.

Most of the data available for this study was in digital form without numerical values being provided on a spreadsheet and selected data was available to the author through the National Technical University of Athens and Egnatia Odos, S.A.. Egnatia Odos S.A. controls the dissemination of certain data due to sensitivity involved at certain construction sites (i.e. the Anthochori tunnelling site had outstanding legal issues that had to be dealt with). Figure 4.14 depicts selected monitoring instrumentation that was employed during tunnel construction for Egnatia Odos.
Figure 4.14 Instrumentation and targets associated with monitoring program of Egnatia Odos; (a) monitoring well and survey target on benchmark, (b) pressure cell (left) and extensometer (right), (c) tunnel wall pressure cell, (d) tunnel wall survey target, (e) surveying the tunnel face, and (f) measurement of targets on tunnel wall (-19 denotes 19 mm of inward displacement at that target location) within Driskos Tunnel.

Configurations of other instrumentation that was used within the tunnels of Egnatia Odos are shown in Figure 4.15. This arrangement of tunnel monitoring instrumentation includes: extensometers (BE), survey target points (or convergence markers)(CB), hydraulic pressure cells (HC) and anchor strain gages (AC). The data from these instruments could not be geo-referenced with any accuracy and as such could not be used within this investigation.
Another data gathering intensive portion of the observation method includes gathering geodetic data in the form of survey monitoring. This monitoring scheme was used in order to verify the adequacy of the adopted geomechanical model and support classification/support system. This data is paramount in making decisions on modifications to design and construction optimisations during the construction phase. A monitoring program for the tunnels of Egnatia Odos included: three-dimensional tunnel wall displacements using optical survey (Figure 4.16), rock mass deformation using sliding deformation meters and inclinometer columns (Figure 4.17) and ground water pressure variations using piezometric cells.
Figure 4.16 Optical survey markers and their locations within the tunnel excavation (Geodata, 2003). Photo taken by author. Note that A-Phase and B-Phase are analogous to top heading and bench excavation phases respectively.

Figure 4.17 Cross-section of twin tunnels showing locations of sliding deformeters and piezometers (Grasso et al., 2003).

Depicted in Figure 4.17 are the locations and arrangement of sliding deformeters with lengths of 25 m inclined at 45 degrees. These deformeters were located on the inner and outer side of the bores. Three piezometers are located at 30 m, 20 m and 10 m radial distances from the bores.
The array of instrumentation allow for constant monitoring of the tunnel and for the designers, contractors and engineers to gain insight as to the behaviour of the rock mass subjected to tunnel excavation.

4.6 Design and Construction of Driskos Twin Tunnels

4.6.1 Introduction

The tunnel construction at the Driskos site consisted of twin, parallel tunnels that are approximately 4570 m in length with a maximum overburden of 220 m. The cross-section’s alignment was introduced in Chapter 2 of this thesis document. In a generic sense, the geological profile shows a gently folded synclinal structure at the centre of the alignment, which elevates to an anticline at the southern end. The Driskos tunnels are 850 m above mean sea level and excavation began in 1999 with excavation being driven from both ends. The faces were split into a 60 m$^2$ top heading followed by a 40 m$^2$ bench. The tunnels were advanced using a Tamrock Para 206 T twoboom and basket drillrig. The north portals were constructed using an umbrella canopy for 50 m followed by 40-50 m cut and cover with another 20-30 m of forepole umbrella (Smith, 2000).

Throughout the excavation process, detailed geological mapping of the face and sidewalls was conducted. Selected sketches and photos as well as associated engineering properties that were amassed and organized by the author through multiple sources are shown in Appendix C. A selected section of tunnel face and sidwall sketches are shown in Figure 4.18.
### Section 4.1 and 4.2 of Driskos Tunnel

**Chainage: 9+400 to 9+600**

**Bore: Right**

<table>
<thead>
<tr>
<th>Properties</th>
<th></th>
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</thead>
<tbody>
<tr>
<td>GSI</td>
<td>45-60</td>
</tr>
<tr>
<td>E</td>
<td>4.1-11.3 GPa</td>
</tr>
<tr>
<td>$\sigma_{ci}$ (UCS)</td>
<td>30-40 MPa</td>
</tr>
<tr>
<td>$\sigma_{cm}$</td>
<td>0.7-3.3 MPa</td>
</tr>
<tr>
<td>Lithology</td>
<td>Sa+SiSa</td>
</tr>
</tbody>
</table>

**Photo of Tunnel Face or Outcrop at Chainage: 9+580**

| Rock Mass Category | III |

**Figure 4.18** Photo, face and sidewall sketches as well as relevant data for Section 4.1 and 4.2 of Driskos tunnel, right bore between chainages 9+400 to 9+600.
Cross-passages connecting the two tunnels occur every 350-400 m. A 180 m ventilation shaft was also constructed near the centre of the tunnel alignment. Tunnel separation is 18.2 m between each bore as seen in the Driskos Northern Portals in Figure 4.19.

In terms of tunnel bore dimensions, a single bore cross-section has dimensions of 9.47 m by 11.0 m with respect to rock mass excavation tolerances. The cross-sections of the bores are arranged in a horseshoe configuration. An idealized cross-section of this nature can be seen in Figure 4.20.
In terms of twin tunnel dimensions, Figure 4.21 outlines the spacing and separation distances for the parallel bores.

**Figure 4.20** Idealized horseshoe cross-section of Driskos Tunnel

**Figure 4.21** Idealized cross-section of Driskos tunnel for twin bores
4.6.2 Support Categories

As described in the previous sections of this chapter, a function of stress, strain and rock mass (i.e. tunnel deformation/tunnel diameter versus rock mass strength/in-situ stress) defines the support requirements for tunnel excavations through rock at a specific site. This exercise was no different for the design of the twin tunnels as part of the Driskos tunnels. The main factors that influenced the design were: lithology, rock mass quality, and the height of overburden (i.e. in situ stress). Based on these results, four rock mass categories (Category II – V, with II being the better rock mass in terms of strength) and corresponding support categories (Figure 4.22) were defined (Egnatia Odos, 1999 and Structural Design S.A., 1999). The percentage of rock masses and support systems corresponding to the overall carriage length for Driskos was:

- Category II – 22%
- Category III – 42%
- Category IV – 27%
- Category V – 8%

Trends:
- Deteriorating Rock Mass Strength
- Increased Support Requirements

Shown in Figure 4.22 is the extent of support measures that were incorporated in the preliminary design for the Driskos tunnels. One can see the increased mechanical support requirements as one goes from Category II to Category V. Included within the support requirements are rockbolts, shotcrete, steel-sets, and forepoles. These support measures will be elaborated upon in subsequent sections within this chapter.
The support measures that correspond to the rock mass categories for Driskos tunnel are summarized in Table 4.1. Category V is further sub-divided into Va and Vb based on the requirement (observational method) of forepoling. The specific characteristics, geometries and material properties associated with these support measures are investigated in the next section.
Table 4.1  Support measures for each rock mass category for Driskos Tunnel (modified after Structural Design S.A., 1999).

<table>
<thead>
<tr>
<th>Support Category</th>
<th>Construction Phases</th>
<th>Distance of unsupported Portion</th>
<th>Support Measures</th>
<th>Additional Measures</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Application Time</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>2 (Top Heading-Benching)</td>
<td>6 m</td>
<td>rockbolts in distribution 1.5x2.0m in crown and sides</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>18 hours from the blasting</td>
<td>3m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>shotcrete</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>steel sets</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>2 (Top Heading-Benching)</td>
<td>3 m</td>
<td>rockbolts in distribution 1.3x1.3m in crown and sides</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>12 hours from the blasting</td>
<td>3m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>shotcrete</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>steel sets</td>
<td></td>
</tr>
<tr>
<td>IV</td>
<td>4 (2 Top Heading-2 Benching)</td>
<td>2 m</td>
<td>rockbolts in distribution 1.2x1.0m in crown and sides, 6m rockbolts in rest of crown</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>During Excavation</td>
<td>4m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>shotcrete</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>steel sets</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>20cm in the crown and the sides and 10cm at the face of top heading</td>
<td>Steel Sets (HEB) at 1m distances</td>
</tr>
<tr>
<td>Va</td>
<td>4 (2 Top Heading-2 Benching)</td>
<td>2 m</td>
<td>rockbolts in distribution 1.5x1.0m in crown, sides &amp; bottom</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>During Excavation</td>
<td>6m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>shotcrete</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>steel sets</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>25cm in the crown and the sides and 10cm at face of top heading</td>
<td>Steel Sets (HEB) at 1m distances</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>forepole umbrella if necessary</td>
<td></td>
</tr>
<tr>
<td>Vb</td>
<td>2 (Top Heading-Benching)</td>
<td>2 m</td>
<td>rockbolts in the basis of the steel sets</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>During Excavition</td>
<td>6m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>shotcrete</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>steel sets</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>25cm in the crown and the sides and 10cm at the face of the top heading; 25cm for invert</td>
<td>Steel Sets (HEB) at 1m distances</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>forepole umbrella</td>
<td></td>
</tr>
</tbody>
</table>
4.6.3 Tunnel Support Measures

A longitudinal cross-section of the Driskos Tunnel design (Category V with forepoles) is seen in Figure 4.23. Here, one can view the steel-sets and the orientation of the forepole umbrella.

![Figure 4.23 Longitudinal cross-section of Driskos showing tunnel support detail (Structural Design, 1999).](image)

It is the weakest rock masses of Category V that were the main focus of this thesis research. This support category corresponds to Section 4 of the Driskos cross-section (i.e. the weakest rock mass within Driskos) as cited in Chapter 2.

4.6.3.1 Forepoles

Forepoles are used to create a protective umbrella ahead of the face adding stabilization ahead of the plastic zone created due to tunnelling effects. These forepoles are grout injected steel pipes (Figure 4.24). They function in a frictional sense in the same manner as pile footings, whereby each individual forepole helps support the entire system of forepoles. The forepoles are installed using a forepole drilling machine/ heavy equipment (Figure 4.24a). Related to Figure
4.23 and Figure 4.24 below depicts the length, spacing and orientation of the forepoles in a longitudinal sense as used in the Driskos tunnels.

![Diagram showing forepole configuration](image)

**Figure 4.24** (a) Forepole configuration for Driskos Tunnel with forepoling machine inset, (b) forepole steel pipes 101 mm diameter by 12 m long, (c) excavated tunnel showing successive forpole umbrella structures.

Table 4.2 below summarizes the key geometries and mechanical properties associated with the steel hollow cylinders that constitute the forepoles that were used.
### Table 4.2 Dimensions and properties associated with forepoles

<table>
<thead>
<tr>
<th>Geometry / Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>101 mm</td>
</tr>
<tr>
<td>Length</td>
<td>12 m</td>
</tr>
<tr>
<td>Overlap</td>
<td>4 m</td>
</tr>
<tr>
<td>Angle (from horizontal)</td>
<td>5.78°</td>
</tr>
<tr>
<td>Spacing</td>
<td>0.3 m around crown</td>
</tr>
<tr>
<td>Young’s Modulus (E)</td>
<td>200 GPa (steel)</td>
</tr>
<tr>
<td>Interaction Friction Angle</td>
<td>35°</td>
</tr>
<tr>
<td>Density</td>
<td>7750 g/m³</td>
</tr>
<tr>
<td>Normal Stiffness</td>
<td>$100 \times 10^6$ N/m²</td>
</tr>
<tr>
<td>Cohesion (Shear Coupling)</td>
<td>$25 \times 10^6$ N/m</td>
</tr>
</tbody>
</table>

#### 4.6.3.2 Grouted Rock Bolts

Rockbolts are used to mechanically stabilize the rock mass around an excavation. The rockbolts must be long enough to protrude past the expected plastic zone (or fractured zone depending on the rock mass) or be fastened securely within this zone. A rockbolt consists of a threaded steel rod that is attached to a mechanical anchor at one end and a face plate unit with a nut at the other end. Once the rockbolt is in place and securely anchored, the plate is placed over the threaded rod and the nut is seated and tightened. In this way, the system is in tension, compressing the rock mass. A fully bonded rockbolt can also be installed for use in more
permanent or weak rock applications. For these rock bolt types, the space between the bolt and the rock is filled with cement or resin grout. **Figure 4.25** depicts the rock bolt pattern or arrangement of bolts for the Driskos Tunnel for rock mass Category V. The bolts were installed in 1.0 m rows and had 1.0 m spacing between them. Individual bolts had 1.2 m spacing within each row. Each row of bolts was also offset by 0.6m from adjacent rows of bolts.

**Table 4.3** summarizes the key geometries and mechanical properties associated with the rock bolts that were used as part of the support measures for the Driskos tunnels. Each bolt was fully bonded and post grouted with no pre-tensioning.

![Diagram of rock bolt configuration](image)

**Figure 4.25**  (a) Rock bolt configuration for Driskos (Category V), (b) rock bolts delivered to tunnelling site prior to installation and (c) rock bolts installed at side wall of tunnel excavation with shotcrete layer evident,
Evident in the figure are the steel sets and the face plate unit with the secured nuts of the rockbolts. A shotcrete layer has also been placed on the tunnel sidewall in Figure 4.25c above.

**Table 4.3** Dimensions and properties associated with rock bolts

<table>
<thead>
<tr>
<th>Geometry / Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>25 mm</td>
</tr>
<tr>
<td>Length</td>
<td>6 m</td>
</tr>
<tr>
<td>Peak Capacity</td>
<td>0.2 MN</td>
</tr>
<tr>
<td>Residual Capacity</td>
<td>0.02 MN</td>
</tr>
<tr>
<td>Spacing</td>
<td>1.2 m x 1.0 m</td>
</tr>
<tr>
<td>Pre-tensioning</td>
<td>0 MN</td>
</tr>
<tr>
<td>Cross-Sectional Area</td>
<td>$4.91 \times 10^{-4}$ m$^2$</td>
</tr>
<tr>
<td>Young’s Modulus (E)</td>
<td>200 GPa (steel)</td>
</tr>
<tr>
<td>Interaction Friction Angle</td>
<td>35$^\circ$</td>
</tr>
<tr>
<td>Density</td>
<td>7750 g/m$^3$</td>
</tr>
<tr>
<td>Normal Stiffness</td>
<td>$100 \times 10^9$ N/m$^2$</td>
</tr>
<tr>
<td>Cohesion (Shear Coupling)</td>
<td>$25 \times 10^6$ N/m</td>
</tr>
</tbody>
</table>

**4.6.3.3 Shotcrete**

Shotcrete is the name given to a concrete or mortar mix that is sprayed onto a surface using compressed air. In the NATM method, it is used as a temporary support measure in combination with rockbolts, rock anchors and/or steel sets, depending on the tunnel design.
Figure 4.26 depicts the application of shotcrete on an excavated face. The specific properties associated with the shotcrete that was used for Driskos are summarized in Table 4.4.

![Figure 4.26 Application of shotcrete to tunnel face](image)

### Table 4.4 Dimensions and properties associated with shotcrete

<table>
<thead>
<tr>
<th>Geometry / Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of Shotcrete layer</td>
<td>25 cm</td>
</tr>
<tr>
<td>Peak Compressive Strength</td>
<td>35 MPa</td>
</tr>
<tr>
<td>Young’s Modulus (E)</td>
<td>30 GPa</td>
</tr>
<tr>
<td>Poisson’s Ratio (v)</td>
<td>0.2</td>
</tr>
<tr>
<td>Normal Stiffness</td>
<td>$100 \times 10^9$ N/m²</td>
</tr>
<tr>
<td>Cohesion (Shear Coupling)</td>
<td>$25 \times 10^6$ N/m</td>
</tr>
<tr>
<td>Residual Compressive Strength</td>
<td>5 MPa</td>
</tr>
<tr>
<td>Peak Tensile Strength</td>
<td>5 MPa</td>
</tr>
<tr>
<td>Residual Tensile Strength</td>
<td>0 MPa</td>
</tr>
</tbody>
</table>
4.6.3.4 Steel Sets – HEB 160

Engineered steel sections comprise a major portion of the temporary support associated with the tunnel excavation. These sections are installed immediately behind the excavated face and assume the shape of the tunnel cross-section. Steel section support is usually added every 1 – 2 m and may be used in conjunction with an elephant’s foot or a sliding joint configuration as part of a steel support ring around the excavation. The locations of the steel sets are shown in the longitudinal cross-section in Figure 4.23. Steel set samples and the method of installation are depicted in Figure 4.27.

![Figure 4.27 Steel set installation as part of temporary tunnel support.](image)

The standard steel sets that were used for Driskos were HEB 160 sections. The dimensions and section properties associated with this section are shown in Table 4.5.
Table 4.5 Dimensions and sectional properties associated with standard HEB 160 steel sections (after Profil Arbed, 2005).

<table>
<thead>
<tr>
<th>Designation</th>
<th>Dimensions</th>
<th>Sectional Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>B x t</td>
<td>G kg/m</td>
</tr>
<tr>
<td></td>
<td>h mm</td>
<td>b mm</td>
</tr>
<tr>
<td></td>
<td>t_w mm</td>
<td>t_f mm</td>
</tr>
<tr>
<td></td>
<td>r mm</td>
<td>A cm²</td>
</tr>
<tr>
<td></td>
<td>I_y cm⁴</td>
<td>W_y cm³</td>
</tr>
<tr>
<td></td>
<td>y₁ cm</td>
<td>y₂ cm</td>
</tr>
<tr>
<td>HEB160</td>
<td>360x10</td>
<td>70.8</td>
</tr>
<tr>
<td></td>
<td>160</td>
<td>160</td>
</tr>
<tr>
<td></td>
<td>8.0</td>
<td>13.0</td>
</tr>
<tr>
<td></td>
<td>15.0</td>
<td>90.3</td>
</tr>
<tr>
<td></td>
<td>4058</td>
<td>356</td>
</tr>
<tr>
<td></td>
<td>5.6</td>
<td>11.4</td>
</tr>
</tbody>
</table>

Table: Designation: Section Plate; Dimensions: B x t kg/m h b t_w t_f r cm²; Sectional Properties: I_y cm⁴ W_y cm³ y₁ cm y₂ cm.

A – sectional area
G – weight per meter
I_y – moment of Inertia (strong axis)
W_y – elastic section modulus

4.7 Tunnelling Issues associated with Driskos Tunnel Construction

Prior to construction, it was predicted that Section 4 of the Driskos Tunnel would be the most problematic due to the weak rock masses within that section (Refer to Chapter 6). As such, the rock mass was designated as a Category IV-V and the initial design complimented this designation (Structural Design S.A., 1999).

However, the deformations that ensued due to tunnel excavation were greater than anticipated. As such, the tunnel temporary design was re-evaluated and modified after an extensive monitoring program by GeoData S.A. (GeoData S.A., 2003) and design recommendations by Panel of Expert reports (Seventh report by Panel of Experts Report, 2000).

The author visited the construction site and photos of the problematic section are shown in Figure 4.28. The identified problem was over stressing of the temporary / primary support that occurred at several locations during excavation and extended over distances of some of metres.
(approximately 10 m). This is typical of the response to be expected and is associated with large deformations due to a combination of high stress and low rock mass strength. As well, the strength of the siltstone-sandstone Flysch is reduced locally by the presence of a high concentration of horizontal bedding crossed by frequent faults. Also contributing to the excessive deformation is the presence of water that contributes to a reduction in rock mass strength.

The deformations that were observed did not exhibit a tendency to stabilise with time. The design, therefore underestimated the rock mass strength for this particular section and as a

**Figure 4.28** Excessive deformations experienced within Section 4 of Driskos Tunnel. (a) Deformed rockbolt face plate as a result of over stressing of the primary support, (b) Spalling of the shotcrete adjacent to a stressed steel set, (c) Sidewall of Driskos Tunnel denoting (in Red) the total amount of inward displacements at selected, monitored target points and (d) Clearer designation of inward displacement in millimetres.
result, the design capacity of the primary support was determined to be too low (Seventh Report by Panel of Experts Report, 2000).

A rigorous monitoring program was implemented, and selected results from this program are located in Appendix E of this thesis. Figure 4.29 is an example of the monitoring (survey) data that was collected on site. As a summary, the major observations associated with the monitoring program as well as their implications to the re-design of Section 4 of the Driskos twin tunnels is (Egnatia Odos, 2001):

a) During the top heading phase of excavation, 210 mm (maximum) of tunnel closure was exhibited by the primary / temporary support;

b) Upon excavation of the parallel bore, 310 mm (maximum) of displacement was captured;

c) Additional primary support measures included: closer spacing of steel set support sections, more and longer fully grouted rockbolts, thicker shotcrete shell, introduction of prestressed cable anchors and the introduction of an invert;

d) Micropiles were also included into the re-design of the temporary support. They were incorporated as a steel reinforced continuous concreted beam element that was laid at the base of the steel ribs along each sidewall;

e) Deformations were on average larger on the outer sidewalls indicating asymmetric tunnel closure behaviour. This was not predicted at the initial design phase for the primary support. More symmetric conditions were achieved after the modification of the sidewall rockbolt pattern was implemented. Another factor that influences this behaviour is a realistic evaluation of the stress field conditions (i.e. K).
Figure 4.29 Radial convergence as captured by survey data from Driskos Left Bore at Chainage 8+503.
4.8 Relevance to Proposed Research Program

This chapter has highlighted the method of stress analysis concerning an excavation, tunnelling techniques associated with weak rock masses and the specific design associated with the tunnels of the Egnatia Odos, chiefly the design of the Driskos tunnels. The modelling that has been undertaken in this investigation complements the Driskos design introduced in this section. The validation data that is used for the numerical modelling is from the robust monitoring program that was implemented during the excavation and monitoring (NATM observational method) of the Driskos twin tunnels. Specific issues concerning this tunnel excavation study have also been captured in the conference papers located in Appendix F and Appendix G as well as the journal papers that are contained within this thesis.

It must be noted that effort to produce the monitoring data to the stage at which they have been presented within this document was not a trivial undertaking. The data set that was obtained by the author from the National Technical University of Athens consisted of graphed data of individual survey points without the data base of numerical values attached to these graphs. The co-ordinates for every single data point that appears on these monitoring data figures (Appendix E) were entered individually by the author. Time and space alignments were determined as well as radial convergence of the tunnel bore(s). As mentioned above, this data was then used for validation purposes for the 3D numerical model that was utilized for this research.
Chapter 5

Numerical Modelling of Weak Rock Tunnelling

5.1 Introduction

The response of a rock mass to the creation of a tunnel cavity (i.e. sequential excavation such as that undertaken in NATM tunnelling) is not an easily defined problem. Many factors concerning the rock mass material must be taken into consideration: discontinuousness, anisotropy, inhomogeneity, inelasticity, intact pieces of rock may deform elastically or fracture, joint surfaces may be created, and individual blocks of rock may undergo rigid body displacements (Hoek, Grabinsky and Diederichs, 1990). Understanding the behaviour of rock masses during excavation requires an accounting of these potential modes and a thorough analysis of the stresses and displacements involved. Numerical modelling or simply modelling, is an analytical tool that attempts to capture this physical behaviour of a material by quantifying it using mathematical techniques (i.e. mathematical representation using numerical modelling).

Currently, the standard practice for analyzing the behaviour of tunnel excavation in rock masses is through the use of 2D plane strain finite element computations. Phase2 (Roecscience, 2004) is such a computation software package used for these purposes. Three-dimensional (3D) numerical analyses are considered sophisticated, costly and time consuming as model construction and input preparation is often complicated. As well, improved accuracy may be incompatible with the level of knowledge of ground conditions as determined through site investigations and tunnel monitoring. However, excavation of a tunnel is uniquely a 3D phenomenon and 3D modelling provides a more realistic appreciation of the tunnel face pre-
convergence. Simulating and capturing these 3D effects in 2D requires unique simplifications. The paper in Chapter 7 addresses aspects of these particular issues.

There are a wide variety of modelling approaches that are commonly used within this discipline. There are primarily four main modelling methods within modern practise. These can be seen in Figure 5.1 below.

Figure 5.1 Diagram showing the four basic methods (two levels each) and therefore, eight different modelling approaches (after Hudson (2001) as cited in Jing, 2003)

As seen in Figure 5.1, the four main modelling methods are comprised of:

a. Method A: Design based on previous design experience;

b. Method B: Design based on simplified methods;

c. Method C: Design based on attempts to capture relevant mechanisms; and

d. Method D: Design based on all-encompassing modelling

Level 1 indicates that there is a 1:1 mechanism mapping within the model (i.e. explicit stress-strain relationship). In Level 2, methods employ a mechanism mapping that is not direct (i.e. use of rock mass classification systems – GSI used in this research project is such an example). The method that is to be employed for a specific investigation is based on the unique
nature of that particular investigation. All techniques have their strengths and limitations. Numerical modelling techniques are valuable tools in analyzing and understanding the complex geomechanics of rock masses. The most commonly applied numerical methods for rock mechanics are summarized in the following sections.

5.1.1 Continuum Methods

Using these methods, the rock mass is treated as a continuum i.e. the input data for the strength and deformation properties are equal in all directions that define a given appropriate constitutive relation for the medium: often used are linear and non-linear elastic, elasto-plastic methods among others (Barla and Barla, 2000). Intact rock properties are scaled down to the rock mass properties by using empirically defined relationships (i.e. as those defined by Hoek and Brown). The most commonly applied numerical methods within continuum rock mechanics problems are the Finite Difference Method (FDM)(i.e. FLAC3D), the Finite Element Method (FEM) and the Boundary Element Method (BEM). In the continuum approach, there is no chance for a failure surface to be explicitly developed as the model elements or blocks cannot separate at the boundaries except on pre-defined boundaries or interfaces.

5.1.2 Discontinuum (Discrete) Methods

Using these methods, the rock mass is represented as a discontinuum and most of the detailed work at the design stage is associated with the characterization of the rock elements, the rock joints and discontinuities. The ‘blocky’ nature and interactions at the joints within the rock mass are considered. These discontinuities may play a key role in the response to tunnel excavation. The most commonly used applied numerical methods are the Discrete Element Method (DEM) and Discrete Fracture Network (DFN) methods. In the Discrete approach, elements are allowed to separate at boundary locations, however, the failure surface (or discontinuity) cannot propagate through the elements; these are still modeled as a continuum.
5.1.3 Continuum and Discontinuum (Hybrid) Methods

There are also methods that combine continuum and discontinuum methods. The choice of numerical technique will depend on the nature of the rock mass material, scale of investigation and problem-specific factors. For example, if the rock mass has many fractures, a discrete approach may be more valid. There are no absolute advantages of one method over the other. The key to the success of such a process is the level of understanding attained in describing the rock mass conditions (in terms of geological/geotechnical, in-situ stress and hydrogeological parameters) and the ability to accurately account for the fundamental components of rock mass behaviour. This is accomplished by using appropriate methods for analysis of stresses and displacements in the rock mass around the tunnel and in the structural components (pre-support measures, primary and final support) (Barla and Barla, 1999). The answers obtained by computer modelling are only as good as the geotechnical input parameters and the constituent models used for analysis. As stated previously, each modelling approach has its unique assumptions and limitations.

5.2 Numerical Modelling Techniques and Choice of 3D Numerical Model

All of the above-mentioned techniques are suitable for analyzing the complex nature of a geomaterial. Most of the numerical modelling packages for soil and rock mechanics are well suited to address the complex geometries of these materials and the various material models (constitutive models). The state of the software is such that one can conduct numerical experiments (parametric analysis) for design, thus developing a qualitative understanding based on quantitative evaluation. Numerical modelling can provide fundamental understanding of the behaviour of a geomaterial, however, these tools should not completely replace empirical design approaches as these have their merits (Jing, 2003).
Numerical models are commercially available to users in the form of computer codes or program packages. A numerical model code is capable of:

a. Solving the equations of equilibrium;
b. Satisfying the strain compatibility equations; and
c. Following certain constitutive models (equations).

All of these are balanced with a set of prescribed boundary conditions within a modelling framework. The numerical code will calculate displacements, changes in stress as well as other tensor quantities of interest.

The choice of the model will be based on the unique nature of the project that requires investigation. For this NATM (or observation method) tunnel excavation investigation in a weak rock material such as Flysch, the numerical project code requirements were as follows:

a. Capable of modelling in three dimensions (3D);
b. Capable of incorporating many constitutive models;
c. Allow non-linear analysis of the geomaterials;
d. Allow the analysis of large (memory) numerical models balanced with acceptable processing time for calculating equations of equilibrium;
e. Able to deal with complex geometries and robust tunnel support scheme;
f. Able to allow for staged excavation steps;
g. Able to access and change the software’s project code; and
h. Cost effective solution.

Balancing these requirements with the software available commercially, FLAC3D (a continuum method, finite difference software package) was clearly the program of choice. Not only does FLAC3D allow for the creation and analysis of complex geometries and configurations, it also incorporates a FISH function that allows users to manipulate FLAC3D’s project code. FISH enables the user to define new variables and functions and offers a unique capability to
users who wish to tailor analyses to suit their specific needs. This numerical modelling software package will be elaborated upon in the next section.

5.3 Numerical Modelling of Weak Rock Tunnelling - Methods of Analysis

Most of the numerical modelling packages for soil and rock mechanics are well suited to address the complex geometries of these materials and the various material and constitutive models. The current state of the commercially available software is such that one can conduct numerical experiments (parametric analysis) for design, thus developing a qualitative understanding based on quantitative evaluation. Numerical modelling can provide fundamental understanding of the behaviour of a geomaterial, however, these tools should not completely replace empirical design approaches as these have their merits (Jing, 2003).

For the purposes of this investigation, Phase2 (Rocscience Inc., 2004) was used for the 2D numerical analysis and FLAC3D (Itasca, 2005) was used for the 3D numerical analysis. Phase2 utilizes the implicit Finite Element Method (FEM)(i.e. solves the mathematical relations) while FLAC3D employs the Finite Difference Method (FDM) (i.e. solves the physics of the problem) in its determinations. Both of these program are widely used in the rock mechanics community for design purposes as well as to capture the behaviour of a tunnel (i.e. stress redistributions and displacements) associated with tunnel excavation. Cai (2008) has investigated the numerical modelling codes for Phase2 and FLAC (2D) (the basis of FLAC3D) on the influence of stress path on tunnel excavation response and these findings will not be repeated here. Chai stated that one software package was not superior to the other, rather he points out the importance of understanding the program codes and selecting the right tool and modelling approach to represent the expected stress path as close to reality as possible. The emphasis in comparison therefore, should not lie in the limitations of the software packages but on the details of how the true physical phenomenon is being modeled within these limitations. The next
sections will introduce the numerical modelling software packages that were utilized in this research.

5.4 Three-Dimensional (3D) Modelling Software Package used in this Investigation

5.4.1 Three-Dimensional (3D) Modelling Software Package FLAC3D

FLAC3D (Itasca, 2005) is an explicit finite difference program that is used to study the mechanical behaviour of a continuous three-dimensional medium as it reaches equilibrium or steady plastic flow. The response observed is derived from a particular mathematical model and from a specific numerical implementation. The mechanics of the medium are derived from general principles (definition of strain, laws of motion), and the use of constitutive equations defining the material. The resulting mathematical expression is a set of partial differential equations, relating mechanical (stress) and kinematic (strain rate, velocity) variables, which are to be solved for particular geometries and properties, given specific boundary and initial conditions. It is the inertial terms that are used as means to reach (in a numerically stable fashion) the equilibrium state. The solid body is divided into a finite difference mesh of 3D group zones. Within a time step calculation cycle, new velocities and displacements are determined from forces and stresses using the equations of motion (Figure 5.2). From these, strain rates are determined from velocities and new stresses from the strain rates. This calculation process is then repeated in each time step.
The main components of the resolution of forces in terms of static equilibrium are formulated on the basis of the equations of motion and static equilibrium. Forces take the form of \( F = ma \):

\[
F(t) = m \times \frac{d\mathbf{u}}{dt}
\]

Eq’n 5.1

where, \( m \) the mass of body;

\( F \) the applied in the solid body force; and

\[
\frac{d\mathbf{u}}{dt}
\]  

the acceleration of mass.

The laws of motion for the continuum are then transformed into discrete forms of Newton’s Law (\( F = ma \)) at the nodes. The resulting system of ordinary differential equations is then solved using an explicit finite difference time-step formulation. Strain rates are defined in terms of velocities and these velocity variations and corresponding space intervals are incorporated within

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**Figure 5.2** Basic explicit calculation cycle for FLAC3D
a discretized medium of constant strain rate elements of tetrahedral shape whose vertices are composed of nodes.

The finite difference equations must be solved within a stable numerical scheme. As such, the nodes within a medium are connected using a spring-mass system. Tolerances are applied in order to ensure stability of this system at each time-step. A simplistic mass-spring system is shown in Figure 5.3.

![Figure 5.3 Idealized form of F = ma using mass and spring analogy. Note that u = displacement, and \( \ddot{u} \) denotes acceleration.](image)

A detailed explanation of the formulation of the 3D explicit finite difference model determination is described fully in the Theory and Background Section of the FLAC3D manual set and will not be repeated here (Itasca, 2005).

It is the inertial terms that are used as a means to reach (in a numerically stable fashion) the equilibrium state. The advantage of using the explicit finite difference formulation is that the numerical scheme stays stable even when the physical system may be unstable. This is particularly advantageous, when modelling non-linear, large strain behavior and actual instability. However, the disadvantage of the time-marching explicit scheme of FLAC3D is that calculation times can be longer than those of implicit formulations; this is balanced against the memory gain from explicit methods that do not have to store multiple arrays of matrices for calculations of equilibrium.
FLAC3D also checks the element state at each time step with respect to the yield criterion. This is conducted for elasticity or plasticity analysis.

The manufacturer of the software package, Itasca, summarizes its unique capabilities succinctly as follows:

“The basis for this program is the well-established numerical formulation used by our two-dimensional program, FLAC.* FLAC3D extends the analysis capability of FLAC into three dimensions, simulating the behavior of three-dimensional structures built of soil, rock or other materials that undergo plastic flow when their yield limits are reached. Materials are represented by polyhedral elements within a three-dimensional grid that is adjusted by the user to fit the shape of the object to be modeled. Each element behaves according to a prescribed linear or nonlinear stress/strain law in response to applied forces or boundary restraints. The material can yield and flow, and the grid can deform (in large-strain mode) and move with the material that is represented. The explicit, Lagrangian, calculation scheme and the mixed-discretization zoning technique used in FLAC3D ensure that plastic collapse and flow are modeled very accurately. Because no matrices are formed, large three-dimensional calculations can be made without excessive memory requirements. The drawbacks of the explicit formulation (i.e., small timestep limitation and the question of required damping) are overcome by automatic inertia scaling and automatic damping that does not influence the mode of failure. FLAC3D offers an ideal analysis tool for solution of three-dimensional problems in geotechnical engineering.”

(Itasca, 2002)

FLAC3D was created primarily for investigations relating to geotechnical engineering. As such, it offers a wide range of capabilities to solve complex problems in geomechanics. The software package incorporates unique numerical representations for the mechanical response of geomaterials. The program has eleven basic built-in material models that are listed in the section following. Each zone in a FLAC3D grid may be assigned a different material property, and a continuous gradient or statistical distribution of any property may be specified (Itasca, 2002).
5.4.2 FLAC3D Constitutive Models - Software

The program has eleven basic built-in material models (Itasca, 2002):

a. The Null Model; (is used to represent material that is removed or excavated)

b. Three (3) Elasticity Models:
   i. Isotropic - is valid for homogeneous, isotropic, continuous materials that exhibit linear stress-strain behaviour;
   ii. Transversely Isotropic - simulated layered elastic media; and,
   iii. Orthotropic Elasticity - material with three mutually perpendicular planes of elastic symmetry;

c. Seven (7) Plasticity Models:
   i. Drucker-Prager Model - model soft clays with low friction angles; not generally recommended for application to geologic materials. Included mainly to permit comparison with other numerical program results;
   ii. Mohr-Coulomb Model - conventional model used to represent shear failure in soils and rocks. (i.e. Mohr-Coulomb failure criterion);
   iii. Ubiquitous-Joint Model - an anisotropic plasticity model that includes weak planes of specific orientation embedded in a Mohr-Coulomb solid;
   iv. Strain-Hardening/Softening Model - representation of nonlinear material softening and hardening behaviour based on prescribed variations of the Mohr-Coulomb model properties (cohesion, friction, dilation, tensile strength) as functions of the deviatoric plastic strain.
   v. Bilinear Strain-Hardening/Softening Ubiquitous-Joint Model - allows representation of material softening and hardening behaviour for the matrix and the weak plane based on prescribed variations of the ubiquitous-joint model properties (cohesion, friction, dilation, tensile strength) as functions of deviatoric and tensile plastic strain.
   vi. Double-Yield Model - represents materials in which there may be significant irreversible compaction in addition to shear yielding; and
vii. Modified Cam-Clay Model. – representation of materials when the influence of volume change on bulk property and resistance to shear need to be taken into consideration (i.e. soft clay).

5.4.3 Constitutive Models – Research Investigation

The constitutive models that were employed during this investigation were Mohr-Coulomb and Strain-Softening plasticity models. The details of how these models are implemented within FLAC3D software are outlined in the FLAC3D manual, Theory and Background (Itasca, 2005). Below is a synopsis of the key principles associated with each plastic model group.

5.4.3.1 Mohr-Coulomb (Elastic, Perfectly Plastic) Model

The Mohr-Coulomb constitutive model or failure condition represents a combination of cohesive and frictional effects that define the strength characteristics of a material. It is the most common failure criterion utilized within the geotechnical engineering field. The Mohr-Coulomb criterion describes a linear relationship between normal and shear stresses (or maximum and minimum principal stresses) at failure (Figure 5.4). Stress states are defined in three dimensions as $\sigma_1 < \sigma_2 < \sigma_3$. The intermediate principal stress has no effect on yield. Failure is said to occur when the Mohr circle (or outer combination of stress limit) becomes just tangent with the failure locus.

![Figure 5.4 Mohr-Coulomb, elastic, perfectly plastic behaviour](image-url)
Many geotechnical computer modelling programs use the Mohr-Coulomb model as part of their strength model. This is why there is a unique requirement to obtain Mohr-Coulomb input parameters when conducting analyses using such software (i.e. obtain equivalent Mohr-Coulomb parameters for non-linear failure envelopes from other failure criteria). This was discussed in Chapter 3 of this document. Within FLAC3D, the Mohr-Coulomb failure envelope is modelled with a shear yield function and a tension cutoff. The position of a particular stress point on this envelope is controlled by a non-associated flow rule for shear failure and an associated rule for tension failure (Itasca, 2005).

5.4.3.2 Strain Softening (Plasticity) Model

The strain softening constitutive model or failure condition represents a stress strain behaviour profile whereby residual strength is exhibited after the peak strength of the material has been reached. In one particular strain softening instance, the initial behaviour can be elastic, the peak strength is reached and an immediate (or percent strain) reduction of the strength is exhibited yielding a residual strength as seen in Figure 5.5.

![Figure 5.5 Strain softening – elasto-plastic behaviour](image)

Within FLAC3D, the strain softening model is based on the Mohr-Coulomb model and therefore, the non-associated shear and associated tension flow rules are still applied to the strain softening model. However, parameters such as strength, cohesion, friction and dilation can be
assigned residual values by the user. This model is ideal for materials which demonstrate a reduction in shear strength when loaded beyond their peak strengths or for other parameters, their initial failure limit.

5.4.4 Structural Elements used in this Investigation

5.4.4.1 PileSELs

PileSELs or pile structural elements are two-noded, straight finite elements with six degrees-of-freedom per node. The stiffness matrix of a PileSEL is identical to that of a beam element. Shear-directed (parallel with the pile’s axis) frictional interaction occurs between the pile and the grid. In addition to skin-friction effects, end-bearing effects can also be modelled. Piles may be loaded by point or distributed loads and are used to model structural-support members, such as forepoles and in other applications, foundation piles, for which both normal- and shear-directed frictional interaction with the rock mass occurs (Itasca, 2005). Further information concerning PileSELs used in this investigation is located in Table A2 in Appendix A.

5.4.4.2 LinerSELs

LinerSELs or liner structural elements are three-noded, flat finite elements that can be attached to the surface of the *FLAC3D* grid. A shear-directed (in the tangent plane to the liner surface) frictional interaction occurs between the liner and the *FLAC3D* grid. Also, in the normal direction, both compressive and tensile forces can be carried, and the liner may break free from and come back into contact with the grid. LinerSELs are used to model thin liners for which both normal-directed compressive/tensile interaction and shear-directed frictional interaction with the geo-medium occurs, such as shotcrete-lined tunnels. The localized co-ordinate system and relevant parameters associated with the liner element are show in Figure 5.6 (Itasca, 2005).
These co-ordinate systems proved to be useful in determining how to extract moments, shear and normal forces from the liner elements.

Figure 5.6 Bending in a liner element depicting (a) stresses acting on a differential element of a homogeneous, linearly elastic plate subjected to plate-bending loading, (b) stress resultants corresponding with these stresses. Also membrane action in a liner depicting (c) stresses that act on a differential element of a homogeneous, linearly elastic plate subjected to plane-stress loading and (d) stress resultants corresponding with these stresses. (note that all stress resultants are drawn acting in their positive sense)(modified after Itasca, 2005).
5.4.5 Interpretation of the FLAC3D Model Results

FLAC3D models evoke a non-linear system over time due to the method’s unique step-wise solution at time-step. Interpretations of results from such analysis may be an onerous task. As such, FLAC3D provides indicators to assess the model state as to whether the system is stable, unstable, or in steady-state plastic flow. In terms of the FLAC3D model used in this research project, Appendix A also comments upon these indicators within the troubleshooting section. Also provided in this appendix are other common “checks” in order to determine if the numerical model is precise and accurate. These indicators of unbalanced force, grid point velocities, plastic indicators and histories are described below (Badr, 2004).

5.4.5.1 Unbalanced Force

Each grid point in a model is surrounded by up to eight zones that allocate forces to that grid point. At equilibrium, the sum of these forces is approximated to be zero (i.e. the forces acting on one side of the grid point virtually balance those acting on the opposite side). Unbalanced forces approaching a constant non-zero value indicate elastic equilibrium (and/or plastic flow) occurring within the model. An extremely low value of unbalanced (i.e. virtually zero) forces indicates that forces balance at all grid points. Nonetheless, steady plastic flow may be occurring, without acceleration. In order to distinguish between these two conditions and actual equilibrium achieved, other indicators included in this section must be examined. For the model that was created for this investigation (refer to Appendix A) the unbalanced force was constantly monitored in order to ensure that the time-steps needed to achieve equilibrium were sufficient and that the unbalanced force approached zero.
5.4.5.2 Grid Point Velocities

The grid point velocities are assessed by plotting out the field of velocities at certain key locations within one’s numerical model. Conversely, one may select key points within the grid and tracking their velocities using histories (described in the following history section). Steady-state conditions are indicated if the velocity histories show horizontal traces in their final stages. If they have all converged to near zero value with respect to their starting values, then the equilibrium condition has been met. If a grid point velocity has converged to a non-zero value, then steady plastic flow is occurring at that grid point. If velocity history plots depict fluctuating velocities, then the system is likely in a transient condition. In order to confirm that continuing plastic flow is occurring, a plot of plasticity indicators should be conducted.

5.4.5.3 Plastic Indicators

FLAC3D can display zones of plasticity (for plasticity models only) in which the stresses satisfy the yield criterion. Such an indication usually denotes that plastic flow is occurring. It is possible, however, for an element to sit on a yield surface without any significant flow taking place. It is therefore important to look at the whole pattern of plasticity indicators to see if a mechanism of failure has developed. Two types of failure mechanisms are indicated by the plasticity state: a) shear failure and b) extensional failure. If a condition of continuing plastic flow has been diagnosed at an artificial boundary (boundary that limits the size of the grid) then the solution is not realistic because the mechanism of failure has been influenced by a non-physical entity (i.e. the induced boundary condition that is artificial in nature). This only applies to the final steady-state solution as intermediate stages may exhibit flow along boundaries.
5.4.5.4 History Plots

Points of interest and their associated variables (i.e. stress, moments, displacements etc.) can be captured within a FLAC3D model using the HISTORY command. These values can then be analyzed in order to determine if the model is “behaving” properly. History plots can be created within FLAC3D (and exported) in order to determine what is happening within the model simulation. This helps the user troubleshoot the model and make adjustments based on sound data interpretation.

5.5 Phase2 (2D) Numerical Modelling Software Package

Phase2 is a 2D, implicit, elasto-plastic finite element method program used to calculate stresses and determine displacements around underground tunnels and can be used to solve a broad range of geotechnical problems. Phase2 uses plane strain analysis whereby two principal in-situ stresses are in the plane of the excavation and the third principal stress is out of plane. As with other finite element regimes, the domain is discretized into a set number of elements and corresponding nodes. Displacements within these finite elements are calculated based on shape functions tied to the nodes of the elements (Rocscience Inc., 2004). Initial in situ stresses, tolerance parameters, material and defect properties are all assigned by the user. Tunnel excavation is simulated by the removal of elements from within an excavation boundary located in an external boundary.

In terms of solution process, convergence and stopping criterion, matrix formulations of algebraic linear equations are used to solve for differential equations. In a simple example, a stiffness matrix relates loads to displacements. Gaussian elimination is used to solve for the system of equations. Criteria for equilibrium are based on the absolute and square root energy criterion as well as force and displacement criterion. These criteria verify that, for a given load step, iterations are ceased when the energy imbalance of the current state becomes a small
fraction of the initial energy imbalance (i.e. energy imbalance on the first iteration). If this condition is not satisfied during a specified maximum number of iterations, the solution process is deemed not to have converged. In terms of force and displacement, iterations are halted when the current force imbalance becomes a small fraction of the total applied force (i.e. current load level) or initial displacement respectively. Plasticity analysis also has its yield criteria. Again, iterative procedures of static equilibrium are used in order to relate the current stress state to that of an elastic solution. If the stress state is in excess of the yield criteria, plastic deformation has taken place (Rocscience Inc., 2004).

5.6 Modelling of Tunnel Excavation

Currently, the standard practice for analyzing the behaviour of tunnel excavation in rock masses is through the use of 2D plane strain finite element computations. Phase2 (Rocscience Inc., 2004) is such a computation software package used for these purposes. Three-dimensional (3D) numerical analyses are considered sophisticated, costly and time consuming as model construction and input preparation is often complicated. As well, improved accuracy may be incompatible with the level of knowledge of ground conditions as determined through site investigations and tunnel monitoring. However, excavation of a tunnel is uniquely a 3D phenomenon and 3D modelling provides a more realistic appreciation of the tunnel face pre-convergence. Simulating and capturing these 3D effects in pseudo-3D (i.e. using 2D models to simulate 3D) requires unique simplifications to account for the real phenomenon of behaviour when dealing with high or isotropic stresses (Chapter 8).

