STABILITY ANALYSIS AND SUPPORT DESIGN OF MANIPURA DIVERSION TUNNELS IN MYANMAR

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การวิเคราะห์เสถียรภาพและการออกแบบค้ำยันของ
อุโมงค์ผันน้ำแมนนิปูร่าในประเทศพม่า

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วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิศวกรรมศาสตรมหาบัณฑิต
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STABILITY ANALYSIS AND SUPPORT DESIGN OF MANIPURA DIVERSION TUNNELS IN MYANMAR

Suranaree University of Technology has approved this thesis submitted in partial fulfillment of the requirements for a Master's Degree.

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บัญญา โซอ์ : การวิเคราะห์เสถียรภาพและการออกแบบค้ำยันของอุโมงค์ผันน้ำแมนนิปูร่าในประเทศพม่า (STABILITY ANALYSIS AND SUPPORT DESIGN OF MANIPURA DIVERSION TUNNELS IN MYANMAR) อาจารย์ที่ปรึกษา : รองศาสตราจารย์ ดร. กิตติเทพ เฟื่องขจร, 112 หน้า

งานวิจัยนี้มีจุดประสงค์เพื่อวิเคราะห์เสถียรภาพและออกแบบค้ำยันสำหรับอุโมงค์คู่ขนานสำหรับระบายน้ำจากโครงการสร้างเขื่อนแมนนิปูร่า โครงการนี้ได้สร้างเขื่อนกั้นแม่น้ำแมนนิปูร่าซึ่งอยู่ห่างจากเมืองกาเล เขตเมืองกาเล ประเทศพม่า เป็นระยะทางประมาณ 52 กิโลเมตร โดยโครงการมีวัตถุประสงค์เพื่อจัดให้มีน้ำสำหรับพื้นที่เกษตรกรรมจำนวน 20,242 เฮคเตอร์ และผลิตกระแสไฟฟ้าจากพลังงานน้ำสำหรับใช้ในพื้นที่นั้น อุโมงค์คู่ขนานมีความยาว 1,050 เมตร โดยมีหน้าตัดเป็นรูปเกือกม้าที่มีความกว้าง 12 เมตร อุโมงค์คู่ขนานนี้มีเส้นทางลอดใต้ภูเขาโดยมีจุดประสงค์เพื่อเบนทิศทางการไหลของน้ำในช่วงที่มีการสร้างตัวเขื่อนแบบหินทิ้ง อุโมงค์ผันน้ำนี้ได้ก่อสร้างผ่านชุดมวลหิน 3 รูปแบบ คือ โซนที่ 1 : เป็นหินชนวนที่มีระดับการผุพังปานกลางถึงมาก โซนที่ 2 : เป็นหินชนวนที่มีระดับการผุพังต่ำถึงปานกลาง และโซนที่ 3 : เป็นหินชนวนที่มีระดับการผุพังต่ำสลับชั้นกับหินทรายเกรย์แวก การประเมินทางธรณีเทคนิคของโซนต่าง ๆ นี้อาศัยข้อมูลที่ได้จากการสำรวจ การสังเกตในภาคสนาม และการทดสอบในห้องปฏิบัติการ

ในงานวิจัยนี้ได้ศึกษาเกี่ยวกับการคุณลักษณะของมวลหิน การประเมินค่าตัวแปรที่เกี่ยวข้องกับมวลหิน การวิเคราะห์เสถียรภาพ และการออกแบบระบบค้ำยันหินรอบอุโมงค์ มวลหินตลอดความยาวของอุโมงค์ได้ถูกจำแนกโดยใช้ระบบการจำแนกมวลหินเชิงประสบการณ์ซึ่งหมายรวมถึง ระบบการจำแนกหินด้วยระบบการให้คะแนน (RMR) ระบบดัชนีคุณภาพมวลหินในอุโมงค์ของ NGI (Q-system) ระบบดัชนีมวลหิน (RMi) ดัชนีความแข็งธรณีวิทยา (GSI) ค่าสัมประสิทธิ์ความยืดหยุ่นของมวลหิน และตัวแปรของ Hook-Brown แนวทางการออกแบบค้ำยันที่ใช้กันอยู่ทั่วไป ได้ถูกนำมาใช้ประกอบกับผลการจำแนกคุณลักษณะของมวลหินในสถานที่จริง การจำลองด้วยแบบจำลองตัวเลข (โดยโปรแกรม $Phase^2$) ถูกนำมาใช้เพื่อประเมินเสถียรภาพของอุโมงค์ทั้งที่มีและไม่มีระบบค้ำยัน โดยที่เกณฑ์การวิบัติของมวลหินที่เสนอโดย Hoek และ Brown ถูกนำมาใช้ประเมินบริเวณที่มีการครากและการเคลื่อนตัวสูงสุดของหินรอบอุโมงค์ ระบบค้ำยันที่แนะนำให้ใช้โดยวิธีเชิงประสบการณ์ (Empirical) ถูกใช้ในการศึกษาด้วยการจำลองเชิงตัวเลข ซึ่งประกอบด้วยหมุดยึดหิน และคอนกรีตพ่นร่วมกันตาข่ายลวด คุณสมบัติของส่วนที่เป็นค้ำยัน เช่น ความยาวของหมุดยึดหิน ระบบการติดตั้งหมุดยึดหิน และความหนาของคอนกรีตพ่น ได้ถูกใช้ตามที่ระบุไว้ในวิธีเชิงประสบการณ์ ก่อนที่จะมีการติดตั้งระบบค้ำยัน พื้นที่ที่เกิดการครากของหินรอบอุโมงค์ได้ถูกสังเกต