5.6.1 2D vs 3D Numerical Modelling

As the effect of an excavation in a rock mass is clearly a 3D phenomenon, the ensuing deformations cannot be simulated directly in 2D finite element (FE) analysis. Theoretically, 2D, axi-symmetric modelling does replicate 3D effects for very simple cases, however it has its
limitations; in high stress fields yielding large plastic zones around the tunnel excavation (i.e. plastic zone is greater than twice the diameter of the tunnel opening) this technique does not captured the true, observed displacements (Vlachopoulos and Diederichs, 2009). Therefore, in a 2D plane strain finite element analysis, one must make assumptions as to the behaviour of the rock mass and introduction of temporary support during the excavation process. Explicit 3D formulations can generate stress paths that represent true stress paths, however, implicit 2D formulations cannot. In implicit formulations only the final equilibrium state results can be captured. In order to compensate for this, conventional approaches or methodologies have been created. This reality is highlighted in Figure 5.7.

Figure 5.7. A 3D model depicting the fact that the 3D model captures the 3D effects at the face, however, the 2D model can only analyze a plane in space with respect to a position relative to the face (i.e. cross-sections A-A and B-B0). The transition in 2D from unexcavated to excavated tunnel is a non-trivial issue. There requirement to develop 2D analogues to a fundamentally 3D phenomenon. One must take into consideration the strength issues provided by the face that are associated with successive excavation in a 2D sense.
If one was to observe a 2D cross-section of the 3D model at A-A, the material is undisturbed and can be sufficiently modeled in 2D as well. At cross-section B-B at a distance from the face (i.e. open tunnel cavity) a 2D analysis at this location is also a non-trivial issue. The complexities occur at the face of the excavation whereby the face is also providing support to the tunnel excavation. A transition from unexcavated rock prior to the face to an open cavity past the face clearly requires a ‘simplification’ in 2D. In this way, 2D analysis must take into account where the analysis is taking place with respect to the face. This process has not been effectively optimized. The methodologies commonly employed in current design practice for 2D modelling to mimic real 3D effects are (Figure 5.7):

a. Straight Excavation,
b. Field Stress Vector / Average Pressure Reduction,
c. Excavation of Concentric Rings, and
d. Face De-stressing.

Karakus et al. (2006) also cites other methods that are employed in this regard. Each of these techniques have been well documented and will not be discussed here in detail. These techniques and concepts have been summarized in Chapter 8 of this document.

5.6.2 Effect of GSI (Rock Strength) on Tunnelling Model

The choice of properties that one assigns to the materials within a model will clearly influence the behaviour of the numerical model. It is therefore imperative to obtain realistic strength values as input parameters to numerical models. In the axisymmetric, 2D finite element (Phase2) simulation that was conducted in Figure 5.8, one can clearly see the effects of varying the GSI of a rock mass. On one side of the tunnel, a GSI of 35 was used, while on the other side of the tunnel a GSI of 45 was assigned. A larger plastic zone of 3 diameters was associated with the lower GSI value than with the 2 diameter plastic zone that was created by the higher GSI designation. This has implications to the expected tunnel displacements and ultimately to the
design of the temporary support for the excavation. The effect of the size of the plastic zone is the topic of the next section.

![Diagram of tunnel behavior](image)

**Figure 5.8.** Yield related closure (no gravity) of two unsupported tunnels at 300m depth: axisymmetric FEM analysis (grid distortion x 10).

### 5.6.3 Effect of Plastic Zone on Tunnel Behaviour

The bullet-shaped plastic zone (**Figure 5.9**) is a phenomenon that has been recorded in the field. It has been observed that there is an influencing effect that that a large ultimate plastic radius has on the rate of development of wall displacements with respect to location along the tunnel (**Chapter 7**). These concepts have been introduced and expanded upon in Vlachopoulos and Diederichs, 2009.
Figure 5.9 A plastic yield zone (shaded, ‘bullet’-shaped zone) developing as tunnel advances to the right from axisymmetric FEM analysis. If the wall yield zone is more than double the tunnel radius (a), it interacts with face yield zone, however, if the maximum plastic zone radius is less than twice the tunnel radius (b), the wall yield zone does not interact with the face yield zone.

5.7 Numerical Models Used in this Investigation

5.7.1 FLAC3D Model

The requirements of the numerical model were highlighted in Section 5.2 of this chapter. These requirements were also balanced against the need to simulate (in a numerical sense) the NATM (or observational method) excavation practices at the Driskos twin tunnel site within a weak rock mass. The FLAC3D model that was created for this investigation is a 3D model incorporating twin tunnel geometry with sequential excavation and support. The full FLAC3D project code and descriptions of this numerical model are located in Appendix A. Note that these files act as a template as to the organization of the numerical model and may not reflect the official numerical runs that were conducted as part of this investigation; these have been submitted to the author’s thesis advisor, Dr. Mark Diederichs, in electronic form for archival purposes.
In a general sense, the numerical model that was created uses FLAC3D GROUP zones in order to create a twin tunnel arrangement embedded within an outer abutment. The material within the tunnels that is excavated was created separately and was subdivided into sub-sections that constituted an excavation step. A forepole support umbrella was installed and allowed to reach equilibrium prior to each excavation step as per the support and excavation steps conducted in the field. At each excavation step, an excavation sub-section block was nulled and steps towards equilibrium were conducted. Following the excavation step, support was added to the tunnel system similar to that of the Driskos support (i.e. shotcrete liner (that incorporated the HEB steel sets) and rockbolts). The excavation of the top heading was conducted first followed by the excavation of the bench material for both, the initial bore and for the twin bore.

The runs that were conducted consisted of supported and unsupported simulations with Mohr-Coulomb and strain softening constitutive models. Input parameters were obtained from the field and Mohr-Coulomb equivalent properties were obtained as described in Chapter 3.

The average model characteristics / statistics per 3D numerical run are as follows:

a. Saved Stage Memory: 407 MB
b. Days to complete entire Supported Run: 3.5 days on dual core computer
c. Number of Grid Points: 132 367
d. Number of Nodes: 35 484
e. Number of Zones: 122 400
f. Number of Structural Elements: 38 440
g. Number of Cycles 127 000

Figures 5.10-5.13 depict various screen captures of the FLAC3D model that was created specifically for this investigation. Appendix A has all the files and specific details associated with the model as well as other model details.
Figure 5.10  Various screen captures detailing (a) the geometry of the twin tunnels, and (b) sequential excavation of tunnels as material to be excavated was subdivided into excavation increments.

Figure 5.11  Various screen captures detailing (a) twin tunnel with first tunnel fully excavated and top heading completed on twin tunnel, (b) plastic yield zone of single tunnel, and tunnel support detail showing tunnel liner (c) and forepoles and rockbolts (d) (material surrounding the support is not depicted).
Figure 5.12 Temporary support detail in relation to support used at Driskos Tunnel Site.

Figure 5.13 Working FLAC3D model sequence and associated FISH files
Figure 5.13 outlines the files that were created by the author in order to construct, manage and run the various 3D numerical models that were used within this investigation. A full explanation is provided in Appendix A.

5.7.2 Phase2 2D Numerical Model

Nominally identical 2D FEM numerical models were developed in order to complement the robust 3D numerical model that was created. The 2D numerical simulations were used in conjunction with the 3D runs that were conducted. The Phase2 interface made it easy to create geometries, optimize meshing, establish boundary conditions, assign material properties, add support and stage the excavation sequence. Selected geometries and results from analyses are depicted in Figures 5.14 – 5.15.

Figure 5.14. Finite Element (FE) mesh of circular tunnel excavation, (a) plane strain and (b) axisymmetric analysis. (c) detail of horseshoe and circular excavation boundaries using 6-noded elements and (d) boundary conditions associated with single, circular tunnel analysis.
Figure 5.15. Geometry, support and selected results of the Phase2 finite element model depicting (a) twin tunnel geometry of horseshoes with invert, (b) supported horseshoe with sequential excavation (1 top heading and 2 bench and invert), and plasticity results surrounding unsupported (c) and supported (d) tunnel excavation.
5.8 Relevance to Research

All of the previous chapters of this document are directly applicable to the modelling portion of this research program as modelling constituted a large portion of the research. In order to accurately ‘define’ a rock mass, one must have an understanding of its physical composition (geology), geologic and stress history (tectonics), and the relevant mechanisms and constitutive laws (and associated variable and parameters) of the rock mass. It is these features that must be accurately captured within a numerical model. There are a certain amount of subjective judgements and assumptions that must be made – which can be based on experience, long-term practices and a solid scientific foundation (Jing, 2003). This research project has tried to accurately model the behaviours that have been noted in the field (as validated by the monitoring data from the field) based on proper use of relevant constitutive relationships and modelling methods combined with the most accurate assessment of the parameters associated with the rock mass and support elements.

Specifically within this project, the 3D numerical model was able to accurately capture three-dimensional (3D) ground reaction considerations for numerical analysis and design purposes of the near face tunnel support issues in weak rock masses, and the weak material behaviour during tunnel excavation. It was through the assessment of the 3D model behaviour combined with the validation data from the field that allowed for the further assessment of longitudinal displacement profile investigations and 2D versus 3D considerations.
Tunnel Behaviour associated with the Weak Alpine Rock Masses of the Egnatia Odos, Driskos Twin Tunnel System

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ABSTRACT: Based on the excessive deformations and support failure encountered during tunnel construction at the Driskos Twin Tunnel site in Northern Greece, this paper provides insight on how tunnels designed in such weak rock environments can be realistically analyzed with a view of determining better analytical tools to predict deformations and improving current design methods. Specific factors that were assessed include, rock strength based on Geological Strength Index (GSI), tunnel deformation, numerical analysis techniques employed, 3D model type, support considerations, dilation, sequencing of tunnel excavation, influence of single bore construction on twin bore and homogenization of tunnel faces. This work involves the use of nominally identical 2D and 3D numerical models of tunnel sequencing for analytical simulation of weak material behaviour and sequential tunnel deformation response with a goal of investigating the strength and deformation of such weak rock masses. These have been used in combination with monitoring data that was obtained in the field during the Driskos Twin Tunnel construction. A discussion of the geological conditions, material property determination, monitoring data and the model calibration strategy is given. This paper provides insight into these issues and poses many more fundamental questions regarding the analysis of tunnel excavation within weak rock masses requiring further investigation.

Key Words: Weak rock masses, tunnel convergence, linear displacement profile (LDP), 2D and 3D numerical modelling techniques for tunnelling.
1.0 INTRODUCTION

Current practice in designing temporary support for tunnel construction in weak rock masses is contingent on a suitable assessment of the rock mass (quality and strength) that is to be encountered throughout the tunnel alignment. The most widely used criteria for estimating rock mass properties is that presented by Hoek and Brown (1997). In tunnelling through weak heterogeneous rock masses such as those of Flysch, it is important to obtain reliable strength estimates of rock materials in order to predict potential tunnelling problems as early as possible in the design process. This is a non-trivial undertaking.

A rock mass classification system framework is also required. The Geological Strength Index (GSI) was used for this investigation (Hoek, 1997; Hoek, Kaiser and Bawden, 1995). This classification system is based on an estimation of the rock mass strength in varying geological conditions and on the rock mass properties proposed by Hoek and Brown (1997). The main criteria associated with the GSI classification system is a detailed engineering geology description of the rock mass that is qualitative in nature. This grew out of the notion that numbers on joints were largely meaningless for the weak and complex rock masses (Marinos et al., 2006). This index is based upon an assessment of the structure, lithology and condition of discontinuity surfaces in the rock mass. The GSI value of a rock mass is incorporated into calculations in order to determine the reduction in the strength of the rock mass as compared with the strength of the intact rock components. An interpretation of GSI (or the applicability of the system itself) for a particular rock mass may vary significantly amongst geologists and tunnel designers.

As well, an assessment of the size of the final plastic zone around a tunnel cavity and its ensuing influence on tunnel deformations has only recently been incorporated into industry standard, 2D design analysis (Vlachopoulos and Diederichs, 2009). Observational design methods have been successfully applied to difficult tunnel conditions (Grasso et al., 2003),
however, these are usually applied only after excessive deformations have been observed in the field.

The backdrop for this research paper is based on the tunnelling that has recently been completed in the Epirus and Western Macedonia regions of Northern Greece, as part of the massive Egnatia Odos Highway construction project. Due to the difficult geological conditions and weak rock masses that were encountered during the construction of the 4.5 km long Driskos Twin Tunnels, excessive deformations and temporary support failures were experienced at various sections of the tunnel alignment during tunnel production. In this way, the Driskos twin tunnel project provides an excellent case study for analyzing excessive tunnel deformations within weak rock masses. Accurate equivalent rock mass performance predictions for tunnels in these materials (including yield and residual strength as well as flow and dilation considerations) is complicated by geomorphologic peculiarities (mixed face conditions, variable orientation or rock masses and structure) such as Flysch materials.

2.0 CASE STUDY: DRISKOS TWIN TUNNEL, EGNATIA ODOS, GREECE

The 670 km long Egnatia Odos Highway is a massive construction project that is currently under construction in Northern Greece in order to open up new, modern and safe roads connecting the countries of the European Union, the Balkans and the Middle East. The highway includes 77 twin tunnels and over 600 bridges along the alignment. The motorway was designed to the specifications of the Trans-European network. Due to the geological setting, many geotechnically unfavourable characteristics were encountered within the Egnatia Odos alignment. The great variety of geological / geotechnical situations imposes the need for different approaches in designing the various components of the highway (Marinos and Hoek, 2001). A geotechnical rock mass model must be defined in order to choose the appropriate geotechnical parameters for the design of cuts, embankments and tunnels. The next section will outline the main geological conditions that were encountered by the construction at the Driskos Twin Tunnel site and
introduce the main geotechnical and rock mass assessment framework that were utilized for this project.

2.1 Geological Environment

The overall geology of Greece and that of the Alpine region has traditionally been described in terms of isopic zones and massifs. These zones are groups of widespread rocks that have shared a common history, both in the ancient environments of deposition of sediments (Greece was a shallow, oxygen rich sea during most of the Triassic, Jurassic, Cretaceous and later) and their faulting and folding. The massifs of metamorphic and plutonic rocks are more resistant to folding and faulting than adjacent sediments. Approximately two thirds of Greece is covered with tectonized limestone. Heterogeneous rock masses, such as Flysch (a tectonically reworked clastic mix), are also abundant. Greece’s geology is still very active as it is located on a converging plate rim between the European and African plate. Figure 1 summarizes the tectonic zones and massifs along the Egnatia corridor.

Figure 1. Tectonic zones and massifs of the Aegean region. These zones are separated by major NW-SE striking thrusts on the Greek mainland and the Ionian Islands (Higgins et al., 1996).
Egnatia Odos traverses the entire width of Greece, crossing almost perpendicularly all the main geotectonic units. Each zone presents unique engineering geology challenges for construction and tunnelling and these have been summarized by Vlachopoulos & Diederichs, 2003 and Marinos and Hoek, 2001. **Figure 2** depicts the typical topography within North-Western Greece and tunnelling works as part of Egnatia Odos.

**Figure 2.** Portals of Tunnels of Egnatia Odos depicting the nature of the challenging geological conditions.

The Driskos Tunnel is situated in a series of varying lithological features of the Ionian tectonic unit adjacent to the Pindos isopic unit. The material is less tectonically disturbed than the Pindos Flysch and therefore, there is an absence of extensive chaotic zones within the Ionian Flysch. The primary rock mass material consists of Flysch; a material that consists of thin to medium bedded alternations of Siltstones and Sandstones (SiSa) as well as medium to thick-bedded Sandstones with interbeded Siltstones (SaSi).

The main lithological formations that were encountered during the excavation of the Driskos Tunnels are as follows (Egantia Odos, 2003):
a. Siltstones with thinly bedded Sandstones (< 10 cm) (Si)

b. Thin to medium bedded alternations of Siltstones and Sandstones (SiSa)

c. Medium to thick-bedded Sandstones with interbeded Siltstones (SaSi)

d. Thick-bedded Sandstones with alternations of thin bedded Siltstones (Sa)

e. Conglomerates (Fc/SiSa)

**Figure 3.** (a) Examples of thin-bedded alternations of Siltstones and Sandstones from Mt. Driskos region (folding of the formations is evident in the right hand photo) (b) Medium to thick-bedded Sandstones and thin-bedded Siltstones. Also evident are angular folds, shear zones along the bedding planes and the fracturing (faults) along axial plains (c) Thick bedded Sandstones with alternations of thin bedded Siltstones.

Selected examples of these rock masses are shown in Figure 3. In order to more effectively study the various formations and rock masses that were encountered by the Driskos tunnel alignment, an idealized Driskos Cross-Section was fabricated and can be seen in Figure 4.

**Figure 4.** Longitudinal topographic profile and idealized cross-section of Driskos Tunnel alignment depicting the rock formations that have been traversed.
The division of these sections was based primarily on the variations in the rock mass material or any geologic discontinuities (upright bedding planes, faults, major joints, degree of tectonism etc.) that differed in adjacent sections along the Driskos cross-section.

2.2 Rock Mass Type and Parameters

In tunnelling through weak heterogeneous rock masses such as those found in Greece, it is important to obtain reliable strength estimates of this material in order to predict potential tunnelling problems as early as possible in the design process. These parameters must be incorporated into an overall rock mass criteria framework (i.e. Hoek-Brown rock mass characterization tool) which are also part of a well-defined rock mass characterization system (i.e GSI for weak heterogeneous rock masses).

Currently, the most widely used criteria for estimating rock mass properties is that presented by Hoek and Brown (1997), and updated by Hoek, Carranza-Torres & Corkum (2002). This generalized Hoek-Brown criterion for intact rock samples approximates the non-linear relationship between maximum axial stress $\sigma_1$ that can be sustained by the sample and the applied confining stress $\sigma_3$. In its generalized form, the following parabolic law defines this relationship:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left( m_b \frac{\sigma_3'}{\sigma_{ci}} + s \right)^a \coth \left( \frac{\sigma_1'}{\sigma_3'} \right)$$

Eq’n 1

where, $\sigma_{ci}$ and $\sigma_3'$ are the maximum and minimum effective stresses at failure respectively, $\sigma_{ci}$ is the uniaxial compressive strength of the intact rock pieces, $m_b$ (or $m_i$ for intact strength) is a Hoek-Brown constant and a parameter deduced from $\sigma_1'$ and $\sigma_3'$ test results of a particular rock type. Constants $s$ and $a$ are unique to the rock mass and are based upon the specific rock mass characteristics.
The Hoek-Brown criterion was the primary characterization tool used for this investigation. As cited above, three parameters are required in order to estimate the strength and deformation properties: the uniaxial compressive strength \( (\sigma_{ci}) \) of the “intact” rock elements; a constant \( m \) that define the frictional characteristics of the rock; and the Geological Strength Index (GSI). GSI was introduced by Hoek et al. (1995), Hoek & Brown (1997) and extended by Hoek et al. (1998) and Hoek et al. (2002). GSI is based upon an assessment of the structure, lithology and condition of discontinuity surfaces in the rock mass. The GSI is estimated through visual examination of the rock mass exposed in tunnel faces. The GSI value of a rock mass is incorporated into calculations in order to determine the reduction in the strength of the rock mass as compared with the strength of the intact rock components. This GSI method was deemed as an appropriate tool for evaluating closely jointed rock masses.

Marinos and Hoek (2000) also developed a GSI table specifically for heterogeneous rock masses such as Flysch. Marinos, V. (2007) also updated the GSI table for Flysch material but this assessment was not available at the design stage of the Driskos Tunnel and is not used within this research investigation. Based on these strength values, categories of rock mass were determined.

Figure 5 shows the 14 sections (as divided by the authors) associated with the Driskos Tunnel site and relevant material properties related to each of these sections. Chainage values are also included in this figure. These values correspond with the original geodetic investigation by Egnatia Odos (1998) and have been used here for ease of reference and cross-referencing purposes. The Driskos Tunnel alignment begins to the West at a chainage of 6+124 (Entrance) (i.e. at kilometer 6 within this region of the Egnatia Odos construction works; at 124 m in). The South-West (SW) tunnel portal begins and continues to 10+727, the mark for the North-West (NW) tunnel portal exit.
Figure 5. Schematic summary section of major geological units crossing the Driskos Tunnel Alignment at depth. Geomechanical properties averaged over lengths shown. Numbered zones represent unique engineering geology.

Also conducted within the Driskos alignment was a site investigation that included 18 boreholes over the 4.5 km tunnel alignment. Selected engineering strength properties from these borehole samples as well as samples collected from outcrops in the area are included in Tables 1-4. A Geological Strength Index (GSI) range of 25–38 and a Modulus of Elasticity (E\text{rm}) of 1400 MPa encompass most of the problematic geological sections that the Driskos tunnels encountered. Site specific Rock Mass Quality categories were then determined based on the strength values obtained in the field. The overburden (in-situ) stress and the strength values of the rock mass assessment predicted that the region with the most squeezing potential (i.e. largest expected strain) would be located between the 8 km and 9 km chainage mark (i.e. the section labeled Section 4 within Figure 5). This is seen in Figure 6.
Table 1. Design values of the uniaxial compressive strength, of intact rock, and of the tensile strength for every rock mass category and lithology. (After Structural Design S.A., 1999).

<table>
<thead>
<tr>
<th>Lithology Rock Mass Category</th>
<th>Compressive Strength $\sigma_{ci}$ (MPa)</th>
<th>Tensile Strength $\sigma_t$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>II</td>
<td>III</td>
</tr>
<tr>
<td>Sa</td>
<td>80</td>
<td>60</td>
</tr>
<tr>
<td>SaSi</td>
<td>60</td>
<td>45</td>
</tr>
<tr>
<td>SiSa</td>
<td>35</td>
<td>30</td>
</tr>
<tr>
<td>Si</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Fc</td>
<td>25</td>
<td>20</td>
</tr>
</tbody>
</table>

Table 2. Uniaxial compressive strength of Flysch, Sandstone and Siltstone as determined for Driskos (Structural Design S.A., 1999).

<table>
<thead>
<tr>
<th>Rock Type / Formation</th>
<th>Designation</th>
<th>Uniaxial Compressive Strength $\sigma_c$ (MPa)</th>
<th>Friction Angle ($\phi$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>Sa</td>
<td>40 - 80</td>
<td>$30^\circ$ - $35^\circ$</td>
</tr>
<tr>
<td>Predominant Sandstone with alternations of Siltstone</td>
<td>SaSi</td>
<td>30 - 60</td>
<td>-</td>
</tr>
<tr>
<td>Predominant Siltstone with alternations of Sandstone</td>
<td>SiSa</td>
<td>10 - 35</td>
<td>-</td>
</tr>
<tr>
<td>Siltstone</td>
<td>Si</td>
<td>7 - 10</td>
<td>$27^\circ$ - $30^\circ$</td>
</tr>
</tbody>
</table>

Table 3. Engineering material parameters associated with relevant rock mass quality categories as determined prior to excavation for Driskos (Omega Kappa, 2003).

<table>
<thead>
<tr>
<th>Rock Mass Quality Category</th>
<th>Rock Type / Formation</th>
<th>GSI</th>
<th>$\sigma_{ci}$ (MPa)</th>
<th>$m_{t}$</th>
<th>$\sigma_{em}$ (MPa)</th>
<th>E (MPa)</th>
<th>Overburden (m)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$p_a$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>III</td>
<td>Si</td>
<td>40</td>
<td>10</td>
<td>9</td>
<td>0.86</td>
<td>1590</td>
<td>100</td>
<td>27</td>
<td>2.7</td>
</tr>
<tr>
<td>IV</td>
<td>Si</td>
<td>40</td>
<td>10</td>
<td>9</td>
<td>0.86</td>
<td>1590</td>
<td>150</td>
<td>27</td>
<td>4.05</td>
</tr>
<tr>
<td>Va</td>
<td>SiSa/Si+SaSi</td>
<td>25</td>
<td>15</td>
<td>11</td>
<td>1.02</td>
<td>918</td>
<td>150</td>
<td>27</td>
<td>4.05</td>
</tr>
<tr>
<td>Vb</td>
<td>SiSa/Si+SaSi</td>
<td>25</td>
<td>15</td>
<td>11</td>
<td>1.09</td>
<td>887</td>
<td>220</td>
<td>27</td>
<td>5.94</td>
</tr>
<tr>
<td>Borehole</td>
<td>Chainage</td>
<td>GSI</td>
<td>Rock Mass Quality Category</td>
<td>Depth of Borehole (m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>----------</td>
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<td>----------------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-15</td>
<td>6+601</td>
<td>58-60</td>
<td>III</td>
<td>115.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-13</td>
<td>6+651</td>
<td>52-54</td>
<td>III</td>
<td>115.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH -9</td>
<td>7+301</td>
<td>45-49</td>
<td>III</td>
<td>65.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH -16</td>
<td>7+901</td>
<td>55-57</td>
<td>III-IV</td>
<td>110.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH -10</td>
<td>8+501</td>
<td>42-44</td>
<td>IV</td>
<td>195.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH -17</td>
<td>8+709</td>
<td>43</td>
<td>III</td>
<td>135.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH -11</td>
<td>8+812</td>
<td>42-44</td>
<td>IV</td>
<td>175.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH -12</td>
<td>9+451</td>
<td>57-59</td>
<td>III</td>
<td>75.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH -4</td>
<td>9+907</td>
<td>49-51</td>
<td>III</td>
<td>50.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH -6</td>
<td>10+551</td>
<td>50-59</td>
<td>III</td>
<td>95.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH -7</td>
<td>10+621</td>
<td>59-62</td>
<td>III</td>
<td>30.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>7+900-8+460</td>
<td>57-68</td>
<td>II-III</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>9+160-9+500</td>
<td>57-68</td>
<td>II-III</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>8+460-9+110</td>
<td>30-37</td>
<td>IV-V</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>9+110-9+160</td>
<td>64-73</td>
<td>II</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 6. Analysis of potential large-strain, squeezing regions along the Driskos tunnel alignment, based on the strength of the rock mass and the depth of overburden. (a) Graph of overburden versus chainage along the Driskos Tunnel alignment and (b) Graph of the calculated percentage strain along the tunnel.
2.3 Tunnel Design and Construction

2.3.1 General

The material through which the Driskos tunnels were bored, cannot be fully defined in terms of well known strength and deformation properties as identified in the previous section (i.e. materials are often discontinuous, inhomogeneous and anisotropic in nature). A design must take into consideration the effects of the disturbance caused by tunnel excavation including stages of excavation not completely confined by the long term support and final lining. It is during this stage that the pre-existing stresses in the rock mass (deviated by the opening of the tunnel) are channelled around the cavity in an arch effect, creating zones of increased stress on the walls of the excavation. The most important task of a tunnel design engineer is to determine how and if an arch effect can be triggered when a tunnel is excavated. The engineer must then ensure that the arch effect is formed by calibrating excavation and stabilization operations (Lunardi, 2000). Understanding this rock mass and support interaction becomes a critical issue.

The purpose of tunnel support is to maintain confinement for the rock mass in order to help the rock mass support itself. Under these confined conditions, the interlocking components of the rock pieces produce a strong and stable rock mass. Care must be taken when excavating the face in order to ensure that confined conditions can be maintained. This is achieved through the immediate installation of support technologies such as (not the same in all cases) fibreglass dowels, spiles, shotcrete, rock bolts and grouting. Again, the initial support systems installed at or in advance of the tunnel face serve to retain the rock mass integrity and provide all of the short term support and permit the ultimate installation of the final lining. Excavation in most tunnels within a weak rock mass is carried out in this staged fashion. As with the New Austrian Tunnelling Method (NATM) or observational technique (methods of excavation and deformation monitoring), a top heading can be excavated and then a bench (or invert sections) may be left in
place for further support. The primary support comes from the initial installation of rock bolts and steel arched rib sections supplemented with shotcrete (Figure 7).

Figure 7. Arrangement of tunnel support and excavation stages for horseshoe tunnel with invert, horseshoe and face support (modified after Grasso et al., 2003).

A typical approach would involve the development of a number of typical cross sections for support design. Each section would be related to an anticipated magnitude of strain (or radial displacement). When advancing through difficult ground the use of the forepoling umbrella arch method is oftentimes employed. For a 10 m span of tunnel, this method would typically involve the installation of 12 m long, 75 mm diameter grouted pipe forepoles at a spacing of 300 to 600 mm. The forepoles are installed every 8 m to provide a minimum overlap of 4 m between successive umbrellas. This method is usually used in combination with other support systems such as steel sets embedded in shotcrete, face stabilization by grouted fibreglass dowels and the use of a temporary invert (bench) to control floor heave (Hoek, 1999). Refer to Figure 7.
2.3.2 Design and Construction of Driskos Twin Tunnels

2.3.2.1 Introduction

The tunnel construction at the Driskos site consists of twin, parallel tunnels that are approximately 4570 m in length with a maximum overburden of 220 m. The cross-section’s alignment is shown in Figure 5. In a generic sense, the geological profile shows a gently folded synclinal structure at the centre of the alignment, which elevates to an anticline at the southern end. The Driskos twin tunnels are on average, 850 m above mean sea level and excavation began in 1999 with excavation being driven from both ends. The faces were split into a 60 m² top heading followed by a 40 m² bench. The tunnels were advanced using a Tamrock Para 206 T two boom and basket drill rig. The North portals were constructed using an umbrella canopy for 50 m followed by 40-50 m cut and cover with another 20-30 m of forepole umbrella (Smith, 2000).

Cross-passages connecting the two tunnels occur every 350-400 m. A 180 m ventilation shaft was also constructed near the centre of the tunnel alignment. Tunnel separation is at most 18.2 m between each bore. In terms of tunnel bore dimmensions, a single bore cross-section has dimensions of 9.47 m by 11.0 m with respect to rock mass excavation tolerances. The cross-section of the bores are arranged in a horseshoe configuration. An idealized cross-section of this nature can be seen in Figure 8.

Figure 8. Driskos Northern Portals during construction in 2003 (left) and Idealized horseshoe cross-section of Driskos Tunnel (right)
2.3.2.2 Support Categories

As described by Hoek (2004), a function of stress, strain and rock mass (i.e. tunnel deformation / tunnel diameter versus rock mass strength/in-situ stress) defines the support requirements for tunnel excavations through rock at a specific site. Hoek also went on to describe the various support methods for temporary support of tunnels within weak rock masses. This exercise was no different for the site-specific design associated with the twin tunnels as part of the Driskos tunnels. The main factors that influenced the design were: lithology, rock mass quality, and the height of overburden (i.e. in situ stress). Based on these results, four rock mass categories (Category II – V, with II being the better rock mass in terms of strength) and corresponding support categories (Figure 9) were defined (Egnatia Odos, 1999 and Structural Design S.A., 1999). The percentage of rock masses and support systems corresponding to the overall carriage length for Driskos is:

Category II – 22%  Trends:
Category III – 42%  Deteriorating Rock Mass Strength
Category IV – 27%  Increased Support Requirements
Category V – 8%

Shown in Figure 9 are the extent of support measures that were incorporated in the preliminary design for the Driskos tunnels. One can see the increased mechanical support requirements from Category II through to Category V. Included within the support requirements are rockbolts, shotcrete, steel-sets, and forepoles.

The support measures that correspond to the rock mass categories for the Driskos Tunnel are summarized in Table 5. Category V was further sub-divided into Va and Vb based on the requirement (observational method) of additional support requirements due to excessive
deformations. The specific characteristics, geometries and material properties associated with these support measures are investigated in the next section.

Figure 9. Original support categories II – V for Driskos Tunnels (Egnatia Odos, 1999).
Table 5 Support measures for each rock mass category for Driskos Tunnel (modified after Structural Design S.A., 1999).

<table>
<thead>
<tr>
<th>Support Category</th>
<th>Construction Phases</th>
<th>Distance of Unsupported Portion</th>
<th>Application Time</th>
<th>Support Measures</th>
<th>Additional Measures</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rockbolts</td>
<td>Shotcrete</td>
<td>Steel Sets</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rockbolts in distribution</td>
<td>Rockbolts in distribution</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3m</td>
<td>10cm in the crown and sides</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.5x2.0m in crown and sides</td>
<td>5cm in the sides</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>15cm in the crown and sides</td>
<td>10cm in the sides</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>20cm in the crown and the sides and 10cm at the face of top heading</td>
<td>Steel Sets (HEB) at 1m distances</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>25cm in the crown and the sides and 10cm at face of top heading</td>
<td>Steel Sets (HEB) at 1m distances</td>
<td>Forepole Umbrella if necessary</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>25cm in the crown and the sides and 10cm at face of top heading</td>
<td>Steel Sets (HEB) at 1m distances</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>25cm for invert</td>
<td>Forepole Umbrella</td>
<td></td>
</tr>
</tbody>
</table>

2.3.2.3 Tunnel Support Measures

A longitudinal cross-section of the Driskos Tunnel design (Category V with forepoles) is seen in Figure 10. Here, one can see the steel-sets and the orientation of the forepole umbrella.
It is the weakest rock masses (and associated support category Category V) that were the main focus of this thesis research. This support category corresponds to Section 4 material of the Driskos cross-section (i.e. the weakest rock mass within Driskos) as cited in Figure 5. This geological section was further divided into sub-sections in order to more precisely define the material according to its structure and associated strength. This is highlighted in the analysis portion of this paper.

2.3.2.4 Tunnel Monitoring and Behaviour

As per the observational method of tunnel design, instrumentation and monitoring play a vital role in verifying design assumptions and calibrating numerical models. As well, monitoring serves as an alert if the initial support or lining is not performing as intended or if the tunnel is in danger of collapse. Deformation is a main factor in controlling the failure and cost-effectiveness of underground excavation. As such, in the last two decades deformation monitoring has become a fundamental requirement for assessing the stability of underground openings and for
quantifying the acceptable risk of rock response. (Kontogianni and Siros, 2002). Monitoring data also provide a wealth of data as to the 3D behaviour of the rock mass, support and the time history associated with excavation. This information can be used to improve geotechnical models and optimize the excavation process.

The monitoring program within the tunnels of Driskos incorporated the use of inclinometers, extensometers, strain gauges, load cells, instrumented rock bolts and standard convergence and deformation measurements (Hindley et al., 2004). Within the concept of the observational method of tunnel construction, monitoring has also played an important role in making design changes to primary support systems. **Figure 11** depicts selected monitoring instrumentation that was employed during tunnel construction for Driskos as well as other tunnels of Egnatia Odos.

**Figure 11.** Instrumentation and targets associated with monitoring program of Egnatia Odos; (a) monitoring well and survey target on benchmark, (b) pressure cell (left) and extensometer (right), (c) tunnel wall pressure cell, (d) tunnel wall survey target, (e) surveying the tunnel face, and (f) measurement of targets on tunnel wall (-19 denotes 19 mm of inward displacement at that target location) within Driskos Tunnel.
The observation method includes the gathering of geodetic data in the form of surveying. This monitoring scheme was used in order to verify the adequacy of the adopted geomechanical model and to support the classification/support system. This data is paramount in making decisions on modifications to design and construction optimizations during the construction phase. A monitoring program for the tunnels of Driskos included three-dimensional tunnel wall displacements using optical survey markers as can be seen in Figure 12.

![Survey Targets](image)

**Figure 12.** Optical survey markers and their locations within the tunnel excavation (Geodata, 2003). Photo taken by author. Note that A-Phase and B-Phase are analogous to top heading and bench excavation phases respectively.

**Figure 13** is an example of the monitoring (survey) data that was collected in the field at Chainage 5+503 for the Left Bore. One can see that each survey point (from **Figure 12**) was used in order to determine the relative closure of the tunnel excavation. Data of this nature is not only valuable in determining rock mass behaviour with a view to modifying the temporary design, this data can also be used to validate numerical models and obtain rock mass parameters through back analysis. This investigation utilized the data from the field in order to validate the 3D numerical model that was created to predict the behaviour of the Driskos Twin tunnels.
Figure 13. Radial convergence as captured by survey data from Driskos Left Bore at Chainage 8+503.
2.3.3 Tunnelling Issues Associated with Driskos Tunnel Construction

Prior to construction, it was predicted that Section 4 of the Driskos Tunnel would be the most problematic due to the weak rock masses within that section. As such, the rock mass was designated as a Category V and the initial design complemented this designation (Structural Design S.A., 1999). This was also predicted based on rock mass strength assumptions and in-situ stresses as seen in Figure 6.

However, the deformations that ensued due to tunnel excavation were greater than anticipated. As such, the tunnel temporary design was re-evaluated and modified after an extensive monitoring program by GeoData S.A. (Grasso et. al., 2003) and design recommendations by a Panel of Experts reports (Hoek and Marinos, 2000).

Photos of the problematic section are shown in Figure 14.

Figure 14. Excessive deformations experienced within Section 4 of Driskos Tunnel. (a) Deformed rockbolt face plate as a result of overstressing of the primary support, (b) Spalling of the shotcrete adjacent to a stressed steel set, (c) Sidewall of Driskos Tunnel denoting (in Red) the total amount of inward displacements at selected, monitored target points and (d) Clearer designation of inward displacement in millimeters.
The identified problem was overstressing of the temporary / primary support that occurred at several locations during excavation and extended over distances of approximately 10 meters over the tunnel alignment. This is typical of the response to be expected and is associated with large deformations due to a combination of high stress and low rock mass strength. The influence on the size of the plastic zone (Figure 15) also plays a key mechanistic role within the expected deformation profile as examine by Vlachopoulos and Diederichs (2009). The larger the ultimate plastic zone, the larger the expected deformations as well as the interaction with the plastic zone ahead of the tunnel excavation (face).

![Figure 15](image)

**Figure 15.** A plastic yield zone (*bullet-shaped, shaded zone*) developing as tunnel advances to the left from axisymmetric FEM analysis. If the wall yield zone is more than double the tunnel radius (a), it interacts with face yield zone, however, if the maximum plastic zone radius is less than twice the tunnel radius (b), the wall yield zone does not interact with the face yield zone.

Records of the convergence measurements showed that stable conditions were only reached 7 months after the commencement of the top heading excavation with an excess convergence that (in several locations over a 300 m segment of the alignment) exceeded 150 mm.

As well, the strength of the siltstone-sandstone Flysch was reduced locally by the presence of a high concentration of parallel bedding crossed by frequent faults. These rock mass features within this chainage can be seen in **Figure 16.** Also contributing to
the excessive deformation was the presence of water that contributed to a reduction in rock mass strength.

**Figure 16.** Photo and sketch of tunnel face within problematic region (Egnatia Odos, 1999).

The deformations that were observed did not exhibit a tendency to stabilise with time. The design, therefore, overestimated the rock mass strength for this particular section and as a result, the design capacity of the primary support was determined to be too low (Hoek and Marinos, 2000).

**Figure 17.** Sketches of tunnel faces during excavation of Driskos Tunnel (modified after Egnatia Odos, 1999).
Another factor influencing the analysis of the rock mass is the variability of the face conditions and the anticipated behaviour of such mixed face conditions. As seen in Figure 17, there is much variability of the face conditions within a limited change distance. Within 60 m (or \(~5.5 \times\) tunnel diameters), the sandstone and siltstone layers go from having a horizontal layering arrangement to a vertical orientation. At a larger scale, however, as presented in Figure 4 previous, the general trends of the larger weak masses are clearly evident. In this way, anisotropy at the tunnel scale is not anticipated. Figure 18 also captures the variability of the ground conditions at depth within a limited scale. The results from the numerical modelling (as described in the following sections) indicated that a ‘homogenization’ of the face at these depths and at these scales is suitable in order to capture the overall behaviour of the tunnel.

Figure 18. Results of intense tectonism in rock masses within the region.

A rigorous monitoring program was implemented in order to capture deformations and modify the support categories. As a summary, the major observations associated with the
monitoring program as well as their implications to the re-design of the temporary support for Section 4 of the Driskos twin tunnels were (Egnatia Odos, 2001):

a) During the top heading phase of excavation, 210 mm of tunnel closure was exhibited by the primary / temporary support;

b) Upon excavation of the parallel bore, 310 mm of displacement was captured;

c) Additional primary support measures included: closer spacing of steel set support sections, more and longer fully grouted rockbolts, thicker shotcrete shell, introduction of prestressed cable anchors and the introduction of an invert;

d) Micropiles were also included into the re-design of the temporary support. They were incorporated as a steel reinforced continuous concreted beam element that was laid at the base of the steel ribs along each sidewall;

Deformations were on average larger on the outer sidewalls indicating asymmetric tunnel closure behaviour. This was not predicted at the initial design phase for the primary support. More symmetric conditions were achieved after the modification of the sidewall rockbolt pattern was implemented. Another factor that influences this behaviour is a realistic evaluation of the stress field conditions.

4.0 METHODS OF ANALYSIS

4.1 Convergence-Confinement and Longitudinal Displacement Profile (LDP)

Convergence-confinement analysis (Panet 1995, 1993; Carranza-Torres and Fairhurst 2000; Duncan-Fama 1993 as well as others) is a commonly accepted tool for preliminary assessment of squeezing potential and support requirements for circular tunnels in a variety of geological conditions and stress states. It is within this convergence-confinement framework that tunnel behaviour was analyzed within this paper, with particular emphasis on the longitudinal displacement profile (LDP) portion of this concept.
In order to determine the appropriate timing for the installation of preliminary support systems or when optimizing the installation of support with a view of specific displacement capacity, it is important to determine the longitudinal closure or displacement profile for the tunnel. A portion of the maximum radial displacements at the tunnel boundary will take place prior to the advancement of the face past a specific point. The tunnel boundary will continue to displace inwards as the tunnel advances further beyond the point in question. This longitudinal profile of closure or displacement versus distance from the tunnel face is called the longitudinal displacement profile or LDP.

Panet (1993, 1995), Panet and Guenot (1982), Chern et al. (1998) and others have proposed empirical solutions for LDP’s based on elastic modelled deformation of varying intensity (correlated to various indices such as the ratio between in-situ stress and undrained cohesive strength, for example). Alternatively, an empirical best fit to actual measured closure data can be used (i.e. based on data from Chern et al, 1998). These solutions have been well documented in Vlachopoulos and Diederichs (2009).

The case study presented above highlights the requirement for a better assessment of rock mass strengths and field values as well as more accurate predictive tools in terms of rock mass behaviour associated with tunnel excavation. As such, the accurate monitoring data that was amassed as part of the Driskos twin tunnel construction were used in this investigation in order to validate the 3D and 2D numerical models that were specifically created for this research study. The data was also used for back analysis purposes.

The themes of this investigation relate to: how well the Driskos monitoring data co-relate to the theoretical empirical formulations, how the selection of GSI affects tunnel design, which LDP does a designer use in order to determine the appropriate tunnel support (i.e. unsupported or supported LDP), the effectiveness of the support, and what the effect is of twin tunnel interaction among other considerations.
4.2 Numerical Models

Within this investigation, Phase2 (Rocscience Inc., 2004-2007) was used for the 2D numerical analysis and FLAC3D (Itasca, 2005) was used for the 3D numerical analysis. Phase2 uses an implicit Finite Element Method (FEM) while FLAC3D employs the Finite Difference Method (FDM) in its determinations. Both of these programs are widely used in the geotechnical and geological engineering industry in order to capture the behaviour of a tunnel (i.e. stress redistributions and displacements) associated with tunnel excavation. Chai (2008) also investigated the numerical modelling codes for Phase2 and FLAC (2D) (the basis of FLAC3D) on the influence of stress path on tunnel excavation response and these findings will not be repeated here. Chai stated that one software package was not superior to the other, rather he points out the importance of understanding the program codes and selecting the right tool and modelling approach to represent the expected stress path as close to reality as possible. The emphasis in comparison therefore, should not lie in the limitations of the software packages but on the details of how the true physical phenomenon is being modelled within these limitations. These modelling considerations have been investigated for deep, high stress circular and horseshoe tunnels in weak rock masses by Vlachopoulos and Diederichs (2009).

4.3 Three Dimensional (3D) Analysis

FLAC3D (Itasca, 2002) is an explicit finite difference program that is used to study the mechanical behaviour of a continuous three-dimensional medium as it reaches equilibrium or steady plastic flow. Selected geometries associated with the 3D model that was developed for the purposes of this investigation can be seen in Figure 19. This model consists of the horseshoe twin tunnel configuration that is associated with the Driskos twin tunnels. The model includes similar excavation geometries incorporating sequential excavation and support. The numerical model that was created uses FLAC3D group zones. The model is 110 m in height and 110 m
wide with a tunnel length of 80 m. In order to reduce boundary effects, the tunnel excavation occurred at the centre of the block surrounded further by a series of abutments. The excavated material within the tunnel was created separately and was subdivided into sub-sections that constituted an excavation step and could be separated into full-face or top heading / bench excavation. At each unsupported excavation step, an excavation sub-section block was nullified and steps were conducted in order to ensure equilibrium conditions were met prior to the next excavation sequence. In terms of support, a forepole support umbrella (pile element) was installed and allowed to reach equilibrium prior to each excavation step as per the support installation and excavation steps conducted in the field. The tunnel lining consisted of a 30 cm thick shotcrete layer that was replicated using liner elements. Support detail is seen in Figure 20.

4.4 Two Dimensional (2D) Analysis

Phase2 is a 2D, implicit, elasto-plastic finite element method program used to calculate stresses and determine displacements around underground tunnels and can be used to solve a broad range of geotechnical problems. Phase2 uses plane strain analysis whereby two principal in-situ stresses are in the plane of the excavation and the third principal stress is out of plane. As with other finite element regimes, the domain is discretized into a set number of elements and corresponding nodes. Displacements within these finite elements are calculated based on shape functions tied to the nodes of the elements (Rocscience Inc., 2004). Initial in situ stresses, tolerance parameters, material and defect properties are all assigned by the user. Tunnel excavation is simulated by the removal of elements from within an excavation boundary located in an external boundary.
Figure 19. Selected geometries associated with 3D (FLAC3D) and 2D (Phase2) numerical models used in this investigation. Also inset is an idealized cross-section of a Driskos tunnel bore.
Figure 20. Support detail within the twin tunnel FLAC3D numerical model (only excavated material and support components are shown).

A selected geometry of one of the 2D numerical models that were developed for the purposes of this investigation can be seen within Figure 19. Again, the Driskos horseshoe twin tunnel case study geometry was replicated for the 2D model. These models mimicked a cross-sectional plane of the 3D models described in the previous section.

4.5 Analysis Details

The runs that were conducted consisted of supported and unsupported simulations with elastic-perfectly plastic models (Mohr-Coulomb constitutive model within FLAC3D), strain softening (residual strength) and elastic models in order to isolate the elastic behaviour. The problematic section within Figure 5 was Section 4.
Figure 21 Sub-sections of geological Section 4 of Driskos referencing Figure 5.