ซึ่งจากผลการศึกษาชี้ให้เห็นว่า มีบางปัญหาเกิดขึ้นเกี่ยวกับเสถียรภาพของอุโมงค์ กล่าวคือหลังจากที่ติดตั้งระบบค้ำยันแล้ว พื้นที่ที่เกิดการครากรอบอุโมงค์และระยะของการเกิดเป็นพลาสติกได้ลดลง อีกทั้งการเคลื่อนตัวสูงสุดของอุโมงค์ยังลดลงอีกด้วย ซึ่งชี้ให้เห็นว่า ระบบการค้ำยันมีความเหมาะสมที่จะทำให้เกิดเสถียรภาพของอุโมงค์ได้ โดยสภาวะที่ดีที่สุดระหว่างผลการออกแบบค้ำยันด้วยวิธีเชิงประสบการณ์ และวิธีทางด้านระเบียบวิธีเชิงตัวเลขนั้น ได้ถูกกำหนดเพื่อใช้ในการออกแบบค้ำยันที่เหมาะสมสำหรับอุโมงค์ผันน้ำแมนนิปูร่า

สาขาวิชา เทคโนโลยีธรณี

ปีการศึกษา 2553

ลายมือชื่อนักศึกษา

ลายมือชื่ออาจารย์ที่ปรึกษา

BANYAR SOE : STABILITY ANALYSIS AND SUPPORT DESIGN OF MANIPURA DIVERSION TUNNELS IN MYANMAR. THESIS ADVISOR : ASSOC. PROF. KITTITEP FUENKAJORN, Ph.D., P.E., 112 PP.

TUNNEL/DESIGN/STABILITY/SUPPORT/DIVERSION/MANIPURA

The objective of this study is to perform stability analysis and support design for two parallel diversion tunnels of the Manipura multi-purposed dam project. The project is located on the Manipura River, 52 km from the Kalay Town, Kalay Township, Sagaing Division, Myanmar. The two parallel tunnels are 1,050 m long with 12 m wide modified horseshoe shape. The diversion tunnels penetrate three different zones of rock mass : zone 1 : moderately to highly weathered slate; zone 2 : slightly to moderately weathered slate; and zone 3 : slightly weathered slate with alternation of greywacke sandstone band. The geotechnical evaluation of these zones is relied on the exploratory data, field observations and laboratory test results.

The proposed research involves rock mass characterizations, evaluation of rock mass parameters and stability analysis, and support design for the rock mass around the tunnels. The rock masses along the tunnels are classified by using empirical rock mass classification systems including rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi) and geological strength index (GSI). Traditional guidelines for the rock support have been used based on the results of the site characterizations. A series of numerical simulations (using Phase^2 code) is performed to assess the stability conditions of the tunnels with and without the support systems. Hoek and Brown failure criterion is used to estimate yielding zone around the tunnels and the maximum displacement. The support systems suggested by empirical methods are employed in numerical modeling. The properties of the support elements, such as bolt length, bolt patterns and thickness of shotcrete are similar to those proposed by the empirical methods. Before support installation, yielding zones are observed. The results indicate that there would be some stability problems for the tunnels. After support installation, the number of yielding zones and the radius of plastic zone are decreased. The maximum displacement is also reduced. This indicates that the applied support systems are adequate to obtain the tunnel stability. Optimization between the empirical and numerical results is made to obtain the suitable support design for the Manipura diversion tunnels.

School of Geotechnology　　　　Student's Signature__________

Academic Year 2010　　　　Advisor's Signature__________

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SYMBOLS AND ABBREVIATIONS

A	=	Empirical constant for equation (5.24)
a	=	Empirical constant for equation (5.20)
B	=	Width of tunnel for equation (5.22)
B	=	Empirical constant for equation (5.25)
Cg	=	Competency factor
D	=	Disturbance factor
D_b	=	Block diameter
D_e	=	Equivalent dimension of excavation
ESR	=	Excavation support ratio
E_i	=	Young's modulus
E_m	=	Deformation modulus of rock mass
FS	=	Safety factor
f_σ	=	Massivity parameter
GSI	=	Geological strength index
Gc	=	Ground condition factor
H	=	Height of the overburden
JC	=	Joint conditions
JP	=	Jointing parameter
J_a	=	Joint alternation number
J_n	=	Joint set number
J_r	=	Joint roughness number

SYMBOLS AND ABBREVIATIONS (Continued)

J_v	=	Volumetric joint count
J_w	=	Joint water reduction number
jA	=	Joint alteration
jL	=	Joint length
jR	=	Joint roughness
k	=	Stress ratio
m_i	=	Hoek and Brown constant of intact rock
m_j	=	Hoek and Brown constant of rock mass
P_{roof}	=	Support pressure
Q	=	NGI tunneling quality index
Q_c	=	Normalization of Q value
Q_N	=	Stress free from Q
RMi	=	Rock mass index
RMR	=	Rock mass rating value
RQD	=	Rock quality designation
SCR	=	Surface condition rating
SR	=	Structure rating
SRF	=	Stress reduction factor.
Sr	=	Size ratio
s_i	=	Hoek and Brown constant of intact rock
s_j	=	Hoek and Brown constant of rock mass
V_b	=	Block volume

SYMBOLS AND ABBREVIATIONS (Continued)

W	=	Width of opening
β	=	Block shape factor
γ	=	Unit weight
υ	=	Poisson's ratio
σ_c	=	Uniaxial compressive strength of intact rock
σ_{cm}	=	Uniaxial compressive strength of rock mass
σ_h	=	Horizontal stress
σ_v	=	Vertical stress
$\sigma_{\theta roof}$	=	Tangential stress at roof
$\sigma_{\theta wall}$	=	Tangential stress at wall

CHAPTER I

INTRODUCTION

1.1 Background of Problems and Significance of the Study

The Manipura multi-purposed dam project is located on the Manipura River, 52 km from the Kalay Town, Kalay Township, Sagaing Division in Myanmar (Figure 1.1). The project is a part of the Multi-purposed Government Funded Schemes and has been implemented under the supervision of Irrigation Department, Ministry of Agriculture and Irrigation in Myanmar. The dam will provide irrigated water for 20,242 hectares and hydro-electricity to the local area. The two parallel tunnels are 1050 m long with 12 m wide modified horseshoe shape (Figure 1.2). They are driven underneath the south bank mountain to divert the water whilst a massive rock-fill dam is built. The diversion tunnels penetrate three distinct zones of rock mass: zone 1 : moderately to highly weathered slate; zone 2 : slightly to moderately weathered slate; and zone 3 : slightly weathered slate with alternation of greywacke sandstone band. Within the confined space of a tunnel, it is difficult and dangerous to deal with stability or water problems which are encountered unexpectedly. Failure to do so will result in an inadequate basis for the design and could be very costly when unexpected problems are sometimes encountered at a later stage in the project. In the case of Manipura diversion tunnel constructions, some tunnel stability problems are expected based on geological investigations.

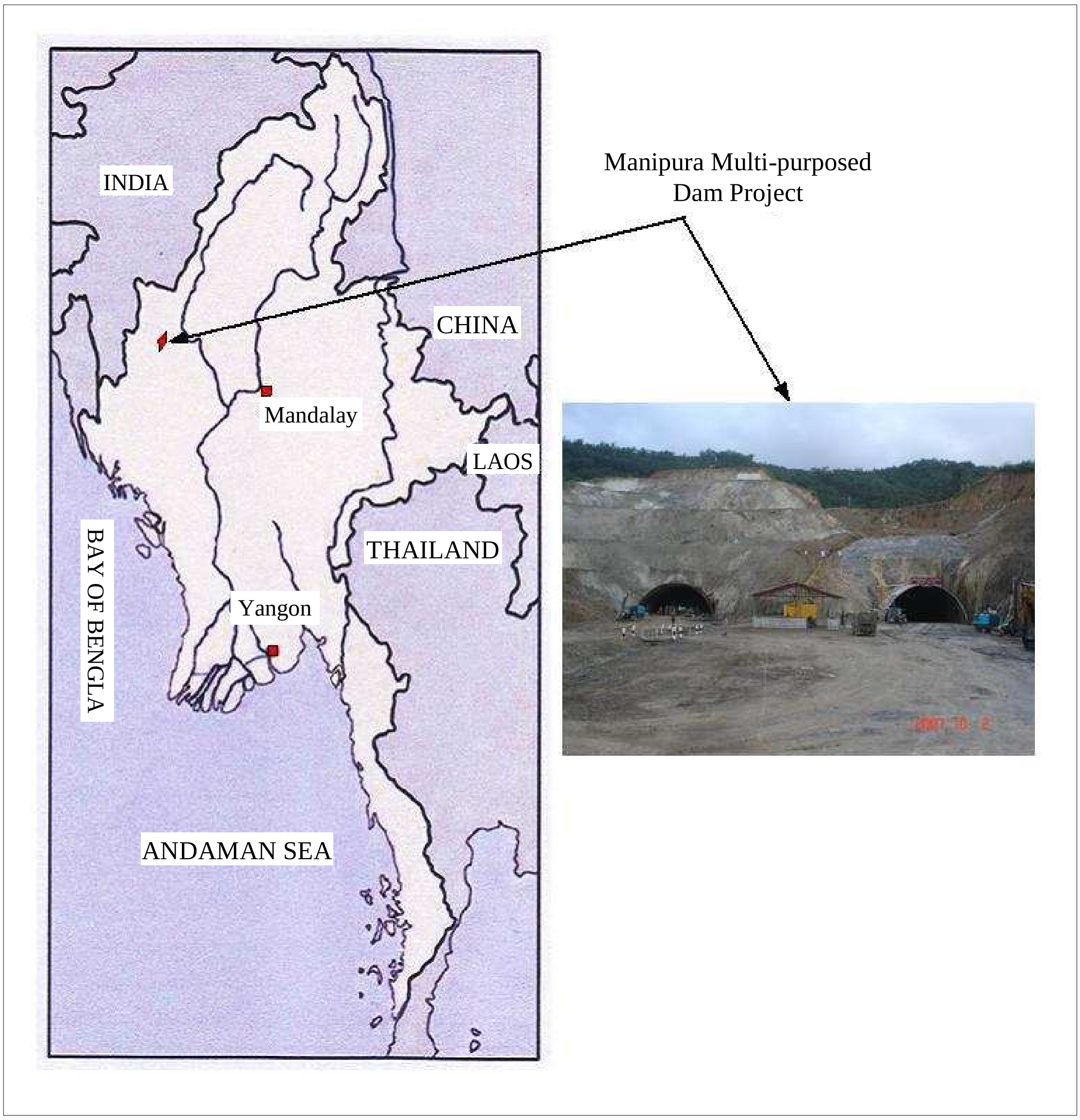

Figure 1.1 Location of Manipura multi-purposed dam project in Myanmar.