Within this section, it was found that there were many geological differentiations that warranted further sub-sections to be classified and assessed. The subsections (4.1, 4.2, 4.3, 4.4 and 4.5) created for this investigation can be seen in Figure 21. The materials and input parameters were selected in order to correspond to these five sub-sections of the geological Section 4 of the Driskos alignment.
Table 6 Relevant factors and rock mass properties for sub-sections of geological Section 4 of Driskos

<table>
<thead>
<tr>
<th>Section Parameter</th>
<th>4.1</th>
<th>4.2</th>
<th>4.3</th>
<th>4.4</th>
<th>4.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chainage</td>
<td>8+385 - 8+500</td>
<td>8+500 - 8+650</td>
<td>8+650 - 8+750</td>
<td>8+750 - 9+000</td>
<td>9+000 - 9+035</td>
</tr>
<tr>
<td>Flysch Category</td>
<td>B-C</td>
<td>B-C</td>
<td>E-F</td>
<td>E-F</td>
<td>B</td>
</tr>
<tr>
<td>Lithology</td>
<td>SiSa</td>
<td>SiSa</td>
<td>SiSa, Si</td>
<td>Si, SiSa</td>
<td>SaSi</td>
</tr>
<tr>
<td>Rock Mass Category</td>
<td>IV-V</td>
<td>IV</td>
<td>III / IV</td>
<td>III / IV</td>
<td>III / IV</td>
</tr>
<tr>
<td>Overburden (m)</td>
<td>145</td>
<td>130</td>
<td>100</td>
<td>180</td>
<td>220</td>
</tr>
<tr>
<td>GSI</td>
<td>18 - 41</td>
<td>21 - 30</td>
<td>22 - 40</td>
<td>31 - 40</td>
<td>35 - 41</td>
</tr>
<tr>
<td>GSI avg</td>
<td>29.5</td>
<td>25.5</td>
<td>31</td>
<td>35.5</td>
<td>38</td>
</tr>
<tr>
<td>$\sigma_{ci}$ (MPa)</td>
<td>37.5</td>
<td>37.5</td>
<td>26.25</td>
<td>26.25</td>
<td>75</td>
</tr>
<tr>
<td>$m_i$</td>
<td>10.3</td>
<td>10.3</td>
<td>7.75</td>
<td>7.75</td>
<td>17</td>
</tr>
<tr>
<td>$E_i$ (MPa)</td>
<td>16125</td>
<td>16125</td>
<td>13453.1</td>
<td>13453.1</td>
<td>28125</td>
</tr>
<tr>
<td>D</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$m_b$</td>
<td>0.83</td>
<td>0.72</td>
<td>0.66</td>
<td>0.77</td>
<td>1.8</td>
</tr>
<tr>
<td>s</td>
<td>0.000396</td>
<td>0.000254</td>
<td>0.000468</td>
<td>0.000772</td>
<td>0.001019</td>
</tr>
<tr>
<td>a</td>
<td>0.52</td>
<td>0.53</td>
<td>0.52</td>
<td>0.52</td>
<td>0.51</td>
</tr>
<tr>
<td>$c$ (MPa)</td>
<td>0.44</td>
<td>0.37</td>
<td>0.29</td>
<td>0.46</td>
<td>1.02</td>
</tr>
<tr>
<td>$\phi$ (degrees)</td>
<td>38</td>
<td>37</td>
<td>36</td>
<td>33</td>
<td>47</td>
</tr>
<tr>
<td>$\sigma_{t}$ (MPa)</td>
<td>-0.017</td>
<td>-0.013</td>
<td>-0.018</td>
<td>-0.026</td>
<td>-0.041</td>
</tr>
<tr>
<td>$\sigma_{c}$ (MPa)</td>
<td>0.62</td>
<td>0.46</td>
<td>0.48</td>
<td>0.65</td>
<td>2.18</td>
</tr>
<tr>
<td>$\sigma_{cm}(MPa)^{+}$</td>
<td>3.1</td>
<td>2.5</td>
<td>2.3</td>
<td>2.9</td>
<td>9.5</td>
</tr>
<tr>
<td>$E_{cm}$ (MPa)</td>
<td>1576.2</td>
<td>1244.1</td>
<td>1442.1</td>
<td>1901.0</td>
<td>4127.7</td>
</tr>
<tr>
<td>$\sigma_{cm}/p_o$</td>
<td>0.84</td>
<td>0.77</td>
<td>0.92</td>
<td>0.64</td>
<td>1.70</td>
</tr>
</tbody>
</table>

+ simplified (original) formula from Hoek (1999)

The parameters or properties associated with each of these materials are located in Table 6. These GSI and strength values were determined independently by the authors. Mohr-Coulomb equivalent properties were obtained using the Hoek-Brown failure criterion (Hoek et al., 2002). Plots of Hoek-Brown and Mohr-Coulomb criteria for these materials are seen in
Figure 22. This figure contains Hoek-Brown plots of (a) major and minor principal stresses, (b) shear and normal stresses as well as Mohr-Coulomb plots of (c) major and minor principal stresses, and (d) shear and normal stresses. Table 7 summarizes the key numerical modelling runs that were conducted for this investigation. Selected results of the major observations are presented in the next section.

Table 7. Selected numerical modeling runs used in this investigation

<table>
<thead>
<tr>
<th>Material(s)</th>
<th>Constitutive Model</th>
<th>Support</th>
<th>Dilation Angle</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1 – 4.5</td>
<td>Elastic, Perfectly Plastic</td>
<td>Unsupported</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>4.1 – 4.5</td>
<td>Elastic, Perfectly Plastic</td>
<td>Supported</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>4.1 – 4.5</td>
<td>Strain Softening (Brittle)</td>
<td>Unsupported</td>
<td>0</td>
<td>Residual Strength = 80%</td>
</tr>
<tr>
<td>4.1 – 4.5</td>
<td>Strain Softening (Brittle)</td>
<td>Supported</td>
<td>0</td>
<td>Residual Strength = 80%</td>
</tr>
<tr>
<td>4.1 – 4.5</td>
<td>Strain Softening (Brittle)</td>
<td>Unsupported</td>
<td>Peak 10</td>
<td>Residual Strength = 80%</td>
</tr>
<tr>
<td>4.1 – 4.5</td>
<td>Strain Softening (Brittle)</td>
<td>Supported</td>
<td>Peak 10</td>
<td>Residual Strength = 80%</td>
</tr>
<tr>
<td>4.1 – 4.5</td>
<td>Perfectly Elastic</td>
<td>Unsupported</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>4.2, 4.4</td>
<td>Elastic, Perfectly Plastic</td>
<td>Supported Top Heading &amp; Invert</td>
<td>0</td>
<td>Install Liner at Face</td>
</tr>
<tr>
<td>4.2, 4.4</td>
<td>Strain Softening (Brittle)</td>
<td>Supported Top Heading &amp; Invert</td>
<td>0</td>
<td>Install Liner at Face</td>
</tr>
<tr>
<td>4.2, 4.4</td>
<td>Strain Softening (Brittle)</td>
<td>Supported Top Heading &amp; Invert</td>
<td>Peak 10</td>
<td>Install Liner at Face</td>
</tr>
<tr>
<td>4.2, 4.4</td>
<td>Elastic, Perfectly Plastic</td>
<td>Supported Top Heading &amp; Invert</td>
<td>0</td>
<td>Install Liner 8m from Face</td>
</tr>
<tr>
<td>4.2, 4.4</td>
<td>Strain Softening (Brittle)</td>
<td>Supported Top Heading &amp; Invert</td>
<td>0</td>
<td>Install Liner 8m from Face</td>
</tr>
<tr>
<td>4.2, 4.4</td>
<td>Strain Softening (Brittle)</td>
<td>Supported Top Heading &amp; Invert</td>
<td>Peak 10</td>
<td>Install Liner 8m from Face</td>
</tr>
</tbody>
</table>
Figure 22. Comparison of Hoek-Brown and Mohr-Coulomb strength Parameters for Subsections 4.1-4.5. Hoek-Brown plots of (a) major and minor principal stresses, (b) shear and normal stresses, Mohr-Coulomb plots of (c) major and minor principal stresses, and (d) shear and normal stresses.
5.0 RESULTS OF ANALYSIS

5.1 Geological Strength Index (GSI)

There is an inherent difficulty of assigning reliable numerical values to rock masses, however, the application of GSI has proven advantages when dealing with weak rock masses such as those found at Driskos. The assignment of a GSI value to a rock mass based on a qualitative assessment of rock mass structure and surface quality is a non-trivial exercise. When assigning a GSI value to a rock mass the most acceptable practice is to assign a range of GSI rather than a single value. The assessment of the strength of the rock mass will have direct implications to the design of the support to be used in a particular excavation.

In the axisymmetric, 2D finite element (Phase2) simulation that was conducted in Figure 23, one can clearly see the effects of varying the GSI of a rock mass. On one side of the tunnel, a GSI of 35 was used, while on the other side of the tunnel a GSI of 45 was assigned.

Figure 23. Yield related closure (no gravity) of two unsupported tunnels at 300m depth using axisymmetric FEM analysis (grid distortion x 10).
A larger plastic zone of 3 diameters was associated with the lower GSI value than with the 2 diameter plastic zone that was created by the higher GSI designation. This has implications to the expected tunnel displacements and ultimately to the design of the temporary support for the excavation. The effect of the size of the plastic zone is the topic of the next section.

Hoek and Marinos (2000) introduced the concept of the determination of support requirements for circular tunnels within a hydrostatic stress field through the use of dimensionless plots of the ratio of tunnel deformation to tunnel radius against the ratio of rock mass strength (as determined through the use of GSI) to the in-situ lithostatic stress. From the results of numerous tunnels excavated in weak rock, the pattern that emerged followed the following empirical formulation:

$$\frac{\Delta R}{R_0} = \left(0.0025 \frac{p_i}{p_0}\right)\frac{\sigma_{cm}}{p_0}\left(2.4\frac{p_i}{p_0} - 2\right)$$

Eq’n 2

where, $\Delta R =$ change in tunnel radius,

$R_0 =$ original tunnel radius

$p_i =$ internal support pressure

$p_o =$ in-situ stress = depth below surface x unit weight of rock mass

$\sigma_{cm} =$ the compressive strength of the rock mass (as determined by GSI)

It was also observed that once the rock mass strength falls below 20% of the in-situ stress level, deformations increased substantially. Unless these deformations are controlled through the installation of adequate support mechanisms, collapse of the tunnel is likely to occur. In this manner, these plots give an excellent indication of the influence of support pressures on tunnel deformation.

The determination of $\sigma_{cm}$ has always been a challenge to designers. It also proved to be a challenge for this research undertaking. An index value for uniaxial rock mass strength is given respectively by Hoek (1999) and Hoek and Marinos (2000):
\[ \sigma_{cm} = 0.019 \sigma_{ci} e^{0.05 \text{GSI}} \]  
Eq’n 3

\[ \sigma_{cm} = \left(0.0034m_i^{0.8}\right)\sigma_{ci} \left(1.029 + 0.025e^{-2m_i}\right)^{\text{GSI}} \]  
Eq’n 4

As well, the value for stress (\(\sigma_{cm}\)) as a function of provided by Hoek and Marinos (2000) is shown below:

\[ \varepsilon = 100x \left(0.02 - 0.025 \frac{p_i}{p_o} \right) \frac{\sigma_{cm}}{p_o} \left[2.4 \frac{p_i}{p_o} - 2\right] \]  
Eq’n 5

Figure 24 Comparison of model data with normalized tunnel closure predicted by Hoek and Marinos 2000 (\(p_i/p_o = \text{normalized support pressure}\)). Rockmass strength is calculated according to three evolutions of this index according to the references shown. The original 1999 version correlates with the modelling data.

For this investigation the three methods seen in Figure 24 were used in order to assess the rock mass strength of the 5 sub-sections of materials (4.1-4.5) utilized in this study. It can be seen that the original version correlated to the expected results of the data strength values the best. The Hoek (1999) version was based on a simple relationship using \(\sigma_{ci}\) and GSI. The Hoek and Marinos (2000) version was an attempt (without real data verification) to incorporate the effect of different frictional properties (\(m_i\)) into the strength determination. The strength determination from such an equation may be too sensitive for low \(m\) values as it tries to capture the range of \(m\) from 7 to 35. Lastly, Hoek et al. (2002) is an approximation of the UCS given by the cohesion
and friction which are a function of confinement that were chosen to fit. In this way, it is implied that UCS is a function of confinement which may not be entirely the case.

Using this relationship (Hoek, 1999), then, Figure 25 is a graph of tunnel deformation to tunnel radius against the ratio of rock mass strength (GSI) to the in-situ stress for the sub-sections of 4.1 – 4.5 in terms of the total displacements observed during the excavation of a single tunnel without the influence of the twin tunnel. Superimposed on the graph are the monitoring values from the field, supported and unsupported values as obtained from the 3D FLAC3D numerical analysis as well as plots of varying support pressures ($p_i/p_o$).

Figure 26 is a similar plot as that of Figure 25, however, this plot takes into consideration the influence of the twin tunnel. One can clearly observe the worst rock mass section (in terms of empirically derived and modelling results) is that of sub-section 4.4. A GSI strength assessment will directly affect the alignment of the rock mass value on the strength axis (x-axis), having a direct impact on support decisions. The rock mass strength for Section 4.4 (and all other sections) does not fall below 20% of the in-situ stress level, thus not requiring an extreme temporary support system to be implemented. Figure 25 demonstrates that the upper and lower bound of the monitoring data that was observed in the field was accurately captured by the numerical modelling simulations. There are some outliers (boxed by a dotted square) that further reinforce the notion that over the entire length of the tunnel, it is impossible to characterize fully all of the conditions that will be encountered. Localized and unidentified (and thus unclassifiable) fault zones as well as other geological peculiarities can add to the degradation of the strength of the rock mass in certain regions. These occurrences can be seen for sub-sections 4.2 and 4.3. In terms of the Driskos construction, when such observances occurred, tunnelling experts were consulted in order to provide advice, modify design and offer quality control with a view to managing the situation and reducing or eliminating such incidents.
Figure 25  Ratio of tunnel deformation to tunnel radius versus the ratio of rock mass strength to In-situ stress for varying support pressures with Driskos monitoring data as well as unsupported and supported 3D modelling analysis for single tunnel.

Figure 26  Ratio of tunnel deformation to tunnel radius versus the ratio of rock mass strength to In-situ stress for varying support pressures with Driskos monitoring data as well as unsupported and supported 3D modelling analysis for single tunnel taking into account the displacements due to parallel (twin) bore excavation.
5.2 Longitudinal Displacement Profile (LDP)

As mentioned previously, the LDP is one of the basic components of the convergence-confinement method. It is a graphical representation of the radial displacement that occurs along the axis of an unsupported cylindrical excavation prior to and past the face. Figure 27 depicts such a profile. The horizontal axis indicates the distance from the face and the vertical axis indicates the corresponding tunnel wall displacement. At a certain distance ahead of the face, the advancing tunnel has no influence on the rock mass and the radial displacement is zero. At approximately 1 diameter distance ahead of the face, the rock mass begins to influence the rock mass. At the face, approximately 30% of the total displacement has already occurred and at a certain distance past the face, the effect of the face is substantially reduced as displacements stabilize.

Figure 27. A typical Longitudinal Displacement Profile for an advancing tunnel within a weak rock mass
The LDP’s associated with Panet (1995), and Unlu and Gercek (2003) are plotted in Figure 28. Also plotted in this figure are the LDP’s associated with 12 monitored sections within the Driskos tunnel. Note that all of the data has been normalized with respect to the maximum supported displacement and the original radius of the tunnel. One can clearly see that the accepted empirical formulations do not correlate well with the data that has been captured in the field; hence, the requirement for further investigations into this behaviour.

Based on this observation and using a series of numerical analyses, Vlachopoulos and Diederichs (2009) introduced a new series of functions defining robust longitudinal displacement profiles (LDP), as a function of maximum normalized plastic radius. This approach takes into consideration the effect that a large ultimate plastic radius has on the rate of development of wall displacements with respect to location along the tunnel for 2D numerical analysis pseudo-capturing 3D effects. Current LDP functions (as seen in Figure 28) are inadequate for tunnel analysis in very week ground at great depth. This approach is valid from the elastic case through to complete plastic closure of the tunnel (as calculated using numerical or analytical solutions). Clearly, the larger and well defined “bullet-shaped”, shaded ultimate plastic zone as seen in Figure 15 significantly influences behaviour and is not accounted for in the elastic approaches cited in Figure 27. The larger the plastic radius, the larger the expected deformations as well as the interaction with the plastic zone ahead of the tunnel face.

Another observation is that the expected behaviour in front of the face should not be expected to be the same as the behaviour past the excavation of the face. As such, LDP calculations should provide two functions; one capturing behaviour prior to the face and, the other as a function of distance past the face. Continuous functions such as Panet (1995) over-idealize the expected behaviour in this regard. This has also been investigated in Vlachopoulos and Diederichs (2009).
5.3 LDP to use for Supported and Unsupported Design Purposes

A legitimate question is which LDP does one use for design of tunnel support purposes? The convergence-confinement method (convention) is based on the LDP from a circular, unsupported excavation. It can be seen in Figure 29 that if one was to plot the 3-dimensionally modelled, unsupported horseshoe LDP for a staged excavation (top heading and bench) the top heading LDP and the bottom heading LDP are not similar in nature. The displacement observed for the bench LDP is approximately twice that of the top heading LDP.

It can be seen from this figure that the incremental deformation when the bench passes the point of monitoring is approximately 50% of the total additional deformation for the bench.
The initial top heading, however, has incremental deformation that is approximately 25% of the total additional deformation. Clearly the same LDP cannot be used for design purposes.

**Figure 29.** FLAC3D model results showing the LDP for section 4.1 of staged excavation (elastic, perfectly plastic model).

The unsupported LDP for sections 4.1 to 4.4 are shown in **Figures 30-33** respectively. Superimposed on these graphs are the FLAC3D modelled, supported LDPs normalized with respect to: a) the maximum supported displacement ($u_{\text{max}(\text{sup})}$) and b) the maximum unsupported displacement ($u_{\text{max}(\text{unsup})}$). Also included on these plots are the normalized displacements associated with the Driskos monitoring data. Chainage locations of monitoring data have been paired up with their respective sections (4.1-4.4). As can be seen, the monitoring data from Driskos track the supported LDP / $u_{\text{max}}$ supported curve.

The displacements at the face for the modelled supported runs are less than the 30% of the total displacement that were anticipated. This can be attributed to the installation of the forepole umbrella above and to 12 m in front of the face and the installation of the liner immediately after the excavation of the face that stiffens the face and provides 100% effective support to the system.
Figure 30. Longitudinal Displacement Profiles associated with unsupported and supported LDPs as well as the LDPs of monitored section 4.1 of the Driskos tunnel.

Figure 31. Longitudinal Displacement Profiles associated with unsupported and supported LDPs as well as the LDPs of monitored section 4.2 of the Driskos tunnel.
Figure 32. Longitudinal Displacement Profiles associated with unsupported and supported LDPs as well as the LDPs of monitored section 4.3 of the Driskos tunnel.

Figure 33. Longitudinal Displacement Profiles associated with unsupported and supported LDPs as well as the LDPs of monitored section 4.4 of the Driskos tunnel.
This stiffening of the face may also be attributed to the shotcrete curing rate that gains 80% of its strength within a 24 hour period, well within the excavation rates for the Driskos project. The effect of the support has also been captured by the 2D Phase2 analysis shown in Figure 34. Here, the Driskos case was simulated using an unsupported run as well as a variety of supported runs of various configurations. Highlighted here is the fact that the use of forepoles, forepoles and liner, and invert and liner (i.e. the most effective support systems controlling deformations) reduce the amount of closure ahead of and at the face. This reduction in anticipated displacements prior to and at the face is the same phenomenon that has been captured in the 3D analysis presented in Figures 30-33.

![Figure 34](image)

**Figure 34.** Phase2 results depicting LDPs from various supported cases.

### 5.4 Twin Tunnel Interaction

FLAC3D modelled LDPs have been plotted (Figure 35-36) for the initial, single tunnel that was excavated superimposed on the LDP of the twin tunnel that was excavated parallel to the initial tunnel. These were plotted for section 4.4 (weakest of the materials) and section 4.3 for the
elastic, perfectly plastic constitutive model, supported and unsupported respectively. In both cases, the LDP of the single (or 1st) tunnel yielded larger displacements than that of the twin (or 2nd) tunnel. The difference is between 10-15% at times. As such, the FLAC3D model does not indicate much twin tunnel interaction. The monitoring data from Driskos does demonstrate an effect on the 1st tunnel due to excavation of the 2nd tunnel parallel to it (Figure 13). There are indications that a weakening or shifting of the 2nd tunnel does occur but this warrants further investigation.

Another interesting observation is the fact that the 3D models did not sufficiently capture the effects of the real, observed twin tunnel interaction in the field. In Figure 13, the sample monitoring data from the Driskos tunneling site at Chainage 8+503 shows that the displacements ‘accelerate’ once the top heading of the right bore (parallel) excavation passes the face of the left bore. This can be seen in the radial closure trend Δ3. This can be attributed to the dissipation of the stresses in the longitudinal direction and not necessarily along the plane strain surface as in 2D numerical models.

![Graph showing relative horizontal compressive / radial closure of tunnel for 3D models with twin tunnel](image)

**Figure 35.** Relative horizontal compressive / radial closure of tunnel for elastic, perfectly plastic - supported - section 4.4 with twin tunnel
5.5 GSI and Effectiveness of Support

In an attempt to determine that the modelled behaviour was consistent with the observed behaviour in the field, the modelled results were compared with the Driskos monitoring data. These graphs can be seen in Figures 37-41 for material sections 4.1-4.5 respectively. The upper bound of the numerical simulations is the unsupported scenario, whereby only the rock mass behaviour is recorded (i.e. the model support is 0% effective as no support was introduced into the model). The lower bound takes into consideration the fully supported numerical model that was built to the specifications of the Driskos Twin Tunnels. This included the introduction of forepoles, rockbolts, and liner with steel sets as described previously (i.e. 100% effective model support).

As can be seen, the results of the FLAC3D model capture the monitored behaviour and trends well. The monitoring data is bound by the two extremes of 0% effective and 100% support.
effective model support. This validates and adds confidence to the numerical model used in this investigation. Displacements predicted by the numerical model with few exceptions were in accordance with the recorded field values.

As well, the selection of GSI strength values also co-relates well for each of the 4 sections. Illustrated in Figure 37, however, the monitoring data at location 8+674 tracks above the upper bound of the model. This reinforces the fact that is impossible to fully characterize all of the conditions that will be encountered during tunnel excavation over the entire length of the project. As mentioned previously, localized and unidentified (and thus unclassifiable) fault zones as well as other geological peculiarities can add to the degradation of the strength of the rock mass. The behaviour observed mimics a pattern of re-accelerated or resumed deformation often related to high tunnel convergence and failures. Excluding any measurement errors in the monitoring data (which are highly uncommon), reasons for these peculiarities can be localized ground conditions, significant changes in the local hydrological conditions (combined with the lithological effects) as well as stress transfer. Kontogianni et al., (2003) cites that under certain

![Figure 37](image-url): Relative radial closure of Driskos Tunnel for elastic, perfectly plastic supported and unsupported FLAC3D runs with Driskos monitoring data for section 4.1.
Figure 38. Relative radial closure of Driskos Tunnel for elastic, perfectly plastic supported and unsupported FLAC3D runs with Driskos monitoring data for section 4.2.

Figure 39. Relative radial closure of Driskos Tunnel for elastic, perfectly plastic supported and unsupported FLAC3D runs with Driskos monitoring data for section 4.3.
Figure 40. Relative radial closure of Driskos Tunnel for elastic, perfectly plastic supported and unsupported FLAC3D runs with Driskos monitoring data for section 4.4.

Figure 41. Relative radial closure of Driskos Tunnel for elastic, perfectly plastic supported and unsupported FLAC3D runs with no Driskos monitoring data available for section 4.5.
conditions high convergence is often associated with re-acceleration in the strain accumulation in neighbouring sections. These appear as steps in the strain curves. Strain may be transferred at distances >2D back along the tunnel outside of the zone of the face effect.

Also adding to the increase in displacements to the monitoring data during the excavation of the bench as seen in Figures 37-40 is the radial (or system) disturbance that is caused to the upper arch system due to the installation of the ‘legs’ of the steel sets immediately after bench excavation. This installation process is shown in Figure 42. Tunnelling support installation equipment cannot access the underside of the arched support without disturbing the temporary support system that has already been installed in the upper arch (top heading) region. It is important therefore, to ensure a connection of limited disturbance between the top heading arched support and the arch legs. In selected designs, a shotcrete invert can be used to stabilize the top heading and assume the load that will eventually be transferred to the legs of the arch.

**Figure 42.** Installation of steel set support in stepped excavation stages. Note that installation of the bench support cannot occur with disturbance to the existing, previously installed support (Modified after Grasso et al., 2003).

A bending moment diagram as extracted from the tunnel liner of the FLAC3D Driskos model for Section 4.2 is seen in Figure 43. This is a typical bending moment pattern that one would expect from such a stress environment; a symmetrical pattern of the moments about the
Figure 43. Bending moment diagram of liner – arched portion – for section 4.2.

Figure 44. Moment, shear and thrust diagram extracted from supported liner for Section 4.2 of Driskos at mid-point (40m) location within excavation.
axis of the tunnel alignment for the arched portion of the supported liner. It can also be seen that the moments approach near zero conditions where the arch portion begins and ends (positions #9 and #25). The sidewall liner connections were then analyzed in conjunction with the upper arched portion in order to determine how stresses and moments were transferred from the arch to the support legs or struts (Figure 44).

The moments and shears within Figure 44 have the most extreme conditions exhibited at the connections between the arched portion and the legs (Δ0.12 MNm for moments and Δ0.12 MN for shear). There is little shear transfer from the arched portion to the struts as seen in the figure. These type of connections may have to be re-assessed in terms of continuity to the arch as well as how the system is seated at the bottom of the tunnel. The arched portion, however, is behaving relatively well.

The effectiveness of the support can be assessed on a determination of the moments shears and thrusts within the liner (and if failed). The concept of designing with the use of support capacity diagrams introduced by Kaiser (1985) and Sauer et al (1994) can also be used in order to determine the effectiveness of the support system.

The support capacity diagrams take into consideration a composite lining based on shotcrete and the steel sections. Within the Driskos temporary support design, HEB 160 steel sets were used with 30 cm of shotcrete. It should be noted that the support capacity diagrams are based on elastic analysis of the composite support elements which implies no tensile or compressive failure of the liner is acceptable. The detailed calculations of moments and forces in the lining elements has been summarized by Carranza-Torres and Diederichs (2008).

Seen in Figure 45 are the plotted support capacity diagrams for section 4.2. It can be observed that the moment-axial thrust points for the support all fall within the capacity curves for the corresponding strength of the support with the exception of the minimal thrust outliers for a
Figure 45. Support capacity diagrams for a factor of safety of 1 and 1.5 for section 4.2 of Driskos
factor of safety of 1. The diagrams indicate minimum overstressing and, hence, the composite lining is should be re-examined and adjusted prior to proceeding to the installation of the final lining. Note that the moment thrust plots show concrete cracking (moment exceeded) only at the connection points between the arch and the struts which is a very likely occurrence.

6.0 Conclusions

The main conclusions of the research that was undertaken are:

- The behaviour of rock tunnelling of weak rock masses such as Flysch can be captured accurately using continuum numerical methods (FLAC3D) of numerical analysis as Driskos Tunnel data from the field co-related well with the results obtained through numerical analysis. In general the deformed Flysch can be analyzed as isotropic over the tunnel scale, although heterogeneity and locally intense anisotropy can lead to inadequate support performance. The field data proved valuable in calibrating and verifying numerical models developed by the author;

- Accurate values for GSI are difficult to obtain for extended, altering zones of weak rock masses as there are often zones whereby localized geological peculiarities may govern tunnel behaviour. An accurate assessment of GSI has a direct impact on input parameters for numerical modelling purposes;

- Through an assessment of the numerical modelling data, it was found that the rock mass strength calculated by Hoek (1999) co-related best to the results obtained in this investigation, vice the same strength parameter (index value for uniaxial rockmass strength) as calculated by Hoek and Marinos 2000 as well as Hoek et. al. 2002;

- Field data from Driskos did not co-relate well with LDP equations as determined by Panet (1995), Unlu and Gereck (2003) and Chern et. al. (1998). This prompted a further investigation into plastic zone development in these weak rock masses as per Vlachopoulos and Diederichs (2009);

- The LDP used for design purposes of the top heading was determined not to be the same as the LDP that was followed by the successive bench. It was found that the incremental
deformation when the bench passes a certain monitoring point is approximately 50% of the total additional deformation whereas for the initial top heading this value is approximately 25% of the total deformation. This has implications as to the selection of a proper LDP for design purposes;

- Normalized Driskos field measurements align well with the 3D modelling supported LDP normalized with the total displacement achieved by the unsupported behaviour vice the supported LDP normalized with the total supported maximum displacement for a single tunnel bore;

- The influence of the size of the plastic zone plays a key mechanistic role within the expected deformation profile or LDP - a new series of functions has been developed to account for this influence;

7.0 Acknowledgements

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Improved Longitudinal Displacement Profiles for Convergence Confinement Analysis of Deep Tunnels

By

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Summary

Convergence-confinement analysis for tunneling is a standard approach for preliminary analysis of anticipated wall deformation and support design in squeezing ground. Whether this analysis is performed using analytical (closed form) solutions or with plane strain numerical models, a longitudinal displacement profile (LDP) is required to relate tunnel wall deformations at successive stages in the analysis to the actual physical location along the tunnel axis. This paper presents a new and robust formulation for the LDP calculation that takes into account the significant influence of ultimate (maximum) plastic radius. Even after all parameters are appropriately normalized, the LDP function varies with the size of the ultimate plastic zone. Larger yield zones take a relatively longer normalized distance to develop, requiring an appropriately calculated LDP. Failure to use the appropriate LDP can result in significant errors in the specification of appropriate installation distance (from the face) for tunnel support systems. Such errors are likely to result in failure of the temporary support. The equations presented here are readily incorporated into analytical solutions and a graphical template is provided for use with numerical modeling.

Keywords: Tunnelling, convergence-confinement, displacement, squeezing, ground reaction

1. Introduction

Convergence-confinement analysis (Duncan-Fama, 1993; Panet, 1993, 1995; Carranza-Torres and Fairhurst, 2000 and others) is a widely used tool for preliminary assessment of squeezing potential and support requirements for circular tunnels in a variety
of geological conditions and stress states. The technique has been well documented and will not be discussed here in detail except to summarize the concept.

An analytical plasticity solution such as that developed by Carranza-Torres and Fairhurst (2000) is applied to a circular opening in an isotropic stress field. An internal pressure, initially equal to the in situ stress is applied on the inside of the excavation boundary. The pressure is incrementally relaxed until the excavation boundary condition is that of zero normal stress. The extent of plastic yielding and thereby, the boundary deformation is calculated at each stage of the process. The result is a continuous representation of the deformation-internal pressure relationship for the tunnel given a particular material strength, deformability, dilation and stress state.

The internal pressure is, of course not a representation of reality but rather a surrogate for the effect of the gradual reduction of the radial resistance provided by the initially present tunnel core (material inside the tunnel boundary) transitioning to an exposed boundary with a progressively distant tunnel face and ultimately a long open tunnel with plane strain conditions. The internal pressure that is coupled with a given boundary displacement is a measure of the amount of support resistance required to prevent further displacement at that point in the progressive tunneling model.

By making the significant and possibly debatable assumption that the support application does not change the material response, estimates of pressure-displacement

![Diagram](image)

**Fig. 1.** Ground reaction (convergence confinement) curve shown with support reaction curve for liner installed at the tunnel face. The longitudinal displacement profile relates the normalized displacement to normalized location along the tunnel axis. Tunnel section is shown for elastic region, and for plastic zone development with and without support.
curves can be compared to estimate the factor of safety against overload (Fig. 1). The support needs to be “installed” at the appropriate location (distance from the face). In order to calibrate the model so that the internal pressures or the displacements are correlated to a real distance from the face, a longitudinal displacement profile or LDP is required as shown in Fig. 1.

2. Longitudinal displacement profiles

In order to determine the appropriate timing for the installation of stiff support or when optimizing the installation of support with specific displacement capacity, for design purposes, it is important to establish the longitudinal closure or displacement profile for the tunnel. A portion of the maximum radial displacements at the tunnel boundary will take place before the face advances past a specific point. The tunnel boundary will continue to displace inwards as the tunnel advances further beyond the point in question. This longitudinal profile of closure or displacement versus distance from the tunnel face is called the longitudinal displacement profile or LDP. An example of a normalized LDP is shown in Fig. 1 and a tunnel specific (without normalization) LDP is illustrated in the example in Fig. 2.

The LDP cannot be calculated using 2D plane-strain models, although the LDP can be used to calibrate staged 2D models in which the inner tunnel core is replaced by incrementally relaxing boundary tractions to simulate a staged ground reaction

![Fig. 2. Use of the longitudinal displacement profile, (LDP), to relate support installation location to nominal wall displacement for use in convergence–confinement ground/support reaction analysis. Schematic analysis based on a 10 m diameter tunnel at depth](image-url)
Fig. 3. Alternative approaches for tunnel modeling: a) 2D plane strain (PHASE2 – Rocscience, 2007) finite element model; b) axisymmetric finite element model with staged excavation; c) 3D finite difference model (FLAC3D – Itasca, 2006)

Fig. 4. Comparison between LDP’s calculated using axisymmetric finite element models (Fig. 3b) and 3D finite difference models (Fig. 3c). An elastic analysis is shown and can be compared to plastic models with different ratios of isotropic in situ pressure $p_0$ and rockmass uniaxial compressive strength ($UCS_{RM}$). Materials B, C, D and E have $p_0/UCS_{RM}$ ratios of 8, 6, 4 and 2, respectively. Sample 3D results are shown in inset. The four tunnels are 5m in radius ($R_T$) and have maximum plastic radii ($R_P$) of approximately 26 m, 18 m, 12 m and 8 m, respectively.
curve such as that shown in Fig. 1. A simple two-dimensional model (Fig. 3a) can be used to calculate the maximum wall displacement \( u_{\text{max}} \) and the maximum radius of the plastic (yielding) zone \( R_p \).

The LDP can be calculated using axisymmetric models for uniform or isotropic initial stress conditions and circular tunnel cross sections (Fig. 3b) of full three-dimensional models for complex loading and geometric conditions (Fig. 3c). This profile can be used to establish a distance-convergence relationship for 2D plane-strain modeling or for analytical solutions (as in Carranza-Torres and Fairhurst, 2000). For the simplified case of a circular tunnel in an isotropic stress field, a comparison between axisymmetric modeling and full 3D analysis (using FLAC3D – Itasca, 2006) is shown in Fig. 4.

There is an important caveat to consider when using numerical analysis to compute longitudinal displacement profiles. When using axisymmetric or full three-dimensional models to determine the longitudinal displacement profile relationship, it is important to consider the excavation rate. A stress front builds ahead of the bullet-shaped plastic zone and influences the rate of plastic zone development. Such models will yield a different apparent longitudinal displacement profile depending on the size of the excavation step. This is clearly shown in Fig. 5, where there is a significant difference between the instantaneous excavation and the 1 m (0.2D) step simulation (other excavation step sizes shown for comparison). For support sequencing, it is important to simulate the actual excavation step size or, if the tunneling is continuous (TBM), to use a small step size.

Fig. 5. Example of Influence of excavation step size (as a ratio of tunnel diameter D) on the modeled longitudinal displacement profile. Instantaneous excavation and elastic solution shown for comparison
3. Review of current LDP approaches

If only two-dimensional models are available or if an analytical convergence-confinement solution is to be used (as in Fig. 1), it is more practical to use an analytical function for the LDP. In order to facilitate analytical calculations of ground response (convergence-confinement), Panet (1995) derived a relationship for the longitudinal displacement profile based on elastic analysis:

\[
\frac{u^*}{u_{\max}} = \frac{1}{4} + \frac{3}{4} \left( 1 - \left( \frac{3}{3 + 4X^*} \right)^2 \right)
\]

where \( X^* = X/R_T \), \( u_R \) is the radial displacement at a specified longitudinal position \( X \), and \( u_{\max} \) is the maximum short term radial displacement distant from the face and corresponding to plane strain analysis of a tunnel cross section. \( R_T \) is the tunnel radius and \( X \) is positive into the tunnel away from the face \((X = 0)\). The position, \( X \), is negative into the rock ahead of the face and is specified along the tunnel centerline.

Numerous other authors have suggested alternative expressions for the elastic longitudinal displacement profile including Unlu and Gercek (2003) who noted that the curve in front of the face and the curve behind the face do not follow a single continuous functional relationship with \( X \). The authors agree with this assertion. The radial deformation profile with respect to distance from the face is accurately predicted for the elastic case to be:

for \( X^* \leq 0 \):

\[
\frac{u^*}{u_{\max}} = \frac{u_R}{u_{\max}} = \frac{u_0}{u_{\max}} + A_a \left( 1 - e^{B_a X^*} \right)
\]

for \( X^* \geq 0 \):

\[
\frac{u^*}{u_{\max}} = \frac{u_R}{u_{\max}} = \frac{u_0}{u_{\max}} + A_b \left( 1 - (B_b/(A_b + X^*))^2 \right)
\]

where \( u_0 \) is the radial displacement at the face location \((X^* = 0)\) and \( A_a, A_b, B_a, B_b \) are functions of Poisson’s Ratio:

\[
\begin{align*}
A_a &= -0.22\nu - 0.19; \quad B_a = 0.73\nu + 0.81 \\
A_b &= -0.22\nu + 0.81; \quad B_b = 0.39\nu + 0.65
\end{align*}
\]  

These preceding equations are for elastic deformation. Panet (1993, 1995), Panet and Guenot (1982), Chern et al. (1998) and other have proposed empirical solutions for longitudinal displacement profiles based on modeled plastic deformation of varying intensity (correlated to various indices such as the ratio between in situ stress and undrained cohesive strength, for example). Alternatively, an empirical ‘best fit’ to actual measured closure data can be used (for example, based on data from Chern et al., 1998):

\[
\frac{u^*}{u_{\max}} = (1 + e^{(-X^*/10)})^{-1.7}
\]

These relationships are summarized in Fig. 6.
Improved LDP related to plastic zone radius

The development of radial deformation, however, is directly linked to the development of the plastic zone as the tunnel advances. Studies by the authors have shown that the longitudinal displacement profile function proposed by Panet (1995) and by Unlu and Gercek (2003) is reasonable for plastic analysis provided that the radius of the plastic zone does not exceed 2 tunnel radii and provided that the yielding zone in the tunnel face does not interact with the developing yield zone around the tunnel walls as illustrated in Fig. 7.

The advancing front of plastic yielding is bullet shaped in three-dimensions and for large plastic zones (radius of plastic zone $R_P > 2 \times R_T$) the shape of this developing yield zone is geometrically similar for increasing maximum plastic radii. There is no reason, therefore, to expect that a single longitudinal displacement profile will suffice for these conditions. In order to account for the influence of increased overall yielding on the shape of the normalized longitudinal displacement profile, the most logical index to relate to the longitudinal displacement profile function is the normalized plastic zone radius, $R^* = R_P / R_T$.

To illustrate this problem, a series of analyses were performed involving a radial tunnel section and an axi-symmetric analysis along the tunnel axis. The first suite of analyses is based on a typical rockmass at 1100 m depth in a weak rockmass (e.g. graphitic phyllite). This is case A1 in Table 1 below. In this case, the initial in situ stress is approximately 10 times the estimated rockmass uniaxial strength ($UCS_{RM}$ or $\sigma_{crm}$ calculated according to Hoek et al., 2002). Five other rockmasses are investigated.

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**Fig. 6.** Comparison of Longitudinal Displacement Profiles (LDP’s) reported in the literature, including Eqs. (1) (Panet, 1995), (2) (Unlu and Gercek, 2003) and (4) (based on Chern et al., 1998)
with increasing intact strength and/or GSI (Marinos and Hoek, 2000) giving a series of representative cases with varying $p_0/UCS_{RM}$ (in situ stress/rockmass strength). The rockmass parameters are summarized in Table 1.

The rockmass strengths are estimated as per Hoek et al. (2002) and the elastic moduli are estimated based on Hoek and Diederichs (2006). A second set of analyses were performed based on rockmass A$_1$ (plastic) and G$_1$ (elastic) in Table 1 with increasing depth. The resultant in situ stress levels are listed in Table 2.

**Table 1.** Rockmass parameters for longitudinal displacement profile analysis using PHASE2 (constant $p_0=28$ MPa)

<table>
<thead>
<tr>
<th></th>
<th>A$_1$</th>
<th>B$_1$</th>
<th>C$_1$</th>
<th>D$_1$</th>
<th>E$_1$</th>
<th>F$_1$</th>
<th>G$_1$</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$p_0/UCS_{RM}$</td>
<td>10</td>
<td>8</td>
<td>6</td>
<td>4</td>
<td>2</td>
<td>1</td>
<td>Elastic</td>
<td></td>
</tr>
<tr>
<td>$\sigma_{cl}$ (MPa)</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>50</td>
<td>75</td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$m_i$</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>7</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GSI</td>
<td>25</td>
<td>35</td>
<td>45</td>
<td>48</td>
<td>60</td>
<td>74</td>
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<tr>
<td>$m$</td>
<td>0.481</td>
<td>0.687</td>
<td>0.982</td>
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<td>1.678</td>
<td>2.766</td>
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<tr>
<td>$s$</td>
<td>0.0002</td>
<td>0.0007</td>
<td>0.0022</td>
<td>0.0031</td>
<td>0.0117</td>
<td>0.0536</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$a$</td>
<td>0.531</td>
<td>0.516</td>
<td>0.508</td>
<td>0.507</td>
<td>0.503</td>
<td>0.501</td>
<td></td>
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<tr>
<td>$E_{RM}$ (MPa)</td>
<td>1150</td>
<td>2183</td>
<td>4305</td>
<td>7500</td>
<td>11215</td>
<td>27647</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$UCS_{RM}$ (MPa)</td>
<td>2.8</td>
<td>3.5</td>
<td>4.7</td>
<td>7</td>
<td>14</td>
<td>28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P_0$ (MPa)</td>
<td>28</td>
<td>28</td>
<td>28</td>
<td>28</td>
<td>28</td>
<td>28</td>
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<td></td>
</tr>
<tr>
<td>$R_F$ (m)</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Fig. 7.** a) Plastic yield zone developing as tunnel advances to the left (axisymmetric FEM analysis). Maximum plastic zone radius is less than twice the tunnel radius and the wall yield zone does not interact with the face yield zone (Panet’s 1995 longitudinal displacement profile is valid); b) wall yield zone more than double the tunnel radius and interacts with face yield zone (Panet’s longitudinal displacement profile is not valid)

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The tunnels were analyzed with Phase2 (Rocscience, 2007) in plane strain cross section to determine the extent of the plastic zone and the maximum radial deformation in each case. In addition, the cases were analyzed via axisymmetric models with 1 m incremental advance to determine the longitudinal displacement profile in each case as in Fig. 3b. The maximum displacements and plastic zone extents were comparable between the radial and longitudinal models. These summary results are presented in Table 3 and the resultant normalized longitudinal displacement profiles are presented in Fig. 8.

By inspection of Fig. 8 it is evident that the longitudinal displacement profile does not correlate with the stress/strength index $p_0/UCS_{RM}$ as the set of curves in both plots represent the same selected values for this ratio and yet have different longitudinal displacement profiles. Analysis of the data, however, shows a direct correlation with the maximum normalized plastic zone, $R_P/RT$, as expected. The correlation between $u_0/u_{max}$ at $X/RT=0$ (at the face) and the maximum plastic radius, $R_p/RT$, is shown in Fig. 9. Ignoring the influence of Poisson’s ratio (negligible compared to plastic yielding) the best fit relationship (independent of material parameters and stress levels) is:

$$u_0/u_{max} = \frac{1}{3} e^{-0.15R^*}$$

where $R^* = R_p/RT$.

The relationships proposed by Unlu and Gercek (2003) correctly illustrate that the behavior ahead of the face ($X < 0$ into the rockmass) does not follow the same continuous function as the behavior (progressive displacement) behind the face ($X > 0$ in the tunnel). Their functions summarized in Eq. (2), do not, however, capture the influence of a large developing plastic zone, nor does Eq. (1) by Panet (1995). Based on the analysis in the preceding discussion, a new set of relationships are presented here.

### Table 2. Rockmass parameters for longitudinal displacement profile analysis using PHASE2 (constant $UCS_{RM} = 2.8$ MPa)

<table>
<thead>
<tr>
<th>$P_0/UCS_{RM}$</th>
<th>$A_2$</th>
<th>$B_2$</th>
<th>$C_2$</th>
<th>$D_2$</th>
<th>$E_2$</th>
<th>$F_2$</th>
<th>$G_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_0$ (MPa)</td>
<td>10</td>
<td>8</td>
<td>6</td>
<td>4</td>
<td>2</td>
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<td>Elastic</td>
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<td></td>
<td>28</td>
<td>22.4</td>
<td>16.8</td>
<td>11.2</td>
<td>5.6</td>
<td>2.8</td>
<td>28</td>
</tr>
</tbody>
</table>

### Table 3. Summary results for tunnel with 2.5 m radius. $R^*$ is the normalized plastic radius ($R_P/RT$) while $u_{max}$ is the maximum radial displacement

<table>
<thead>
<tr>
<th>$P_0/UCS_{RM}$</th>
<th>$A_2$</th>
<th>$B_2$</th>
<th>$C_2$</th>
<th>$D_2$</th>
<th>$E_2$</th>
<th>$F_2$</th>
<th>$G_2$</th>
</tr>
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<tbody>
<tr>
<td>Constant $P_0$</td>
<td>$A_1$</td>
<td>$B_1$</td>
<td>$C_1$</td>
<td>$D_1$</td>
<td>$E_1$</td>
<td>$F_1$</td>
<td>$G$</td>
</tr>
<tr>
<td>$R^*$</td>
<td>7.5</td>
<td>5.1</td>
<td>3.5</td>
<td>2.3</td>
<td>1.5</td>
<td>1.2</td>
<td>1</td>
</tr>
<tr>
<td>$u_{max}$ (m)</td>
<td>2.14</td>
<td>0.571</td>
<td>0.154</td>
<td>0.0495</td>
<td>0.0148</td>
<td>0.00367</td>
<td>0.0753</td>
</tr>
<tr>
<td>Constant $UCS_{RM}$</td>
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<td>$E_2$</td>
<td>$F_2$</td>
<td>$G$</td>
</tr>
<tr>
<td>$R^*$</td>
<td>7.5</td>
<td>6.3</td>
<td>5.0</td>
<td>3.3</td>
<td>2.2</td>
<td>1.6</td>
<td>1</td>
</tr>
<tr>
<td>$u_{max}$ (m)</td>
<td>2.14</td>
<td>1.25</td>
<td>0.632</td>
<td>0.242</td>
<td>0.0585</td>
<td>0.00167</td>
<td>0.0753</td>
</tr>
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</table>
that capture the influence of large plastic zone development on the longitudinal displacement profile. Best fit curves to the modeling results are shown in Fig. 10.

Equation (5) gives the relationship between normalized maximum plastic radius (at tunnel completion), $R^*$, and normalized closure $u_0^* = u_0/u_{\text{max}}$ at the face ($X^* = 0$). Equations (6) and (7) give the best fit longitudinal displacement profile for $X^* \leq 0$ and
As a function of normalized maximum plastic zone radius. The correlation with 2D model data is shown in Fig. 10.

\[
u = \frac{u_0}{u_{\text{max}}} = e^{X^*} \quad \text{for} \quad X^* < 0 \quad \text{(in the rockmass)} \tag{6}
\]

where \( u_0^* = u_0/ u_{\text{max}} \) is given by Eq. (5).

\[
u^* = 1 - (1 - u_0^*) \cdot e^{-X^*/R^*} \quad \text{for} \quad X^* \geq 0 \quad \text{(in the tunnel)} \tag{7}
\]

where \( R^* = R_p/R_T \).

5. Application

The new relationships summarized in Eqs. (5), (6) and (7) can be used to correlate displacement to position (along the tunnel) in order to accurately sequence support installation in staged 2D plane strain analyses (simulated tunnel advance through tunnel boundary relaxation). For 2D analysis, \( u_{\text{max}} \) and \( R_p \) need to be calculated prior to the sequenced analysis. The sequencing of the plane strain analysis can be accomplished through a core replacement technique (repeated replacement of the material within the tunnel core results in a force imbalance at the tunnel boundary that is resolved to equilibrium during subsequent convergence increment), or by progressively relaxing the tunnel boundary tractions from in situ to zero in stages. These two techniques are illustrated in Fig. 11. In these cases, the convergence axis values or the
convergence values at each analysis stage can be converted to location along the tunnel using the LDP obtained from Eqs. (5), (6) and (7).

The equations can be directly incorporated into convergence-confinement analysis as maximum displacement and maximum plastic zone radius are primary

Fig. 10. Correlation between model data (from Table 3) and best-fit longitudinal displacement profiles (Eqs. (6) and (7)): a) constant $p_0$ analysis; b) constant $UCS_{RM}$ analysis
outputs of the analytical process. The convergence or normalized closure axis or analysis parameter can be converted using the LDP. The implications of using an elastic LDP (such as in Eq. (1)) in cases with large yield zones \( R^* > 2 \) and wall convergence \( R^k > 2 \) is illustrated in Fig. 12. In this example \( R_T = 5 \text{ m} \),

![Convergence Confinement Analysis of Deep Tunnels](image)

**Fig. 11.** Approaches for plane strain simulation of tunnel advance: a) replacement of tunnel core with unstressed elastic material (tunnel core reaction is shown as dashed line – core replacement results in a force imbalance which is resolved to equilibrium during subsequent convergence increment); b) incremental reduction (dashed line) of tunnel boundary tractions to simulate progressive advance.

![Convergence Confinement Analysis of Deep Tunnels](image)

**Fig. 12.** Example comparison of Factor of Safety (F.S.) obtained through simple analytical convergence-confinement analysis using 3 different LDP’s to guide support installation. Support (see text) is installed at 2.5 m from the face in all cases. \( R^* = 5 \) in this example.
the calculated maximum normalized plastic radius $R_p/R_T = 5$ and the maximum convergence, $u_{\text{max}}$, is 1.4 m (using the methodology of Carranza-Torres and Fairhurst 2000 and elastic modulus estimation based on Hoek and Diederichs, 2006). In this example, a system of 20 cm I-Beam
arches at 0.5 m spacing is installed at a nominal 2.5 m from the face. The stiffness and capacity of this support system is calculated according to Hoek and Brown (1980).

If the elastically derived equation ("Panet" for example) is used in this example the recommended timing of support installation (here shown with at 2.5 m or $x^* = 0.5$ giving a Factor of Safety of 6.7) will be erroneous and non-conservative. If the generic empirical formula ("Chern") from Eq. (3) is used, the apparent Factor of Safety, for installation at 2.5 m, drops considerably to 1.25. If the LDP from Eqs. (5), (6) and (7) ("Vlachopoulos and Diederichs") is used, then the Factor of Safety for installation at 2.5 m drops to 0.6 and support failure is predicted. Recommendation for completed support installation will be after significantly more convergence has taken place (possibly necessitating the use of sliding joints in the support rings as discussed in Shubert, 1996; Hoek et al., 2008).

The foregoing example is based on the assumption that the unsupported LDP can be used to appropriately locate the point of support installation. An alternative approach is to perform the analytical or numerical analysis with the design support pressure (reduced by the desired factor of safety). The final deformation, $u_{\text{max}}$, and the final plastic radius, $R_p$, of the supported tunnel can be used in Eqs. (5), (6) and (7).

6. Conclusion

Using a series of numerical analyses, a new series of functions defining robust longitudinal displacement profiles, as a function of maximum normalized plastic radius, has been developed. This approach takes into consideration the effect that a large ultimate plastic radius has on the rate of development of wall displacements with respect to location along the tunnel. Previous LDP functions are inadequate for tunnel analysis in very weak ground at great depth. This approach is valid from the elastic case through to complete plastic closure of the tunnel (as calculated using numerical or analytical solutions).

Equations (5), (6) and (7) define a relationship for a complete LDP ($X \leq 0$) that can be incorporated directly into analytical solutions or used for calibration of staged numerical models. As an alternative to these equations, Fig. 13 provides a graphical template for this purpose. The equations and graphical tools presented here are for short term displacements occurring as a function of tunnel advance only. Where time-dependent squeezing is anticipated, this approach will need some modification following the guidance provided by Pan and Dong (1991). Care must be taken, in this case, to appreciate whether the time-dependent squeezing is accompanied an increase in or is independent of the plastic radius.

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Special thanks are due to Drs. Evert Hoek, Carlos Carranza-Torres, Brent Corkum and Giordano Russo for their insightful discussions on this topic as well. We are also grateful to Matt Lato for early assistance with this work. This work has been funded by the Natural Science and Engineering Research Council of Canada as well as the Province of Ontario via the PREA program.
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Issues Related to Two-Dimensional (2D) Tunnel Analysis

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ABSTRACT: In spite of the gradual development of 3D analysis packages utilizing finite element models or finite difference algorithms for stress-strain calculations, 2D analysis is still used as the primary tool for tunnel behaviour and tunnel support analysis and design. 2D finite element analysis or analytical convergence confinement solutions, for example, depend on an independence between the ground reaction curve and the support resistance. In addition, the longitudinal displacement profile, prior to support is assumed to be independent of the support effect. Also it is assumed that non-isotropic stresses and non-circular geometries can be handled in the same way as circular tunnels in isotropic conditions. The process involves generating a ground reaction curve (internal tunnel wall resistance versus tunnel closure) and calibrating this using a standardized longitudinal displacement profile (LDP). This paper examines the validity of these assumptions and the error inherent in these extensions to 2D tunnel analysis. Anisotropic stresses and lagged (staged) excavation present a particular problem. Solutions are proposed for support LDP’s in simplified conditions.

Key Words: Weak rock masses, tunnel convergence, linear displacement profile (LDP), 2D and 3D numerical modelling techniques for tunnelling.

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1.0 Introduction

Tunnelling is an inherently three-dimensional process. The advancing tunnel face creates a complex three-dimensional stress path as explored by Eberhardt (2001) and also generates a three dimensional bullet-shaped zone of plasticity in soft rock. This developing plasticity or yielding zone, combined with the elastic closure of the surrounding rock mass creates a wall displacement profile (Figure 1) that is non-linear, develops partially before the advancing face and continues for a number of tunnel diameters before equilibrium conditions are achieved. This profile, known as the longitudinal displacement profile (LDP) is a function of tunnel radius and the extent of the ultimate plastic radius. This relationship is explored in detail by Vlachopoulos and Diederichs (2009) for the axisymmetric condition (tunnel geometry and stress).

Figure 1. Profile view of radial displacements ($u_r$) for an unsupported tunnel near the face.
This chapter examines issues related to two dimensional modelling, both as plane strain and as axisymmetric configurations and the relationships to true three-dimensional effects. In order to accurately simulate the loading of support or the effects of sequential excavation, the two dimensional model must capture the pre-face conditions, the state of displacement and plasticity at the face and the subsequent development of deformation and yielding as shown in Figure 2. It is important to examine these issues as two dimensional modelling is still very much state of practice for tunnel engineering analysis (Hoek et al., 2008).

Figure 2. Highlights the requirement to develop 2D analogues to a fundamentally 3D phenomenon. Must take into consideration the strength issues provided by the faces that are associated with successive excavation in a 2D sense.
The basic premise of 2D tunnel modelling is that the tunnel boundary moves (normally inwards) progressively as the tunnel face passes the model section. Ultimately a stable tunnel closure is reached (for elasto-plastic analysis without strain softening and without ground surface interaction). This inward displacement of the tunnel boundary can be simulated by replacing the “rock” inside the tunnel with an outward pressure $p_i$ (initially equivalent to the in situ pressure $p_o$) and reducing this internal pressure to zero over a number of model steps as shown in Figure 3.

![Figure 3](image)

**Figure 3.** Relationship between internal pressure, plastic radius, tunnel closure and position.

This reduction of the internal “support pressure” results in a redistribution of stress within the model and can lead to yielding of the rock mass around the tunnel. A
plastic zone initiates in front of the tunnel or some time after the passage of the tunnel face and grows to a maximum, coincident with the maximum tunnel closure. Internal pressure and plastic zone radius are linked to tunnel closure (radial displacement). Closure, in turn is linked to the actual axial position in the tunnel, relative to the tunnel face, through the longitudinal displacement profile or LDP.

Vlachopoulos and Diederichs (2009) made significant improvements to the calculation of the LDP for tunnels with extensive plastic zone development. These developments were directly intended for unsupported, full face circular tunnels in isotropic stress conditions. As described in the next section, this technique is readily extrapolated to staged excavation and supported tunnels in practice. This chapter will examine the limitations and caveats associated with this approach.

2.0 Model Approaches

A number of modelling approaches are discussed in this chapter. The following is a brief discussion of the model construction and execution. For the purposes of this investigation, Phase2 (Rocscience Inc., 2004) was used for the 2D numerical analysis and FLAC3D (Itasca, 2005) was used for the 3D numerical analysis. Phase2 utilizes the implicit Finite Element Method (FEM) (i.e. solves the mathematical relations) while FLAC3D employs the Finite Difference Method (FDM) (i.e. solves the physics of the problem) in its determinations. Both of these program are widely used in the rock mechanics community for design purposes as well as to capture the behaviour of a tunnel (i.e. stress re-distributions and displacements) associated with tunnel excavation. Chai (2008) has investigated the numerical modelling codes for Phase2 and FLAC (2D) (the
basis of FLAC3D) on the influence of stress path on tunnel excavation response and these
findings will not be repeated here. Chai stated that one software package was not
superior to the other, rather he points out the importance of understanding the program
codes and selecting the right tool and modelling approach to represent the expected stress
path as close to reality as possible. The emphasis in comparison therefore, should not lie
in the limitations of the software packages but on the details of how the true physical
phenomenon is being modelled.

2.1 3D Finite Difference Plasticity Model

FLAC3D (Itasca, 2005) is an explicit finite difference program that is used to
study the mechanical behaviour of a continuous three-dimensional medium as it reaches
equilibrium or steady plastic flow. This model has been discussed elsewhere in this paper
and is reviewed here. The response observed derives from a particular mathematical
model and from a specific numerical implementation. The mechanics of the medium are
derived from general principles (definition of strain, laws of motion), and the use of
constitutive equations defining the material. The resulting mathematical expression is a
set of partial differential equations, relating mechanical (stress) and kinematic (strain rate,
velocity) variables, which are to be solved for particular geometries and properties, given
specific boundary and initial conditions. It is the inertial terms that are used as means to
reach (in a numerically stable fashion) the equilibrium state. The solid body is divided
into a finite difference mesh of 3D zones. Within a cycle, new velocities and
displacements are determined from forces and stresses using the equations of motion. Strain rates are determined from velocities and new stresses from the strain rates.

The advantage of using the explicit finite difference formulation is that the numerical scheme stays stable even when the physical system may be unstable. This is particularly advantageous, when modelling non-linear, large strain behaviour and actual instability. FLAC3D checks the element state at each time step with respect to the yield criterion. However, the disadvantage of the time-marching explicit scheme of FLAC3D is that calculation times can be longer than those of implicit formulations although memory requirements are reduced as explicit methods do not have to store matrices for calculations of equilibrium.

The 3D models that were developed for the purposes of this investigation can be seen in Figure 4. These models consist of circular and horseshoe excavation geometries incorporating sequential excavation and support. The numerical models that were created uses FLAC3D group zones. The models were 110 m in height and 110 m wide with a tunnel length of 100 m (depth). The excavated material within the tunnel was created separately and was subdivided into sub-sections that constituted an excavation step and could be separated into full-face or top heading/bench excavation. At each excavation step, an excavation sub-section block was nulled and steps were conducted in order to ensure equilibrium conditions were met prior to the next excavation sequence. The tunnel lining consisted of a 30 cm thick shotcrete layer that was replicated using liner elements.
Figure 4. Geometry and boundary conditions of the FLAC3D Finite Difference Model
2.2 Phase2 – Plane Strain 2D Numerical Modelling

Phase2 is a 2D, implicit, elasto-plastic finite element method program used to calculate stresses and determine displacements around underground tunnels and can be used to solve a broad range of geotechnical problems. PHASE2 (RocScience 2005) is based on the finite element formulation and the strain-softening/hardening formulations described in Owen and Hinton (1980) and Chen (1982), respectively. The load stepping and iterative plastic solution described by these authors is used here. Phase2 uses plane strain analysis whereby two principal in-situ stresses are in the plane of the excavation and the third principal stress is out of plane. As with other finite element regimes, the domain is discretized into a set number of elements and corresponding nodes. Displacements within these finite elements are calculated based on shape functions tied to the nodes of the elements (Rocscience Inc., 2004). Initial in situ stresses, tolerance parameters, material and defect properties are all assigned by the user. Tunnel excavation is simulated by the removal of elements from within an excavation boundary located in an external boundary.

In terms of solution process, convergence and stopping criterion, matrix formulations of algebraic linear equations are used to solve for differential equations. In a simple example, a stiffness matrix relates loads to displacements. Gaussian elimination is used to solve for the system of equations. Criteria for equilibrium are based on the absolute and square root energy criterion as well as force and displacement criterion. These criteria verify that, for a given load step, iterations are ceased when the energy imbalance of the current state becomes a small fraction of the initial energy imbalance.
(i.e. energy imbalance on the first iteration). If this condition is not satisfied during a
specified maximum number of iterations, the solution process is deemed not to have
converged. In terms of force and displacement, iterations are halted when the current
force imbalance becomes a small fraction of the total applied force (i.e. current load
level) or initial displacement respectively. Iterative procedures of static equilibrium are
used in order to relate the current stress state to that of an elastic solution. If the stress
state is in excess of the yield criteria, plastic deformation has taken place (Rocscience
Inc., 2004). The 2D models that were developed for the purposes of this investigation
can be seen in Figures 5 and 6. Both fixed displacement and constant pressure boundary
conditions are used in this analysis.

![Figure 5](image)

**Figure 5.** a) PHASE2 model with fixed boundary conditions some distance
(normally expressed as a multiple of tunnel radii) from the tunnel. Detail of b) circular
and c) horseshoe tunnel geometries.
2.3 Phase2 – Axisymmetric 2D Numerical Modelling

Phase2 also allows computation in axisymmetric mode. In this formulation, a two-dimensional model geometry is used with a single axis of symmetry about which the model is assumed to be three dimensional through geometric rotation. In this formulation the stress-strain relationships (with respect to deformation in the plane) are a function of radial distance from the axis of symmetry (see Brady and Brown 1993, for example). In these analyses, the tunnel is advanced in steps from the bottom up in a model that is centered on the tunnel axis as shown in Figure 7. The excavation is done incrementally by removing the tunnel material. Vlachopoulos and Diederichs (2009) showed that a tunnel advance increment below 0.4D was sufficient to simulate continuous excavation without error. These models advance at a rate of 0.2D or 2m for a 10m tunnel (5m radius).
2.4 Review of Selected Methods of Analysis for Capturing 3D Effects in 2D

As the effect of an excavation in a rock mass is clearly a 3D phenomenon, the ensuing deformations cannot be simulated directly in 2D finite element plane strain analysis. 2D, axi-symmetric modelling does replicate 3D effects for very simple cases (circular geometry and isotropic material and stress). In 2D plane strain, the progressive displacement of the tunnel boundary must be recreated in accordance with the appropriate linear displacement profile. If done correctly, this will capture the progressive development of loads and displacements in tunnel geometries and in support elements that respond in the radial plane (liners and bolts for example, but no forepoles and face
support). The LDP is recreated implicitly in 2D plane strain. The methodologies commonly employed in current design practice for 2D modelling to mimic real 3D effects are (Figure 8):

- Straight Excavation;
- Field Stress Vector / Average Pressure Reduction;
- Excavation of Concentric Rings; and,
- Face De-stressing (with or without softening).

Karakus (2006) also cites other methods that are employed in this regard.

### 2.4.1 Straight Excavation

This method is simply the full-face excavation usually associated with hard rock masses of a homogeneous nature and with primitive tunnel geometries. Initially, the material properties of the geo-material as well as the simplistic tunnel shape are input.

In the next sequence of 2D numerical modelling, the material is excavated in its entirety allowing instantaneous displacements to be determined without giving any other consideration to possible influencing mechanisms (i.e. support provided by the face, 3D plasticity effects in front of the face, capturing of preconvergence etc.). This is a simplistic excavation technique that does not take into account the stress re-distribution and ensuing deformations that occur during the sequential excavation and advance of the tunnel that can be more effectively simulated in 3D numerical models.
Figure 8. Methods or advanced strategies used in 2D Numerical Analysis in order to approximate the uniquely 3D behaviour associated with rock tunnel excavation.
2.4.2 Average Pressure Reduction (Convergence-Confinement Method)

Convergence-confinement analysis or the stress relief method (Panet 1995, 1993; Carranza-Torres and Fairhurst 2000; Duncan-Fama 1993 and others) is a widely used tool for preliminary assessment of squeezing potential and support requirements for circular tunnels in a variety of stress states and geological conditions. An internal pressure ($p_o$), initially equal to the in-situ stress is applied on the inside of the excavation boundary. The pressure is incrementally relaxed until the excavation boundary condition is effectively zero normal stress. The extent of plastic yielding and thereby, the boundary deformation is calculated at each stage of the process. The result is a continuous representation of the deformation-internal pressure relationship for the tunnel given a particular material strength, deformability, dilation and stress state. The internal pressure is, not a direct representation of real effects, however, it is a substitute for the effect of the gradual reduction of the resistance due to the effect of the distancing supporting tunnel face. The internal pressure that is coupled with a given boundary displacement is a measure of the amount of support resistance required to prevent further displacement at that point in the progressive tunnelling model. The stress is applied normal to the inner boundary and idealizes the progressive stress state. This is also referred to as the load step method, as there is an incremental reduction of tunnel boundary tractions that simulate advance.

2.4.3 Field Stress Vector

In cases where the initial stresses are not isotropic, the boundary pressure in the convergence-confinement technique must be replaced with a traction vector with shear ($\tau_o$) and normal ($n_o$) components is applied to each tunnel boundary element to replace
the in-situ stress acting on the element plane pre-tunnel. In this technique, in terms of the Ground Reaction Curve (GRC) (i.e. convergence versus internal support pressure), there is an incremental reduction (dashed line) of tunnel boundary tractions that simulate progressive tunnel excavation advance. This technique has been recently incorporated into Phase2 and will be used here.

2.4.4 Concentric Disks of Excavation Method

This is an outdated method that excavates the tunnel cavity in stages concentrically from the centre of the tunnel to the outer boundaries of the desired tunnel diameter (or shape). This can be seen in Figure 8 for a circular tunnel. Each excavation disk that is nulled in this system of excavations represents a different stage of tunnel advancement. In a 3D sense, the excavation of the central disk represents a weakening of the material ahead of the excavated face while the final ring that is excavated represents the open cavity and passing of the face past that location. This method can also be combined with softening or distressing of the material whereby one would reduce the Modulus of Elasticity (E_i) of the core material from its original value, E. This method is still used in practice but will not be discussed further here.

2.4.5 Face replacement or destressing.

Plane strain simulation of tunnel advance in this method involves the replacement of the tunnel core with unstressed, elastic material during each step. The tunnel boundary is allowed to converge during the subsequent model step until the stresses reestablish in the tunnel core and a temporary equilibrium is reached. The face is then replaced again and the process is repeated. In this way the tunnel works its way down the pressure-
displacement (ground-reaction) curve in a series of steps. This method was favoured in the past as the stress-vector technique was difficult to incorporate manually into a model. These two methods will be compared here.

The face replacement method can be made more efficient by progressive softening of each successive core replacement. Softening the face on its own will not create a response as the model functions on the basis of stress equilibrium (resetting the stiffness does not create a force imbalance in the model and therefore no direct response). Softening combined with face replacement (or distressing) results in an efficient excavation sequence simulation (Figure 9).

![Figure 9](image.png)

**Figure 9.** Combined face replacement (distressing) with material softening. The dashed lines represent the stress path of the inner tunnel core. Solid line is the response of the outer region.
2.5 Applying the Longitudinal Displacement Profile

The Longitudinal Displacement Profile (LDP) is one of the three basic components of the convergence-confinement method. A characteristic LDP diagram indicates that there is an amount of axial displacement at some distance ahead of the face (i.e. a zone of influence prior to excavation of the core beyond the face) and at a certain distance behind the face that the amount of displacement approaches a constant value (Carranza-Torres and Fairhurst, 2000). As shown by Vlachopoulos and Diederichs (2009) the normalized LDP ($d/d_{max}$ vs $X/R_t$) is a function of the ultimate plastic radius.

The first step in the analysis process is to determine the maximum plastic radius via a simple plane strain analysis of the unsupported tunnel or through an analytical solution such as that given by Carranza-Torres and Fairhurst (2000). Next, the longitudinal deformation profile can be calculated using the methodology of Vlachopoulos and Diederichs (2009). Alternatively, an axisymmetric model can be used for this purpose, facilitated by the assumed isotropic stresses and circular profile. A longitudinal deformation profile for an unsupported tunnel is developed as shown by the solid line (“Disp. vs Location”) in Figure 10.

A 2D finite element plane strain analysis was then applied to the full face construction sequence (unsupported). The technique of progressive face replacement (distressing) described in the previous section was applied in this case. At the end of each stage in the 2D model, the tunnel wall will have moved a certain distance.
Figure 10. Ground reaction curve, “Disp. vs Support Pressure” and corresponding longitudinal displacement profile “Disp vs Distance (unsupported)”. Normalized plastic radius \( R_p/R_t = 8 \) in this example. Point symbols and number ID’s represent corresponding stages in plane strain model (related symbols are linked horizontally between two curves as shown for stage 11 by dotted line).

In addition, there will be a certain pressure or traction on the surface (applied by the reloaded core in this case or applied directly in the stress-vector approach). The incremental displacement-pressure value pairs collectively define the Ground Reaction Curve (white diamonds on “Disp. vs Support Pressure” curve in Figure 10). Each stage can also be associated with a point on the LDP defining locations along the tunnel using the longitudinal deformation profile (shown for stage 11 by the dotted line). Model stages can be adjusted in “space” along the tunnel by adjusting the pressure increments or face replacement modulus values.
This methodology has been used in one form or another throughout the tunnelling industry. This technique works as intended for unsupported circular tunnels with isotropic stress fields - the same assumptions implicit in axisymmetric analysis (Figure 11).

![Figure 11](image)

**Figure 11.** Axisymmetric model equivalent to the 2D plane strain analysis in Figure 10

Inaccuracies can be expected when the plane strain analysis includes anisotropic stresses, staged support, non-circular geometries and staged excavations. The rest of this chapter will explore the significance of these inaccuracies and possible solutions.

### 3.0 Boundary Conditions and 2D Method Comparison

The comparisons that follow in the rest of this chapter were conducted using supported and unsupported simulations with elastic and elastic-perfectly plastic models (Mohr-Coulomb constitutive model within FLAC3D and Phase2). The materials and
input parameters were selected in order to span the spectrum of the ratio of rock mass strength to in situ stress and strain considerations. The suite is similar to that used in Vlachopoulos and Diederichs 2009. The parameters or properties associated with each material B1, C1, D1 and E1 are located in Table 1. As can be seen, materials B1, C1, D1 and E1 have $p_0/\text{UCS}_{RM}$ (in-situ pressure to rock mass uniaxial compressive strength) ratios of 8, 6, 4 and 2 respectively. Mohr-Coulomb equivalent properties and rock mass strengths were estimated as per Hoek et al. (2002) and the elastic moduli were estimated based on Hoek and Diederichs (2007).

Table 1. Parameters used for 2D and 3D model comparisons

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<th>D1</th>
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*Values obtained using Rocklab software based on Hoek et al. 2002.
3.1 Boundary Conditions

It is important to establish the influence of 2D boundary conditions to ensure that valid comparisons can be made. Two options are explored here - fixed displacement (=0) outer boundary conditions some distance away from the excavation (as in Figure 5) and free boundary conditions with an in-situ boundary traction applied (Figure 6).

The results of the FLAC3D analysis (Figure 4a) are also compared here with the analytical solution for displacements into a circular tunnel (as per Brady and Brown 1993 for example). The comparison is shown in Figure 12 illustrating that the boundary conditions and mesh accuracy are acceptable for the 3D models.

Figure 12. Elastic LDP results calculated using FLAC3D circular tunnel models (isotropic stresses and elastic properties as per Table 1. Comparison with 2D analytical solution for final displacement is shown.
Figure 13 illustrates the comparison between the FLAC3D LDP’s, normalized with respect to the respective analytical solution for maximum elastic closure, with the 2D solutions based on fixed boundary conditions 6 12 and 32 radii from the tunnel and with free boundary conditions 12 radii from the tunnel. The latter is exact (coincident with the analytical solutions as is the FLAC3D results. Since the kinematic control of a free boundary is more difficult when the tunnel is not circular, fixed 2D boundaries at 16 to 20R are used for the rest of this work.

Figure 13. Elastic LDP results calculated using FLAC3D circular tunnel models (isotropic stresses and elastic properties as per Table 1. Comparison with 2D analytical solution for final displacement is shown.

Finally, it is necessary to compare the normalized LDP’s from the FLAC3D analyses with the axisymmetric models used in this chapter and to compare both to accepted analytical formulations for the longitudinal displacement profile. It can be seen from Figure 13 that the normalized LDP’s from the FLAC analysis are independent of
elastic modulus. Figure 14 shows that this normalized profile is coincident with the analytical formulation by Unlu and Gercek (2003). The axisymmetric analysis with a fixed boundary at 30R shows good correlation.

![Normalized Radial Closure](image)

**Figure 14.** Elastic LDP results calculated using FLAC3D circular tunnel models (isotropic stresses and elastic properties as per Table 1.) Comparison with 2D axisymmetric solution and analytical result.

### 3.2 3D and Axisymmetrical LDP’s

The plastic LDP’s for the FLAC 3D models are now compared with the equivalent axisymmetric 2D results in Figure 15 illustrating that they are acceptably coincident. The semi-analytical LDP function proposed by Vlachopoulos and Diederichs 2009 was based on axisymmetric modelling. In Figure 16, the FLAC 3D results are compared with this function. The developing plastic radii from the FLAC3D models are also shown in this figure. The LDP functions are based on the final value of $R_p/R_t$. 
Figure 15. Plastic LDP results calculated using FLAC3D circular tunnel models. Comparison with 2D axisymmetric solution.

Figure 16. Plastic LDP results calculated using FLAC3D circular tunnel models. Comparison with 2D analytical (calc) solution from Vlachopoulos and Diederichs 2009. Development of plastic radii in FLAC3D models is shown.
3.3 Comparison of 2D Plane Strain Methodologies

An idealized 2D model with a circular tunnel, 6-noded triangular elements arranged in an expanding radial grid with fixed boundaries at 32R from the tunnel, and isotropic stress conditions is initially used to compare the Ground Reaction Curves generated using the stress-vector (pressure) method and the face-replacement (modulus) method described in Section 2. In this comparison, 20 steps are used to regenerate the GRC. In order to provide similar load/displacement steps, the “modulus” method is executed first and the internal pressure increments from this analysis are used as input into the “pressure” method. The “internal normal pressure” is queried at the tunnel boundary after each stage. The displacements are given directly. Figure 17 shows that the process is not sensitive to the method used.

![Figure 17](image)

**Figure 17.** Comparison of Ground Reaction Curve for idealized circular tunnel plane strain analysis.
Next the same comparison is made using a more practical grid (randomly generated - 3 noded, boundaries at 32R) for both the circular and horseshoe geometries. Results in Figure 18 shows that the “pressure” method is less sensitive to element type (more deviation between roof, floor, wall) in the circular case, and both are subject to deviations caused by non-ideal geometry (in the case of the horseshoe). It is important to keep this level of inherent error in mind when evaluating the effects of support, stress ratio, sequencing, etc. The average convergence-confinement (GRC) results are compared for two shapes and two methods in Figure 19.

![Figure 18](image.png)

**Figure 18.** Comparison of GRC’s generated using 2D plane strain analysis, Left: horseshoe tunnel. Right: circular tunnel. Isotropic stress field = 28MPa, material C from Table 1. “Modulus” refers to the face replacement method while “Pressure” refers to the stress vector method.
Figure 19. Comparison of average GRC’s generated using 2D plane strain analysis using two methods: “Modulus” refers to the face replacement method while “Pressure” refers to the stress vector method.

4.0 Limitations of the 2D, LDP based simulation of 3D tunnelling.

This section will summarize a series of investigations to determine the limitations of 2D FEM modelling to simulate 3D tunnel advance using the LDP approach outlined in Section 2.0.

4.1 Excavation Shape

Figure 20 compares the plastic zone development and the associated LDP’s for the circle and horseshoe tunnels under hydrostatic stress. Two tunnel strength/stress ratios are used here. This comparison, combined with Figure 19 demonstrates that within the limits of error inherent in the FEM analysis, the LDP plane strain analysis procedure
outlined in Section 2.0 is valid for non-circular shapes, even if the LDP is based on the correlated LDP functions of Vlachopoulos and Diederichs (2009) for circular tunnels. The validity of this approach is likely reduced as the aspect ratio of the non-circular opening increases.

Figure 20. Comparison of LDP’s for circular and horseshoe tunnels. Bottom: Plastic zones are shown for the two tunnel shapes and material C. Plastic zone for FLAC 3D (circle) is shown in long section.
4.2 In-Situ Stress Ratio

The LDP procedures developed for 2D modelling are based on an isotropic stress field ($k=1$). A brief examination is performed here to determine whether this is a significant limitation for the approach. Figure 21 represents results for material C under a horizontal stress ratio of 1.5 (28 MPa vertical stress for deeper tunnels). The normalized deformation profiles for the walls and for the roof/floor are different. The stress ratio axial to the tunnel has a minimal impact. Using the LDP function derived for isotropic stress (Vlachopoulos and Diederichs 2009) does not work for either horizontal or vertical plastic radius. However, the LDP derived for the case $k=1$ does seem to follow the deformation profile for the vertical direction (direction of maximum yield).

![Figure 21. Comparison of 3D LDP’s with derived functions based on k=1.5](image_url)
Figure 22 illustrates the same comparison with $k=0.67$ (same vertical stress). Here again the LDP’s are different for different directions even though each LDP is normalized to itself. The LDP for the case of $k=1$ best approximates the deformation profile for the horizontal direction (maximum yield). This is consistent with the previous example.

Figure 22. Comparison of 3D LDP’s with derived functions based on $k=0.67$

Clearly, these examples show a deviation from the assumptions inherent in this process and point to the need for 3D analysis. For practical purposes, it seems that using the LDP based on yield for $k=1$ can be used to calibrate 2D models using the direction of maximum yield and movement. More investigation here is warranted.
4.3 Sequential Excavation

One assumption that is generally accepted in practice is that once a 2D sequenced model is calibrated based on the LDP for a single excavation phase, each subsequent stage can use the same sequence of face replacement or pressure reduction to simulate the 3D advance (of a bench after a top heading for example). This assumption is demonstrated schematically in Figure 23. Here an LDP has been established for a single unsupported opening. Through the methodology in Section 2.0, this LDP is translated to a sequence of internal pressure reduction increments to stage the excavation in 2D. This is valid as shape is not a major influence. The next step, however, involves using the same sequence of internal pressure reduction to “excavate” the bench. Only selected steps are shown (20 steps in all in this example).

![Figure 23](image)

**Figure 23.** Selected excavation steps (using internal pressure (stress vector) reduction. Same sequence is used for top heading and bench.

In order to evaluate the validity of this practice, a series of sequenced (top heading and bench) excavation simulations were analyzed using FLAC3D as shown in Figure 24.
Figure 24. FLAC3D model LDP’s of sequenced excavation (as per inset).

Figure 25 illustrates a potential difficulty with the standard 2D approach. The normalized LDP’s for different timings of bench excavation differ considerably from the single excavation (in terms of percentage of displacement at the passing of the bench). The convergence confinement approach (analytical or in 2D plane strain) assumes that the excavation stages are independent. This is not the case here as the approaching bench softens the ongoing response of the initial top heading excavation (as the tunnel face moves on). Figure 26, on the other hand shows that for practical purposes, the same sequence may be used for a second excavation stage provided the distance between the stages is sufficient for them to act independently.
Figure 25. Normalized LDP’s from FLAC3D. Left: normalized to individual maximum displacements. Right: normalized to top heading only.

Figure 26. Left: Full face excavation compared with top heading and bench excavated 20R apart. Right: Normalized LDP’s for Segment 1 and 2 from left image (isolated heading and bench).
5.0 Support Modelling

The primary purpose of convergence-confinement analysis using LDP’s is to properly locate support installations within the deforming tunnel to optimize the liner resistance without exceeding the liner strength. This approach is demonstrated in Figure 27 for an analytical solution. The methodology is the same (using Section 2.0) for plane strain analysis. This section will examine the installation of a 30cm concrete lining with 160mm steel at 1m. This support will be installed at different steps in the model.

Figure 27. Screen shot from RocSupport software (Rocscience) illustrating the use of Vlachopoulos and Diederichs (2009) LDP for locating support installation. Bilinear line represents liner response. Curved line is ground response.

5.1 Excavation Shape

Figures 28 and 29 illustrate LDP’s generated through FLAC3D analysis for different excavation shape, liner configurations and support sequencing. It can be seen from these analyses, that the LDP’s for the supported and unsupported tunnels are similar up to a point just before installation as long as the liner is several radii from the face.
Figure 28. Unsupported and supported LDP’s for horseshoe tunnel, generated with FLAC3D. (Material D)

Figure 29. Unsupported and supported LDP’s for horseshoe tunnel, generated with FLAC3D. (Material C and D)
When the liner is less than 3 radii from the face, the error incurred in using the unsupported LDP to locate the installation point for the liner may become unacceptable. In order to address this problem, at least for the case of isotropic stresses, a series of axisymmetric runs with staged liners was analyzed. The normalized LDP’s are shown in Figure 30. The LDP’s are consistently sigmoidal in nature and relatively insensitive to material type when normalized.

Figure 30. Supported LDP’s generated through axisymmetric FEM analysis. Materials B,C,D and E were used here. Support installed at different distances from the face.
A version of a sigmoid function was developed here to provide a best fit LDP as a function of face distance and support installation position:

\[
\frac{u}{u_{\text{max}}} = \frac{1}{1 + e^{0.6 \left[ 1 - 0.1 \left( \frac{S}{R} \right) \left( \frac{S}{R} - \frac{S}{R} \right) \right]}}
\]

Eq’n 1

where X is the distance from the face, R is the tunnel radius and S is the distance between the face and the support. The function is shown against the model data in Figure 31. This function can be used to iteratively position support in a 2D model to achieve the correct ratio between face displacement and final supported displacement.

Figure 31. Same data as Figure 30 with best fit sigmoid function overlain.
6.0 Implications

The primary purpose of convergence-confinement analysis using calibration via LDP’s is to properly locate support installation. In squeezing ground, early installation of a liner can result in overloading of the liner and failure. Late installation will incur excess ground displacement and ground disintegration. It is therefore critical to properly “locate” the point of support installation within a staged 2D excavation model.

In conventional convergence-confinement analysis, the unsupported LDP is used to correlate the ground reaction curve (displacement vs internal pressure) or 2D analysis stages (face replacement or pressure reduction stages) with location along the tunnel. In the simple example in Figure 32, a 30cm concrete liner with 160mm steel sets @ 1m spacing provides an estimated 15MN/m of hoop thrust capacity in a circular liner (Hoek and Brown 1980). The GRC analysis (as per Carranza-Torres and Fairhurst 2000) combined with the LDP of Vlachopoulos and Diederichs (2009), show that this liner, installed at 2m from the face in a 5m radius tunnel with properties of material C, will have a factor of safety of 1.

Consider, however, the axisymmetric analysis results presented in Figure 33. In this analysis, the ground reaction prior to support is not independent of the support as in the previous analysis. A series of 8 analyses are summarized in this plot showing the final liner load versus the installation distance to the face. In addition, the ultimate plastic zone and tunnel wall displacement are also affected by installation distance. This analysis shows that the minimum distance between face and liner should be 8m and that the liner analyzed in the previous example (installed at 2m) would fail.
Figure 32. Ground reaction analysis of a lined tunnel (liner at 2m).

Figure 33. Summary of axisymmetric analyses of tunnel liner installed at different distances from the face.
Another example of the importance of support placement is given by the following. A 5m radius tunnel is excavated in material C conditions at 1200m. The horizontal stress ratio is 1.25. A 30cm concrete liner with 160mm steel sets at 1m is installed. For simplicity in this schematic example, the concrete is assumed to achieve full strength immediately. Typical concrete curing rates yield over 80% of maximum strength and stiffness after 28 days and this can be incorporated into the sequencing. The sensitivity of the composite liner to installation location (with respect to the face) is shown in Figure 34. The moment-thrust capacity envelope is given for a factor of safety equal to 1. Clearly it is important to accurately locate this installation point within a staged 2D numerical or analytical model.

Figure 34. Moment-Thrust data and capacity for tunnel liner.
If the two dimensional model can be calibrated using the techniques and abiding by the cautions in this paper, it is a very powerful tool. Consider the case shown in Figure 35.

**Figure 35.** Example analysis of a twin tunnel problem - a) unsupported, b) support configuration in 2D model and c/d) supported and unsupported displacements (x10). Unsupported LDP used for a) and supported LDP (Eq’n 1) used to locate support in b) at 4m from the face.
Taking two materials with strength to in situ stress ratios of 3.2 and 5.8 and using the strength - closure plot of Hoek and Marinos 2000, it is possible to analyze and plot the results of a number of progressively more complex analyses based on the twin tunnel example shown in Figure 35. Here forepoles are modelled as a slightly stiffer zone above the tunnel and the liner is modelled as a material rather than a liner element.

The results of the example analyses are compiled in Figure 36 for the unsupported case, the supported case and for single and twin tunnels (with and without dilation post yield. This example is merely to illustrate the utility of 2D models provided that there is an understanding of the limitations as discussed in this paper.

Figure 36. Compiled 2D FEM results for the tunnel analysis in Figure 35.
7.0 Conclusions

The conventional approach of 2D tunnel analysis, calibrating excavation stages with an LDP derived from simple 3D calculations based on an unsupported circular tunnel in isotropic stresses, has been examined in detail in this paper with the following conclusions:

- Boundary conditions are important to analysis of squeezing ground problems. Fixed boundaries should be a minimum of 10 radii from the tunnel or 3 plastic radii away from the plastic zone.

- For simple tunnel geometries, the 2D LDP and GRC is not sensitive to the choice of face replacement or pressure reduction technique but is sensitive to the step size (face too soft or pressure increment too great).

- Tunnel shape is not an important factor provided the aspect ratio of tunnel geometry is not extreme.

- Non-isotropic stresses render the standard LDP approach inaccurate. For mild values of stress ratio, k, some assumptions and adjustments can be made to make the approach practically viable.

- Sequenced excavation such as top heading and bench excavation poses a problem for the LDP approach unless the second excavation stage is distant from the first.

- A new LDP is required for stiff liners installed within 2 to 6 radii of the face. For installations closer than 2 radii, 3D analysis may be required.

- It is critical to correctly locate the installation step within a staged 2D modelling sequence to prevent overloading or excess deformations.
8.0 References


Chapter 9
Conclusions and Recommendations

9.1 General

The objectives of this research study were to determine effective methods of modelling tunnel construction within weak masses with a view of improving these analytical techniques for design purposes (to include accurate tunnel displacement predictions and ultimately optimization of temporary support design measures). In the accomplishment of this aim, the specific contributions and conclusions are cited below. As well, recommendations in order to improve this research investigation are also listed along with future work that the author hopes to undertake in the near future.

9.2 Areas of Contribution

The specific areas of contribution to the field that were addressed by this investigation are summarized below:

- Shared the lessons learned from the Driskos Case Study and provided a confirmation that 3D continuum methods accurately captured tunnel convergence as determined by field measurements;
- Produced a retrospective evaluation of the engineering geology along the Driskos Tunnel alignment and, in particular, developed a subdivision within the weakest/deepest unit for use in design back analysis;
- Demonstrated that the GSI system for characterization is appropriate for design if the ground is consistent through the design domain. GSI based performance prediction becomes difficult in heterogeneous terrain;
- Demonstrate the effectiveness of a semicircular arch support system while demonstrating the difficulties with staged excavation and linear liner elements appended to the top heading arch system;

- Assessment of the potential and limitations of 2D numerical methods of tunnelling in weak rock based on 3D numerical analysis (with validation of the 3D model from the field); and,

- Developed, using a series of numerical analyses, a new series of functions defining robust longitudinal profiles as functions of maximum plastic radius. Functions have been incorporated in commercially available numerical modelling software;

### 9.3 Conclusions

The main conclusions of the research that was undertaken are:

- Three-dimensional (3D) modelling should be used when attempting to design near face support for tunnelling projects in complex rock masses versus conducting two-dimensional (2D) analysis incorporating pseudo-3D effects;

- The behaviour of rock tunnelling of weak rock masses such as Flysch can be captured accurately using continuum numerical methods (FLAC3D) of numerical analysis as Driskos Tunnel data from the field co-related well with the results obtained through numerical analysis. In general the deformed Flysch can be analyzed as isotropic over the tunnel scale, although heterogeneity and locally intense anisotropy can lead to inadequate support performance. The field data proved valuable in calibrating and verifying numerical models developed by the author;

- Accurate values for GSI are difficult to obtain for extended, altering zones of weak rock masses as there are often zones whereby localized geological peculiarities may govern
tunnel behavior. An accurate assessment of GSI has a direct impact on input parameters for numerical modelling purposes;

- Through an assessment of the numerical modelling data, it was found that the rock mass strength calculated by Hoek (1999) co-related best to the results obtained in this investigation, vice the same strength parameter (index value for uniaxial rockmass strength) as calculated by Hoek and Marinos 2000 as well as Hoek et. al. 2002;

- Field Data from Driskos did not co-relate well with LDP equations as determined by Panet (1995), Unlu and Gercek (2003) and Chern (1998). This prompted a further investigation into plastic zone development in these weak rock masses as per Vlachopoulos and Diederichs (2009);

- The LDP used for design purposes of the top heading was determined not to be the same as the LDP that was followed by the successive bench. It was found that the incremental deformation when the bench passes a certain monitoring point is approximately 50% of the total additional deformation whereas for the initial top heading this value is approximately 25% of the total deformation. This has implications as to the selection of a proper LDP for design purposes;

- Normalized Driskos field measurements align well with the 3D modelling supported LDP normalized with the total displacement achieved by the unsupported behaviour vice the supported LDP normalized with the total supported maximum displacement for a single tunnel bore;

- The influence of the size of the plastic zone plays a key mechanistic role within the expected deformation profile or LDP - a new series of functions has been developed to account for this influence;
The boundary peripheries (i.e. boundary conditions) to be used for these weak rock materials in 2D (based on 3D, validated result comparisons) should be at least 10 radii away for the tunnel excavation zone otherwise, the boundaries negatively impact the results of the numerical analysis;

There is little difference between the results obtained from 2D staged analysis using internal pressure or stress vector reduction and that using face replacement and softening;

The LDP method for calibrating staged analysis can be used for non-circular openings;

An LDP developed based on circular tunnel geometry, isotropic stress, and full face excavation cannot be used accurately to calibrate 2D models with stress anisotropy or lagged excavation steps; and,

For isotropic stresses, a new supported tunnel LDP function has been developed to approximate the influence of support on the tunnel displacement profile. This can be used to correctly place liner support in a 2D model.

9.4 Limitations of Current Research

As with any undertaking of this nature, there were certain limitations that the author had to work within. Below is a list of such limitations:

The author had to work within the design constraints and implemented support of the original Driskos tunnel design (as built);

GSI was the main rock mass classification model that was reasonable based on the inability to obtain samples of Flysch from the field; and,

No reliable stress measurements were obtained or made available to the author.
9.5 Recommendations and Future Work

The recommendations based on the results, observations, experiences and conclusions of this investigation are listed below. There are many aspects of tunnelling in weak rock that warrant further investigation. As such, future work can concentrate (but are not limited to) the following areas of research:

- A more rigorous investigation is required in order to model the effects associated directly at the face by monitoring, surveying and assessing mechanisms at the face;

- Determine the specific mechanisms of how forepoles can be modelled in 2D and 3D numerical analysis. These can also be complimented by physical testing of forepoles individually as well as in a arch arrangement. These mechanisms are poorly understood at the present time and optimization of these arrangements have not been achieved;

- The 3D influence of pillar width or twin tunnel spacing warrants further investigation in order to determine the influence of the excavation of the parallel bore on tunnel convergence. 2D numerical models can then be modified in order to account for these 3D effects;

- An assessment of the behaviour associated with the homogenization of mixed face conditions; and,

- Develop improved calibration functions and methods to allow 2D models to be used more accurately for support sequencing.
References


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Appendix A

FLAC3D Numerical Model
Appendix A

FLAC3D Numerical Model

1.0 Introduction

This appendix outlines the FLAC3D (Fast Lagrangian Analysis of Continua in 3 Dimensions) numerical project code that was developed specifically for this research venture. The code was written in an acceptable FLAC3D format using text files (Microsoft Notepad) for the programming code as well as for specific FISH protocols. The investigation called for the development of a twin tunnel system that was designed to incorporate sequential excavation steps and the introduction of specific support configurations in order to duplicate numerically those NATM excavation and support activities that were undertaken in the field for the Driskos twin tunnels.

The appendix is organized such that all of the programming codes and files are presented sequentially as called by the FLAC3D version 3.0+ program. Additional insertions have been made in order to highlight the specific function of each file. Note that the values and form of the files that are included here serve as a template only and are not specifically the end product of the runs that have been completed as part of this research project. These official runs have been saved in electronic form and have been submitted to the thesis advisor (Dr. Mark Diederichs) for storage and archival purposes.

2.0 FLAC3D Numerical Model Sequence and Organization

In order to organize the FLAC3D project code into manageable sections, a Master file was developed that called other project code files as required in order to complete the modelling sequence. Figure A1 outlines the procedure that was followed by the programming logic in order to conduct the pre-processing and processing of the numerical code.

In simple terms, the Master File first called upon the geometry file in order to create the mesh and build the specific model. Next, the boundary conditions were added
Figure A1 Working FLAC3D Model Sequence and Associated FISH Files
in order to apply the correct constraints and applied loads to the model. A rename file was then called in order to divide the material to be excavated into excavation steps. The appropriate constitutive model could then be called upon (Mohr Coulomb, Strain Softening etc.) in order to apply the correct material properties to the model. A history file was evoked that placed specific monitoring points within the model. These were used in order to analyze the behaviour of the model at certain locations. The sequencing of the model through the processing stage was achieved by the run file. This set-up the conditions of excavation and support in a sequential manner. The run files also called upon other FISH codes in order to implement the correct tunnel support (i.e. forepoles, liner and bolts). Using this approach facilitated the isolation of specific subroutines and allowed for a standardization of parameters to be used within the model. This modularization of the programming also allowed for interchangeable material property files as well as any other sequencing or project code modifications that were required. It also helped with troubleshooting the model and isolating the files that were causing localized or gross errors. Each of the major programming files will be presented in the following sections.