Rock mass classification systems are very useful tools for the preliminary design stage of a project, when very little detailed information on rock mass is available (Genis, Basarir, Ozarslan, Bilir, and Balaban, 2007). To classify the rock mass quality, rock mass classification systems including rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi), and geological strength index (GSI) are applied in this study. Their rating values are used to evaluate the tunnel support systems and rock mass parameters. These empirical methods have been originally obtained from many tunneling case studies and they have been applied to many construction designs. However, these empirical methods cannot adequately calculate stress distributions, support performance and deformational behavior around a tunnel. Therefore, 2D finite element software, Phase^2 version 6.0, will be used for numerical simulations. The rock mass parameters determined by empirical equations are utilized as input data for numerical modeling (using Phase^2 code). The comparison will be made the results obtained from empirical methods with numerical method to assess the reasonable support systems.

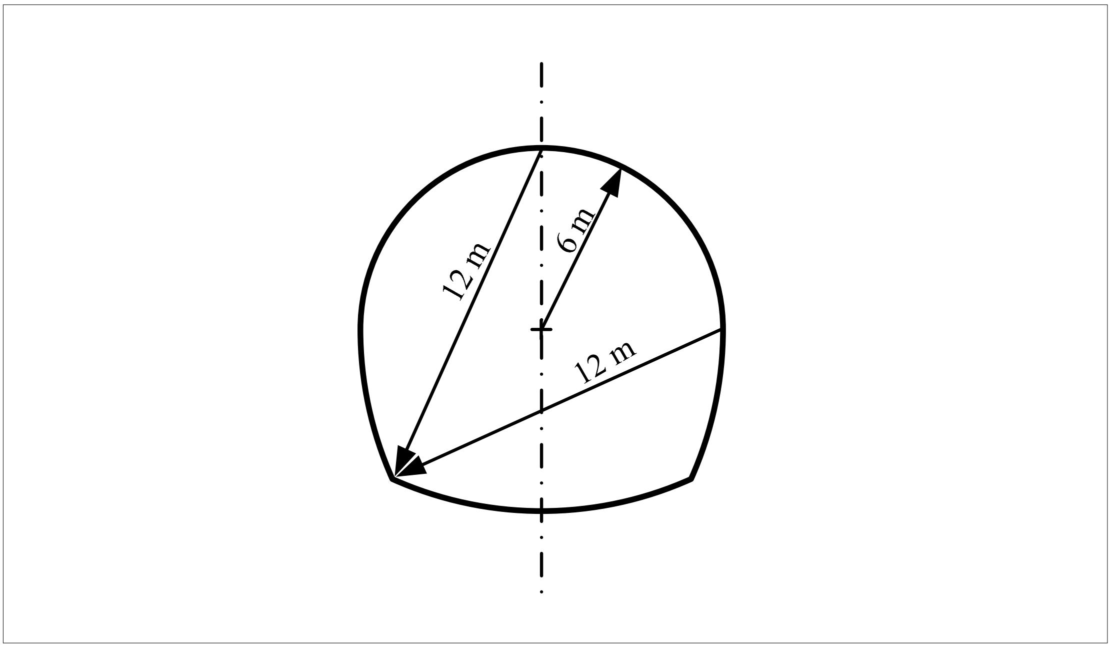

Figure 1.2 Cross-section of the Manipura diversion tunnels (Modified horseshoe shape).

1.2 Research Objectives

The objective of the proposed research is to perform stability analysis and support design for the two parallel diversion tunnels of the Manipura multi-purposed dam project. The proposed research involves conducting a design methodology of the actual diversion tunnel and comparing the support design results obtained from empirical methods with numerical method. The effort comprises the characterizations of rock mass by using rock mass classification systems, determination of input parameters, stability analysis, support design by empirical approaches and numerical method (using Phase^2 code). The feasible support designs are assessed by comparing the results with those obtained from the empirical and numerical methods.

1.3 Research Methodology

This research consists of six main tasks: literature review, geological data collection, rock mass characterizations, geotechnical rock mass parameter estimation and stability analysis, support design (empirical methods and numerical method) and comparisons, and discussions, conclusions and thesis writing. The research methodology is illustrated in Figure 1.3.

1.3.1 Literature Review

Literature review has been carried out to study the rock mass classification systems, estimation of rock mass parameters and stability analysis, determination of support systems by using empirical methods and numerical method (using Phase^2 code), and case studies in Myanmar. The sources of information are from journals, technical reports and conference papers. A summary of the literature review will be given in the thesis.

1.3.2 Geological Data Collection

Engineering Geology Office 2, Irrigation Department, Ministry of Agriculture and Irrigation in Myanmar, carried out the preliminary geological investigation in 2004 by drilling four vertical boreholes along the tunnel alignment with total depth of 239 m and continuous investigations have been performed. The geotechnical parameter evaluation of the Manipura diversion tunnels is relied on the exploratory data, field observations and laboratory test results.

1.3.3 Rock Mass Characterizations

The rock masses along the tunnel alignment are classified by using the rock mass classification systems; rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi), and geological strength index (GSI). Their rating values are used to evaluate the rock mass parameters and support designs for the Manipura diversion tunnels.