3.0 FLAC3D Programming Files Used

The project codes and function of each of the files and sub files have been included in this section. Again, note that the values and form of the files that are included here serve as a template only and are not specifically the end product of the runs that have been completed as part of this research project.

3.1 Master File

This is a master file (denoted as Master.txt) is the central file that is called upon by FLAC3D in order to begin the numerical model. It can be used to run the various files in a semi-automated fashion. One can comment out the parts that are not required depending on the requirements and what has changed in the model. For example, if the geometry of the model remains the same, there is no requirement to regenerate the model geometry every time the model is run. The advantage of this method is that one does not have to recreate geometries, boundary conditions etc. every time the model is run.
### 3.1.1 Master File Project Code

; This file acts as a master call file for the entire project  
; Each command can be commented out I need to restart from a mid-point in the model  
; This saves rebuilding the geometry for every permutation of the model  
;NOTE HOW COMMENTS HAVE BEEN INSERTED IN PROGRAM FILES BY USING THE “;”

```plaintext
call Driskos_geometry.txt ;build initial geometry
;rest driskos_tunnel.sav

call driskos_rename.fis ;rename the tunnel segments
;save driskos_rename.sav

;rest driskos_rename.sav

call driskos_histories_incl_twin.txt ;call the histories file

call driskos_prop_ss-01.txt ;Choose either Strain Softening or
;rest driskos_ss_plastic.sav ;Hoek-Brown parameters by commenting
;out the unwanted version

;call driskos_prop_hb-01.txt
;rest driskos_hb_plastic.sav

;rest driskos_ss_plastic.sav
;rest driskos_hb_plastic.sav

;call driskos_prop.txt ; This is a Mohr Model

call driskos_run.txt ;this file will restore the appropriate equilibrium file
;and run through the mining sequence including the
;Implementation of the support elements
```

### 3.2 Geometry File

This is the main geometry file used to build the model (driskos_geometry-01.txt). The model consists of specific block zones that create the geometry of two twin tunnels with abutments and with tunnel openings with material to be excavated. Zoning of the model has been conducted in order to tie all of the nodes and grids together and for the reduction of boundary/edge effects.
3.2.1  Geometry File Project Code

; Revised coding to allow for greater flexibility in the geometry
; creation process.

; Each of the variables is labelled and all of the values for geometry
; can be created from this initial function
; subsequent files (such as boundary conditions) refer to some of
; these variables.
; dealt with resolution of Flysch and abutments using ratio function
; FISH code used to define geometry and variable parameters

define _input

xstart=0.0 ; starting position upon which all other
ystart=0.0 ; co-ordinates are derived
zstart=0.0

width1=11.0/2.0 ; tunnel half-width
width2=18.2/2.0 ; tunnel separation half-width
width_abut=40.0 ; width of X-direction abutments

x0=xstart
x1=x0+width1 ; right hand wall of tunnel
x2=x1+width2 ; right abutment of the tunnel command

tun_length=80.0 ; tunnel length MUST ALSO CHANGE
; BENCHYRES
y0=ystart ; start point for the tunnel length
y1=y0+tun_length ; end point for the tunnel length

height1=3.2 ; height of bench
height2=9.5 ; height to top of tunnel excavation
height3=width2 ; height to tunnel block abutment
height_abut=40.0 ; height of model abutment

z0=zstart ; bottom of tunnel
z1=z0+height1 ; bench of the tunnel
z2=z0+height2 ; top of the tunnel
z3=z2+height3 ; top of the block
z4=z0-height3 ; bottom of the block

; benchxres=width1/1.0 ; number of zones in local "x" direction
; benchyres=tun_length/1.0 ; number of zones in y direction
; benchzres=height1/1.0 ; number of zones in local "z" direction
; abutres=width2/1.0 ; number of zones in external box
benchxres=6 ;number of zones in local "x" direction
benchyres=100 ;number of zones in y direction
benchzres=6 ;number of zones in local "z" direction
abutres=9 ;number of zones in external box

tundiares=12 ;# of zones along tun diameter

x4=-x2 ;identify left side of tunnel sections
x3=x4-width_abut ;left hand abutment of the model

z5=z3+height_abut ;top of the external block
z6=z4-height_abut ;bottom of the external block

abutxres=12 ;x-abut uses an average block width
abytyres=50 ;y-abut is tied to the model length
abutzres=6 ;z-abut uses an average block height
abutxres2=benchxres*2 ;needs to be linked to the width of the bench
abutzres2=benchzres ;needs to be linked to the height of the bench
abutxres3=10 ; needs to be linked to abut res3
abutzres3=10

disttotwin=2*(width1+width2) ; distance between similar points to twin tunnel
end

__input

;build the bottom right corner of the tunnel
gen zone radtunnel p0 x0 y0 z1 p1 x0 y0 z4 p2 x0 y1 z1 p3 x2 y0 z1 &
  p4 x0 y1 z4 p5 x2 y1 z1 p6 x2 y0 z4 p7 x2 y1 z4 & ;exterior of box
  p8 x0 y0 z0 p9 x1 y0 z1 p10 x0 y1 z0 p11 x1 y1 z1 &
  p12 x1 y0 z0 p13 x1 y1 z0 & ;interior of box defined
  size benchxres benchyres benchzres abutres &
  ratio 1.0 1.0 1.0 1.0 &
  group flysch fill group bench ;exterior called flysch, interior bench

;build the top right corner of the tunnel
gen zone radcylinder p0 x0 y0 z1 p1 x2 y0 z1 p2 x0 y1 z1 p3 x0 z0 &
  p4 x2 y1 z1 p5 x0 y1 z3 p6 x2 y0 z3 p7 x2 y1 z3 & ;exterior of box
  p8 x1 y0 z1 p9 x0 y2 z0 p10 x1 y1 z0 p11 x0 y1 z2 &
  size benchxres benchyres tundiares abutres &
  ratio 1.0 1.0 1.0 1.0 &
  group flysch fill group aphase ;exterior called flysch, interior aphase
gen zone reflect dip 90 dd 90 ori x0 y0 z0 ;reflect half-tunnel

; ABBUTMENTS

; top external brick attaches to flysch grouping
gen z brick p0 x4 y0 z3 p1 x2 y0 z3 p2 x4 y1 z3 p3 x3 y0 z5 &
p4 x2 y1 z3 p5 x3 y1 z5 p6 x2 y0 z5 p7 x2 y1 z5 &
size abutxres abutyres abutzres3 &
ratio 1 1 1.25 &
group external

; bottom external brick
gen z brick p0 x3 y0 z6 p1 x2 y0 z6 p2 x3 y1 z6 p3 x4 y0 z4 &
p4 x2 y1 z6 p5 x4 y1 z4 p6 x2 y0 z4 p7 x2 y1 z4 &
size abutxres2 abutyres abutzres3 &
ratio 1 1 0.8 &
group external

; lower left external brick
gen z brick p0 x3 y0 z6 p1 x4 y0 z4 p2 x3 y1 z6 p3 x3 y0 z1 &
p4 x4 y1 z4 p5 x3 y1 z1 p6 x4 y0 z1 p7 x4 y1 z1 &
size abutxres3 abutyres abutzres2 &
ratio 0.8 1 1 &
group external

; upper left external brick
gen z brick p0 x3 y0 z1 p1 x4 y0 z1 p2 x3 y1 z1 p3 x3 y0 z5 &
p4 x4 y1 z1 p5 x3 y1 z5 p6 x4 y0 z3 p7 x4 y1 z3 &
size abutxres3 abutyres abutzres &
ratio 0.8 1 1 &
group external

gen zone reflect dip 90 dd 90 ori x2 y0 z0 ;reflect second tunnel

plot block group

gen merge 0.1
attach face ; ensures that there is continuity along each of the boundaries and faces of the zones

geom_test
save driskos_tunnel.sav

The next section depicts how the geometry was defined by the project code above (Figure A2).
3.2.2 Geometric Parameters used within Geometry File

- Reference Point $x_0, y_0, z_0$
- First Reflection Plane
- Second Reflection Plane
- Zone: Block
  - Top external brick
  - Upper left external brick
  - Lower left external brick
- Zone: Block
  - Bottom external brick
- Zone: radcylinder
- Zone: radtunnel
- Height 1
- Height 2
- Height 3
- Width 1
- Width 2
- Abutments
- Flysch

Figure A2 Geometric parameters associated with model geometry
3.2.3 Geometries Created in FLAC3D using Geometry File

The following figures are screen captures from FLAC3D that highlight the creation of the twin tunnel geometry. For this particular model, 165,600 zones were created and 168,929 grid points. The bottom figure depicts a cross-section within the model within the first or left hand side tunnel bore. The top heading and bench material has been separated in this way in order to allow for sequential excavation of the tunnel.

Figure A3 Geometry created within FLAC3D using driskos_geometry-01.txt file.
3.3 Rename File

This FISH program file (driskos_rename.fis) is a sub-routine that takes the material within the top heading and the bench and sub-divides these sections into smaller segments. The thickness of these segments is associated with the mining or excavations steps that are required. It is these smaller segments that are deleted sequentially in order to mimic the excavation process. The naming convention can be changed by modifying the variable “sname” and the mining step thickness can be altered by modifying the “segment_thick” variable.

3.3.1 Rename File Project Code

; Modified to allow the renaming to be linked to the tunnel length
; through the use of the num_segs variable
; if the names of the tunnels in the initial geometry is changed
; this file will have to be updated for the "sname" variable

define _rename

segment_thick=4.0 ;how long is each advance
num_segs=tun_length/segment_thick ;how many advances needed for the model

loop i(1,num_segs)
y_name=0.0+segment_thick*(i-1)
y_name2=y_name+segment_thick
sname='aphase'+string(i)
command
   group sname range x x4 x2 y y_name y_name2 group aphase
endcommand
endloop

loop i(1,num_segs)
y_name=0.0+segment_thick*(i-1)
y_name2=y_name+segment_thick
sname='bench'+string(i)
command
   group sname range x x4 x2 y y_name y_name2 group bench
endcommand
endloop
3.3.2 Renamed Sections Created in FLAC3D

The end product of the rename command can be seen in Figure A4. The material that is to be excavated is subdivided into smaller sub-sections (denoted by various colour schemes in figure – note that the colour schemes cannot be altered by the user within the FLAC3D software). An internal subroutine within FLAC3D designates the colours that are associated with each excavation unit.
3.4 History File

A history FISH file (driskos_histories_incl_twin.fis) was used in order to monitor certain locations or place holders within the model. The histories monitor displacements, tensors; stresses, moments and other block properties. Most modelling locations were at the center of the model, furthest away from the boundary effects. More monitoring sections were used for the histories associated with the support elements. These were entered within the run file after the implementation of the various support elements. The monitoring positions are standardized to the tunnel geometry and are used to monitor the primary tunnel as well as the twin tunnel through the excavation sequence.

Figure A4 Renaming and subdividing material to be excavated into sub sections.
3.4.1 History File Project Code

plot show 'Base'

; Fish function to monitor the histories of various parameters of tunnel model
; Specific functions are provided to monitor the 5 survey locations at the site
; general values are provided to monitor displacement as a function of depth
; and the area between the tunnels.
; the monitoring positions are standardized to the tunnel geometry

define _surveyed_locations
    ; input x y and z for each of the surveyed locations relative
    ; to the local co-ordinate system
    ; must ensure that the history point lands in the final wall or
    ; else the data will stop when the zone is excavated.

; Monitoring in the centre (y-direction) of the model - 1st Excavation Tunnel

surveyx1=x0-width1  ; Survey Point 1 - Bench Top Left
surveyy1=tun_length/2
surveyz1=z0+height1

surveyx2=x0        ; Survey Point 2 - Crown Top
surveyy2=tun_length/2
surveyz2=z0+height2

surveyx3=x0+width1 ; Survey Point 3 - Bench Top Right
surveyy3=tun_length/2
surveyz3=z0+height1

surveyx4=x0-width1 ; Survey Point 4 - Bottom Left
surveyy4=tun_length/2
surveyz4=z0

surveyx5=x0+width1 ; Survey Point 5 - Bottom Right
surveyy5=tun_length/2
surveyz5=z0

surveyx6=x0        ; Survey Point 6 - Centre - Bench level
surveyy6=tun_length/2
surveyz6=z0+height1
; TWIN TUNNEL

; Monitoring in the centre (y-direction) of the model - Twin Excavation Tunnel

surveyx7=x0-width1+disttotwin ; Survey Point 1 Position of Twin Tunnel - Bench
   Top Left
surveyy7=tun_length/2
surveyz7=z0+height1

surveyx8=x0+disttotwin      ; Survey Point 2 Position of Twin Tunnel - Crown
   Top
surveyy8=tun_length/2
surveyz8=z0+height2

surveyx9=x0+width1+disttotwin ; Survey Point 3 Position of Twin Tunnel - Bench
   Top Right
surveyy9=tun_length/2
surveyz9=z0+height1

surveyx10=x0-width1+disttotwin ; Survey Point 4 Position of Twin Tunnel - Bottom
   Left
surveyy10=tun_length/2
surveyz10=z0

surveyx11=x0+width1+disttotwin ; Survey Point 5 Position of Twin Tunnel - Bottom
   Right
surveyy11=tun_length/2
surveyz11=z0

surveyx12=x0+disttotwin      ; Survey Point 6 Position of Twin Tunnel - Centre - Bench
   level
surveyy12=tun_length/2
surveyz12=z0+height1

end
_surveyed_locations

; Initial Tunnel

his gp xdisp surveyx1 surveyy1 surveyz1 ; history #1
his gp ydisp surveyx1 surveyy1 surveyz1; history #2
his gp zdisp surveyx1 surveyy1 surveyz1; history #3
his gp xdisp surveyx2 surveyy2 surveyz2; history #4
his gp ydisp surveyx2 surveyy2 surveyz2; history #5
his gp zdisp surveyx2 surveyy2 surveyz2; history #6

his gp xdisp surveyx3 surveyy3 surveyz3; history #7
his gp ydisp surveyx3 surveyy3 surveyz3; history #8
his gp zdisp surveyx3 surveyy3 surveyz3; history #9

his gp xdisp surveyx4 surveyy4 surveyz4; history #10
his gp ydisp surveyx4 surveyy4 surveyz4; history #11
his gp zdisp surveyx4 surveyy4 surveyz4; history #12

his gp xdisp surveyx5 surveyy5 surveyz5; history #13
his gp ydisp surveyx5 surveyy5 surveyz5; history #14
his gp zdisp surveyx5 surveyy5 surveyz5; history #15

his gp xdisp surveyx6 surveyy6 surveyz6; history #16
his gp ydisp surveyx6 surveyy6 surveyz6; history #17
his gp zdisp surveyx6 surveyy6 surveyz6; history #18

;Twin Tunnel

his gp xdisp surveyx7 surveyy7 surveyz7; history #19
his gp ydisp surveyx7 surveyy7 surveyz7; history #20
his gp zdisp surveyx7 surveyy7 surveyz7; history #21

his gp xdisp surveyx8 surveyy8 surveyz8; history #22
his gp ydisp surveyx8 surveyy8 surveyz8; history #23
his gp zdisp surveyx8 surveyy8 surveyz8; history #24

his gp xdisp surveyx9 surveyy9 surveyz9; history #25
his gp ydisp surveyx9 surveyy9 surveyz9; history #26
his gp zdisp surveyx9 surveyy9 surveyz9; history #27

his gp xdisp surveyx10 surveyy10 surveyz10; history #28
his gp ydisp surveyx10 surveyy10 surveyz10; history #29
his gp zdisp surveyx10 surveyy10 surveyz10; history #30

his gp xdisp surveyx11 surveyy11 surveyz11; history #31
his gp ydisp surveyx11 surveyy11 surveyz11; history #32
his gp zdisp surveyx11 surveyy11 surveyz11; history #33
his gp xdisp surveyx12 surveyy12 surveyz12; history #34
his gp ydisp surveyx12 surveyy12 surveyz12; history #35
his gp zdisp surveyx12 surveyy12 surveyz12; history #36

define _general_histories

yhis1=5 ;y-location of first general monitoring station
yhis2=10 ;y-location of second general monitoring station
yhis3=15 ;y-location of third general monitoring station
yhis4=20 ;y-location of fourth general monitoring station
yhis5=25 ;y-location of fifth general monitoring station

xhis0=0
xhis1=x1
xhis2=x1+1
xhis3=x1+2
xhis4=x1+4
xhis5=x2

zhis0=z1
zhis1=z2
zhis2=z2+1
zhis3=z2+2
zhis4=z2+4
zhis5=z2+8

end
_general_histories

; these histories result in a grid of spaced displacement tracking points
; to observer the locations of these points, do a plot of "history location"
; locations can be altered through the values above
; copying and pasting a set can allow for other things to be monitored such as
; stresses without re-typing the whole thing. Just grab a set, copy, paste, and replace
; the "gp xdisp" with "zone smin" for example.
; more variables can be defined if more locations are needed as long as the grammar is
; kept consistent to avoid interference with variables from other FISH functions.

; monitor X-displacements along the wall of the first tunnel - within the PILLAR between
tunnels
his gp xdisp xhis1 yhis1 zhis0; history #37
his gp xdisp xhis2 yhis1 zhis0; history #38
his gp xdisp xhis3 yhis1 zhis0; history #39
his gp xdisp xhis4 yhis1 zhis0; history #40
his gp xdisp xhis5 yhis1 zhis0; history #41
his gp xdisp xhis1 yhis2 zhis0; history #42
his gp xdisp xhis2 yhis2 zhis0; history #43
his gp xdisp xhis3 yhis2 zhis0; history #44
his gp xdisp xhis4 yhis2 zhis0; history #45
his gp xdisp xhis5 yhis2 zhis0; history #46

his gp xdisp xhis1 yhis3 zhis0; history #47
his gp xdisp xhis2 yhis3 zhis0; history #48
his gp xdisp xhis3 yhis3 zhis0; history #49
his gp xdisp xhis4 yhis3 zhis0; history #50
his gp xdisp xhis5 yhis3 zhis0; history #51

his gp xdisp xhis1 yhis4 zhis0; history #52
his gp xdisp xhis2 yhis4 zhis0; history #53
his gp xdisp xhis3 yhis4 zhis0; history #54
his gp xdisp xhis4 yhis4 zhis0; history #55
his gp xdisp xhis5 yhis4 zhis0; history #56

his gp xdisp xhis1 yhis5 zhis0; history #57
his gp xdisp xhis2 yhis5 zhis0; history #58
his gp xdisp xhis3 yhis5 zhis0; history #59
his gp xdisp xhis4 yhis5 zhis0; history #60
his gp xdisp xhis5 yhis5 zhis0; history #61

;monitor vertical displacements along the back of the first tunnel
his gp zdisp xhis0 yhis1 zhis1; history #62
his gp zdisp xhis0 yhis1 zhis2; history #63
his gp zdisp xhis0 yhis1 zhis3; history #64
his gp zdisp xhis0 yhis1 zhis4; history #65
his gp zdisp xhis0 yhis1 zhis5; history #66

his gp zdisp xhis0 yhis2 zhis1; history #67
his gp zdisp xhis0 yhis2 zhis2; history #68
his gp zdisp xhis0 yhis2 zhis3; history #69
his gp zdisp xhis0 yhis2 zhis4; history #70
his gp zdisp xhis0 yhis2 zhis5; history #71

his gp zdisp xhis0 yhis3 zhis1; history #72
his gp zdisp xhis0 yhis3 zhis2; history #73
his gp zdisp xhis0 yhis3 zhis3; history #74
his gp zdisp xhis0 yhis3 zhis4; history #75
his gp zdisp xhis0 yhis3 zhis5; history #76

his gp zdisp xhis0 yhis4 zhis1; history #77
his gp zdisp xhis0 yhis4 zhis2; history #78
3.4.2 History File Monitoring Point Locations

The location of the monitoring points coincides with the geomatic monitoring of the Driskos tunnel (modelling validation data) at the locations that are noted in Figure A5 and Table A1. Table A1 summarizes the location of the Driskos survey points and the relative x, y and z, co-ordinates within the FLAC3D Driskos geometry file.

![Figure A5 Locations of monitoring points at mid-length (y-direction) for each bore.](image)

<table>
<thead>
<tr>
<th>Survey Point</th>
<th>x</th>
<th>y</th>
<th>z</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>x0-width1</td>
<td>y0 or y0+segment_thick</td>
<td>z0+height1</td>
</tr>
<tr>
<td>2</td>
<td>x0</td>
<td>y0 or y0+segment_thick</td>
<td>z0+height2</td>
</tr>
<tr>
<td>3</td>
<td>x0+width1</td>
<td>y0 or y0+segment_thick</td>
<td>z0+height1</td>
</tr>
<tr>
<td>4</td>
<td>x0-width1</td>
<td>y0 or y0+segment_thick</td>
<td>z0</td>
</tr>
<tr>
<td>5</td>
<td>x0+width1</td>
<td>y0 or y0+segment_thick</td>
<td>z0</td>
</tr>
</tbody>
</table>

Table A1. Locations of Survey points and co-responding co-ordinates within numerical model.
3.5 Material Properties Files

The material properties files (driskos_prop_XX-01.txt) are used in order to assign the input parameters for the material properties of the (Flysch) material and abutments. The file also designates the type of constituent model to be used (Mohr, Strain Softening etc.). This file has all of the properties laid out for the constitutive model. One can input as many materials as required using different properties. One can use these properties in the same model, or use each one as a place-holder for a specific property set by commenting out property designations that are not required. Inputs are in accordance to the FLAC3D code and are straightforward in nature. Residual strengths are dealt with by using tables based upon properties that one requires to generate (i.e. peak to residual strength designations). The variable “_mat1strain” is used to change the strain interval; 1% has been used in this application and can be modify as required. All of the post-peak property tables are linked to the initial inputs and are generate automatically. For the strain softening model shown in the next section, the model will run 1500 cycles under elastic conditions to even out any instability followed by another 1500 cycles under plastic conditions to ensure that there are no further instabilities.

3.5.1 Material Property File Project Code (Strain Softening Example)

`; Add material properties to model by allocating properties to all of the created geometry
`; For each material, the inputs have been separated from the calculated values
`; except for tensile strength which is often related to cohesion
`; One can either keep a ratio, or put in an explicit number
`; properties for Material 1 are based upon the Driskos data
`; conversion generated by Rocklab version 1.021 with the tunnel application
`; and a depth of 50m with a 0.025MN/m3 unit weight.
`; Average values are used from TABLE 1 when a range was supplied.

define _rockprop
;===================================================================
;
; Properties for Flysch
;===================================================================
;inputs

Emat1=1.645e9 ;Young's Modulus of Flysch
_mat1poiss=0.25 ;Poisson's ratio for Flysch
_mat1coh=0.159e6 ;material cohesive strength
;friction angle
_mat1fric=38.1 ;density based on gravity = 10
_mat1dens=2500 ;strain interval from peak to residual
_mat1strain=0.01 ;1% from peak to residual)
_mat1ten= mat1coh/2.0 ;material tensile strength
;(assume 1/2 cohesion to start)
_mat1_coh_ratio=0.8 ;% of intact strength for residual strength –
;(near mohr-coulomb to start)
_mat1_ten_ratio=0.0 ;% of intact strength for residual strength –
;(tensile strength to zero)
_mat1_fric_ratio=1.0 ;% of intact strength for residual strength
_mat1dil=0.0 ;intact dilation angle
_mat1dilr=0.0 ;residual dilation angle
_mat1sh=Emat1/(2*(1+_mat1poiss)) ;calculated shear modulus
_mat1bu=_mat1sh*((2*(1+_mat1poiss))/(3*(1-2*_mat1poiss))) ;calculated bulk modulus
_mat1cohr=_mat1coh*_mat1_coh_ratio ;calculated residual cohesion
_mat1tenr=_mat1ten*_mat1_ten_ratio ;calculated residual tensile strength
_mat1fricr=_mat1fric*_mat1_fric_ratio ;calculated residual friction angle
end

_end

; give elastic properties to the materials
;each of these commands requires a range function in order to function

model elastic
 prop bu _mat1bu sh _mat1sh dens _mat1dens
;set large ;large strain mode (i.e. gridpoint calculations are updated as they move)
set grav 0 0 -10 ;set gravity = 10 [if you change this, densities and stresses need to be changed to match]
set mech damp loc ;default damping mode is used for static analysis
his unbal ;track unbalanced forces
call driskos_boundary.txt

solve ratio 1e-5 step 1500 ;arbitrary number of steps to reach initial equilibrium

save driskos-ss-elastic.sav
provide the strain softening parameters to the various groups of materials
;only need to give the 'model ss' command once unless it is desired to keep
;the abutments elastic or use a different constitutive model
;need to ; out any unneeded groupings. Tables can be left uncommented out as
;they are ignored if they are not used.
;it is assumed that the post-peak behaviour can be simulated by 3-points
;if need a more detailed post-peak behaviour, simply add in intermediate
;strain,strength pairs into the tables.
;as with the elastic properties, if I need to use more than one material I need to
;uncomment out the one I need and then supply a range to each material
;command.

model ss

prop bu _mat1bu sh _mat1sh coh _mat1coh tens _mat1ten dil _mat1dil fric _mat1fric &
c tab 1 tt ab 2 ft ab 3 dt ab 4

table 1 0.0,_mat1coh _mat1strain _mat1cohr 10._mat1cohr
table 2 0.0,_mat1ten _mat1strain _mat1tenr 10._mat1tenr
table 3 0.0,_mat1fric _mat1strain _mat1fricr 10._mat1fricr
table 4 0.0,_mat1dil _mat1strain _mat1dil 10._mat1dil

solve ratio 1e-5 step 1500 ;arbitrary number of steps to reach a plastic equilibrium
;keeping an eye on the unbalanced force plot will tell me
;if enough cycles were performed

ini xdis 0 ;reset all displacements since they are numerical instabilities
ini ydis 0 ;and do not represent the steady state
ini zdis 0

save driskos-ss-plastic.sav

3.6 Boundary Conditions

The boundary conditions file (driskos_boundary.txt) is required in order to implement the boundary conditions into the model. It is called by the material properties file into the numerical modelling sequence and uses inputs from the initial geometry file prior to the initial equilibrium cycling that is performed. The main variables of concern are the depth at which the tunnel are located (in meters) and the s1 ratio and s2 ratio which are the ratios of the horizontal stresses to the vertical stresses respectively.
The model defines the boundaries (fixed and free) and the stress field that the model is exposed to.

### 3.6.1 Boundary Conditions File Project Code

; File needed to implement the boundary conditions
; into the model. Linked to the initial geometry file
; changes can be made to the stress ratios and directions
; but boundary locations are defined in the initial geometry file

define _bounds ;define boundaries and stress field
;the variables created for the boundary result in a 0.1m
tolerance on all sides to ensure proper capture of boundaries
xll=x3-0.1 ;left side of left boundary
xll=x3+0.1 ;right side of left boundary
xhl=x2+(width1*2.0)+(width2*2.0)+width_abut-0.1 ;left side of left boundary
xhh=x2+(width1*2.0)+(width2*2.0)+width_abut+0.1 ;right side of left boundary
yll=y0-0.1 ;near side of near boundary
ylh=y0+0.1 ;far side of near boundary
yhl=y1-0.1 ;near side of far boundary
yhh=y1+0.1 ;far side of far boundary
ymidl=tun_length/2-1.7 ; midpoint location
ymidh=tun_length/2+1.7 ; midpoint location
zll=z6-0.1 ;bottom of bottom
zlh=z6+0.1 ;top of bottom
zhl=z5-0.1 ;bottom of top
zhh=z5+0.1 ;top of top

**depth=-105** ;surface is a positive number with surface = 0
;the conversion to the compression negative form is handled by variable sv

sv=0.025e6 ;overburden stress in MPa/meter (assumed 2500kg/m^3*10m/s^2) can be updated to
;use 9.81 for gravity if desired

s1ratio=1.25 ;relationship between x-stress and vertical
s2ratio=1.5 ;relationship between y stress and vertical
s1grad=s1ratio*sv ;stress gradient for S1
s2grad=s2ratio*sv ;stress gradient for s2
\[ s1 = sv \times s1\text{ratio} \times \text{depth} \] ; max in-situ stress at reference depth (assume centerline of tunnel=depth)
\[ s2 = sv \times s2\text{ratio} \times \text{depth} \] ; int in-situ stress at reference depth
\[ s3 = sv \times \text{depth} \] ; vertical in-situ stress at reference depth
\[ s3\text{top} = s3 + z5 \times sv \] ; vertical in-situ stress at top boundary of the model
; calculated by subtracting the ground above reference to the ; top of model

end

bounds

; initialize stresses

; Fixed - rollers on both sides in y direction
ini sxx s1 grad 0 0 s1\text{grad} ; initialize the stresses in x direction (assumed to be s1)
ini syy s2 grad 0 0 s2\text{grad} ; initialize the stresses in y direction (assumed to be s2)
ini szz s3 grad 0 0 sv ; initialize the vertical stresses
ini xvel 0 ; reset all velocities
ini yvel 0
ini zvel 0

; apply boundary conditions
fix x range x xll xlh ; fix the opposite boundary to the applied stresses
fix y range y yll ylh ; fix the opposite boundary to the applied stresses
fix y range y yhl yhh ; fix the opposite boundary to the applied stresses
fix z range z zll zlh ; fix the opposite boundary to the applied stresses
apply sxx s1 grad 0 0 s1\text{grad} ra x xhl xhh ; apply stresses to opposite side of fixed boundary
apply szz s3\text{top} ra z zhl zhh ; apply stresses to opposite side of fixed boundary

3.6.2 Boundary Conditions and Constraints used within FLAC3D Code

The following Figure A6 depicts the boundary conditions that have been implemented by the project code described above. One must ensure that these constraints are accurate and realistic in nature. Care should also be taken to avoid any boundary effects from the peripheries of the numerical model. Much troubleshooting occurred with this file in order to ensure that these conditions were accurate. The model is fixed in place on 4 sides (both y faces and an x and z face) while being subjected to an overburden stress and gradient stresses on an x and z face.
Overburden pressure $s_{3\text{top}} = s_3 + z_5 x \sigma_v$ where $s_3 = \sigma_v \times \text{depth}$

**Figure A6** Boundary Conditions for Numerical Model

Cross-Section to be used in 2D numerical simulations
3.7 Run File

This run file (driskos_run.txt) is the heart of the command files for the excavation and support sequencing for the numerical model. It is used to conduct all of the required sequencing once the model has attained equilibrium. When support is required, a FISH function for the implementation of that support is called (i.e. forepoles, liner, and rockbolt sequences). Subsequent tunnel advances only requires one to invoke the correct _install command to install the required support. Once the tunnel reaches the end of excavation, the support automatically transitions to the second tunnel. In this way, the excavation sequence for the tunnel can be altered to a variety of mining options (i.e. excavation from all portals simultaneously). After running a selected number of steps (5 in this case), the file will generate a save file. A 2000 cycle placeholder has been used in order to define the number of cycles performed based upon the model reaching equilibrium at each excavation and support step.

3.7.1 Run File Project Code

; This file is used to run the mining sequence.
; INCLUDES MATERIAL PROPERTIES, EXCAVATION, HISTORIES AS WELL AS SUPPORT
; 80 m long tunnel
; NO UPPER BOLT SEQUENCE
; NEED TO INTRODUCE LINER, FOREPOLE AND BOLT MONITORING

;set movie avi step 100 file movie_file.avi
;movie start

plot show 'Base'

;Install the first set of forepoles prior to mining
call driskos_forepoles-02.fis
step 2000 ;do some more cycles to ensure stability
save step0.sav

del ra group aphase1 ;extract first part of tunnel
call driskos_liner-01.fis ;install shotcrete in first tunnel segment
;call driskos_bolting-01.fis ;NO UPPER WALL BOLTING - install pattern bolting in first segment
_install_fp ;install forepoles for next segment
step 2000
save step1.sav

del ra group aphase2
 _install_liner ;install shotcrete
 _install_bolts ;install pattern support
 _install_fp ;install forepoles
step 2000
;save step2.sav

del ra group aphase3
 _install_liner ;install shotcrete
 _install_bolts ;install pattern support
 _install_fp ;install forepoles
step 2000
;save step3.sav

del ra group aphase4
 _install_liner ;install shotcrete
 _install_bolts ;install pattern support
 _install_fp ;install forepoles
step 2000
;save step4.sav

del ra group aphase5
 _install_liner ;install shotcrete
 _install_bolts ;install pattern support
 _install_fp ;install forepoles
step 2000
save step5.sav

del ra group aphase6
 _install_liner ;install shotcrete
 _install_bolts ;install pattern support
 _install_fp ;install forepoles
step 2000
;save step6.sav

del ra group aphase7
 _install_liner ;install shotcrete
 _install_bolts ;install pattern support
 _install_fp ;install forepoles
step 2000
;save step7.sav
del ra group aphase8
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
step 2000
;save step8.sav

del ra group aphase9
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
step 2000
;save step9.sav

del ra group aphase10
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
step 2000
save step10.sav

del ra group aphase11
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
step 2000
;save step11.sav

del ra group aphase12
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
step 2000
;save step12.sav

del ra group aphase13
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
step 2000
;save step13.sav

del ra group aphase14
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
step 2000
;save step14.sav

del ra group aphase15
   _install_liner ;install shotcrete
   _install_bolts ;install pattern support
   _install_fp ;install forepoles
step 2000
save step15.sav

del ra group aphase16
   _install_liner ;install shotcrete
   _install_bolts ;install pattern support
   _install_fp ;install forepoles
step 2000
;save step16.sav

del ra group aphase17
   _install_liner ;install shotcrete
   _install_bolts ;install pattern support
   _install_fp ;install forepoles
step 2000
;save step17.sav

del ra group aphase18
   _install_liner ;install shotcrete
   _install_bolts ;install pattern support
;Do not install forepoles so that it does not exceed the boundaries for the tunnel
step 2000
;save step18.sav

del ra group aphase19
   _install_liner ;install shotcrete
   _install_bolts ;install pattern support
step 2000
;save step19.sav

del ra group aphase20
   _install_liner ;install shotcrete
   _install_bolts ;install pattern support
step 2000
save step20.sav
; Bench Excavation from 1st Tunnel

`del ra group bench1
call driskos_liner_walls-01.fis
call driskos_bolting_walls-01.fis ; installs bolts on sidewalls
step 2000
;save step21.sav`

`del ra group bench2
_install_liner_walls
_install_bolts_walls
step 2000
;save step22.sav`

`del ra group bench3
_install_liner_walls
_install_bolts_walls
step 2000
;save step23.sav`

`del ra group bench4
_install_liner_walls
_install_bolts_walls
step 2000
;save step24.sav`

`del ra group bench5
_install_liner_walls
_install_bolts_walls
step 2000
save step25.sav`

`del ra group bench6
_install_liner_walls
_install_bolts_walls
step 2000
;save step26.sav`

`del ra group bench7
_install_liner_walls
_install_bolts_walls
step 2000
;save step27.sav`
del ra group bench8
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step28.sav

del ra group bench9
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step29.sav

del ra group bench10
  _install_liner_walls
  _install_bolts_walls
step 2000
save step30.sav

del ra group bench11
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step31.sav

del ra group bench12
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step32.sav

del ra group bench13
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step33.sav

del ra group bench14
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step34.sav

del ra group bench15
  _install_liner_walls
  _install_bolts_walls
step 2000
save step35.sav

del ra group bench16
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step36.sav

del ra group bench17
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step37.sav

del ra group bench18
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step38.sav

del ra group bench19
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step39.sav

del ra group bench20
  _install_liner_walls
  _install_bolts_walls
step 2000
save step40.sav

;movie finish

;******************************************************************************
; Twin Tunnel Excavation
;******************************************************************************
  _install_fp ;install forepoles for this segment
step 1000

del ra group twin_aphase1 ;extract first part of tunnel
  _install_liner
  ;_install_bolts ;install pattern bolting in first segment
install_fp ;install forepoles for next segment
step 2000
;save step41.sav
del ra group twin_aphase2
  _install_liner ;install shotcrete
 ;_install_bolts ;install pattern support
 _install_fp ;install forepoles
 step 2000
 ;save step42.sav

del ra group twin_aphase3
  _install_liner ;install shotcrete
 ;_install_bolts ;install pattern support
 _install_fp ;install forepoles
 step 2000
 ;save step43.sav

del ra group twin_aphase4
  _install_liner ;install shotcrete
 ;_install_bolts ;install pattern support
 _install_fp ;install forepoles
 step 2000
 ;save step44.sav

del ra group twin_aphase5
  _install_liner ;install shotcrete
 ;_install_bolts ;install pattern support
 _install_fp ;install forepoles
 step 2000
 save step45.sav

del ra group twin_aphase6
  _install_liner ;install shotcrete
 ;_install_bolts ;install pattern support
 _install_fp ;install forepoles
 step 2000
 ;save step46.sav

del ra group twin_aphase7
  _install_liner ;install shotcrete
 ;_install_bolts ;install pattern support
 _install_fp ;install forepoles
 step 2000
 ;save step47.sav

del ra group twin_aphase8
  _install_liner ;install shotcrete
 ;_install_bolts ;install pattern support
;install forepoles
step 2000
;save step48.sav

del ra group twin_aphase9
;install shotcrete
;install pattern support
;install forepoles
step 2000
;save step49.sav

del ra group twin_aphase10
;install shotcrete
;install pattern support
;install forepoles
step 2000
;save step50.sav

del ra group twin_aphase11
;install shotcrete
;install pattern support
;install forepoles
step 2000
;save step51.sav

del ra group twin_aphase12
;install shotcrete
;install pattern support
;install forepoles
step 2000
;save step52.sav

del ra group twin_aphase13
;install shotcrete
;install pattern support
;install forepoles
step 2000
;save step53.sav

del ra group twin_aphase14
;install shotcrete
;install pattern support
;install forepoles
step 2000
;save step54.sav
del ra group twin_aphase15
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
solve ratio 1e-5 step 1500
save step55.sav

del ra group twin_aphase16
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
step 2000
;save step56.sav

del ra group twin_aphase17
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  _install_fp ;install forepoles
step 2000
;save step57.sav

del ra group twin_aphase18
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
  ;_install_fp ;Do not install forepoles so that it does not exceed the boundaries fo the tunnel
step 2000
;save step58.sav

del ra group twin_aphase19
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
step 2000
;save step59.sav

del ra group twin_aphase20
  _install_liner ;install shotcrete
  ;_install_bolts ;install pattern support
step 2000
save step60.sav

;***********************************************************************
; Bench Excavation from Twin Tunnel
;***********************************************************************
del ra group twin_bench1
  _install_liner_walls

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_install_bolts_walls
step 2000
;save step61.sav

del ra group twin_bench2
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step62.sav

del ra group twin_bench3
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step63.sav

del ra group twin_bench4
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step64.sav

del ra group twin_bench5
  _install_liner_walls
  _install_bolts_walls
step 2000
save step65.sav

del ra group twin_bench6
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step66.sav

del ra group twin_bench7
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step67.sav

del ra group twin_bench8
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step68.sav
del ra group twin_bench9
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step69.sav

del ra group twin_bench10
  _install_liner_walls
  _install_bolts_walls
step 2000
save step70.sav

del ra group twin_bench11
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step71.sav

del ra group twin_bench12
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step72.sav

del ra group twin_bench13
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step73.sav

del ra group twin_bench14
  _install_liner_walls
  _install_bolts_walls
step 2000
;save step74.sav

del ra group twin_bench15
  _install_liner_walls
  _install_bolts_walls
step 2000
save step75.sav

del ra group twin_bench16
  _install_liner_walls
  _install_bolts_walls
step 2000
3.7.2 Run File Sequencing in FLAC 3D

The run file sequence implements supports (with their own properties and geometries; to be discussed the following sections) and deletes zones or blocks of material to be excavated at a rate determined by the rename FISH function. Equilibrium is then achieved through steps of the finite difference FLAC3D program code prior to the next excavation or support step. Figure A7 notes various screen captures during a numerical model run.
3.8 Forepole Support Installation

This file (driskos_forepole-0#.txt) uses the predefined geometry of the twin tunnel sequence and installs forepoles in an umbrella arch configuration prior to each excavation step. The sequence that has been followed is that of the Driskos tunnel construction sequence. As seen in the file code, this series of commands is a completely automated support installation routine; the angle of installation, the length of the bolts, and their positions are all automated. The primary variables in this file are: a) num_fp which determines how many forepoles will be installed in the umbrella arch (i.e. around the circumference of the upper tunnel wall based on the desired spacing, b) fp_length which is the length of each forepole, c) fp_angle that defines the angle (in the y-direction) that the forepoles are installed in and d) _fpadvance that designates the ring-to-ring offset.
based upon the pre-selected (4m) excavation steps. Once this function is invoked the first time, one only needs to invoke the _install_fp command and the forepole installation will automatically advance to the next umbrella sequence (in the y-direction) with offsetting rings. This file will also transition to the other tunnel at the appropriate time as the length of tunnel is also referenced within the file code. If you need to have both tunnels going at once, you can control the offset between the tunnels by adjusting the “if fpcount > num_segs + xx” variable. Making the “xx” less than 0 will result in some overlapping of the installation.

3.8.1 Forepole Support File Project Code

; This function is used to replicate the forepoling support
; system used for the tunnel model. Parameters for in and out
; of plane spacing are required as well as the angle of insertion.
; The number of forepoles required is defined at the beginning of the
; file. That value is then taken to determine the spacing.
; Perimeter and x-sectional area are defined for a bolt with an outer
; diameter of 114mm and an inner diameter of 101mm (for this run)

; Definition of variables necessary to define how the forepoles are installed.

define _fp_parameters ;this function is used to allocate the proper number of
;forepoles and thier positions. Info is used in
;_install_forepoles function

fpcount=1 ;starting ID for the first fore pole. This is incremented for each one
;inserted
num_fp=60 ;Total number of forepoles required around the circumference of
;the tunnel this assumes that they are only required on the arched portion.

fpz0=z1
fp_length=12.0 ;length of forepole in meters

fpx0=x0
fpy0=y0
fpy00=fpyo
fpy00=fpyo
fz0=z1

fp_angle=5.78*0.01745 ;convert the 5.78 degree angle to a radian measure
;this angle needs to be in radians

radius=height2-height1+0.1
;radius of the arch in the tunnel from the geometry section
radius2=(height2-height1+0.1)+sin(fp_angle)*fp_length
;the radius of the toe of the forepoles

_fpadvance=segment_thick ;increments the next set of forepole installations
;based upon the round advance from the _renaming
;function

end
_fp_parameters ;execute the function to load the variables into memory

;===============================================================
; Function used to actually install the forepoles
;===============================================================

define _install_fp

if fpcount < (num_segs-2+0.1) ;assume that once the first tunnel is fully excavated
;that the fore-poling
;jumps over to the second tunnel
;the 3 takes into account the three last excavation
;steps that do not require forepole installation

loop i(1,num_fp) ;loop through the # of bolts

_rot=pi/(num_fp-1) ;define the rotational increment around the tunnel
;circumference
fpx1=fpx0+0.01+radius*sin(_rot*(i-1)-pi/2) ;calculate the x pos of the collar
fpy1=fpy0 ;calculate the y pos of the collar
fpz1=fpz0+radius*cos(_rot*(i-1)-pi/2) ;calculate the z pos of the collar
fpx2=fpx0+0.001+radius2*sin(_rot*(i-1)-pi/2) ;calculate the x pos of
;the toe
fpz2=fpz0+radius2*cos(_rot*(i-1)-pi/2) ;calculate the y pos of
;the toe
fpy2=fpy1+fp_length*cos(fp_angle) ;calculate the z pos of the toe
;command

sel pile id fpcount begin fpx1 fpy1 fpz1 end fpx2 fpy2 fpz2 nseg 10 ;make the forepole

(endcommand
endloop
command

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sel pile id fpcount prop emod 200e9 xarea 0.002195 per 0.179 slide on &
cs_scoh 25e6 cs_nk 100e9 cs_sfric 35 dens 7750
   ;give it properties
endcommand

count=count+1
   ;increment the ID counter
fpy0=fpy1+_fpadvance
   ;set the start point for the next set of forepoles
endif

if fpcount > (num_segs-2+0.1) ;once the value of k is greater than the number of
   ;segments in the first tunnel it will reset the
   ;positions to go to the second tunnel

   loop i(1,num_fp) ;loop through the # of bolts
      _rot=pi/(num_fp-1) ;define the rotational increment around the
         ;tunnel circumference
      fpx1=x2*2.0+0.01+radius*sin(_rot*(i-1)-pi/2) ;calculate the x pos of the
         ;collar
      fpy1=fpy00
      ;calculate the y pos of the collar
      fpz1=fpz0+radius*cos(_rot*(i-1)-pi/2)  ;calculate the z pos of the collar
      fpx2=x2*2.0+0.001+radius2*sin(_rot*(i-1)-pi/2) ;calculate the x pos of
         ;the toe
      fpz2=fpz0+radius2*cos(_rot*(i-1)-pi/2)  ;calculate the y pos of
         ;the toe
      fpy2=fpy1+fp_length*cos(fp_angle) ;calculate the z pos of the toe
         ;command
      sel pile id fpcount begin fpx1 fpy1 fpz1 end fpx2 fpy2 fpz2 nseg 10
         ;make the forepole
   endcommand
   endloop
command

   sel pile id fpcount prop emod 200e9 xarea 0.002195 per 0.179 slide on &
cs_scoh 25e6 cs_nk 100e9 cs_sfric 35 dens 7750
   ;give it properties
endcommand

fpy0=fpy1+_fpadvance
   ;set the start point for the next set of forepoles
endif
fpcount=fpcount+1
   ;counter to determine when to go from tunnel 1 to tunnel 2
   ;relate the installation to the number of segments in the tunnel
end
_install_fp
forepoles
   ; need to invoke this function every time you want to advance the
   ;do not call the entire file as this will reset the counters, only
command
   ; the '_fp_install' command is required and it will
   ;automatically update everything.
3.8.2 Forepole Support File Sequencing in FLAC3D

This file sequence implements supports in the form of forepoles using pile elements within the FLAC3D program. These pile elements are expanded on fully in the following sections within this Appendix (i.e. FLAC3D specific coded elements; Pile SEL). **Figure A8** notes various screen captures during a numerical model run.

**Figure A8** (a) Orientation of installation of Forepoles for Driskos Tunnel, (b) configuration within FLAC3D depicting forepole installation, and (c) Overlapping forepoles with bolting sequence with reference to real life tunnel excavation practice.
3.9 Bolting Support Installation

Two FISH files are used (driskos_bolting.fis and driskos_bolting_walls.fis) to install the bolting of the arch (upper top heading) and walls (after bench has been excavated) respectively. The installation is tied to the geometry and called upon within the run file. The bolts are installed in a staggered pattern based upon the excavation interval. The spacing of the bolts is controlled by the num_bolt1 variable based on the bolt spacing and the circumference of the tunnel as in the Drikos tunnel. The second part of the function will offset the pattern and consists of one less bolt in the arch, and one extra bolt in the walls. This FISH function is also tied to the number of excavation steps and automatically transition from one tunnel to the other once the num_segs threshold is reached. Simultaneous extraction can be accommodated by adjusting the offset value in the second half of the function.

3.9.1 Rockbolt Support File Project Code

3.9.1.1 driskos_bolting.fis file

; This file contains all of the necessary commands to input the offset bolt spacing for the Driskos tunnels.
; Automatically advances the offset bolting pattern based upon the position of the advancing face by calling in the "_install_bolts" function at each mining step
; piles SELs are used as they allow for the rockbolt logic.
; residual strength of rockbolts cannot be modelled directly.
; Need to supply grout strength parameters for the bolts to get correct loading and response.

;============================================================================================================================
; Definition of variables to define how the bolts are installed.
;============================================================================================================================
define _bolt_parameters ;this function is used to allocate the proper number of bolts and their positions. Info is used in _install_bolts function

bolt_count=99 ;Tracking ID to advance the bolting cycle.
num_bolt1=16 ;Total number of bolts in the master ring
num_bolt2=num_bolt1-1 ;number of bolts in the offset ring.

bolt_offset=1.0 ;ring offset for each bolt
boltyo=yo+0.5 ; fixed reference for close side of tunnel
bolty0=boltyo ; temporary y reference that changes as the tunnel advances
bolty00=boltyo ; secondary reference needed to reset the geometry back to
the start of the second tunnel

bolty01=bolty0+bolt_offset ; position marker for offset ring in second tunnel
boltz0=z1

bolt_length=6.0 ; length of the bolt in meters

bolt_radius=x1 ; radius of the arch in the tunnel from the geometry section
bolt_radius2=x1+bolt_length ; the radius of the toe of the bolts

_boltadvance=segment_thick ; increments the next set of bolt installations

end

_bolt_parameters ; execute the function to load the variables into memory

;=================================================================

; Function used to actually install the bolts
;=================================================================

define _install_bolts

bolt_count=bolt_count+1 ; counter to determine when to go from tunnel 1 to tunnel 2

if bolt_count<num_segs+99.5 ; assume that once the first tunnel is fully excavated
end

loop m(1,2) ; loops needed to allow for offset pattern in the

loop n(1,num_bolt1) ; main bolting line

_rot=pi/(num_bolt1-1)
brtxt1=0.01+bolt_radius*sin(_rot*(n-1)-pi/2)
bolty1=bolty0+bolt_offset*2*(m-1)
boltz1=boltz0+bolt_radius*cos(_rot*(n-1)-pi/2)
boltz2=boltz0+bolt_radius2*cos(_rot*(n-1)-pi/2)

command

sel pile id bolt_count begin boltx1 bolty1 boltz1 end boltx2 bolty1 boltz2

nseg 8
endcommand
endloop
loop p(1,num_bolt2) ;offset bolting line
  _rot=pi/(num_bolt2)
boltx1=0.01+bolt_radius*sin(_rot*(p-1)-pi/2.0+_rot/2.0)
bolty2=bolty1+bolt_offset
boltz1=boltz0+bolt_radius*cos(_rot*(p-1)-pi/2.0+_rot/2.0)
boltx2=boltx0+0.001+bolt_radius2*sin(_rot*(p-1)-pi/2.0+_rot/2.0)
boltz2=boltz0+bolt_radius2*cos(_rot*(p-1)-pi/2.0+_rot/2.0)
command
  sel pile id bolt_count begin boltx1 bolty2 boltz1 end boltx2 bolty2 boltz2
nseg 8
endcommand
endloop

command
  sel pile id bolt_count prop emod 200e9 xarea 4.91e-4 per 0.09 slide on &
  cs_scoh 25e6 cs_sk 100e9 cs_sfric 35 dens 7750 tyield 0.2e6 rockbolt on
endcommand

bolty0=bolty0+_boltadvance ;set the start point for the next set of bolts
endif

if bolt_count>num_segs+99.5 ;once the value of k is greater than the number of
  segements in the first tunnel
  ;it will reset the positions to go to the second tunnel
loop m(1,2) ;loops needed to allow for offset pattern in the
  ;bolting
  loop n(1,num_bolt1) ;main bolting line
    _rot=pi/(num_bolt1-1)
boltx1=x2*2+0.01+bolt_radius*sin(_rot*(n-1)-pi/2)
bolty1=bolty00+bolt_offset*2*(m-1)
boltz1=boltz0+bolt_radius*cos(_rot*(n-1)-pi/2)
boltx2=x2*2+0.001+bolt_radius2*sin(_rot*(n-1)-pi/2)
boltz2=boltz0+bolt_radius2*cos(_rot*(n-1)-pi/2)
  command
    sel pile id bolt_count begin boltx1 bolty1 boltz1 end boltx2 bolty1 boltz2
  nseg 8
  endcommand
  endloop
endloop

loop p(1,num_bolt2) ;offset bolting line
  _rot=pi/(num_bolt2)
boltx1=x2*2+0.01+bolt_radius*sin(_rot*(p-1)-pi/2.0+_rot/2.0)
bolty2=bolty1+bolt_offset
boltz1=boltz0+bolt_radius*cos(_rot*(p-1)-pi/2.0+_rot/2.0)
boltx2=x2*2+0.001+bolt_radius2*sin(_rot*(p-1)-pi/2.0+_rot/2.0)
boltz2=boltz0+bolt_radius2*cos(_rot*(p-1)-pi/2.0+_rot/2.0)

command
    sel pile id bolt_count begin boltx1 bolty2 boltz1 end boltx2 bolty2 boltz2
nseg 8
endcommand
endloop
end

command
    sel pile id bolt_count prop emod 200e9 xarea 4.91e-4 per 0.09 slide on &
    cs_scoh 25e6 cs_sk 100e9 cs_sfric 35 dens 7750 tyield 0.2e6 rockbolt on
endcommand

bolty00=bolty00+_boltadvance ;set the start point for the next set of bolts
endif
end

3.9.1.2 driskos_bolting_walls.fis file

; This file contains all of the necessary commands to input the offset
; bolt spacing for the Driskos tunnels.
; Automatically advances the offset bolting pattern based upon the position
; This function is used to put bolts into the walls of the bench using the same pattern as
; before.

;==========================================================================
; Definition of variables to define how the bolts are installed.
;==========================================================================
define _boltw_parameters
    ;bolts and their positions. Info is used in _install_bolts
    ;function

    boltw_count=199 ;Tracking ID to advance the bolting cycle.
    num_boltw1=3 ;Total number of bolts in the master ring
    num_boltw2=3 ;number of bolts in the offset ring.
boltw_offset=1.0 ;ring offset for each bolt

boltwy0=y0+0.5 ;fixed reference for close side of tunnel
boltwy0=boltwy0 ;temporary y reference that changes as the tunnel advances
boltwy00=boltwy0 ;secondary reference needed to reset the geometry back to
    the start of the second tunnel

boltwy01=boltwy0+boltw_offset ;position marker for offset ring in second tunnel
boltwz0=z1
boltw_spacing=1.2

boltw_length=6.0 ;length of the bolt in meters

boltw_radius=x1
boltw_radius2=x1+boltw_length

_boltwadvance=segment_thick ;increments the next set of bolt installations

end

_boltw_parameters ;execute the function to load the variables into memory

; Function used to actually install the forepoles
;===================================================================
deﬁne _install_bolts_walls

boltw_count=boltw_count+1 ;counter to determine when to go from tunnel 1 to
    ;tunnel 2

if boltw_count<num_segs+199.5 ;assume that once the first tunnel is fully excavated
    ;before jumping to other side

loop m(1,2) ;loops needed to allow for offset pattern in the
    ;bolting

    boltwx1=-x1-0.01
    boltwx2=-x1-0.01-boltw_radius
    boltwx3=x1+0.01
    boltwx4=x1+0.01+boltw_radius
    boltwy1=boltwy0+boltw_offset*2*(m-1)
    boltwz1=boltwz0-boltw_spacing
    boltwz2=boltwz0-boltw_spacing

    command
        sel pile id boltw_count begin boltwx1 boltwy1 boltwz1 end boltwx2
        boltwy1 boltwz1 nseg 8
        sel pile id boltw_count begin boltwx1 boltwy1 boltwz2 end boltwx2
        boltwy1 boltwz2 nseg 8

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sel pile id boltw_count begin boltwx3 boltwy1 boltwz1 end boltwx4
boltwy1 boltwz1 nseg 8
sel pile id boltw_count begin boltwx3 boltwy1 boltwz2 end boltwx4
boltwy1 boltwz2 nseg 8
endcommand
boltwx1=-x1-0.01
boltwx2=-x1-0.01-boltw_radius
boltwx3=x1+0.01
boltwx4=x1+0.01+boltw_radius
boltwy2=boltwy1+boltw_offset
boltwz1=boltwz0-boltw_spacing/2.0
boltwz2=boltwz1-boltw_spacing
boltwz3=boltwz2-boltw_spacing
command
sel pile id boltw_count begin boltwx1 boltwy2 boltwz1 end boltwx2
boltwy2 boltwz1 nseg 8
sel pile id boltw_count begin boltwx1 boltwy2 boltwz2 end boltwx2
boltwy2 boltwz2 nseg 8
sel pile id boltw_count begin boltwx1 boltwy2 boltwz3 end boltwx2
boltwy2 boltwz3 nseg 8
endcommand
endloop
command
sel pile id boltw_count prop emod 200e9 xcarea 4.91e-4 per 0.09 slide on
&
cs_scoh 25e6 cs_sk 100e9 cs_sfric 35 dens 7750 tyield 0.2e6 rockbolt on
endcommand
boltwy0=boltwy0+_boltwadvance ;set the start point for the next set of bolts
endif
if boltw_count>num_segs+199.5 ;once the value of k is greater than the number of
segs in the first tunnel
;it will reset the positions to go to the second tunnel
loop m(1,2)
;loops needed to allow for offset pattern in the
;bolting
boltwx1=x2*2-x1-0.01
boltwx2=x2*2-x1-0.01-boltw_radius
boltwx3=X2*2+x1+0.01
boltwx4=x2*2+x1+0.01+boltw_radius
boltwy1=boltwy00+boltw_offset*2*(m-1)
boltwz1=boltwz0-boltw_spacing
boltwz2=boltwz1-boltw_spacing

command
    sel pile id boltw_count begin boltwx1 boltwy1 boltwz1 end boltwx2
    boltwy1 boltwz1 nseg 8
    sel pile id boltw_count begin boltwx1 boltwy1 boltwz2 end boltwx2
    boltwy1 boltwz2 nseg 8
    sel pile id boltw_count begin boltwx1 boltwy1 boltwz3 end boltwx2
    boltwy1 boltwz3 nseg 8
    boltwy1 boltwz2 nseg 8
endcommand

    boltwx1=x2*2-x1-0.01
    boltwx2=x2*2-x1-0.01-boltw_radius
    boltwx3=x2*2+x1+0.01
    boltwx4=x2*2+x1+0.01+boltw_radius
    boltwy2=boltwy1+boltw_offset
    boltwz1=boltwz0-boltw_spacing/2.0
    boltwz2=boltwz1-boltw_spacing
    boltwz3=boltwz2-boltw_spacing

command
    sel pile id boltw_count begin boltwx1 boltwy2 boltwz1 end boltwx2
    boltwy2 boltwz1 nseg 8
    sel pile id boltw_count begin boltwx1 boltwy2 boltwz2 end boltwx2
    boltwy2 boltwz2 nseg 8
    sel pile id boltw_count begin boltwx1 boltwy2 boltwz3 end boltwx2
    boltwy2 boltwz3 nseg 8
    boltwy2 boltwz2 nseg 8
    boltwy2 boltwz3 nseg 8
endcommand

endloop

command
    sel pile id boltw_count prop emod 200e9 xcarea 4.91e-4 per 0.09 slide on
    cs_scoh 25e6 cs_sk 100e9 cs_sfric 35 dens 7750 tyield 0.2e6 rockbolt on
endcommand

boltwy00=boltwy00+_boltwadvance   ;set the start point for the next set of bolts
endif

end
3.9.2 Rockbolt Support File Sequencing in FLAC3D

This file sequence implements supports in the form of rockbolts using pile elements (SELs) within the FLAC3D program. These pile elements are expanded on fully in the following sections within this Appendix (i.e. FLAC3D specific coded elements; Pile SEL). The previous figure, Figure A8, notes various screen captures during a numerical model run and highlights in black the rockbolts that have been installed. Note that depending on the rock class within Driskos tunnel, some designs call for rockbolts on the top heading (Class IV) and some only have forepole installation and sidewall bolt installation (Class V+). Figure A9 denotes the orientation and offsetting installation of the rockbolt pattern. Rockbolts are installed in rows at 1.0 m spacing and are offset by 0.6 m (within this bolting sequence).

---

**Figure A9** Denotes bolting sequence (spacing and offset) for tunnel roof and walls.

3.10 Tunnel Temporary Support Liner Installation

Two FISH files are used (driskos_liner.fis and driskos_liner_walls.fis) to install the shotcrete temporary support throughout the tunnel bore after excavation for lining of the arch and walls respectively. The installation is tied to the geometry and called upon within the run file. Similar to the other support functions, these are cross-referenced with the advance rate (step) and will skip to the other tunnel when the end of the excavation of the first tunnel is reached. Thresholds are placed on the surface to which the shotcrete
has been applied and is limited to the inside of the tunnel excavation. Note that the temporary support liner within the model runs of this thesis have been combined with the support provided by the HEB steel sets also used for support purposes.

3.10 Rockbolt Support File Project Code

3.10.1.1 driskos_liner.fis

; This file contains the necessary commands to create the shotcrete liner elements in FLAC3D. The liner is set to follow the tunnel advance and as of now, is only set up to apply the liner to the arched portion of the top heading. The liner_wall file deals with the lining of the walls after bench excavation. Liner elements in FLAC3D are elastic. If a non-linear response is desired it must be built into the model as actual zones.

; Definition of variables necessary to define how the shotcrete liner is installed.

define _lin_parameters ;this function is used to allocate the variables needed to define how the shotcrete gets applied
lincount=1 ;starting ID for the first liner segment. This is incremented for each one inserted
linx0=x0 ;reference location for the liner to center on the tunnel
liny0=y0+0.01 ;fixed reference for close side of tunnel
liny00=liny0 ;temporary y reference that changes as the tunnel advances
liny00=liny00 ;secondary reference needed to reset the geometry back to the start of the second tunnel
linz0=z1+0.1 ;vertical reference so that liner is only installed on the arch on the first pass
lin_advance=segment_thick ;increments the counter so advance the tunnel
end _lin_parameters ;execute the function to load the variables into memory

; Installation of the arched portion of the liner.

define _install_liner
if lincount < num_segs+0.5 ;if the counter determines that the first tunnel is still being excavated, apply the correct shotcrete section
    linx1=linx0-x1-1 ;limit the range of the wall to be included
    linx2=linx0+x1+1
end
liny1 = liny0  ; start the liner at the beginning of the tunnel
liny2 = liny0 + lin_advance - 0.1  ; limit the liner to the end of the tunnel minus a small threshold
linz1 = linz0  ; this prevents liner from being installed on the face
linz2 = z2 + 1  ; limit the liner to the arched portion of the tunnel

command
    sel liner id lincount range x linx1 linx2 y liny1 liny2 z linz1 linz2
    ; make the liner
    sel liner id lincount prop iso = (35e9, 0.2) thick = 0.25  ; 25cm thick install
    sel liner id lincount prop cs_nk = 7.4e10 cs_sk = 7.4e10 cs_scoh = 1e20
endcommand

liny0 = liny1 + lin_advance  ; set up the next liner advance place holder
endif

if lincount > num_segs + 0.5  ; if the tunnel is now on the second half
    linx1 = x2 * 2 - x1 - 1  ; uses the centroid of the second half as a reference
    ; (2 * x2 = center of second tunnel)
    linx2 = x2 * 2 + x1 + 1
    liny1 = liny0
    liny2 = liny0 + lin_advance - 0.1
    linz1 = linz0
    linz2 = z2 + 1
    command
        sel liner id = lincount range x linx1 linx2 y liny1 liny2 z linz1 linz2
        ; make the liner
        sel liner id = lincount prop iso = (35e9, 0.2) thick = 0.25  ; 25cm thick install
        sel liner id = lincount prop cs_nk = 7.4e10 cs_sk = 7.4e10 cs_scoh = 1e20
    endcommand

    liny0 = liny1 + lin_advance  ; set up the next liner advance place holder
endif

lincount = lincount + 1  ; increment the counter so that the shotcrete advances with the tunnel
end

__install_liner
sel node fix y xr zr range y -0.1 0.1  ; only at the faces and outside model
    ; need to repeat this command at the end of the first tunnel and again at the start of the second tunnel
3.10.1.2 driskos_liner_wall.fis

; This second liner file is used to apply shotcrete to the walls of the bench following
; the excavation of the arch.

; Definition of variables necessary to define how the shotcrete liner is installed.

define _lin_wall_parameters
lincountw=1  ; starting ID for the first bench segment
linwx0=x0  ; reference location for the liner to center on the tunnel
linwyo=y0+0.01 ; fixed reference for close side of tunnel
linwy0=linwyo ; temporary y reference that changes as the tunnel advances
linwy00=linwy0 ; secondary reference needed to reset the geometry back to the start
; of the second tunnel
linw_advance=segment_thick  ; increments the counter so advance the tunnel
end

.lin_wall_parameters  ; execute the function to load the variables into memory

; Installation of the arched portion of the liner.

define _install_liner_walls
if lincountw < num_segs+0.5 ; if the counter determines that the first tunnel is still being
; excavated, apply the correct shotcrete section
    linwx1=linwx0-x1-1 ; limit the range of the wall to be included
    linwx2=linwx0-x1+1
    linwx3=linwx0+x1-1 ; limit the range of the wall to be included
    linwx4=linwx0+x1+1
    linwy1=linwy0  ; start the liner at the beginning of the tunnel
    linwy2=linwy0+linw_advance-0.1 ; limit the liner to the end of the tunnel minus
; a small threshold
    linwz1=z0+0.1 ; this prevents liner from being installed on
; the face
    linwz2=z1-0.1 ; limit the liner to the arched portion of the tunnel
command
    sel liner id lincountw range x linwx1 linwx2 y linwy1 linwy2 z linwz1 linwz2
linwz2
    sel liner id lincountw range x linwx3 linwx4 y linwy1 linwy2 z linwz1 linwz2
3.10.2 Temporary Support Liner File Sequencing in FLAC3D

This file sequence implements the liner support in the form of a liner SEL within the program. These liner elements are expanded on fully in the following sections within this Appendix (i.e. FLAC3D specific coded elements; Liner SEL). The previous figure, Figure A8(c), notes various screen captures during a numerical model run and highlights in red the liner that has been installed. Figure A10 also depicts a liner that has been
installed in the first tunnel after top heading excavation and part-way through bench excavation sequence. Note that only the excavation and liner are shown; these are within the tunnel geometry as previously shown in Figure 1.4.

Figure A10  Support Liner within FLAC3D (Liner SEL).

4. Elements used in FLAC3D Model

Elements or zones in FLAC3D are the basic elements that are used to construct geometries. These are constructed and connected to the grid system within the program code. These zones can then be grouped and assigned properties. Structural elements (SEls) or support elements are also included in FLAC3D formulation; the numerical formulation that supports the structural-element logic. Structural elements can either be independent of or coupled to the grid representing the solid continuum. This section (Table A2) summarizes the elements (zones) and structural elements used in the formulation of the numerical model and provides insight to these specialized, pre-programmed elements within the FLAC3D software. All element properties and a more detailed description is provided in the FLAC3D Version 3.0 user’s guide (Itasca, 2005).
<table>
<thead>
<tr>
<th>Type and Name of Element</th>
<th>Diagram of Element</th>
<th>Description of Element</th>
</tr>
</thead>
</table>
| **Radial Tunnel**  
*Radtunnel*  
Primitive Mesh Shape | ![Diagram](image)  
*Description*  
These zones are created within a 3D volume with the **GENERATE zone** command. This command can be used independently to create a zoned model of an explicit primitive shape (or grid-connectivity type). The primitive shapes presented here have been used in the geometry file in order to create the desired twin tunnel configuration as required. The meshes used range from a simple brick shape to a complex radial tunnel shape. Several **GENERATE zone** commands were given in order to connect these primitive together to build the FLAC3D grid. The characteristics of the shape (e.g., global coordinate positions, number of zones) are defined by specifying the keywords after the shape keyword (i.e. dimensions d1, d2 etc.). |  |
| **Brick**  
*Brick*  
Primitive Mesh Shape | ![Diagram](image)  
*Description*  
These zones are created within a 3D volume with the **GENERATE zone** command. This command can be used independently to create a zoned model of an explicit primitive shape (or grid-connectivity type). The primitive shapes presented here have been used in the geometry file in order to create the desired twin tunnel configuration as required. The meshes used range from a simple brick shape to a complex radial tunnel shape. Several **GENERATE zone** commands were given in order to connect these primitive together to build the FLAC3D grid. The characteristics of the shape (e.g., global coordinate positions, number of zones) are defined by specifying the keywords after the shape keyword (i.e. dimensions d1, d2 etc.). |  |
| **Radial Cylinder**  
*Radcylinder*  
Primitive Mesh Shape | ![Diagram](image)  
*Description*  
These zones are created within a 3D volume with the **GENERATE zone** command. This command can be used independently to create a zoned model of an explicit primitive shape (or grid-connectivity type). The primitive shapes presented here have been used in the geometry file in order to create the desired twin tunnel configuration as required. The meshes used range from a simple brick shape to a complex radial tunnel shape. Several **GENERATE zone** commands were given in order to connect these primitive together to build the FLAC3D grid. The characteristics of the shape (e.g., global coordinate positions, number of zones) are defined by specifying the keywords after the shape keyword (i.e. dimensions d1, d2 etc.). |  |
| **Pile Structural Elements**  
PileSEls | ![Diagram](image)  
*Description*  
These zones are created within a 3D volume with the **GENERATE zone** command. This command can be used independently to create a zoned model of an explicit primitive shape (or grid-connectivity type). The primitive shapes presented here have been used in the geometry file in order to create the desired twin tunnel configuration as required. The meshes used range from a simple brick shape to a complex radial tunnel shape. Several **GENERATE zone** commands were given in order to connect these primitive together to build the FLAC3D grid. The characteristics of the shape (e.g., global coordinate positions, number of zones) are defined by specifying the keywords after the shape keyword (i.e. dimensions d1, d2 etc.). |  |
| **Liner Structural Elements**  
LinerSEls | ![Diagram](image)  
*Description*  
These zones are created within a 3D volume with the **GENERATE zone** command. This command can be used independently to create a zoned model of an explicit primitive shape (or grid-connectivity type). The primitive shapes presented here have been used in the geometry file in order to create the desired twin tunnel configuration as required. The meshes used range from a simple brick shape to a complex radial tunnel shape. Several **GENERATE zone** commands were given in order to connect these primitive together to build the FLAC3D grid. The characteristics of the shape (e.g., global coordinate positions, number of zones) are defined by specifying the keywords after the shape keyword (i.e. dimensions d1, d2 etc.). |  |
5.0 Comments on Numerical Model and Trouble Shooting

The numerical model that has been presented in this Appendix took over one year to develop. The complex geometry and installation of support elements at specific locations was not a trivial task. The reader of this document is asked to keep in mind that the initial model took at least 4 days to complete its series of computations for a supported run. The author consulted with his thesis advisor, Dr. Mark Diederichs, as well as with Itasca Canada in order to develop the completed numerical model. As with any numerical model, there are many considerations that one must take into account; ranging from model construction, constituent model, boundary conditions etc. This model was developed in order to represent the true construction of a tunnel in the field. These geometries and tunnelling practices had to be captured by the numerical model in order to ensure that validation data collected at the Driskos tunnel site was applicable.

All of the recommendations for the monitoring and testing of the model were conducted as presented in Chapter 5 of this thesis report (i.e. monitoring of boundary conditions, vectors and unbalanced forces as well as the application of history points at specific locations). Further to these checks and balances, Chapter 3 of the Itasca Manual (Itasca, 2005), Problem Solving with FLAC3D was consulted in order to ensure that all of the recommended practices were followed within the limitations of this software application.

Below are selected (a very small sample of problems has been cited here) examples of issues that surfaced during the development stage of model creation.

5.1 Boundary Conditions and Boundary Effects Example

Figure A11 below depicts a cross-section of the first bore tunnel. Excavation of the top heading has been completed and the excavation of the bench is almost complete. The colours show the state of plasticity for the rockmass. There is a blowout of stresses (vectors) at the boundary as the magnitude of the velocity at the boundary gridpoints is two orders of magnitude larger than expected. The boundary conditions used were 3 applied forces act faces in the x, y, and z, direction with 3 fixed boundaries opposite to
these applied surfaces. As can be seen, the boundary conditions caused an unrealistic stress effect at the entrance and exit of the tunnel portal.

**Figure A11** Block State of Plasticity for Selected Boundary Conditions

Troubleshooting of the system began by trying to determine what real boundary conditions could be used for the model. An early attempt was to fix the boundary at all boundary locations at the centre of the model (i.e. to act as a membrane) as well as fixed boundaries on an x and z face. **Figure A12** below depicts these fixity conditions. The same “blowout” effect at each of the y faces was experienced using these boundary conditions.

**Figure A12** Numerical model showing locations of fixity
This situation was rectified by applying a fixed condition on both of the y faces (as shown in the boundary conditions section of this Appendix). Also seen here is a line (shown within the circled section) that indicates that elements in that region are triangular in shape and that another boundary has been added. This effect is not real. The graphics interface for the program produces this error and it can be overcome by a simple (if one knows this from the start) adjustment of the cutting plane configuration.

Errors such as those presented in Figure A12 also had to be investigated and solved. Here, an illegal geometry issue crashed the running program. The issue had to do with the forepoles that exceeded the boundaries of the model (circled portion in figure). As a result, forepoles were installed up to a point just prior to the boundary of the exit portal for each of the tunnel bores.

Figure A12  Illegal Geometry error during numerical model run.
Figures A13 and A14 depict the errors that presented themselves during model running. It was determined that both these errors presented themselves only when a plot of the plasticity (Block State) was turned on during the processing stage of the modelling process. This was a limitation of the FLAC3D software. The author required to monitor the plasticity zone, however, this was conducted after the full run without keeping the plot plasticity command active during any particular run.

**Figure A13** Block Plot Error during numerical model run.

**Figure A14** Runtime Error during numerical model run.
Figure A15 and A16 depict the real challenge of ensuring that element geometries and boundary conditions are accurate and constantly monitored. All geometries had to be checked during the post-processing stage in order to ensure that the tolerances and installation of support elements were conducted correctly. It was determined that installation of liner elements at 90° to one-another presented another numerical error.

Figure A15  Geometry tolerance of liner system.

Figure A16  Geometry tolerance of boundary conditions and liner system.
5.2 List of other Issues that Arose During Model Construction and Running

The previous section highlighted but a selected few of the issues that arose during numerical model construction and running of the model within FLAC3D. The real issues associated with developing a realistic and accurate model are too numerous to expand upon here. Listed in this section, however, are a short list of other issues that were also investigated and solved during model construction and implementation:

a. Had to determine exact locations of monitoring points within model and Driskos tunnel;
b. Data management due to size of files was an issue;
c. Length of time in order to run the complete model;
d. Hoek-Brown model within FLAC3D not accepting parameters such as bulk modulus etc.;
e. Block plot errors when monitoring the block state of the material while running a simulation;
f. Model error due to null command for excavation. Recommend using delete function which also frees memory;
g. Troubleshooting associated with orientation of all support elements and their configurations;
h. Large Strain versus small strain issues;
i. Determination of how to place histories within support elements;
j. Shotcrete failures and displacements of floor that caused more surfaces to be shotcreted;
k. Runtime errors: pure virtual function call;
l. Unbalanced force checks;
m. Failure of sidewall shotcrete to be installed;
n. Stresses, moments and interface checks of support elements and grid;
o. Determining orientation due to global and local co-ordinate systems; and
p. Extraction of Moments from elements not a trivial issue;
Appendix B

Technical Site Visits to Multiple Tunnelling Sites
Appendix B

Technical Site Visits to Multiple Tunnelling Sites

1.0 Introduction

This Annex includes material from multiple site visits that were conducted by the author as part of the overall field study for the region (accompanied at times with Dr. Diederichs and Dr. Marinos). The visits were conducted in September 2003, December 2003 and December 2007. Selected materials gathered from these site visits appear within the document proper, other appendices highlighting the geology of the region and within this appendix. Cited here are:

a. Photos of tunnel construction (and other relevant works) at multiple sites as part of Egnatia Odos taken in September 2003; and

b. Itinerary of Tunneling Works for Egnatia Odos as well as other tunneling sites within Greece, conducted in December 2007.

2.0 Program Outline for Technical Site Visit – September 2003

The following section is a series of photographs that were taken at multiple construction sites along the Egnatia Odos. The author was escorted by Dr. Vassilis Marinos to each of the sites. The three day technical site visit lasted from September 21-23, 2003. The sequence of photos depicted follow in chronological order. Each photo also consists of a number that can be cross referenced with their location and relevant details as cited in the table following the photos.
Outline

- Photos and Technical points amassed during site visits of selected tunnels along the Egnatia Odos, Greece.

EGNATIA ODOSEGNATIA ODOS

- A massive construction project that is currently under construction in Northern Greece.
- Upgrade of the current highway.
- Old alignment follows the ancient Egnatia Road, an 800km route constructed by the Romans for military purposes in the 2nd century B.C.

- 680 km motorway (East-West)
- Total of 77 twin road tunnels
- Overall combined single carriageway length of 98 km
- 60 tunnels are bored or blasted tunnels, remainder are cut-and-cover
- 7% of the overall highway will be carried through tunnels
- 30% of the total estimated construction cost.
- Estimated overall budget of the project is $3.2 billion (US), 60% EU, 40% Greek government.

EGNATIA ODOS

- Athens

Day 1
- Dodoni Tunnel

Day 2
- Tunnels: Ioannina-Metsovo

Day 3
- S3 Tunnel Veria

Day 3 - S3 Tunnel Veria
Day 2 - Tunnels: Ioannina-Metsovo
Day 1 - Dodoni Tunnel

Scale 100 km
End of Day 1
Technical Tour of Selected Tunnels of the Egnatia Odos, Northern Greece

Nikolaos Vlachopoulos, M.Eng., P.Eng.
GeoEngineering Centre at Queen’s RMCC
Geology and Geological Engineering Department
Queen’s University
Kingston, Ontario

Technical Tour of Selected Tunnels of the Egnatia Odos, Northern Greece

Day 2

Egnatia Odos
End of Day 2
Technical Tour of Selected Tunnels of the Egnatia Odos, Northern Greece

Day 3

Nicholas Vlachopoulos, M.Eng., P.Eng.
GeoEngineering Centre at Queen’s
Geology and Geological Engineering Department
Queen’s University
Kingston, Ontario

Technical Tour of Selected Tunnels of the Egnatia Odos, Northern Greece

Day 3
End of Day 3
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<td>Near Exit of Dodoni Tunnel</td>
<td>Tectonized Limestone (azvestolithos) with Karstic features</td>
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<td>130</td>
<td>Exit to Dodoni Tunnel (East Side-Left Bore)</td>
<td>Sign</td>
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<td>131</td>
<td>Portals Dodoni Tunnel (East Side)</td>
<td>Embedded in Limestone</td>
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<td>132</td>
<td>Dodoni Tunnel (East Side-Left Bore)</td>
<td>Steel Sets (HEB)</td>
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<td>133-153</td>
<td>Dodoni Tunnel (East Side-Left Bore)</td>
<td>Facing Excavation (2.5 km from East Side) Full Face (NATM)</td>
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<td>Dodoni Tunnel (East Side-Left Bore)</td>
<td>Final Lining (Re-bar) (Oplismos)</td>
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<td>159-161</td>
<td>Dodoni Tunnel (East Side-Left Bore)</td>
<td>Rock Bolts (Angiria)</td>
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<td>Dodoni Tunnel (East Side-Right Bore)</td>
<td>Newer Type Rock Bolts (Expanding)</td>
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<td>Re-bar Installing Eqpt (&quot;Helona&quot;)</td>
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<td>Dodoni Tunnel (East Side-Right Bore)</td>
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<td>5 Umbrella's approx 60m Water Drains Overhead</td>
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<td>Facing</td>
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<td>Facing Material - Limestone Thinly Layered with Clay</td>
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<td>202-204</td>
<td>Dodoni Tunnel (West Side)</td>
<td>Limestone, Brecciated, Layered</td>
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<td>205-208</td>
<td>Egnatia Odos Near Dodoni Tunnel (West)</td>
<td>Cuts in Flysch - Green - geo net Photo #207 - Failure of Embankment/Cut</td>
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<td>209-212</td>
<td>Flysch Outcrop near Egnatia Odos Near Dodoni Tunnel (West)</td>
<td>Outcrop of alternating Sandstones and Siltstones Sandstone = Thicker Layer = Psamitis Siltstone = Thinner Layers = iliolithos Jeep</td>
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<td>Egnatia Odos Near Dodoni Tunnel (West)</td>
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<td>214</td>
<td>S1 or S2 Tunnel Entrance Portal (West Side)</td>
<td>Very Weak Flysch, mainly siltstone Plies reinforced concrete bored for stabilization of portal failure</td>
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<td>S1 or S2 Tunnel Entrance Portal (West Side)</td>
<td>Right Bore cover and Cut (Flysch)</td>
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<td>S1 or S2 Tunnel Entrance Portal (West Side)</td>
<td>Portal West Side - Right and Left Bore</td>
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<td>Tyria - Paramythia Portion (East Side)</td>
<td>Completed Portion of Egnatia</td>
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<td>219-220</td>
<td>Between S1 and S2 Portion 1.1.6</td>
<td>Bridge - Egnatia</td>
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<td>221-224</td>
<td>Between S1 and S2 Portion 1.1.6</td>
<td>Outcrop of Very Brecciated Limestone (discintrigated) Ravelling Conditions</td>
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<td>Portion of Egnatia Highway</td>
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<td>230-231</td>
<td>Portion 1.1.6 - Future Tunnel</td>
<td>Tunnel Will be excavated through slide area</td>
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<td>material consists of huge limestone blocks on Flysch</td>
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<td>of low thickness</td>
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<td>West of Portion 1.1.6</td>
<td>Portal East Side in Flysch</td>
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<td>233-238</td>
<td>Krika - East Side Portals</td>
<td>Portal - Cut and Cover in Limestone</td>
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<td>Krika - West Side Portals</td>
<td>Portal - Cut and Cover in Limestone</td>
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<td>Portal Failure - Left Bore Supported with Piles</td>
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<td>Mesovouni - West Side Portals - Right Bore</td>
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<td>Mesovouni - West Side Portals</td>
<td>In Limestone</td>
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<td>255-258</td>
<td>Mesovouni (1.1.3) - West Side Portals</td>
<td>Photo taken from Mesovouni Tunnel</td>
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<td>Krika Tunnel - West Side Portals</td>
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<td>S2 Incomplete Tunnel (Left Bore) Travelling E-W</td>
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<td>Near S2 East Portal</td>
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<td>Bridge Between S1 (East) and S2 (West)</td>
<td>In Limestone</td>
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<td>282-283</td>
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<td>S1 Tunnel - East Portal - Left Bore</td>
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<td>S1 Tunnel - East Portal - Left Bore</td>
<td>Forepole Detail</td>
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<td>290-292</td>
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<td>293</td>
<td>S1 Tunnel - Right Bore - Travel E-W</td>
<td>GOATS</td>
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<td>23-Sep-03</td>
<td>299-300</td>
<td>Ioannina</td>
<td>From Egnatia Office</td>
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<td>Driskos Tunnel - Left Bore - West Side (Sect 2.3)</td>
<td>In Flysch - Siltstone and Sandstone</td>
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<td>Driskos Tunnel - Left Bore - West Side (Sect 2.3)</td>
<td>Excessive Deformations beyond Line B (From Final Lining)</td>
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<td>314-318</td>
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<td>Shotcrete Applicator</td>
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<td>Driskos Tunnel - Left Bore - West Side (Sect 2.3)</td>
<td>Liner Detail</td>
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<td>Driskos Tunnel - Left Bore - West Side (Sect 2.3)</td>
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<td>Undermined Section</td>
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<td>322-323</td>
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<td>Driskos Tunnel - Left Bore - West Side</td>
<td>Undermined Shotcrete Mat'l</td>
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<td>Driskos Tunnel - Left Bore - West Side</td>
<td>Instrumentation - Top - Pressure Cell</td>
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<td>Instrumentation - Bottom - Extensometers</td>
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<td>Outside of Driskos Tunnel - Left Bore - West Side</td>
<td>Forepoling Machine</td>
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<td>329</td>
<td>Egnatia Construction Mngr Office - Driskos</td>
<td>Yianni Papadatos &amp; Robin Miles</td>
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<td>Technical Support Team - Egnatia Odos</td>
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<td>330-333</td>
<td>Near Driskos - Exit - East Side (Sect 2.3)</td>
<td>Outcrops of Angular Folds of Siltstone</td>
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<td>334-335</td>
<td>Near Driskos - Exit - East Side</td>
<td>Egnatia Sign</td>
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<td>336</td>
<td>Near T6 Tunnel (Sect 2.4)</td>
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<td>Near T6 Tunnel (Sect 2.4)</td>
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<td>338-339</td>
<td>Tunnel T8 - West Side - Portal (Sect 2.4)</td>
<td>Siltstones, Conglomerates</td>
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<td>340-345</td>
<td>Tunnel T8 - West Side - Right Bore</td>
<td>Tunnel Face - Bench Excavation</td>
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<td>Explosives Used</td>
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<td>346-347</td>
<td>Tunnel T8 - West Side - Right Bore (Looking West)</td>
<td>Tunnel Detail</td>
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<td>248-352</td>
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<td>Tunnel T6 - East Side (Sect 2.4)</td>
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<td>Tunnel T8 - West Side - Left Bore</td>
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<td>355-362</td>
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<td>Benchmark after Explosives</td>
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<td>Tunnel T8 - West Side - Right Bore</td>
<td>Blasting Caps and Explosives</td>
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<td>364</td>
<td>Tunnel T8 - West Side - Right Bore</td>
<td>FEL - With Chains</td>
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<td></td>
<td>365-366</td>
<td>Tunnel T8 - West Side - Right Bore</td>
<td>Steel Sets</td>
</tr>
<tr>
<td></td>
<td>367</td>
<td>Driskos Tunnel Exit (East Side)</td>
<td>From Distance</td>
</tr>
<tr>
<td></td>
<td>368-373</td>
<td>Section 3.1 (Close to Anthohori)</td>
<td>Berlin Wall - Cut and Cover</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Weak Siltstone, Failures Apparent</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Thrust Zone, Anchored Slope</td>
</tr>
<tr>
<td>Date</td>
<td>Image #</td>
<td>Location</td>
<td>Details</td>
</tr>
<tr>
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<td>---------------------------------------------------------------------------</td>
<td>---------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>23-Sep-03</td>
<td>374</td>
<td>Section 3.1</td>
<td>Second, Completed Cut and Cover Tunnel</td>
</tr>
<tr>
<td></td>
<td>375</td>
<td>Anthohori Tunnel - West Side (Section 3.2)</td>
<td>In Weak Siltstone, Thrust Zone</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Flysch, Most Movement, Most Challenging</td>
</tr>
<tr>
<td></td>
<td>376-377</td>
<td>Anthohori Tunnel - West Side - Right Bore</td>
<td>Detail of Successive Forepoling Sections</td>
</tr>
<tr>
<td></td>
<td>378-379</td>
<td>Anthohori Tunnel - West Side - Right Bore</td>
<td>Invert Detail - Floor of Tunnel</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bench Material Detail</td>
</tr>
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<td></td>
<td>380-381</td>
<td>Anthohori Tunnel - West Side - Right Bore</td>
<td>Siltstone</td>
</tr>
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<td>382-383</td>
<td>Anthohori Tunnel - West Side - Right Bore</td>
<td>Tunnel Face - Bench Excavation</td>
</tr>
<tr>
<td></td>
<td>384-385</td>
<td>Votonosi Bridge - Sect 3.2 (Between Votonosi Tunnel and Anthohori Tunnel)</td>
<td>Anchor Installation</td>
</tr>
<tr>
<td></td>
<td>386-390</td>
<td>Anthohori Tunnel - East Side - Right Bore</td>
<td>Repairs of Invert</td>
</tr>
<tr>
<td></td>
<td>391-392</td>
<td>Anthohori Tunnel - East Side - Right Bore</td>
<td>Anchors outside Portals</td>
</tr>
<tr>
<td></td>
<td>393-394</td>
<td></td>
<td>Egnatia Sign / Map</td>
</tr>
<tr>
<td>STA 396-398</td>
<td>401</td>
<td>Metsovo Interchange</td>
<td>Egnatia Alignment</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>In Red Siltstone</td>
</tr>
<tr>
<td></td>
<td>402</td>
<td>Anilio Tunnel - West Side</td>
<td>Sandstone and Siltstone (Mainly Siltstone)</td>
</tr>
<tr>
<td></td>
<td>403-406</td>
<td>Anilio Tunnel - West Side - Right Bore</td>
<td>Tunnel Support Measures Details</td>
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<td>407-417</td>
<td>Anilio Tunnel - West Side - Right Bore</td>
<td>Tunnel Face Excavation</td>
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<td></td>
<td></td>
<td>Face Stabilization using Hard FibreGlass</td>
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<td></td>
<td></td>
<td>Forepoling</td>
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<td>418</td>
<td>Anilio Tunnel - West Side - Right Bore</td>
<td>looking West to Tunnel Face</td>
</tr>
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<td></td>
<td>419</td>
<td>Anilio Tunnel - West Side - Right Bore</td>
<td>micro piles - part of initial support design</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>micropiles deemed unnecessary further excavation</td>
</tr>
<tr>
<td></td>
<td>421-440</td>
<td>Anilio Tunnel - West Side - Left Bore</td>
<td>Tunnel and Face Details</td>
</tr>
<tr>
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<td></td>
<td>Active Forepoling - 8m</td>
</tr>
<tr>
<td>STA 428</td>
<td></td>
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<td>Active Forepoling - 8m</td>
</tr>
<tr>
<td>STC 430</td>
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<td>Active Forepoling - 8m</td>
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<td>STA 441-445</td>
<td></td>
<td>Anilio Tunnel - West Side - Left Bore</td>
<td>Instrumentation - Extensometers and Pressure Cell</td>
</tr>
<tr>
<td></td>
<td>447-453</td>
<td>Anilio Tunnel - West Side - Left Bore</td>
<td>Forepoling</td>
</tr>
<tr>
<td></td>
<td>454</td>
<td>Anilio Tunnel - West Side - Left Bore</td>
<td>Anchors</td>
</tr>
<tr>
<td></td>
<td>455</td>
<td>Anilio Tunnel - West Side - Left Bore</td>
<td>Egnatia Regional Office</td>
</tr>
</tbody>
</table>
3.0 Program Outline for Technical Site Visit – December 2007

The program was organized by the Geotechnical Engineering Department at the National Technical University of Athens (NTUA), by Professor Paul Marinos. This program is undertaken annually by the Graduate Students within the Geotechnical Department (tunnelling specialty) at NTUA. The author took part in this excursion as he was in Greece working with NTUA from September 2007 – January 2008. During this time, he also visited the Driskos tunneling site.

The site visit outlined in this section was conducted from December 11\textsuperscript{th} to 16\textsuperscript{th}. It is part of the Tunnel Design course that the author audited while at NTUA. The tunneling sites that were visited were part of Egnatia Odos, the National Highway and the Greek National Railway. Assignments were conducted at each site, while student presentations and technical talks were held in the evenings.

This outline was translated from Greek. The following is an excerpt form the program outline as given to all staff and students prior to departure:

NATIONAL TECHNICAL UNIVERSITY OF ATHENS
INTERDEPARTMENTAL PROGRAM OF POSTGRADUATE STUDY
“PLANNING AND CONSTRUCTION OF UNDERGROUND WORKS”
The Faculties of Mining and Metallurgists
and Civil Engineers of NTUA

PROGRAM
EDUCATIONAL EXCURSION OF - EXERCISES OF COUNTRYSIDE OF ACADEMIC YEAR 2007-2008
TUNNELS UNDER CONSTRUCTION IN GREECE
Tuesday 11/12/2007

| Departure: 07:30 NTUA (Buildings of Civil Engineering) |
| Itinerary: Athens - Bourla - Tithorea - Bourla - Karditsa. |
| Overnight stay: Karditsa Hotel “ARNI PALACE”, Telephone: 2441022161 |
| Fax: 2441073080 |

**Main Objects**

1. **Tunnel Knimidas** in St. Konstantinos. Tunnel of Motorway adjacent with active crack. In-situ rock formations are limestones and dolomites.


**Exercises on-site**

- Exercise of estimate of movement at length of active crack.
- Syntax of geological tunnel (Kallidromo) from recognition of appearances in the countryside of morphology
- Determination of geotechnical parameters in rock masses
- Estimate of convergences
- Characterization GSI, RMR, \( s \), \( m_i \) in various rock masses

**Additional discussion subjects during travel time:**

- Tunnels of western “Regional Hymettus”
- Landslide of Malakasa area and her confrontation: the drainage tunnels (and the creation of counterpoise)
- Karst arrangement between Yliki Lake-Kopais Lake
- Draining of Kifissos River (in Viotia region) to Yliki lake with the tunnel Karditsa of Viotia prefecture.
- Activity of Euboean gulf, Arkitsa Crack - Kamma Vourla- Thermopyles
- Junction of Maliakos Gulf area with “sunk” tunnel (solution that was today abandoned)
- Tunnel at Orthrys [ERGOSE] and subjects of planning and methods of manufacture (that has recently commenced)

**Presentations (evening, at the hotel)**

The tunnel of Orthrys (Balasi -Marilia)
Wednesday 12/12/2007

<table>
<thead>
<tr>
<th>Itinerary:</th>
<th>Karditsa-Moyzaki-Drakotrypa-Pindos-Moyzaki-Larisa-Tempi-Beroia-Grebenai-Filippaioi</th>
</tr>
</thead>
</table>

| Overnight stay: Filippaioi: “Agnanti] of Begka”. [Tel] 24620 85339 Fax: 24620 25678 |

**Main object**

1. **Tunnel of deviation of Acheloos River**
   - Forehead of Drakotrypas. Opening up of rock with TBM that has been recently removed because of serious problems. Continuation with conventional method. Differentiation of behavior of formations (flysch), thin-bedded limestones, cherts, mixed phases. Site visit continues in Pindos geological unit, above from the region of tunnel (weather and time permitting). Identification of rockmass and tectonic structures.