1.3.4 Geotechnical Rock Mass Parameter Estimation and Stability Analysis

Geotechnical rock mass parameters such as Hoek and Brown constants, deformation modulus of the rock mass and uniaxial compressive strength of the rock mass are estimated by empirical equations. The stability for all sections of the tunnels has been evaluated in terms of stand-up time of rock mass, estimation of maximum unsupported span and safety factor.

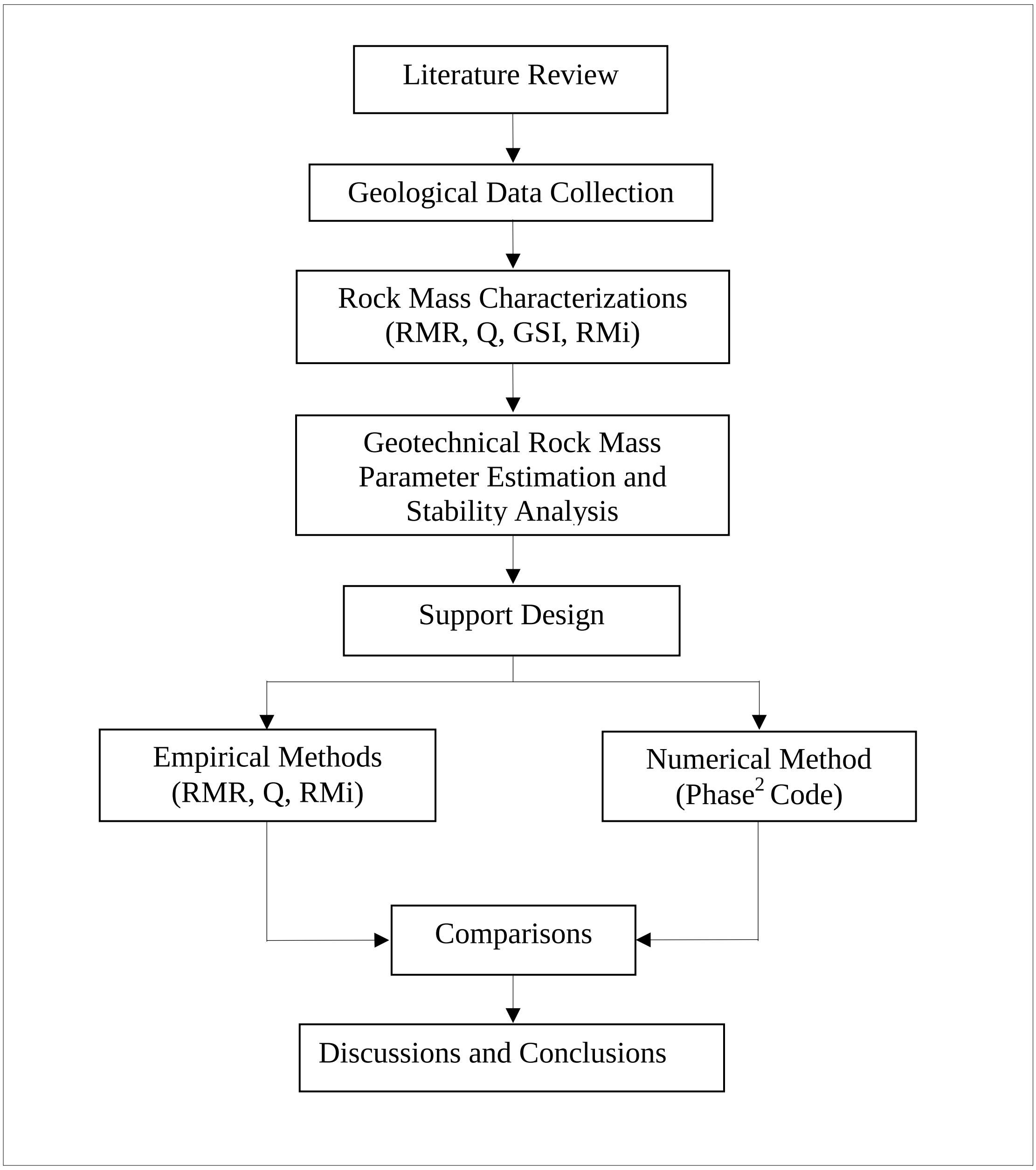

Figure 1.3 Research methodology.

1.3.5 Support Design and Comparisons

The empirical methods including rock mass rating system (RMR), NGI tunneling quality index (Q system) and rock mass index (RMi) are used to determine the support systems for the diversion tunnels dealing with their rating values.

The performances of the support elements suggested from empirical methods are analyzed by numerical method. A series of numerical simulations (using Phase^2 code) are performed to assess the stability conditions of the tunnels with and without the support systems. Results obtained from empirical methods are compared with the support systems obtained from the numerical method (Phase^2 code).

1.3.6 Discussions, Conclusions and Thesis Writing

The research results will be concluded and provided the recommended support systems for the Manipura diversion tunnels.

All research activities, methods, and results will be documented and complied in the thesis. The research or findings will be published in the conferences, proceedings or journals.

1.4 Scope and Limitations of the Study

Extensive literature review of the design methodology of the Manipura diversion tunnels is conducted. The shapes of the Manipura diversion tunnels are modified horseshoe shape. The engineering geological properties of the two tunnels are assumed the same. The two tunnels have been constructed by using drill-and-blast technique. The geological investigation of the Manipura diversion tunnels is relied on the exploratory data, field observations and laboratory test results. The comparison of the results obtained from empirical methods and numerical method will be made.

1.5 Thesis Contents

Chapter I introduces the thesis by briefly describing the background of problems and significance of the study. The research objectives, methodology, scope and limitations are identified. **Chapter II** summarizes the results of the literature review. **Chapter III** describes the geological data collection including geology of the project area, engineering geology and laboratory test results. **Chapter IV** presents the characterizations of rock mass by using rock mass classification systems. **Chapter V** discusses the estimation of geotechnical rock mass parameters by using empirical equations and stability analysis. **Chapter VI** describes the evaluation of support designs for the diversion tunnels. Estimating the feasible support designs of the tunnels are divided into 3 tests, including (1) support designs by using empirical methods, (2) numerical simulations (using $Phase^2$ code), and (3) comparisons the results obtained from empirical methods with numerical method. **Chapter VII** concludes the research results and provides recommendations for future research studies.

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CHAPTER II

LITERATURE REVIEW

2.1 Introduction

This chapter summarizes the results of literature review carried out to improve an understanding of rock mass characterizations, stability analysis and support design of tunnels. Topics relevant to this research involve rock mass classification systems including rock mass rating system (RMR), NGI tunneling quality index (Q system), geological strength index (GSI), and rock mass index (RMi), Deere's rock quality designation (RQD), numerical modeling (Phase^2 code) and published papers.

2.2 Rock Mass Classification Systems

The rock mass characterization processes are normally used to assess the rock mass quality in accordance with the existing engineering rock mass classification systems. The result becomes effective parameters for the application of the tunnel stability and design. The four methods of quantitative rock mass classification systems including rock mass rating system (RMR), NGI tunneling quality index (Q system), geological strength index (GSI), and rock mass index (RMi) will be applied in this research.

2.2.1 Rock Mass Rating System (RMR)

Bieniswski (1973) initially developed the rock mass rating system (RMR), otherwise known as the geomechanics classification. It was modified over the years as more case histories, became available and to conform to international standards and procedures (Bieniawski, 1979).

Bieniawski provided the system as the most common quantitative method for describing the quality of the rock mass for tunneling. Uniaxial compressive strength of intact rock (UCS), rock quality designation (RQD), spacing of discontinuities, conditions of discontinuities, ground water condition and orientation of discontinuities are utilized parameters. After the determination of the important ratings of the each parameter, they are summed to describe the basic RMR rating of the rock mass. In tunneling, the rating value must be made adjustment for discontinuity orientation. The rock mass rating classification table (Bieniawski, 1989) will be used in this research. Bienawski (1979) has described a chart to check the stand-up time of rock mass and to estimate maximum unsupported span of underground openings. In 1989, Bieniawski has provided guidelines for the selection of rock support for horseshoe shaped tunnel excavated by the drill-and-blast technique.

2.2.2 NGI Tunneling Quality Index (Q system)

The Q system of rock mass classification was developed in Norway by Barton, Lien, and Lunde (1974), all of the Norwegian Geotechnical Institute. Its development represented a major contribution to the subject of rock mass classification for a number of reasons: the system was proposed based on the analysis of 212 tunnel case histories from Scandinavia, it is a quantitative classification system and it is an engineering system facilitating the design of tunnel supports. The Q system is based on a numerical assessment of the rock mass quality using six different parameters:

1) Rock quality designation
2) Joint set number
3) Joint roughness number
4) Joint alternation number
5) Joint water reduction number
6) Stress reduction factor

These six parameters are combined to express the ground quality with respect to stability and rock support in underground opening in the following equation:

$$Q = \frac{RQD}{Jn} \cdot \frac{Jr}{Ja} \cdot \frac{Jw}{SRF} \tag{2.1}$$

where RQD is rock quality designation, J_n is joint set number, J_r is joint roughness number, J_a is joint alternation number, J_w is joint water reduction number and SRF is stress reduction factor. The rock quality can range from Q = 0.001 to Q = 1,000 on a logarithmic rock mass quality scale.

Geol, Jethwa, and Paithankar (1995) suggested the parameter (Q_N) for stress free form Q. In order to calculate Q_N, SRF is taken 1 and the equation becomes:

$$Q_N = \frac{RQD}{J_n} \cdot \frac{Jr}{Ja} \cdot J_w \tag{2.2}$$

Relating the Q index with the stability and support requirements of underground excavations, Barton et al. (1974) have defined an additional parameter that is called the equivalent dimension of excavation (D_e). This dimension is obtained by dividing the span, diameter or wall height of excavation by a quantity called the excavation support ratio (ESR). Hence:

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio, ESR}} \tag{2.3}$$

The value of ESR is the so-called excavation support ratio. It ranges between 0.5 and 5. For the diversion tunnel, the excavation support ratio (ESR) is defined as 1.6. The value of ESR is related to the intended use of the excavation and to the degree of security, which is influence on the support system to be installed to maintain the stability of the excavation. The equivalent dimension (D_e) plotted against the value of Q is used to define a number of support categories in a chart published in the original paper (Barton, et al., 1974). This chart has later been updated to directly give the support. Grimstad and Barton (1993) made another update to reflect the increasing use of steel fiber, reinforced shotcrete in underground excavation support. The Q-values and support are related to the total amount of support (temporary and permanent) in the roof. The diagram is based on numerous tunnel support cases.

The use of the Q classification system can be of considerable benefit during the feasibility and preliminary design stages of a project, when very little detailed information on the rock mass and, its stress and hydrologic characteristics is available (Palmstrom and Broch, 2006).