2. **Tunnels of Egnatia Road**
   - Tunnels Veria-Leucopetra-Polymylos in partially metamorphosed and metamorphic rocks (today in operation):
     - Tunnel [S]3 Leucopetra (tunnel in marginal stability; its slopes required support. (phyllitis formation in relaxed structure).
     - Tunnel [S]4 Leucopetra: Chaotic material, adaptations of mapping, special planning of tunnel.
     - Tunnels in good condition: Gneiss formations ([S]5-[S]13)

**Exercises on the site**

- Exercise of choice of method of opening up and choice [TBM] (“Tree of decisions” exercise)
- Comparison of geological forecasts and discoveries at the perforation of tunnel of Acheloos River. Mapping of geological section.
- Estimate of stability in the depth of current forehead (hypertexts of big thickness) in the tunnel of Acheloos River.

**Additional discussion subjects during travel time**

- Thessalian plain, the overexploitation of underground waters and the work of deviation of Acheloos River.
- The railway and road tunnels: (Tempi, Platamonas).
**Presentations (evening, at the hotel)**

Tunnel of Kallidromo (by students)

Convergence Curves (P. Fortsakis, PhD cand. of Geotechnical Sector of NTUA).

<table>
<thead>
<tr>
<th>Thursday 13/12/2007</th>
</tr>
</thead>
<tbody>
<tr>
<td>Itinerary: Filippaioi-Kipouryio- “Passage of Bear” - Virgin Mary-Metsobo</td>
</tr>
<tr>
<td>Overnight stay: Metsobo Hotel “Galaxy” Tel. 2656041123, fax: 2656041124, “Kassaros” tel: 26560 41800, fax: 26560 41262</td>
</tr>
</tbody>
</table>

**Main objects**

1. **Tunnels of Egnatia Road**
   - Tunnels in molassa formations: Probabilities for gravitational form instability-excessive excavations
     - Tunnel Benetikon (completed - final stages)
     - Tunnel Karntza
     - Tunnel Koilomatos (placement of membranes)
     - Tunnel Laggadia
     - Tunnel Overcomes
     - Tunnel Zigkras (in construction)
     - Tunnel Acorns (completion of excavation)
     - Tunnel Saws (final investment)
     - Tunnel Aghantero
     - Tunnel of Aghia Paraskevi
     - Tunnel of Aghia Triada (construction of phase A)
   - Instability orifices in the tunnels in molassa. Choices of suitable solutions:
     - Tunnels Karantza, Koilomatos
   - Landslip in the formation of molassa (in attrition phase) after the exit of tunnel of Latch.
     - Partial change in mapping.
   - Tunnel of Latch. Tunnel in serpentines (entry).

**Exercises on-site**

- Quantitative characterization of molassa rockmass.
- Resistance of serpentines
• Trial of resistance of unbreakable rock ([s]ci)
• Determination of rockmass quality ([s]cm, E) via the GSI, mi, [s]ci,
• Estimate of deformities.

Presentations (evening, at the hotel)
Tunnel of Acheloos River (by students)
Tunnels in gneiss formations and instability of tunnel slopes [S]3 (students)
Demonstration of program “Dips and Unwedge” (Dr. Ch. Sarogloy)

<table>
<thead>
<tr>
<th>Date</th>
<th>Itinerary: Metsobo- Ioannina</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friday 14/12/2007</td>
<td>Overnight stay: Ioannina Hotel “DU LAC”, Telephone: 2651059100, Fax: 2651059200</td>
</tr>
</tbody>
</table>

Main object

1. Tunnels of Egnatia Road
   ✓ Case of poor designing of Malakasi Tunnel G (old tunnel that today has been repaired by Egnatia HM)
   ✓ Tunnel Malakasi B (“Parthenonas”) in serpentines and schist-cherts.
   ✓ Case of “Large Excavation”. The danger of substantial landslide and the how to eliminate the risk with suitable choice of way of designing/constructing passages.
   ✓ The second sector of tunnel Metsobo. Tunnel with the higher section on Egnatia Road in serpentines and flysch and in the area of an ophiolitic cover that is collapsed. Successful convergences with “succumbing” support.
   ✓ Tunnel Anilio in flysch. The location of the tunnel was chosen for the passage of the road so that it avoids the big landslide (Anilio). Tunnel in line with active tectonism phases and in various types of flysch. Instability of Eastern orifice.
   ✓ The change of mapping out in a section of Metsobitiko tunnel (to avoid landslides in the southern section) and tunnels Stubble, Two Tops, Krimno, Bytonosio. The geological environment of the area is flysch (good quality in terms of stability of its rockmass).
   ✓ Excavation at Anthochorio in chaotic flysch. Subjects of planning (collapse of a section of Pindos geological unit above the Ionian geological unit). Stability of orifices in slopes.
   ✓ Change of initial mapping out of regions with old landslips in Peristeri.
Exercises on-site

- Mapping exercise of geological section including elements of surface and drillings (tunnel Anilioy)
- Exercise of estimate of convergences (Tunnel Anthochorio or Anilioy)
- Exercise of estimate of tolerance of convergences for the planning of slipping frames
- Characterization of rockmass of flysch: GSI, mi, \(s\), 

Additional discussion subjects during travel time:

The landslislide at the Eastern slope of Metsobitiko and the choices in the mapping out of Egnatia Road.

Excavations in slopes that are dominated by flysch rockmass. Choice at places with solution “cut and cover”

High bridges at Metsobo and Botonosio

Bridge at Metsobitiko

Tunnel “Driskos”

Presentations (evening, at the hotel)

Tunnels in Molassa (by students)

Tunnels of Latch and Virgin Mary (by students)

Subjects of anisotropy (Mr. Victor)

Database on analysis of tunnels – V. Marinos, PhD cand.

Saturday 15/12/2007

Itinerary: Ioannina- Dodoni-Aghia Anastasia-Dodoni-Ioannina

Overnight stay: Ioannina Hotel “DU LAC”, Telephone: 2651059100, Fax: 2651059200

Main object

1. Tunnels of Egnatia Road
   - Tunnel Dodoni. Limestone, application of full-frontage excavation. Karst voids
   - Tunnel [S]2. Tunnel in flysch of the Ionian geological unit. Unstable situation in the western section of tunnel and challenges of landslides in slopes

2. Tunnels of “Railway Egnatia”
   - Tunnel Kosmiron. Karst Limestones and choice of method of construction – TBM choice
   - Tunnel of Aghia Anastasia
Exercises on the spot

- Mapping of geological section of tunnel Kosmira from the Geological Chart 1:50.000 of Institute of Geological and Mineral Research, Athens, Greece.
- Estimate of conditions of orifice of tunnel Aghia Anastasia.

Presentations (evening, at the hotel)

Tunnel Metsobo (by students)
Tunnels in flysch (by students)
Northern mapping out of Metsobo. Tunnels in “soft” flysch (by students)
Presentation of Program of [peperasmenon] elements Phase (G. Prountzopoulos, PhD cand. of Geotechnical Sector).

Sunday 16/12/2007

Itinerary: Ioannina-Antirrio-Rio-Athens

Discussion subjects during travel time:

Tunnels [ERGOSE] of Northern Peloponness of Aeghio, Platanis, Trapeza.
Tunnels of Kakia Skala. Tunnels in active tectonic area.
Cable bridge of Rio-Antirrio
Landslip at Panagopoula and future construction of bypass with tunnel.

RETURN IN ATHENS IN THE EVENING.

Equipment: Each student should bring galoshes and boots for fieldwork, raincoat suitable for field work as well as clothes to keep warm in low temperatures. Also: Notepad, pencil, rubber, colourful pencils, calculator, ruler, paper, as well as logarithmic paper.

It is recommended you bring a geological hammer and compass (especially for the mineralogists of the group, the trip will be an absolute delight, so bring your enthusiasm!).

PARTICIPANTS OF THE FIELDTRIP EDUCATIONAL EXERCISES IN UNDER MANUFACTURE TUNNELS IN GREECE, 2007-2008

PROFESSORS
1. Pavlos G. Marinos, Professor, Faculty of Civil Engineers
2. Georgios Tsiampaos, Associate Professor, Faculty of Civil Engineers
3. Michalis Sakellariou, Professor, Faculty of Urban Planners.
4. Major Nikolaos Vlachopoulos, Assistant Professor, Civil Engineering Dept., R.M.C, Canada.

POSTGRADUATE STUDENTS -SECOND CYCLE STUDENTS- PHD candidates- MEMBERS [NTUA]
(Triantafyllidou Despina (Secretary of Postgraduate program), Marinos Vasileios, Saroglou Charalampos).
Appendix C

Geological Mapping of Tunnel Face and Sidewalls
Appendix C

Geological Mapping of Tunnel Face and Sidewalls

1.0 Introduction

This Appendix contains photos and geological sketches of tunnel faces and sidewalls as captured during the excavation process at the Driskos Twin Tunnel Construction Site as provided by Egnatia Odos S.A. through a multitude of tunnelling logs and other relevant documents. The data contained within these tables has been amassed, consolidated and selected by the author. The consolidated format is also uniquely the author’s. Each table corresponds to a range of excavation based on the chainage of the tunnel. Where photos of the face were available, they were inserted into the corresponding table, otherwise an outcrop of same GSI within the chainage area was selected. Also included within the table are relevant properties associated with the rock mass such as the Geological Strength Index (GSI), Modulus of Elasticity (E), Intact Uniaxial Compressive Stress ($\sigma_{ic}$), Uniaxial Compressive Stress of Rock Mass ($\sigma_{cm}$), the lithology and the Rock Mass Category. The Legend for the cross-section data sheets is seen in Figure C1 below.

![Figure C1 Legend for cross-section data sheets](image)
### Section 1 of Driskos Tunnel

**Chainage:** 7+400 to 7+600

**Bore:** Left

<table>
<thead>
<tr>
<th>Properties</th>
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<tbody>
<tr>
<td>GSI</td>
<td>45<del>52(70%)-32</del>40(30%)</td>
</tr>
<tr>
<td>$E$</td>
<td>1.8-6.2 GPa</td>
</tr>
<tr>
<td>$\sigma_{ci}$ (UCS)</td>
<td>25-30 MPa</td>
</tr>
<tr>
<td>$\sigma_{cm}$</td>
<td>2.4-7.7 MPa</td>
</tr>
</tbody>
</table>

**Lithology:** SiSa

**Rock Mass Category:** III(70%)-IV(30%)

- **Photo of Tunnel Face or Outcrop at Chainage:** 7+400 to 7+600
- **Sidewall of Tunnel at Chainage:** 7+400 to 7+600
### Section 1 of Driskos Tunnel

**Chainage: 7+400 to 7+600**

**Bore: Right**

<table>
<thead>
<tr>
<th>Properties</th>
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<tbody>
<tr>
<td>GSI</td>
<td>45<del>52(70%)-32</del>40(30%)</td>
</tr>
<tr>
<td>$E$</td>
<td>1.8-6.2 GPa</td>
</tr>
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<td>25-30 MPa</td>
</tr>
<tr>
<td>$\sigma_{cm}$</td>
<td>2.4-7.7 MPa</td>
</tr>
</tbody>
</table>

**Lithology**

| SiSa |

**Rock Mass Category**

| III(70%)-IV(30%) |

---

Photo of Tunnel Face or Outcrop at Chainage: 7+400 to 7+600

Tunnel Face at Chainage: 7+428

Tunnel Face at Chainage: 7+452

Tunnel Face at Chainage: 7+475

Tunnel Face at Chainage: 7+525

Sidewall of Tunnel at Chainage: 7+400 to 7+600
Section 1&2 of Driskos Tunnel  
**Chainage: 7+600 to 7+800**  
**Bore: Left**

<table>
<thead>
<tr>
<th>Properties</th>
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</thead>
<tbody>
<tr>
<td>GSI</td>
<td>40~50</td>
</tr>
<tr>
<td>E</td>
<td>3.0-6.7 GPa</td>
</tr>
<tr>
<td>( \sigma_{ci} ) (UCS)</td>
<td>30-45 MPa</td>
</tr>
<tr>
<td>( \sigma_{cm} )</td>
<td>4.2-10.4 MPa</td>
</tr>
<tr>
<td>Lithology</td>
<td>SaSi/SiSa</td>
</tr>
<tr>
<td>Rock Mass Category</td>
<td>III(70%)-IV(30%)</td>
</tr>
</tbody>
</table>

Photo of Tunnel Face or Outcrop at Chainage: 7+600 to 7+800

![Tunnel Face at Chainage: 7+624](image1)

![Tunnel Face at Chainage: 7+669](image2)

![Tunnel Face at Chainage: 7+727](image3)

![Tunnel Face at Chainage: 7+735](image4)

Sidewall of Tunnel at Chainage: 7+600 to 7+800
Section 2 of Driskos Tunnel

Chainage: 7+600 to 7+800

Bore: Right

### Properties

<p>| | |</p>
<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
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</tr>
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<td>$\sigma_{ci}$ (UCS)</td>
<td>30-45 MPa</td>
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<tr>
<td>$\sigma_{cm}$</td>
<td>4.2-10.4 MPa</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Lithology</th>
<th>SaSi/SiSa</th>
</tr>
</thead>
</table>

| Rock Mass Category | III(70%)-IV(30%) |

---

**Photo of Tunnel Face or Outcrop at Chainage: 7+600 to 7+800**

**Tunnel Face at Chainage: 7+619**

**Tunnel Face at Chainage: 7+625**

**Tunnel Face at Chainage: 7+652**

**Tunnel Face at Chainage: 7+719**

**Sidewall of Tunnel at Chainage: 7+600 to 7+800**
### Section 2&3 of Driskos Tunnel

**Chainage: 7+800 to 8+000**

**Bore: Left**

<table>
<thead>
<tr>
<th>Properties</th>
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<tbody>
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<td>GSI</td>
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<tr>
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<td>$\sigma_{ci}$ (UCS)</td>
<td>45-60 MPa</td>
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<td>$\sigma_{cm}$</td>
<td>8.1-29.5 MPa</td>
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**Lithology**

<table>
<thead>
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<th>SaSi</th>
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</table>

**Photo of Tunnel Face or Outcrop at Chainage: 7+800 to 8+000**

**Rock Mass Category** II (70%)-III (30%)

**Tunnel Face at Chainage: 7+850**

**Tunnel Face at Chainage: 7+904**

**Tunnel Face at Chainage: 7+966**

**Tunnel Face at Chainage: 7+981**

**Sidewall of Tunnel at Chainage: 7+800 to 8+000**
### Section 2&3 of Driskos Tunnel

**Chainage: 7+800 to 8+000**

**Bore: Right**

<table>
<thead>
<tr>
<th>Properties</th>
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<tbody>
<tr>
<td>GSI</td>
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<tr>
<td>E</td>
<td>5.0-13.8 GPa</td>
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<tr>
<td>$\sigma_{ci}$ (UCS)</td>
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</tr>
<tr>
<td>$\sigma_{cm}$</td>
<td>8.1-29.5 MPa</td>
</tr>
<tr>
<td>Lithology</td>
<td>SaSi</td>
</tr>
<tr>
<td>Rock Mass Category</td>
<td>II(70%)-III(30%)</td>
</tr>
</tbody>
</table>

**Photo of Tunnel Face or Outcrop at Chainage: 7+800 to 8+000**

**Tunnel Face at Chainage: 7+817**

**Tunnel Face at Chainage: 7+891**

**Tunnel Face at Chainage: 7+978**

**Tunnel Face at Chainage: 7+990**

**Sidewall of Tunnel at Chainage: 7+800 to 8+000**
### Section 3 of Driskos Tunnel
**Chainage: 8+000 to 8+200**

**Bore:** Right

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>GSI</td>
<td>60–65(70%)-45–55(30%)</td>
</tr>
<tr>
<td>E</td>
<td>5.0-13.8 GPa</td>
</tr>
<tr>
<td>$\sigma_{ci}$ (UCS)</td>
<td>45-60 MPa</td>
</tr>
<tr>
<td>$\sigma_{cm}$</td>
<td>8.1-29.5 MPa</td>
</tr>
<tr>
<td>Lithology</td>
<td>SaSi</td>
</tr>
<tr>
<td>Rock Mass Category</td>
<td>II(70%)-III(30%)</td>
</tr>
</tbody>
</table>

**Photo of Tunnel Face or Outcrop at Chainage:** 8+000 to 8+200

**Tunnel Face at Chainage:**
- 8+019
- 8+063
- 8+090
- 8+140

**Sidewall of Tunnel at Chainage:** 8+000 to 8+060
### Properties

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<td>(E)</td>
<td>5.0-13.8 GPa</td>
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<tr>
<td>(\sigma_{\text{ci}}) (UCS)</td>
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<tr>
<td>(\sigma_{\text{cm}})</td>
<td>8.1-29.5 MPa</td>
</tr>
<tr>
<td>Lithology</td>
<td>SaSi</td>
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### Rock Mass Category

II(70%)-III(30%)
Section 3&4.1& 4.2 of Driskos Tunnel

Chainage: 8+400 to 8+600

Bore: Right

Properties

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Lithology: SaSi

Rock Mass Category: IV(70%)-V(30%)

Photo of Tunnel Face or Outcrop at Chainage: 8+400 to 8+600

Tunnel Face at Chainage: 8+447

Tunnel Face at Chainage: 8+459

Tunnel Face at Chainage: 8+521

Tunnel Face at Chainage: 8+555

Sidewall of Tunnel at Chainage: 8+400 – 8+460
Section 4.2 & 4.3 of Driskos Tunnel

Chainage: 8+600 to 8+800

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<td>IV(70%)-V(30%)</td>
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Photo of Tunnel Face or Outcrop at Chainage: 8+600 to 8+800

Tunnel Face at Chainage: 8+612

Tunnel Face at Chainage: 8+669

Tunnel Face at Chainage: 8+686

Tunnel Face at Chainage: 8+694

Sidewall of Tunnel at Chainage: 8+600 to 8+660
## Section 4.3 & 4.4 of Driskos Tunnel

**Chainage: 8+800 to 9+000**

**Bore: Right**

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<td>Lithology</td>
<td>SaSi</td>
</tr>
<tr>
<td>Rock Mass Category</td>
<td>IV (70%)–V (30%)</td>
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### Lithology

- Photo of Tunnel Face or Outcrop at Chainage: 8+800 to 9+000
- Tunnel Face at Chainage: 8+817
- Tunnel Face at Chainage: 8+827
- Tunnel Face at Chainage: 8+932
- Tunnel Face at Chainage: 8+969
- Sidewall of Tunnel at Chainage: 8+800 to 8+860
Section 4.5&5 of Driskos Tunnel

Chainage: 9+000 to 9+085

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<td>Sa-An</td>
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<td>Rock Mass Category</td>
<td>II(70%)-III(30%)</td>
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Photo of Tunnel Face or Outcrop at Chainage: 9+000 to 9+085

Tunnel Face at Chainage: 9+005

Tunnel Face at Chainage: 9+084

Tunnel Face at Chainage: 9+097

Tunnel Face at Chainage: 9+132

Sidewall of Tunnel at Chainage: 9+000 to 9+200
### Section 5&6 of Driskos Tunnel

**Chainage:** 9+085 to 9+200  
**Bore:** Left

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**Lithology:** SaSi

**Rock Mass Category:** II(70%)-III(30%)

---

![Photo of Tunnel Face or Outcrop at Chainage: 9+085 to 9+200](image)

**Tunnel Face at Chainage:** 9+005  
**Tunnel Face at Chainage:** 9+084  
**Tunnel Face at Chainage:** 9+097  
**Tunnel Face at Chainage:** 9+132

---

**Sidewall of Tunnel at Chainage:** 9+000 to 9+200
### Section 6 of Driskos Tunnel

**Chainage: 9+200 to 9+400**

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<td>$\sigma_{ci}$ (UCS)</td>
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<td>$\sigma_{cm}$</td>
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**Lithology**

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**Rock Mass Category**

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**Photo of Tunnel Face or Outcrop at Chainage: 9+200 to 9+400**

**Tunnel Face at Chainage: 9+256**

**Sidewall of Tunnel at Chainage: 9+200 to 9+400**

**Tunnel Face at Chainage: 9+275**

**Tunnel Face at Chainage: 9+370**
### Section 6 of Driskos Tunnel

**Chainage: 9+200 to 9+400**

**Bore: Right**

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<tr>
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**Lithology**

SaSi

**Rock Mass Category**

II(70%)-III(30%)

---

**Photo of Tunnel Face or Outcrop at Chainage: 9+200 to 9+400**

**Tunnel Face at Chainage: 9+216**

**Tunnel Face at Chainage: 9+276**

**Tunnel Face at Chainage: 9+311**

**Tunnel Face at Chainage: 9+356**

**Sidewall of Tunnel at Chainage: 9+200 to 9+400**
### Properties

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<td>Sa+SiSa</td>
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</table>

| Rock Mass Category | III         |

### Lithology

- Sa+SiSa

### Bore: left

#### Chainage: 9+400 to 9+600

- Photo of Tunnel Face or Outcrop at Chainage: 9+400 to 9+600
- Tunnel Face at Chainage: 9+410
- Tunnel Face at Chainage: 9+485
- Tunnel Face at Chainage: 9+518
- Tunnel Face at Chainage: 9+547

#### Sidewall of Tunnel at Chainage: 9+400 to 9+600
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<td>E</td>
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**Photo of Tunnel Face or Outcrop at Chainage:**

**Tunnel Face at Chainage:**

**Sidewall of Tunnel at Chainage:**
## Properties

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<td>E</td>
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<td>$\sigma_{ci}$ (UCS)</td>
<td>20-40 MPa</td>
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<tr>
<td>$\sigma_{cm}$</td>
<td>2.8-19.6 MPa</td>
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<tr>
<td>Lithology</td>
<td>SaSi/SiSa+Si</td>
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### Rock Mass Category

- II(70%)-III(30%)
### Section 8 of Driskos Tunnel

**Chainage: 9+600 to 9+800**

**Bore: Right**

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<th>Properties</th>
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<tbody>
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<tr>
<td>E</td>
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<td>(\sigma_{ci}) (UCS)</td>
<td>20-40 MPa</td>
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<tr>
<td>(\sigma_{cm})</td>
<td>2.8-19.6 MPa</td>
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<tr>
<td>Lithology</td>
<td>SaSi/SiSa+Si</td>
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| Rock Mass Category               | II(70%)-III(30%)  |

#### Photo of Tunnel Face or Outcrop at Chainage: 9+600 to 9+800

#### Rock Mass Category

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<td>9+781</td>
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#### Sidewall of Tunnel at Chainage: 9+600 to 9+800

C-21
## Section 8&9&10 of Driskos Tunnel

**Chainage: 9+800 to 10+000**

**Bore: Left**

<table>
<thead>
<tr>
<th>Photo of Tunnel Face or Outcrop at Chainage: 9+800 to 10+000</th>
<th>Properties</th>
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<td></td>
<td>E</td>
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<td>2.5-15 GPa</td>
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<td>2.8-19.6 MPa</td>
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<thead>
<tr>
<th>Tunnel Face at Chainage: 9+828</th>
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### Section 8&9&10 of Driskos Tunnel

**Chainage: 9+800 to 10+000**

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| Rock Mass Category     | II(50%)-III(50%)    |

**Photo of Tunnel Face or Outcrop at Chainage: 9+800 to 10+000**

**Tunnel Face at Chainage:**
- 9+825
- 9+860
- 9+889
- 9+939

**Sidewall of Tunnel at Chainage: 9+800 to 10+000**
**Section 10 of Driskos Tunnel**

**Chainage: 10+000 to 10+100**

**Bore: Left**

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**Lithology**

- **SiSa**

**Rock Mass Category**

- **III**

---

**Photo of Tunnel Face or Outcrop at Chainage: 10+000 to 10+100**

**Tunnel Face at Chainage: 10+063**

**Tunnel Face at Chainage: 10+128**

**Tunnel Face at Chainage: 10+142**

**Tunnel Face at Chainage: 10+167**

**Sidewall of Tunnel at Chainage: 10+000 to 10+200**
### Section 10&11 of Driskos Tunnel

**Chainage: 10+000 to 10+200**

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<td>$\gamma$</td>
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#### Photo of Tunnel Face or Outcrop at Chainage:

- Tunnel Face at Chainage: 10+063
- Tunnel Face at Chainage: 10+128
- Tunnel Face at Chainage: 10+142
- Tunnel Face at Chainage: 10+167

#### Sidewall of Tunnel at Chainage: 10+000 to 10+200
Section 10&11 of Driskos Tunnel  
**Chainage: 10+000 to 10+200**

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<td></td>
<td>$\phi$</td>
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<td></td>
<td>$m_i$</td>
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<td>$\gamma$</td>
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**Photo of Tunnel Face or Outcrop at Chainage:**

**Tunnel Face at Chainage:**

**Sidewall of Tunnel at Chainage:**
### Section 11 of Driskos Tunnel

**Chainage: 10+100 to 10+200**

**Bore: Left**

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<tbody>
<tr>
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<td>$E$</td>
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<tr>
<td>$\sigma_{cm}$</td>
<td>5.4-6.9 MPa</td>
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**Lithology**: Sa+SiSa

**Rock Mass Category**: I/IV-V

**Photo of Tunnel Face or Outcrop at Chainage: 10+100 to 10+200**

- Tunnel Face at Chainage: 10+063
- Tunnel Face at Chainage: 10+092
- Tunnel Face at Chainage: 10+128
- Tunnel Face at Chainage: 10+142
- Tunnel Face at Chainage: 10+167

**Sidewall of Tunnel at Chainage: 10+000 to 10+200**
### Section 11&12 of Driskos Tunnel
**Chainage: 10+200 to 10+400**

**Bore: Left**

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<td>( \sigma_{cm} )</td>
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<td>Fa/SiSa</td>
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<td>Rock Mass Category</td>
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**Photo of Tunnel Face or Outcrop at Chainage:**

- Tunnel Face at Chainage: 10+219
- Tunnel Face at Chainage: 10+228
- Tunnel Face at Chainage: 10+269
- Tunnel Face at Chainage: 10+389

**Sidewall of Tunnel at Chainage: 10+200 to 10+400**
Section 11&12 of Driskos Tunnel  
**Chainage: 10+200 to 10+400**  
**Bore: Right**

<table>
<thead>
<tr>
<th>Photo of Tunnel Face or Outcrop at Chainage:</th>
<th>Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GSI</td>
</tr>
<tr>
<td></td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{ci}$ (UCS)</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{cm}$</td>
</tr>
<tr>
<td></td>
<td>Lithology</td>
</tr>
<tr>
<td></td>
<td>Rock Mass Category</td>
</tr>
</tbody>
</table>

Tunnel Face at Chainage: 10+205  
Tunnel Face at Chainage: 10+253  
Tunnel Face at Chainage: 10+290  
Tunnel Face at Chainage: 10+368  
Sidewall of Tunnel at Chainage: 10+200 to 10+400
Section 12&13 of Driskos Tunnel  
Chainage: 10+400 to 10+600  
Bore: Left

<table>
<thead>
<tr>
<th>Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>25-50</td>
</tr>
<tr>
<td>E</td>
<td>0.65-1.6 GPa</td>
</tr>
<tr>
<td>$\sigma_{ci}$ (UCS)</td>
<td>8~15 MPa</td>
</tr>
<tr>
<td>$\sigma_{cm}$</td>
<td>0.5-6 MPa</td>
</tr>
</tbody>
</table>

Lithology: Si/SiSa  
Rock Mass Category: IV(50%)/V(50%)

Photo of Tunnel Face or Outcrop at Chainage: 10+400 to 10+600

Rock Mass Category

Tunnel Face at Chainage: 10+464  
Tunnel Face at Chainage: 10+519  
Tunnel Face at Chainage: 10+553  
Tunnel Face at Chainage: 10+596

Sidewall of Tunnel at Chainage: 10+400 to 10+600
# Section 12&13 of Driskos Tunnel

**Chainage:** 10+400 to 10+600

**Bore:** Right

### Properties

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>GSI</strong></td>
<td>25-50</td>
</tr>
<tr>
<td><strong>E</strong></td>
<td>0.65-1.6 GPa</td>
</tr>
<tr>
<td><strong>(\sigma_{ci}) (UCS)</strong></td>
<td>8~15 MPa</td>
</tr>
<tr>
<td><strong>(\sigma_{cm})</strong></td>
<td>0.5-6 MPa</td>
</tr>
<tr>
<td><strong>Lithology</strong></td>
<td>Si/SiSa</td>
</tr>
<tr>
<td><strong>Rock Mass Category</strong></td>
<td>IV(50%)/V(50%)</td>
</tr>
</tbody>
</table>

### Images

- **Photo of Tunnel Face or Outcrop at Chainage: 10+400 to 10+600**
- **Tunnel Face at Chainage:10+409**
- **Tunnel Face at Chainage:10+453**
- **Tunnel Face at Chainage:10+512**
- **Tunnel Face at Chainage:10+528**
- **Sidewall of Tunnel at Chainage: 10+400 to 10+600**

---

C-31
### Section 13&14 of Driskos Tunnel

**Chainage: 10+600 to end**

**Bore:**

<table>
<thead>
<tr>
<th>Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>45-50</td>
</tr>
<tr>
<td><strong>E</strong></td>
<td>4.7-6.7 GPa</td>
</tr>
<tr>
<td><strong>σ_{c1} (UCS)</strong></td>
<td>40~45 MPa</td>
</tr>
<tr>
<td><strong>σ_{cm}</strong></td>
<td>7.9-10.4 MPa</td>
</tr>
<tr>
<td><strong>Lithology</strong></td>
<td>SaSi/Sa+SiSa</td>
</tr>
</tbody>
</table>

**Rock Mass Category**

- **III**

---

**Photo of Tunnel Face or Outcrop at Chainage: 10+600 to end**

**Tunnel Face at Chainage: 10+638**

**Tunnel Face at Chainage: 10+668**

**Tunnel Face at Chainage: 10+697**

**Tunnel Face at Chainage: 10+712**

**Sidewall of Tunnel at Chainage: 10+600 to end**
Appendix D

Geological Strength Index (GSI)
Appendix D

Geological Strength Index (GSI)

1.0 Introduction

This Appendix contains the Geological Strength Index (GSI) tables (Table D1 and Table D2) that were used as part of this investigation. Note that only Table D1 was available to the designers at Driskos and therefore was the one used throughout this investigation.

This index is based upon an assessment of the structure, lithology and condition of discontinuity surfaces in the rock mass. The GSI is estimated through visual examination of the rock mass exposed in tunnel faces. The GSI value of a rock mass is incorporated into calculations in order to determine the reduction in the strength of the rock mass as compared with the strength of the intact rock components.

Also included are geo-referenced photos of outcrops as they relate to the Driskos Twin Tunnel site. These photos allow the reader to gain an appreciation of the GSI values associated with a particular outcrop. It should also be noted that GSI values commonly improve with depth due to confining pressures.

All of the GSI values that are found within this Appendix were determined by the author through multiple site visits to the Driskos Twin Tunnels site. These values were then used within the numerical analysis portion of the thesis.
### Table D1. GSI estimates for heterogeneous rock masses such as Flysch (Marinos, P. and Hoek, E., 2000)

<table>
<thead>
<tr>
<th>Surface Conditions of Discontinuities (Predominantly bedding planes)</th>
<th>Very Good</th>
<th>Good</th>
<th>Fair</th>
<th>Poor</th>
<th>Very Poor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very rough, fresh unweathered surfaces</td>
<td>70</td>
<td>60</td>
<td>50</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td>Roughly weathered surfaces</td>
<td>60</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
</tr>
<tr>
<td>Smooth, moderately weathered surfaces</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>Very smooth, occasionally slickensided or fillings with compact fragments</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>10</td>
<td>5</td>
</tr>
<tr>
<td>Very smooth, slickensided or highly weathered surfaces</td>
<td>30</td>
<td>20</td>
<td>10</td>
<td>5</td>
<td>2</td>
</tr>
</tbody>
</table>

**COMPOSITION AND STRUCTURE**

- **A.** Thick bedded, very blocky sandstone
  
  The effect of pelitic coatings on the bedding planes is minimized by the confinement of the rock mass. In shallow tunnels or slopes these bedding planes may cause structurally controlled instability.

- **B.** Sandstone with thin interlayers of siltstone

- **C.** Sandstone and siltstone in similar amounts

- **D.** Silty sandstone or siltstone with sandstone layers

- **E.** Weak siltstone or clayey shale with sandstone layers

- **F.** Tectonically deformed, intensively folded/faulted, sheared clayey shale or siltstone, with broken and deformed sandstone layers forming an almost chaotic structure

- **G.** Undisturbed silt or clayey shale with or without a few very thin sandstone layers

- **H.** Tectonically deformed silty or clayey shale forming a chaotic structure with pockets of clay. Thin layers of sandstone are transformed into small rock pieces.

---

: Means deformation after tectonic disturbance
**Table D2.** Most Recent, updated GSI estimates for heterogeneous rock masses such as Flysch (Marinos, V., 2007)
Figure D.1. Flysch Formation (relevant information inset)

Figure D.2. Flysch Formation (relevant information inset)
Figure D.3. Flysch Formation (relevant information inset)

Figure D.4. Flysch Formation (relevant information inset)
Figure D.5. Flysch Formation (relevant information inset)

Figure D.6. Flysch Formation (relevant information inset)
Figure D.7. Flysch Formation (relevant information inset)

Figure D.8. Flysch Formation (relevant information inset)
SaSi
GSI: 45-50
Location: Top of NE Entrance
39°41'47.15"N
20°58'06.68"E
Elevation: 714 m

Figure D.9. Flysch Formation (relevant information inset)

SaSi
GSI: 35-45
Location: Top of NE Entrance
39°41'47.15"N
20°58'06.68"E
Elevation: 714 m

Figure D.10. Flysch Formation (relevant information inset)
SiSa
GSI: 35-45
Location: Top of NE Entrance
39°41'47.15"N
20°58'06.68"E
Elevation: 714 m

Figure D.11.  Flysch Formation (relevant information inset)

SiSa
GSI: 30-35
Location: Top of NE Entrance
39°41'46.41"N
20°58'03.48"E
Elevation: 694 m

Figure D.12.  Flysch Formation (relevant information inset)
Figure D.13. Flysch Formation (relevant information inset)

Figure D.14. Flysch Formation (relevant information inset)
Figure D.15. Flysch Formation (relevant information inset)

Figure D.16. Flysch Formation (relevant information inset)
Figure D.17. Flysch Formation (relevant information inset)

Figure D.18. Flysch Formation (relevant information inset)
Figure D.19.  Flysch Formation (relevant information inset)

Figure D.20.  Flysch Formation (relevant information inset)
Day 3

SiSa
GSI: 33-37
Location: West of SW Entrance on way to Southern Portals
39°39’26.07”N  20°57’04.29”E
Elevation: 613 m

Figure D.21. Flysch Formation (relevant information inset)

Figure D.22. Flysch Formation (relevant information inset)
SaSi
GSI: 35-40
Location: West of SW Entrance on way to Southern Portals
39°39′25.56″N 20°57′19.62″E
Elevation: 613 m

Figure D.23. Flysch Formation (relevant information inset)

SaSi
GSI: 40-50
Location: West of SW Entrance on way to Southern Portals
39°41′44.00″N 20°58′06.60″E
Elevation: 698 m

Figure D.24. Flysch Formation (relevant information inset)
Figure D.25. Flysch Formation (relevant information inset)

Figure D.26. Flysch Formation (relevant information inset)
Transition Zone SiSa to SaSi
GSI: 38-47
Location: Over Southern Portals traveling North
39°39’26.59”N  20°57’35.58”E
Elevation: 709 m

Figure D.27. Flysch Formation (relevant information inset)

Transition Zone SiSa to SaSi
GSI: lower 35-40 upper 50
Location: Over Southern Portals traveling North
39°39’28.71”N  20°57’38.20”E
Elevation: 735 m

Figure D.28. Flysch Formation (relevant information inset)
SiSa
GSI: 35-40
Location: Over Southern Portals traveling North
39°39’28.71”N  20°57’38.20”E
Elevation: 735 m

Figure D.29.  Flysch Formation (relevant information inset)

SaSi
GSI: 25-32
Location: Over Southern Portals traveling North Valley Intersection (1600 m)
39°40’03.25”N  20°58’23.27”E
Elevation: 955 m

Figure D.30.  Flysch Formation (relevant information inset)
SaSi
GSI: 25-32
Location: Over Southern Portals traveling North Valley Intersection (1600 m)
39°40'03.25"N  20°58'23.27"E
Elevation: 955 m

Figure D.31. Flysch Formation (relevant information inset)

GSI: 30
Location: Over Southern Portals traveling North Valley Intersection (1600 m)
39°40'01.93"N  20°58'23.81"E
Elevation: 797 m

Figure D.32. Flysch Formation (relevant information inset)
SiSa
GSI: 30-35
Location: Over Southern Portals traveling North on Hwy (2800 m)
39°40'49.90"N 20°57'58.97"E
Elevation: 824 m

Figure D.33. Flysch Formation (relevant information inset)

Conglomerate
Location: Over Southern Portals traveling North on Hwy (2300 m)
39°40'32.90"N 20°57'39.19"E
Elevation: 893 m

Figure D.34. Conglomerate (relevant information inset)
Appendix E

Tunnel Monitoring Data from Driskos Twin Tunnel Construction
LEFT BORE - CHAINAGE 8+657

HORIZONTAL MOVEMENT PARALLEL TO THE DIRECTION OF BORING

Date (Time) vs. Displacement (mm)

-100 0 100 200 300 400 500

Distance from the Face (m)

Displacement (mm)

LEFT BORE - CHAINAGE 8+657

HORIZONTAL MOVEMENT PERPENDICULAR TO THE DIRECTION OF BORING

Date (Time) vs. Displacement (mm)

-100 0 100 200 300 400 500

Distance from the Face (m)

Displacement (mm)

LEFT BORE - CHAINAGE 8+657

RELATIVE CONVERGENCE

Date (Time) vs. Displacement (mm)

-100 0 100 200 300 400 500

Distance from the Face (m)

Displacement (mm)

LEFT BORE - CHAINAGE 8+657

VERTICAL DISPLACEMENT

Date (Time) vs. Displacement (mm)

-100 0 100 200 300 400 500

Distance from the Face (m)

Displacement (mm)
LEFT BORE - CHAINAGE 8+674
HORIZONTAL MOVEMENT PARALLEL TO THE DIRECTION OF BORING

HORIZONTAL MOVEMENT PERPENDICULAR TO THE DIRECTION OF BORING

RELATIVE CONVERGENCE

VERTICAL DISPLACEMENT
Appendix F

Conference Paper

ROCK MECHANICS IN HIGHLY DEFORMED GROUND: TUNNELLING IN GREECE
Nicholas Vlachopoulos, MAsc., P.Eng and Dr. Mark S. Diederichs, PhD., PEng.
GeoEngineering Centre at Queen’s - RMC, Queen’s University, Kingston, Ontario, Canada

ABSTRACT
Greece is a country born of intense tectonic processes. Highly deformed and altered sediments and low grade metamorphic rock masses dominate the near surface environment creating a variety of technical challenges for tunnelling and slope stability related to modern infrastructure. This paper gives an overview of the key technical issues associated with the Egnatia Odos Highway Tunnelling Projects in Northern Greece, including: the prediction of tunnel squeezing in weak heterogeneous and often chaotic rock masses; accurate rock mass strength estimations of these materials; dealing with geomorphologic peculiarities such as limestone flysch materials, phyllitic schists, shear zones in hard rocks, weathered zones in crystalline rocks, etc.; tunnelling in karstic terrain; material sampling for engineering purposes; portal stability problems and support design in heavily jointed, heavily fractured or brecciated rock masses.

1. INTRODUCTION
The Egnatia Odos Highway is a massive construction project that is currently under construction in Northern Greece. The project is an upgrade of the current highway across the north of Greece. The old alignment follows the ancient Egnatia Road, an 800km route constructed by the Romans for military purposes in the 2nd century B.C.

Upon completion, the new 680 km Egnatia motorway (Figure 1) will have a total of 77 twin road tunnels with an overall combined single carriageway length of 98 km. 60 of these tunnels are bored or blasted tunnels. The remainder are cut-and-cover (Egnatia Odos AE 2001). As such, over 7% of the overall highway will be carried through tunnels, incurring 30% of the total estimated construction cost. The estimated overall budget of the project is $3.2 billion (US), 60% being funded by the European union and 40% by the Greek government (Silva et al.,2002).

The Egnatia Highway is being constructed in order to open up new, modern and safe roads connecting the countries of the European Union, the Balkans and the East. The motorway was designed to the specifications of the Trans-European network. It traverses the entire width of Greece, crossing almost perpendicularly the main geotectonic units (as will be discussed in the Section 2). Thus, many geotechnically unfavourable characteristics were encountered when deciding how to align the Egnatia Highway.

Also affecting the alignment were various environmentally sensitive areas and locations of high archaeological interest. The great variety of geological/geotechnical situations imposes the need for different approaches in designing the various components of the highway.

A geotechnical rock mass model must be defined in order to choose the appropriate geotechnical parameters for the design of cuts, embankments and tunnels. This paper will outline the main geological conditions that are being encountered by the construction of the Egnatia Motorway and introduce the main geotechnical mass models that were (and are being) developed for this project.

2. GEOLOGICAL ENVIRONMENT
2.1 Geologic History
The overall geology of Greece and that of the Alpine region has traditionally been described in terms of isopic zones and massifs. These zones are groups of widespread rocks that have shared a common history, both in the ancient environments of deposition of sediments (Greece was a shallow, oxygen rich sea during most of the Triassic, Jurassic, Cretaceous and later) and their faulting and folding. The massifs of metamorphic and plutonic rocks are more resistant to folding and faulting than adjacent sediments. .
With the alpine orogeny (the formation of the Alps), limestone was lifted all over Greece and folded. Therefore, approximately two thirds of the area of Greece is covered with limestone as well as many other karst phenomena. Many heterogeneous rock masses, such as flysch, are also abundant. Greece's geology is still very active as it is located on a converging plate rim between the European and African plate (Higgins et al., 1996). Figure 2 depicts the tectonic zones and massifs of the Aegean region.

2.2 Zones spanned by the Egnatia Highway

The motorway passes through almost all of the geologic history of Greece by crossing nine major geotectonic units. Due to these geologic units, tunnelling works are to be executed in a variety of rock mass conditions ranging from very good quality rock masses such as massive limestones or granites/gneisses to very poor rock masses such as clay shales/mudstones. Each geologic unit exhibits different particularities in terms of weak rock masses and the potential for unstable arrangements of rocks. From west to east, the Egnatia Odos can be subdivided into the following five parts based on the tectonic regimes and geotechnical issues involved in the highway construction. These 5 zones are identified in Figure 3 and described as follows.
Region 1.
From Igoumenitsa to Metsovitikos River – Ionian geotectonic unit. This part is characterised by flysch and alternations of various carbonate formations (mainly limestone) with very limited occurrence of schist and local occurrences of anhydrite and gypsum. Big faults, large overthrusts and folded rocks are also present.

Region 2.
a) From Metsovitikos River to Metsovo tunnel – Pindos geotectonic unit. This area consists of various forms of flysch and is characterized by intense folding, heavily sheared with numerous overthrusts. The massive degree of tectonic deformation at some locals degrades the quality of the rock mass, and;
b) from Metsovo tunnel to Panagia region – Nappe of Pindos ophiolites. The predominant rock mass in this area is composed of ophiolites. These structures exhibit much heterogeneity of weathering and occurrence of shear zones. There is also a presence of weak flysch.

Region 3.
From Panagia to Siasta. This region consists of thick-bedded conglomerates, sandstones and marls (molassic formations). There is not any dramatic decrease in geotechnical qualities due to the absence of tectonic shearing in this region.

Region 4.
a) From Siatista to Lefkopetra - Pelagonian geotectonic units. This area contains predominantly hard rocks such as marbles, gneisses and granites. Fault zones are very localized; and
b) From Lefkopetra to Veria – Axios to Almopia geotectonic units. This area is characterized by the presence of phyllites, limestones and ophiolites broken by sheared zones and overthrusts.

Region 5.
a) From Aliakmon River to Axios River flood plane to Thessaloniki region. This entire region consists of alluvial fill that exhibits insufficient natural compaction and;
b) Section east of Thessaloniki to eastern border. Within this region, the Egnatia highway passes through hard crystalline marbles, gneisses, granites (locally weathered and locally crosscut by faults with sheared zones within the rock mass) and areas of marls and sandstones.

2.3. Weak Rock Masses

As described by Marinos and Hoek (2001), the dominant weak rock masses along the Egnatia Motorway, often in chaotic and heterogeneous forms are:
Various forms of flysch (depending on the proportion of weak siltstone-clayey members and the tectonic deformation);
Ophiolites (depending on the weathering and tectonic deformation);
Phyllitic schists (depending on tectonic pre-shearing and weathering);
Sheared zones in hard rocks; and
Weathered zones in gneisses, granites and schists.

A selection of the major tunnels located on the Egnatia motorway and associated geological formations are listed in Table 1.

Table 1. Major Tunnels on the Egnatia Motorway
(Egnatia Odos A.E., 2001)

<table>
<thead>
<tr>
<th>Region</th>
<th>Tunnel Name</th>
<th>Length (m)</th>
<th>Construction method</th>
<th>Geological formations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epirus</td>
<td>Odrin</td>
<td>3,360</td>
<td>Tunneling</td>
<td>Limestone</td>
</tr>
<tr>
<td>Epirus</td>
<td>Drosos</td>
<td>4,690</td>
<td>Tunneling</td>
<td>Flysch formations</td>
</tr>
<tr>
<td>Epirus</td>
<td>Tági</td>
<td>2,130</td>
<td>Tunneling</td>
<td>Flysch formations</td>
</tr>
<tr>
<td>Epirus</td>
<td>Krimnos</td>
<td>1,080</td>
<td>Tunneling</td>
<td>Flysch formations</td>
</tr>
<tr>
<td>Epirus</td>
<td>Vrsi Anilo</td>
<td>2,135</td>
<td>Tunneling</td>
<td>Flysch formations</td>
</tr>
<tr>
<td>Epirus</td>
<td>Metsovo</td>
<td>3,650</td>
<td>Tunneling</td>
<td>Ophiolite / Flysch</td>
</tr>
<tr>
<td>Thyessaly</td>
<td>Panagia</td>
<td>2,700</td>
<td>Tunneling</td>
<td>Gabbros / Ophiolite / Hornstone</td>
</tr>
<tr>
<td>Western Macedonia</td>
<td>Syrtio</td>
<td>1,500</td>
<td>Tunneling</td>
<td>Ophiolite / Molassic</td>
</tr>
<tr>
<td>Western Macedonia</td>
<td>Koiloma</td>
<td>1,080</td>
<td>Tunneling</td>
<td>Molassic</td>
</tr>
<tr>
<td>Western Macedonia</td>
<td>Verojë</td>
<td>1,210</td>
<td>Tunneling</td>
<td>Molassic</td>
</tr>
<tr>
<td>Central Macedonia</td>
<td>STO</td>
<td>2,240</td>
<td>Tunneling</td>
<td>Marls / Granites</td>
</tr>
<tr>
<td>Central Macedonia</td>
<td>Pangaio</td>
<td>1,650</td>
<td>Tunneling</td>
<td>Marls</td>
</tr>
</tbody>
</table>

The geologic/geotectonic setting described in Section 2.2 results in very difficult engineering conditions with weak rock masses that requires site specific investigations and the use of sophisticated design methods. The design of underground excavations in these materials (primarily flysch) requires knowledge of the mechanical properties of the rock masses in which these excavations are carried out. The next section will deal with estimating the mechanical properties of such weak heterogeneous rock masses that are common throughout Greece.