2.2.3 Rock Mass Index (RMi)

The rock mass index (RMi) was first presented by Palmstrom in 1995 and has been further developed and presented in several papers. It is a volumetric parameter indicating the approximate uniaxial compressive strength of a rock mass. The RMi value is applied as input for estimating rock support and input to other rock engineering methods (Palmstrom, 2009). The RMi system has some input parameters similar to those of the Q system. Thus, the joint and jointing features are almost the same.

The input parameters used can be determined by commonly used field observations and measurements. The RMi value can be calculated as follow:

For Jointed rock,

$$RMi = \sigma_c \times JP \tag{2.4}$$

where σ_c is uniaxial compressive strength of intact rock, JP is the jointing parameter combines by empirical relations JC (joint conditions) and V_b (block volume) in the following exponential equation derived from strength tests on large jointed rock samples:

$$JP = 0.2\sqrt{JC}\ V_b^{D} \qquad (D = 0.37\ JC^{-0.2}) \tag{2.5}$$

where JC = jR × jL/jA (jR = joint roughness, jA = joint alteration, and jL = joint length).

For massive rock,

$$RMi = \sigma_c \times f_\sigma \text{ (applied for cases where } f_\sigma > JP) \tag{2.6}$$

where f_σ is called the massivity parameter, given as $f_\sigma = \sigma_c\ (0.05/D_b)^{0.2}$ (D_b = block diameter). In most cases, $f_\sigma \approx 0.5$.

The RMi requires more calculations than the RMR and the Q system, but the spreadsheets have been developed, from which the RMi value and the type(s) and amount of rock support can be found directly. For the estimation of RMi value and RMi support design, RMi-calc., version 2 and RMi support, version 3.1 (Palmstrom, 2001) will be used in this research.

2.2.4 Geological Strength Index (GSI)

The geological strength index (GSI), introduced by Hoek, Kaiser, and Bawden (1995), is a system of rock mass characterization that has been developed in engineering rock mechanics to meet the need for reliable input data, particularly those related to rock mass properties required as inputs into numerical analysis or closed form solutions for designing tunnels, slopes or foundations in rocks. The rock mass characterization is straightforward and it is based upon the visual impression of the rock structure, in terms of blockiness, and the surface condition of the discontinuities indicated by joint roughness and alteration. The combination of these two parameters provides a practical basis for describing a wide range of rock mass types, with diversified rock structure ranging from very tightly interlocked strong rock fragments to heavily crushed rock masses. Based on the rock mass descriptions, the value of GSI is estimated from the contours.

Due to lack of the parameters to describe surface conditions of the discontinuities and the rock mass structure in the GSI system, two terms namely, structure rating, SR, based on volumetric joint count (J_v) and surface condition rating, SCR, estimated from the input parameters (e.g., roughness, weathering and infilling) were suggested by Sonmez and Ulusay (1999). The modified quantitative GSI system chart (Sonmez, 2001) is used in this research.

The basic input consists of estimates or measurements of the uniaxial compressive strength (σ_c) and a material constant (m_i) that is related to the frictional properties of the rock. Ideally, these basic properties should determined by laboratory testing as described by Hoek and Brown (1997) but, in many cases, the information is required before laboratory tests have been completed and the condition that the laboratory testing is not available. To meet this need, Marions and Hoek (2000) reproduced the tables that can be used to estimate values for these parameters.

2.3 Deere's Rock Quality Designation (RQD)

In 1964 Deere proposed a quantitative index of rock mass quality based upon core recovery by diamond drilling, but it was not until 1967 that the concept was presented for the first time in a published form Deere, Hendron, Patton, and Cording (1967). It has come to be very widely used and has been shown to be particularly useful in classifying rock masses for the selection of tunnel support.

The RQD is defined as the percentage of core recovered in intact pieces of 100 mm or more in length in the total length of a borehole (Deere, 1989). Hence:

$$\text{RQD (\%)} = 100 \times \frac{\text{Length of core in pieces} \geq 100 \text{ mm}}{\text{Length of borehole}} \tag{2.7}$$

Palmstrom (1982) has suggested that when core is unavailable, the RQD can be estimated from the number of joints (discontinuities) per unit volume with the following equation:

$$\text{RQD} = 115 - 3.3J_v \tag{2.8}$$

where J_v is the total number of joints per cubic meter (volumetric joint count). The RQD is used as a standard parameter in drill core logging and forms a basic element of the two major rock mass classification systems such as rock mass rating system (RMR) and NGI tunneling quality index (Q system).

2.4 Numerical Method

In order to evaluate the stress and deformation around the tunnel, numerical method (Phase^2 code), will be used. Phase^2, version 6.0, is a finite element program, developed by Rocscience (2007) and permits two-dimensional study of the non-linear deformation of rocks using Hoek-Brown failure criterion or Mohr-Coulomb failure criterion. In this program, automatic mesh around the tunnel is generated and, deformations and stresses are computed based on the elasto-plastic analysis. Convergence-confinement is a procedure that allows the load imposed on a support installed behind the face of tunnel to be estimated. The input parameters are unit weight of rock, Young’s modulus, Poisson’s ratio, uniaxial compressive strength of intact rock, in-situ stresses, Hoek-Brown constants or Mohr-Coulomb constants and tunnel radius.