3. ESTIMATION OF ROCK MASS PROPERTIES

The most widely used criteria for estimating rock mass properties is that presented by Hoek and Brown (1997). In applying the Hoek-Brown criterion, three parameters are required in order to estimate the strength and deformation properties. These include:
1. The uniaxial compressive strength ($\sigma_u$) of the "intact" rock elements within the rock mass;
2. A constant $m_i$ that define the frictional characteristics of the rock elements; and
3. The Geological Strength (GSI) that relates the properties of intact rock elements to those of the overall rock mass. The GSI was introduced by Hoek, Kaiser and Bawden (1995), Hoek and Brown (1997) and extended by Hoek, Marinos and Benissi (1998) and Hoek et al. (2002).

In tunnelling through weak heterogeneous rock masses such as those in Figure 4, it is important to obtain reliable strength estimates of this material in order to predict potential tunnelling problems as early as possible in the design process.
Figure 4. Examples of (top) phyllite from region 4, (mid) deformed marl from region 3 and (bottom) heavily sheared siltstone flysch from region 2.

The most important component of the Hoek-Brown system for rock mass strength determination is the process of reducing the material constants \( m \) and \( \sigma_0 \) from their “laboratory” values to realistic in situ values. Each of the three parameters required to estimate the strength of a rock mass will be introduced in the following sections.

3.1 Uniaxial Compressive Strength of Intact Rock

The difficulty of obtaining an “intact” core sample from a heterogeneous rock mass poses a challenge in obtaining samples for laboratory testing. Samples obtained from rock masses as shown in Figure 4, will most definitely contain discontinuities in the form of bedding and joints or schistosity planes. Samples will likely also contain several component rock types. As a result, any laboratory tests carried out on core samples will be more representative of the rock mass rather than the intact rock components. Using these results will give unrealistically low values for rock mass strength (Hoek and Marinos, 2000).

In order to obtain a more accurate strength reading, Marinos and Hoek (2001) suggested using the Point Load Test on samples. The specimens used for testing in this case are irregular pieces or pieces broken from the core. The point load index \( I_\text{p} \) can be calculated using:

\[ I_\text{p} = \frac{P}{D^2} \]  

(1)

where, \( P \) is the load on the points; and \( D \) is the distance between the points.

The uniaxial compressive strength of the intact rock \( (\sigma_0) \) samples can be reasonably estimated by multiplying the point load index \( (I_\text{p}) \) by 24. Where it is not possible to obtain samples for Point Load Testing, a qualitative description (Table 2) of the rock material is the only alternative in order to estimate the uniaxial compressive strength.

Table 2. Field estimates of uniaxial compressive strength of intact rock.

<table>
<thead>
<tr>
<th>Grade*</th>
<th>Term</th>
<th>Uniaxial Comp. Strength (MPa)</th>
<th>Field estimate of strength</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>R6</td>
<td>Extremely Strong</td>
<td>&gt; 250</td>
<td>Specimen can only be chipped with a geological hammer</td>
<td>Fresh basalt, chert, diabase, gneiss, granite, quartzite</td>
</tr>
<tr>
<td>R5</td>
<td>Very strong</td>
<td>100 - 250</td>
<td>Specimen requires many blows of a geological hammer to fracture it</td>
<td>Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite, phyllite, talc</td>
</tr>
<tr>
<td>R4</td>
<td>Strong</td>
<td>50 - 100</td>
<td>Specimen requires more than one blow of a geological hammer to fracture it</td>
<td>Limestone, marble, sandstone, schist</td>
</tr>
<tr>
<td>R3</td>
<td>Medium strong</td>
<td>25 - 50</td>
<td>Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer</td>
<td>Concrete, phyllite, schist, siltstone</td>
</tr>
<tr>
<td>R2</td>
<td>Weak</td>
<td>5 - 25</td>
<td>Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer</td>
<td>Chalk, claystone, potash marl, siltstone, shale, rock salt,</td>
</tr>
<tr>
<td>R1</td>
<td>Very weak</td>
<td>1 - 5</td>
<td>Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife</td>
<td>Highly weathered or altered rock, shale</td>
</tr>
<tr>
<td>R0</td>
<td>Extremely weak</td>
<td>0.25 - 1</td>
<td>Indented by thumbnail</td>
<td>Stiff fault gouge</td>
</tr>
</tbody>
</table>

*Grade classifications are based on the nature and quality of the rock material.
3.2 Hoek-Brown Constant $m_i$

This parameter has a significant influence on the strength characteristics of a rock mass. The constant $m_i$ depends on the frictional characteristics of the component minerals of an intact rock sample. This parameter can only be determined by triaxial testing of core samples or estimated by Hoek and Brown's (1997) qualitative description of the rock material.

3.3 Geological Strength Index (GSI)

This index is based upon an assessment of the structure, lithology and condition of discontinuity surfaces in the rock mass. The GSI is estimated through visual examination of the rock mass exposed in tunnel faces. The GSI value of a rock mass is incorporated into calculations in order to determine the reduction in the strength of the rock mass as compared with the strength of the intact rock components.

A table developed by Hoek and Marinos (2000) can be used to estimate the GSI value for typical jointed rock masses. Due to the numerous engineering projects under construction in heterogeneous rock masses, attempts have been made in order to provide better and enhanced engineering geology tools. To this end, Marinos and Hoek (2000) also developed a GSI table specifically for heterogeneous rock masses such as flysch and is shown in Table 3.

3.4 Flysch

Flysch consists of variations of clastic sediments that are associated with orogenesis. It is characterized by repeating alternations of sandstone and fine grained (siltstones, silty shales and clayey shales) layers. The overall thickness of flysch is very large. The rock mass consists of frequent bedding discontinuities and is affected by reverse (and consequent normal) faults and thrusts. These factors combine to degrade the geotechnical quality of the heterogeneous flysch rock mass. Therefore, large (hundred to a few thousand meters) sheared or even chaotic rock masses can be found throughout northern Greece. For heterogeneous rock masses such as flysch, it is necessary to consider the selection of $m_i$ and $\sigma_{ci}$.

Due to a presence of weak and strong materials within these rock masses, Marinos and Hoek (2000) suggested using a weighted average of the intact strength properties of the strong and weak layers. Their suggested values are listed in Table 4.

Table 3: GSI estimates for heterogeneous rock masses such as flysch (Marinos and Hoek 2000)
Table 4. Suggested proportions of values of $\sigma_c$ and $m_i$ for each rock type to be included in calculations for rock mass property determination for flysch. (Marinos and Hoek 2000)

<table>
<thead>
<tr>
<th>Flysch type</th>
<th>Use weighted average of components after adjusting sandstone values:</th>
</tr>
</thead>
<tbody>
<tr>
<td>A and B</td>
<td>Use values for sandstone (ss) beds</td>
</tr>
<tr>
<td>C</td>
<td>Reduce ss by 20%, full values for siltstone</td>
</tr>
<tr>
<td>D</td>
<td>Reduce ss by 40%, full values for siltstone</td>
</tr>
<tr>
<td>E</td>
<td>Reduce ss by 40%, full values for siltstone</td>
</tr>
<tr>
<td>F</td>
<td>Reduce ss by 60%, full values for siltstone</td>
</tr>
<tr>
<td>G</td>
<td>Use values for siltstone or shale</td>
</tr>
<tr>
<td>H</td>
<td>Use values for siltstone or shale</td>
</tr>
</tbody>
</table>

3.5 Mechanical Properties of Rock Mass

Once the parameters $m_i$, $\sigma_c$, and GSI have been defined, the next step is to estimate the mechanical properties of the rock mass. These estimates can be determined in accordance with the procedure defined by Hoek and Brown (1997) and will not be repeated here. For tunnels at depths greater than 30 m, the rock mass surrounding the tunnel is confined and its properties are calculated on the basis of a minor principal stress or confining pressure of $0<\sigma_3<0.25\sigma_c$. For shallow tunnels, a minor principle stress range of $0<\sigma_3<\sigma_v$ is used, where $\sigma_v = \text{depth} \times \text{unit weight of rock mass}$.

4. TUNNEL SQUEEZING

When tunnelling through heterogeneous rock masses the potential for tunnel squeezing must be accounted for. An estimation of potential tunnel squeezing can be appreciated with the incorporation of the estimated mechanical properties of the rock mass. A useful parameter for evaluating potential tunnel squeezing problems is the uniaxial compressive strength of the rock mass ($\sigma_{cm}$). It can be calculated using the rock mass strength parameters of $\sigma_c$, $m_i$, and GSI:

$$\sigma_{cm}=(0.0034m_i^{0.6})\sigma_c(1.029+0.025e^{(-0.1m_i)})^GSI$$  \hspace{1cm} (2)

The ratio of the uniaxial compressive strength $\sigma_{cm}$ of the rock to the in situ stress $p_0$ can be used as an indicator of potential tunnel squeezing problems (Hoek, 1999). This ratio can be plotted against the percentage strain ($\varepsilon = 100 \times \text{the ratio of the tunnel closure to tunnel diameter}$) of the tunnel in order to develop the approximate relationship between strain and the degree of difficulty associated with tunnelling through squeezing rock.

$$\varepsilon\% = \alpha(1 - p_i/p_0)\frac{\sigma_{cm}}{p_0}^{-(3p_i/p_0+1)/(3.8p_i/p_0+0.54)}$$  \hspace{1cm} (3)

where $\varepsilon$ is the normalized strain of the tunnel walls and $\alpha = 0.15$. For face "strain" use $\alpha = 0.1$ (compare to simple axisymmetric FEM model in Figure 5). $p_i$ in these equations is the support pressure or ultimate distributed capacity mobilized in the support system (acting radial to the tunnel and perpendicular to the face). $p_0$ represents an average stress level (hydrostatic equivalent).

![Figure 5: Yield related closure (no gravity) of two unsupported tunnels at 300m depth: axisymmetric FEM analysis (grid distortion x 10).](image)

Few problems are expected below a tunnel strain, et of 1, normally mandating basic support such as shotcrete and rockbolts. Support components are layered onto this basic system as anticipated strain increases. For strain values between 1 and 2.5, typical support design would add light steel sets or lattice girders to the shotcrete and bolts of the previous case. These steel sets become heavy reinforcement above $\varepsilon =2.5$. In addition some buttressing or pre-reinforcement (doweling) of the face may be required. Above a normalized strain of 5, heavy umbrella support is required ahead of the face (forepoling). Face buttressing and staged excavation may be mandatory. Extreme squeezing and very difficult tunnelling conditions are expected at values of et in excess of 10 and yielding support elements (sliding joints that maintain support pressure over larger deformations) are required.

A typical approach would involve the development of a number of typical cross sections for support design. Each section would be related to an anticipated magnitude of strain. A number of typical support system configurations in use along the Egnatia project are shown in Figure 6 and Figure 7.

All of the Egnatia tunnels will ultimately be lined with a fully formed concrete lining (Figure 8). This lining is designed to resist ultimate creep pressures and the restoration of hydraulic head.
The forgoing discussion is really aimed at the initial support systems installed at or in advance of the tunnel face. These systems serve to retain the rockmass integrity and provide all of the short term support and permit the ultimate installation of the final lining.

5. FURTHER RESEARCH

This preliminary assessment method must be considered inadequate for final design purposes. Where significant potential squeezing problems have been identified, the tunnel should be subjected to numerical analysis techniques. Even with such sophisticated tools as 3D finite element or finite difference models, the problem remains to define the constitutive behaviour (peak and post yield) for these very complex materials. In addition, the process of modelling near-face rock-support interaction is non-trivial. The mechanisms of rock-support interaction related to forepoling and much of the face or near-face reinforcement system (Figure 8) are poorly understood at this time. Design decisions such as those discussed in this paper are currently possible but a true optimized and mechanics-based design process for the forepole umbrella, for example, is not at present fully developed.
The impact of deformations and instabilities ahead of the tunnel face on the subsequent stability of the overall tunnel (both short and long term) is a key mechanistic problem requiring further study (Figure 9).

In collaboration with the National Technical University of Athens, this current research will be utilize a databank, of in situ strength parameters, stresses, measured strains, failure documentation and operational observations etc. along the Egnatia project, to provide a framework for the development of and calibration/validation of constitutive models for these composite and tectonically deformed rockmasses. In addition the database analysis will aid in the optimization of rock-support interaction models for face restraint.

These models will be incorporated into state of the art continuum and discontinuum numerical codes. This modeling work will establish the appropriate instability mechanics for the near-face environment in a classified suite of neo-tectonic rockmasses and provide a more fundamental understanding of the rock support interactions at work in these environments.

Guidance will be provided for the use of conventional design tools and numerical codes for optimization of these support systems.

ACKNOWLEDGEMENTS

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Appendix G

Conference Paper

1. INTRODUCTION

The Egnatia Odos Highway is a 680 km modern motorway that is currently under construction in Northern Greece. Egnatia Odos is part of the Trans-European road network of highways within Europe and follows the ancient Egnatia Road, constructed by the Romans in the 2nd century B.C.

The new 680 km Egnatia motorway (Figure 1) is a closed, dual motorway. Upon completion, the route will have a total of 73 twin road tunnels with an overall combined single carriageway length of 98 km. Sixty of these tunnels are bored or blasted tunnels. The remainder are cut-and-cover (Egnatia Odos AE 2001). Over 7% of the highway will be carried through tunnels, incurring 30% of the total estimated construction cost. The estimated overall budget of the project has risen from US $3.2 billion (Silva et al. 2002, Sperelakis et al. 2003) to $7.7 billion (Egnatia Odos AE 2006), shared by the European union and by the Greek government.

The Egnatia Highway traverses the entire width of Northern Greece, crossing almost perpendicularly the main geotectonic units. Many geotechnically unfavourable characteristics were encountered along the alignment of the Egnatia Highway.

Figure 1. Alignment of Egnatia Highway in Northern Greece and spatial setting of the major geological zones (modified after Egnatia Odos AE 2001).

2. GEOLOGICAL ENVIRONMENT

2.1 Geologic History

The geology Greece’s Alpine region has been divided into massifs and isopic zones of deformed sediments and volcanics. The latter are groups of lithologically and structurally similar rocks. Greece was a shallow, oxygen rich sea during most of the...
Triassic, Jurassic, Cretaceous with tectonic faulting and folding taking place subsequently. Massifs are metamorphic and plutonic rocks are more resistant to folding and faulting than adjacent sediments. Approximately two thirds of Greece is covered with tectonized limestone. Heterogeneous rock masses, such as flysch (a tectonically reworked clastic mix), are also abundant. Greece’s geology is still very active as it is located on a converging plate rim between the European and African plate (Higgins et al. 1996). Figure 1 summarizes the tectonic zones and massifs along the Egnatia corridor.

Each zone presents unique engineering geology challenges for construction and tunnelling (Vlachopolous & Diederichs 2003). Region 1 is a flysch with alternations of various carbonate formations and with limited occurrence of schist as well as anhydrite and gypsum. Large faults, large overthrusts and folded rocks are also present. Region 2 consists of variable and intensely folded and sheared flysch with resultant low quality engineering rockmasses the weakest being the Ionian Geotectonic Unit). Weathered and sheared ophiolites (ancient sea floor volcanics) are also present. Region 3 consists of thick-bedded mollasic conglomerates, sandstones and marls. Due to the absence of tectonic shearing rockmass quality is generally good. Region 4 contains hard rocks such as marbles, gneisses and granites with local fault zones. Sub-sections of this area are also characterized by the presence of phyllites, limestones and ophiolites broken by sheared zones and overthrusts. Region 5 consists of uncompacted alluvium on top of crystalline marbles, gneisses, faulted and areas of marls and sandstones.

3. ESTIMATION OF ROCK MASS PROPERTIES

The Hoek and Brown criterion is the primary characterization tool on this project. Three parameters are required in order to estimate the strength and deformation properties: the uniaxial compressive strength \( \sigma_{ci} \) of the “intact” rock elements; a constant \( m_i \) that define the frictional characteristics of the intact rock; and the Geological Strength Index (GSI) that relates properties of intact rock elements to those of the rock mass. GSI was introduced by Hoek et al. (1995), Hoek & Brown (1997) and extended by Hoek et al. (1998) and Hoek et al. (2002). New correlations to other schemes are proposed by Tzamosa & Sofianos (2007).

In tunnelling through weak heterogeneous rock masses such as those in Figure 2, it is important to obtain reliable strength estimates of this material in order to predict potential tunnelling problems as early as possible in the design process.

3.1 Uniaxial Compressive Strength of Intact Rock

The difficulty of obtaining an “intact” core sample from a heterogeneous rock mass poses a challenge in obtaining samples for laboratory testing. Samples obtained from rock masses as shown in Figure 2, will most definitely contain discontinuities in the form of bedding and joints or schistosity planes. Samples will likely also contain several component rock types. As a result, any laboratory tests carried out on core samples will be more representative of the rock mass rather than the intact rock components. These results will give unrealistically low values for rock mass strength (Hoek & Marinos, 2000).

In order to obtain a more accurate strength reading, it is possible to use the Point Load Test (Franklin 1985, Thuro & Plinninger 2001) on irregular samples. The specimens used for testing in this case are hand samples or pieces broken from the core. The point load index \( I_S \) can be calculated using:

\[
I_S = \frac{P}{D^2} \tag{1}
\]

where, \( P \) is the load on the points; and
\( D \) is the equivalent distance between points.
\( \sigma_{ci} \) is estimated as 24\( I_S \) (although the coefficient 24 varies for different rock types).
3.2 Hoek-Brown Constant $m_i$

This parameter has a significant influence on the strength characteristics of a rock mass. The constant $m_i$ depends on the frictional characteristics of the component minerals of an intact rock sample. This parameter can only be determined by triaxial testing of core samples or estimated by Hoek & Brown’s (1997) qualitative description of the rock material.

3.3 Geological Strength Index (GSI)

GSI is an index based upon an assessment of the structure, lithology and condition of discontinuity surfaces in the rock mass. The GSI value of a rock mass is incorporated into calculations in order to determine the reduction in the strength of the rock mass as compared to intact rock components. A relaxation and damage parameter $D$ (Hoek et al. 2002) can be used to modify the effect of GSI and is described in detail in Hoek & Diederichs (2006) but is not used here ($D=0$) for undisturbed or carefully excavated tunnel environments. A table presented in Hoek & Marinos (2000) and Marinos & Hoek (2000) can be used to estimate the GSI value for typical rock masses, or a revised version (Table 2) can be used for heterogeneous rock masses such as those encountered on Egnatia.

The rockmass strength envelope is given as:

$$\sigma_1 = \sigma_3 + \sigma_{ci} \left( m_b \frac{\sigma_3}{\sigma_{ci}} + s \right)^a$$  \hspace{1cm} (2a)

where for $D=0$:

$$m_b = m_i e^{28 \frac{GSI-100}{GSI-100}}$$ \hspace{1cm} (2b)

$$s = e^{-9 \frac{GSI}{100}}$$ \hspace{1cm} (2c)

$$a = \frac{1}{2} + \frac{1}{6} \left( e^{\frac{15 - GSI}{15}} - e^{-\frac{20}{3}} \right)$$ \hspace{1cm} (2d)

Hoek & Diederichs (2006) provided an improved relationship for rockmass modulus ($D=0$):

$$E_{rm} = E_i \left( 0.02 + \frac{1}{1 + e^{(60-GSI)/11}} \right)$$ \hspace{1cm} (3)

In the case of many of the rocks in this region, intact modulus is not available but can be related to the intact strength as per Hoek & Diederichs (2006). Finally a standardized index value for rockmass uniaxial strength (Hoek & Marinos 2000) can be obtained from:

$$\sigma_{cm} = \left( 0.0034 m_i^{0.8} \right) \sigma_{ci} \left( 1.029 + 0.025 e^{-0.2m_i} \right)^{GSI}$$ \hspace{1cm} (4)

An alternate formulation is given in Hoek et al. (2002) but not used on Egnatia or in this work.

Table 2. GSI estimates for heterogeneous rockmasses such as flysch (Marinos & Hoek 2001)
3.4 Flysch

Flysch consists of laminated elastic sediments, associated with orogenesis, characterized by alternating sandstone and fine grained (siltstones, silty shales and clayey shales) layers. The thickness of flysch along the project alignment is large and is affected by faults and thrusts. For heterogeneous rock masses such as flysch, it is necessary to carefully consider selection of $m_i$ and $\sigma_{ci}$. Marinos & Hoek (2001) suggested using a weighted average of the intact strength properties of the strong and weak layers as well as the modified Table 2.

4. TUNNEL SQUEEZING

An estimation of potential tunnel squeezing (change in tunnel radius, $R$) can be calculated using the rock mass strength parameters of $\sigma_{cm}$ and an estimate of the support pressure $p_i$ normalized to the average rock insitu pressure (prior to tunnelling) $p_0$:

$$\frac{\Delta R}{R} = \left( 0.002 - 0.0025 \frac{p_i}{p_0} \right) \left( \frac{\sigma_{cm}}{p_0} \right)^{2.4} \frac{L^2}{p_0}$$

(5)

According to Hoek (2002) and Hoek et al. (2006) few problems are expected below a “tunnel strain” or normalized radial closure of 0.01 or 1%, normally mandating shotcrete and rockbolts. Support components are layered onto this basic system as anticipated strain increases. For normalized closure values between 0.01 and 0.025, typical support design would add steel sets or lattice girders to the shotcrete and bolts of the previous case. These steel sets become heavy reinforcement above 0.025.

In addition some buttressing or pre-reinforcement (doweling) of the face may be required. Above a normalized closure of 0.05, heavy umbrella support is required ahead of the face (forepoling) although this is often proactively implemented at lower predicted strains and was used extensively in the Driskos Tunnel. Face buttressing and staged excavation may be mandatory. Extreme squeezing and very difficult tunnelling conditions are expected at values in excess of 0.10 and yielding support elements (sliding joints that maintain support pressure over larger deformations) are required.

5. TUNNEL SUPPORT AND MONITORING

Tunnel support in weak rock mass conditions involved the development of a number of typical cross sections for support design. Each section would be related to an anticipated magnitude of normalized radial closure.

When advancing through difficult ground the use of the forepoling umbrella arch method is oftentimes employed (Figure 3). This method is usually used in combination with other support systems such as steel sets embedded in shotcrete, face stabilization by grouted fibreglass dowels and the use of a temporary invert (bench) to control floor heave.

![Figure 3. (top) Primary support system for weak rock tunnelling (courtesy E. Hoek), including pre-support beyond the face. (bottom) an example of forepole (arrows) and support in Driskos Tunnel, Egnatia.](image)

The monitoring program within the tunnels of Egnatia Odos incorporates the use of inclinometers, extensometers, strain gauges, load cells, instrumented rock bolts and standard convergence and deformation measurements. Within the concept of the observational method of tunnel construction, monitoring has also played an important role in making design changes to primary support systems. This data will be vital for the calibration of numerical models as part of this research study.

6 DRISKOS TUNNEL

The Driskos twin tunnel is 2 x 4.7 km long and is situated west of Metsovo, the rock units and their average estimated properties are shown in Figure 4. For rockmasses as soft as some of these units, it is important to estimate the elastic modulus values correctly for displacement prediction. The approach of Hoek et al. (2002) is compared with the updated Hoek & Diederichs (2006) in Figure 5.
Figure 5. Min. and max. estimated rockmass modulus using Hoek et al. 2002 (HCC) and Hoek & Diederichs 2006 (HD). Zones correspond to Figure 4.

Figure 4. Schematic summary section of major geological units crossing the Driskos Tunnel Alignment at depth. (modified from Egnatia Odos. S.A. 2001). Geomechanical properties averaged over lengths shown. Numbered zones represent unique engineering geology. Bottom plot gives a smoothed summary of the ratio of rockmass strength (Equation 4) to overburden pressure (max and min ratio).

A particularly difficult section (5) is the focus of this study. Typical monitoring data from a deep and weak section of this tunnel zone is shown in Figure 6. The data is from the first bore and data begins after the first bench passes the monitoring point. It is the goal of this research to use this data along the Driskos Tunnel to calibrate predictive tools for weak rockmasses. Such a comparison is premature at the time of writing although an example of the numerical analysis being employed follows.

Figure 6. Example of monitoring data from Driskos tunnel. Total wall to wall closure is shown with absolute roof displacement. Progress of benches in the left bore and the right ("twin") tunnel are shown. Instrumentation is installed after the bench passes the section.
7. NUMERICAL ANALYSIS

For scoping calculations, a 2D finite element program (Rocscience 2006) was employed. The model was analyzed with progressively softening face and staged excavation of the top heading and bench. The excavation of the single tunnel and the subsequent twin tunnel was analyzed.

A typical support scenario is employed for this example. Forepoles are simulated by a stiffened arch above the tunnel, installed as the face is “advanced”. Bolts are installed at the face along with shotcrete and arch composite liner (modelled here as a meshed material although liner elements could also be used. For this example, rockmass properties on higher end of quality for Driskos Zone 5 are used (GSI=33, $\sigma_{ci}$=25MPa, $m_i$=11). Two depths of cover are shown here (105 and 195 metres). The tunnels were analyzed with and without support as well was with plastic (no dilation) and strain softening (dilation parameter 10% of frictional parameter). Horizontal stresses are 1.25 times the overburden. The model geometry and sample results are shown in Figure 7.

![Figure 7](image1)

**Figure 7.** a) FEM mesh and unsupported tunnel geometry. b) left bore with support and staging shown. c) comparative yield and displacements in unsupported left bore after gull excavation of right bore. d) displacements with support (no dilation). Twin tunnel is to the right in all analyses.

![Figure 7](image2)

**Figure 8.** Summary results from the example section in Figure 7. FEM results (dots) are compared with empirical/analytical prediction from Equation 5.

A number of monitoring points were sampled. The average radial vertical closure (one half of wall to floor closure) and one half of the wall to wall closure (horizontal closure) were assessed as well as the vertical deflection of the roof with respect to the centreline (springline). Results are shown in Figure 8 for different points under different conditions of support and dilation. The results are comparable to the predictions (for unsupported isotropic stress conditions) from Equation 5 although the influence of support systems, anisotropic convergence and dilation is clearly illustrated.

A three dimensional model has been constructed using the finite difference element codes within FLAC3D (Itasca, 2005). The model consists of 64000 zones, abutments that are used to minimize the boundary affects and an interior, horseshoe tunnel excavation portion (Figure 9).

![Figure 9](image3)

**Figure 9.** Detail of FLAC3D numerical model of twin (horseshoe) tunnels (axial length is compressed in figure).
The tunnel excavation has been sub-divided into top heading extraction and bottom bench excavation. This has been done in order to more closely simulate the true excavation sequence as practiced by industry. Further, portions of this investigation include the determination of the effects of twin tunnel excavation and interaction. As such, symmetry could not be used in order to optimize computation time. A typical section and twin tunnel geometry was used (Figure 10).

![Figure 10: Typical tunnel geometry after Lambropoulos (2005) A flat floor shown here although curved floor was used in some sections. Twin tunnel configuration shown at bottom. Support varies (see also Figure 7).](image)

Support, in the form of forepoles, rockbolts and liners has also been incorporated into the numerical model. This detail can be seen in Figure 11. Figure 11 depicts only the excavated portions of the numerical model. In the forefront, one can see that five excavation steps have been performed on the West tunnel and support has been added to the excavated portions of this tunnel.

The stresses and displacements are monitored throughout the 3D model, however, in order to minimize boundary effects, the planes of interest lie at the centre of the tunnel, longitudinally along the axis of excavation.

Constitutive models are being investigated including plasticity, strain softening and dilative flow, anisotropic plasticity. The strain dependency of dilation and the relationship to GSI and confinement will be studied as this is key to support response prediction. Scoping results are shown in Figure 12 and 13 for a simple plastic solution using similar parameters (Mohr Coulomb equivalent) to the analysis in Figure 7 and 8. These displacements are comparable to the typical monitored results in Figure 6 accounting for support adjustments in Figure 8 (full 3D supported model results will be employed as well). Note that the displacements in Figure 8 do not include pre-face displacements or top-heading excavation and pre-bench displacements. The analysis shown is for more favourable rockmass conditions than the specific case in Figure 6. Detailed sections will be analyzed with location specific geomechanical parameters. One important influence illustrated in these analyses is the extensive deformation that occurs in the first tunnel due to the passage of the second (twin) tunnel heading and bench. 2D analysis (Figure 8) and this 3D analysis suggest a critical threshold for tunnel interaction as a function of rockmass strength, depth and inter-tunnel separation. The deeper, weaker sections of Driskos tunnel exceed this threshold.

![Figure 11. Support detail within the twin tunnel FLAC3D numerical model (only excavated material and support components are shown).](image)

![Figure 12. Model displacement of roof (absolute vertical displacements). Results for unsupported model shown (depth 195m).](image)

8. FURTHER RESEARCH

2D and 3D FEM analysis incorporating staging and rock-support interaction, will be calibrated as discussed here using high quality monitoring data that has been obtained from the tunnels of Egnatia...
ACKNOWLEDGEMENTS

Many thanks to Dr. Evert Hoek, Dr. Paul Marinos and Vassilis Marinos for past and future collaboration and to Egnatia Odos S.A. This research is funded, in part, by NSERC and by PREA.

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Figure 13. Radial closure between two sidewall measuring points (total horizontal closure / 2) normalized to single completed tunnel. Monitoring results (Fig. 6) only capture displacements after the passage of 1st bench.

Even with such sophisticated tools as 3D finite element or finite difference models, the problem remains to define the constitutive behaviour (peak and post yield) for these very complex materials. Research is ongoing to determine the proper determination of these parameters. In addition, the process of modelling forepole rock-support interaction is non-trivial. The mechanisms of rock-support interaction related to forepuling and much of the face or near-face reinforcement system are poorly understood at this time. Design decisions such as those discussed in this paper are currently possible but a true optimized and mechanics-based design process for the forepole umbrella, for example, is not at present fully developed.

The impact of deformations and instabilities ahead of the tunnel face on the subsequent stability of the overall tunnel (short and long term) is a key mechanistic problem requiring study. The ultimate aim of this research is to provide guidance for the use of conventional design tools and numerical codes for optimization of these support systems.
Appendix H

Calculations of Equivalent Properties in Lining Elements
Appendix H

Calculations of Equivalent Properties in Lining Elements

1.0 Introduction

This Appendix outlines a procedure, in addition to the procedure outlined by Hoek in his Kersten Lecture (Hoek, 2008), as to how to calculate equivalent properties associated (i.e. Modulus of Elasticity and Moment of Inertia) with tunnel support that consists of steel sets embedded in shotcrete. These parameters will ultimately be used to determine moments and forces within the temporary support. In such a case, a designer requires to understand the contribution of each of these support elements and to be able to adjust the number and dimensions of each to accommodate the loads imposed on the lining. The arrangement and spacing that has been used in these series of examples is seen below in Figure H1. Arched steel sets compose the roof of the structure while vertical sections comprise the wall support. The support has 1.0 m spacing.

The derivations that have been included in this Appendix utilize a purely mechanics of materials, structural approach. The steel sets that have been used in Driskos tunnel were HEB160. These can be seen in Figure H2.

![Figure H1](image)

**Figure H1** Support Details associated with steel set support
### Geometrical and Statistical Values for HEB 160 Steel Sets (Profil Arbed, 2007)

**Design Parameters:**
- **L:** span of the SFB beam in meters
- **q:** design load in kN/m
  \[ q = q_{y} + 2G + v_{y} + 3R \]
- **Validity Criteria:**
  - Steel grade 500S
  - Symmetrical loaded beam
  - Support length of the hollow core slab = 7 cm
  - Load ratio G/P = 50/100
  - Beam weight included in dead load G
  - Deflection under live load $P < L/300$
  - Transverse deflection of the bottom flange < 1.5 mm
  - Elastic-plastic design
  - Ideal elastic-plastic material behaviour

**Application Example:**
- **Pinned:** 5 x 7.2 m
- **Live load p:** 5 kN/m²
- **Dead load q:** 1.2 kN/m²
- **Slab thickness:** approximately 20 cm

**Users' Choice:**
- **I.B. beam span:** 6 m
- **Slab span:** 7.2 m (beam distance)
- **Depth of the HC slab:** 29.6 cm ($q_{y} = 3.6$ kN/m²)

### Calculated Values:
- **Line load from $p$:** $G = 7.2 - 12.2 = 36$ kN/m
- **Line load from $q$:** $q_{y} = 2.2 - 36 = 102.2$ kN/m

### Derived From the Annexed Design Tables:
- **HEB 200 + 460 x 15:** 147.1 kN/m
- **HEB 220 + 430 x 15:** 167.9 kN/m

**N.B.:** Please observe the minimum tonnage required for section delivery.

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**Diagram:**
- A steel beam with dimensions and forces indicated.

---

**Figure H2** Geometrical and statistical values for HEB 160 Steel Sets (Profil Arbed, 2007).
Figure H3 Arrangement of Steel Support (HEB 160) embedded in 30 cm of shotcrete. Note that the rockwall is to the bottom of the diagram.

Figure H3 depicts a scenario whereby HEB 160 steel sets are embedded in 300 mm of shotcrete. Isolated here is a single steel set of support that is installed at 1.0 m spacing for initial, temporary support purposes. Pressures (P) are being exerted from the rock wall face at the bottom of the figure.

2.0 Arch Structure

The arrangement of the support for the top portion of the excavated tunnel is shown in Figure H4. It is assumed that the support is in pure compression (i.e. compressive stress field) and that the axial loads induced from the rock mass are applied at the centroid of the support beam.
Figure H4  Arrangement of HEB 160 Steel Sets within Shotcrete assuming that the effective area is 100 cm (i.e. effective area is the 1 m span).

The values used for the Modulus of Elasticity for Steel and Shotcrete ($E_s$ and $E_c$) as well as the area of Steel and Shotcrete are as follows: ($A_s$ and $A_c$)

\[
E_s = 200 \text{GPa} \quad A_s = 90.3 \text{cm}^2
\]

\[
E_c = 30 \text{GPa} \quad A_c = 2,909.7 \text{cm}^2
\]

A composite Modulus of Elasticity can be determined through an average that depends to a large extent on the effective area of the shotcrete.

\[
\frac{A_s E_s + A_c E_c}{A_s + A_c} = E_{\text{composite}}
\]

Eq’n 1

Calculating this Modulus of Elasticity for our example, one obtains:

\[
= \frac{(90.3 \text{cm}^2) \times (200 \text{GPa}) + [(100 \times 30) - 90.3] \text{cm}^2 \times (30 \text{GPa})}{90.3 + [(100 \times 30) - 90.3] \text{cm}^2}
\]

\[
= \frac{18,060 + 87,291}{3,000 \text{cm}^2}
\]

\[
\approx 35 \text{GPa}
\]
Figure H5  Arrangement of HEB 160 Steel Sets within Shotcrete assuming that the effective area is determined using a 45 degree angle from the edges of the embedded steel section.

In structural engineering, the effective area is conventionally calculated in terms of a 45 degree influence of the structure embedded in concrete, as such, this would give one a 60 cm zone of influence (Figure H5 and Figure H6) vice the assumed 100 cm (from steel set to steel set).
In this scenario, the modulus of elasticity is greater, as there are zones of pure shotcrete that influence behaviour between the steel sets (Figure H7).

\[
\frac{A_s E_s + A_c E_c}{A_s + A_c} = E_{\text{composite}}
\]

\[
= \frac{(90.3 \times 200) + (1,709.7 \times 30)}{1,800 \text{cm}^2}
\]

\[
= \frac{18,060 + 51,291}{1,800}
\]

\[
\approx 38 \text{GPa}
\]

These zones of influence can be highlighted through the use of the profile view provided in Figure H7 below. If the area of influence is assumed to be between each of the steel sets (a), then \(E_{\text{composite}}\) is to be assumed to be 35GPa. If however, one was to take a more conservative approach (b), there are zones of influence for the steel sets whereby the combined \(E_{\text{composite}}\) is 38 GPa however, there are gaps in these zones where only shotcrete provides support (\(E_c = 30 \text{ GPa}\)).
Figure H7 Profile view depicting 1 m spacing of steel sets embedded in shotcrete support. Modulus of Elasticities are shown for 100 m zone of influence (a) and 60 cm of influence (b).

3.0 Determining the accuracy of Modulus of Elasticities Calculated

In order to check the simplified $E_{\text{composite}}$ for pure Compression:

Let $P = 5,000 \text{KN}$ \hspace{1cm} $E_{\text{composite}} = 35 \text{GPa}$

$$\sigma = \frac{P}{A_{\text{composite}}} = \frac{5,000,000 \text{N}}{1,000 \times 300 \text{mm}^2} = 16.67 \text{MPa}$$

$$\sigma_{\text{concute}} = 30 \text{MPa}$$

$$\sigma_{\text{steel}} = 250 \text{MPa}$$

$$\delta = \frac{PL}{AE_{\text{composite}}} = \frac{(5,000,000 \text{N}) \times (5,000 \text{mm})}{(1,000 \times 300 \text{mm}^2) \times (35,000 \text{MPa})} = 2.38 \text{mm}$$

Determining Forces:

$$P_s = P \left( \frac{E_s A_s}{E_s A_s + E_c A_c} \right) = 5,000 \text{KN} \left[ \frac{200 \text{GPa} \times 90.3 \text{cm}^2}{(200 \times 90.3) + (30 \times 2,909.7 \text{cm}^2)} \right]$$

$$= 5,000 \text{KN} \left[ \frac{18,060}{18,060 + 87,291} \right] = 5,000 \text{KN} \cdot 0.1714 = 857 \text{KN}$$
\[ P_e = P\left(\frac{E_c A_c}{E_s A_s + E_c A_c}\right) = 5,000\text{KN} \left[\frac{87.291}{18,060 + 87.291}\right] = 5,000\text{KN}[0.8286] = 4,143\text{KN} \]

Therefore, \[ \delta = \delta_s = \frac{P L}{E_s A_s} = \frac{(857,000\text{N}) \times (5,000\text{mm})}{(200,000\text{MPa})(90.3 \times 10^2 \text{mm}^2)} = 2.37\text{mm} \]

Therefore, \[ \delta = \delta_c = \frac{P c L}{E_c A_c} = \frac{(4.143 \times 10^3 \text{N}) \times (5,000\text{mm})}{(30 \times 10^3 \text{MPa}) \times (2,909.7 \times 10^2 \text{mm}^2)} = 2.37\text{mm} \]

\[ \delta \text{ Checked O.K. Therefore, } E_{\text{composite}} = 35\text{GPa} \text{ O.K.} \]

4.0 Support for Walled Sections

These sections are under a combination of compressional and flexural influence. The structural analogue for such a scenario is a beam subjected by combined loadings. In the following derivation, it is assumed that the compressive load is much larger than the transverse load.

1. Compressive Load \((\sigma_1)\): use the pure compression of beam as conducted in Section 2.0.
2. Transversal Load \((\sigma_2)\): need to determine “new” Neutral Axis of this Composite configuration. Therefore, the Neutral Axis for the section, \(Y\), can be determined (as depicted in Figure H8).

\[ Y = \frac{\sum (E_s A_s) y_i}{\sum (E_i A_i)} \quad \text{Eq’n 2} \]

\[ = \frac{\left[(200\text{GPa} \times 90.3\text{cm}^2) \times 18.6\text{cm}\right] + 30\text{GPa} \left[A_1 \times y_1 + A_2 y_2 + A_3 \times 15\text{cm} + A_4 \times 6.5\text{cm}\right]}{2(200 \times 90.3) + 30(A_1 + A_2 + A_3 + A_4) 	imes 2} \]

\[ = 2,909.7\text{cm}^2 \]

Now are required to calculate the combine Moment of Inertial for the material about the “new” neutral axis:

\[ I_{\text{composite}} = I_{\text{steel}} + A_{\text{steel}} \times (186 - Y)^2 + \sum_{i=1}^{4} \left[ (I_i + A_i \times (d_i)^2) \right] \times 2 \quad \text{Eq’n 3} \]
If bending moment due to lateral pressure (transversal load) is known, then,

\[ \sigma_{\text{flexural}} = \frac{M\bar{Y}}{I_{\text{composite}}} \quad \text{or} \quad \sigma_{\text{flexural}} = \frac{M(300\text{mm} - \bar{Y})}{I_{\text{composite}}} \quad \text{Eq’n 4} \]

Therefore, the required tunnel support for the walled sections is the superposition of \( \sigma_1 \pm \sigma_2 \).

**Figure H8** Determination of Neutral Axis for Walls of Tunnel Support.
Appendix I

Driskos Case Study Paper
Selected Modelling Results
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Relative Horizontal Compressive / Radial Closure of Tunnel for All Section 4.4 - Unsupported Runs

Relative Horizontal Compressive / Radial Closure of Tunnel for All Section 4.4 - Supported Runs

Relative Vertical Displacement of Tunnel for All Section 4.4 - Unsupported Runs

Relative Vertical Displacement of Tunnel for All Section 4.4 - Supported Runs
Relative Horizontal Compressive / Radial Closure of Tunnel for - Elastic - Unsupported Runs

Relative Vertical Displacement of Tunnel for - Elastic - Unsupported Runs

History Point at 40m

- A-Phase Excavation Reaches Hist Pt
- A-Phase Excavation Complete
- Excavation of First Tunnel Complete
- Excavation of Twin Tunnel Complete
- Bench Excavation Reaches Hist Pt
- A-Phase Excavation Passes beside Hist Pt
- A-Phase Excavation Complete
- Excavation of Twin Tunnel Complete
- Bench Excavation Passes beside Hist Pt
- Excavation of First Tunnel Complete
- A-Phase Excavation Complete

Time Steps

Displacement (mm)
Appendix J

2D vs 3D Paper
Selected Modelling Results
## 2D versus 3D Paper

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Support = 30cm shotcrete + 160St.set @ 1m.
## 2D versus 3D Paper
### Modelling Runs Table

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### Additional Runs – Lagging Liner and Invert

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Support = 30cm shotcrete + 160St.set @ 1m.
Relative Horizontal Compressive / Radial Closure of Tunnel for Material D1 - Unsupported, Circular Runs - Mohr & Elastic

Relative Vertical Displacement of Tunnel for Material D1 - Unsupported, Circular Runs - Mohr & Elastic

Relative Horizontal Compressive / Radial Closure of Tunnel for Material E1 - Unsupported, Circular Runs - Mohr & Elastic

Relative Vertical Displacement of Tunnel for Material E1 - Unsupported, Circular Runs - Mohr & Elastic
Relative Horizontal Compressive / Radial Closure of Tunnel
for Material C1 - Circular SUPPORTED - for liner installed 8m back

- Full Face Excavation of Tunnel Reaches History Point
- Run #17, Circular, D1
- Run #40, liner installed 8m back

Relative Vertical Displacement of Tunnel
for Material C1 - Circular SUPPORTED - for liner installed 8m back

Excavation Complete

History Point at 40m of 100m long tunnel

Relative Horizontal Compressive / Radial Closure of Tunnel
for Material D1 - Circular SUPPORTED - for liner installed 8m back

- Full Face Excavation of Tunnel Reaches History Point
- Run #16, Circular, D1
- Run #39, liner installed 8m back

Relative Vertical Displacement of Tunnel
for Material D1 - Circular SUPPORTED - for liner installed 8m back

Excavation Complete

History Point at 40m of 100m long tunnel
Relative Horizontal Compressive / Radial Closure of Tunnel for Materials B1, C1, D1 and E1 - Unsupported, Circular Runs #1-4

Relative Vertical Displacement of Tunnel for Materials B1, C1, D1 and E1 - Unsupported, Circular Runs #1-4
Relative Horizontal Compressive / Radial Closure of Tunnel
for **Material D1** for Various Horizontal Stress Ratios

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Relative Vertical Displacement of Tunnel
for **Material D1** for Various Horizontal Stress Ratios

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<th>Kp</th>
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Relative Horizontal Compressive / Radial Closure of Tunnel
for Materials B1, C1, D1 and E1 - Unsupported, Circular Runs #22-25

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Relative Vertical Displacement of Tunnel
for Materials B1, C1, D1 and E1 - Unsupported, Circular Runs #22-25

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**Time Steps**

**Displacement (mm)**
Relative Horizontal Compressive / Radial Closure of Tunnel
for Material D1 Supported, Horseshoe Runs

History Point at 40m of 100m long tunnel

Excavation Complete

Run #38 Horseshoe Forepoles with Liner
Run #19 Horseshoe Liner Only
Run #27 Horseshoe Forepoles NO liner
Run #43 Horseshoe with Invert at face
Run #45 Horseshoe with Invert at 8m back
Run #41 Horseshoe with liner 8m back
Run #19 Horseshoe Liner Only

Full Face Excavation of Tunnel Reaches History Point

Displacement (mm)

Time Steps

Relative Vertical Displacement of Tunnel
for Material D1 Supported, Horseshoe Runs

History Point at 40m of 100m long tunnel

Excavation Complete

Run #21 Horseshoe Ka=0.67, Kp=1.5 Liner Only
Run #37 Horseshoe Forepoles NO liner
Run #38 Horseshoe Forepoles with Liner
Run #19 Horseshoe Liner Only

Full Face Excavation of Tunnel Reaches History Point

Displacement (mm)

Time Steps
Relative Horizontal Compressive / Radial Closure of Tunnel for Material C1 Unsupported, Horseshoe FF and TB Runs

-250 -200 -150 -100 -50 0 20000 40000 60000 80000 100000 120000

Time Steps
Displacement (mm)

History Point at 40m of 100m long tunnel

Note: Steps for each are for computation steps and not relative or comparative location of Face

Relative Vertical Displacement of Tunnel for Material C1 Unsupported, Horseshoe FF and TB Runs

-250 -200 -150 -100 -50 0 10000 20000 30000 40000 50000 60000

Time Steps
Displacement (mm)

History Point at 40m of 100m long tunnel

Note: Steps for each are for computation steps and not relative or comparative location of Face
Relative Horizontal Compressive / Radial Closure of Tunnel for Materials C1 & D1 Unsupported, Horseshoe Runs

- Run #7 Horseshoe, $K_a=1$, $K_p=1$ (C1 Mat'l)
- Run #8 Horseshoe, $K_a=1$, $K_p=1$ (D1 Mat'l)

Relative Vertical Displacement of Tunnel for Materials C1 & D1 Unsupported, Horseshoe Runs

- Run #7 Horseshoe, $K_a=1$, $K_p=1$ (C1 Mat'l)

Relative Horizontal Compressive / Radial Closure of Tunnel for Material C1 - Supported and Unsupported Circular Runs

- Run #2 Circle, Unsupported - Elastic
- Run #17 Circle, Supported - Liner
- Run #2 Circle, Unsupported

Relative Vertical Displacement of Tunnel for Material C1 - Supported and Unsupported Circular Runs

- Run #23 Circle, Unsupported - Elastic
- Run #17 Circle, Supported - Liner
- Run #2 Circle, Unsupported