2.5 Review of Papers

Basarir, Ozsan, and Karakus (2005) suggested that more reliable support design could be achieved by using the finite element method together with the empirical methods. A case study was carried out at the diversion tunnel project of Guledar dam site, which was located at the North of Ankara, Turkey. Based on the collected information in the field and rock properties determined in the laboratory, rock masses were characterized by means of rock mass classification systems (RMR, Q, RMi and GSI). These classification systems were also employed to estimate support requirements for the diversion tunnel. Convergence-confinement method was employed to perform stability analysis. Based upon the performed stability analysis it was decided to use the support systems recommended by rock mass classification systems. Finite element analysis was utilized to assess the stability of the tunnel and evaluate the performance of support recommended by the empirical methods. The strength parameters necessary for finite element analysis were estimated from the empirical methods and input into the finite element code (Phase^2).

The empirical methods recommend the utilization of bolt and shotcrete as support elements for sandstone formation at Guledar diversion tunnel project. Convergence-confinement and numerical methods showed that small deformations occur and a limited plastic zone develops around the tunnel. When the recommended support systems by the empirical methods were applied, these yielded elements disappeared in the finite element analysis. The empirical methods indicate that substantial support was necessary for diabase formation and both convergence-confinement and numerical methods agreed that the size of the plastic zone and the deformations increase and reach their maximum values for this formation. However, after installation of support elements recommended by the empirical method, the finite element analysis showed that there was not any yielded element and plastic zone around the tunnel. The results proved that the empirical and numerical methods agree with each other. Thus, it is suggested that when designing a support system for a tunnel driven in rock mass, empirical and numerical methods are to be used together (Basarir, et al., 2005). However, the validity of the proposed support system, obtained from combination of empirical and numerical modeling should be verified by comparing predictions with actual measurements during construction.

Ozsan and Basaria (2002) carried out the support capacity estimation of the diversion tunnel at Urus dam site located in the central part of Turkey on the Suveri River. The project area is in weathered tuff and weak zone. Tunneling in weak rock requires some special considerations, since misjudgment in support design results in costly failures. There are several ways of estimating rock support pressure and selecting support. However, all systems suffer from their characteristic limitations in achieving objectives. Thus, it is more useful to use different methods for estimating support pressure and type of support. The support pressure (p_i) was established by three different methods. These methods are the (1) empirical methods based on rock mass rating system (RMR) and rock mass quality index (Q system), (2) ground support interaction analysis (GSIA) and (3) numerical methods, namely, Phase^2 finite element program. Rock masses were characterized in terms of RSR, RMR, Q system and GSI. Finally, the required support system was proposed and evaluated by different methods in the highly weathered tuff and weak zone of the diversion tunnel.

Oo (2003) evaluated the rock mass quality and estimated support design of Ye Nwe diversion tunnel by using rock mass classification systems. The project is located on Ye Nwe River, near Myochaung village, Kyauktaga Township, Bago Diversion, Myanmar. The four methods of engineering rock mass classification schemes, RMR, Q, RMi and GSI, are independently applied to assess the geological input data with practical experience and engineering judgment. According to the engineering geological data obtained from those characterizations, the rock mass quality is defined as the assigned rating values, which enable to evaluate the in-situ rock mass strength, deformation modulus, Mohr-coulomb parameters and Hoek-Brown parameters. The rock mass strength was estimated by the empirical method. The required initial rock support for each structural region have been recommended by RMR, Q and RMi, and then correlated with each other.

Rasouli (2009) studied at Garmi Chay Diversion Tunnels Project in the northwest of Iran focusing on stabilization analysis and support design. The diversion tunnel of the dam has a diameter of 5.5 m and a length of 420 m and was driven in slightly to highly weathered micaschist and trachy andesite rock units. The tunnel alignment was divided into three geotechnical zones. For every zone, support capacity of rock masses was evaluated by means of empirical and numerical methods. The rock mass classification systems (RMR, Q, GSI, RSR, SRC and RMi), the convergence-confinement method and a 2D finite element computer software, Phase^2 were used for empirical and numerical method, respectively. According to the results acquired from these methods, some stability problems were expected in the tunnel.

The support system, suggested by empirical method, was applied and its performance was evaluated by means of numerical modeling. After installation, the support suggested by Phase^2 program, the thickness of plastic zone and deformations around the tunnel decreased significantly. Consequently, the agreement of these methods with each other was resulted and using combination of them was recommended for more reliable support design (Rasouli, 2009).

CHAPTER III

GEOLOGICAL DATA COLLECTION

3.1 Introduction

The careful exploration of local engineering geological conditions is an important phase of investigation for tunneling. It is a prerequisite to the successful and optimized design of engineering structures and underground excavations. Accordingly, a site investigation can foresee and provide against difficulties that may arise during construction because of the uncertainties of the ground and/or other local conditions. Investigation should not cease once construction begins. It is essential that the prediction of ground conditions that constitute the basic design assumption be checked as construction proceeds and designs modified accordingly if conditions are revealed to be different from those predicted. In the case of the Manipura diversion tunnel constructions, four vertical boreholes along the tunnel alignment with total depth of 239 m were drilled by Engineering Geology Office 2, Irrigation Department, Ministry of Agriculture and Irrigation, Myanmar, as a preliminary geological investigation in 2004 and continuous investigations have been performed. TECHO Corporation Pte. Ltd. carried out the probe drilling investigations (horizontal drilling inside the tunnel) as a contract in 2009 (Figure 3.1). The geotechnical parameters are evaluated base on the exploratory data, field observations and laboratory test results.

Figure 3.1 Probe drilling investigation by TECHO Corporation Pte. Ltd.

3.2 Geology of Project Area

The tunnels are in the Chin Flysch aged of Tertiary as a part of the Rakhine-Chin-Naga Hill ranges (Win and Aung, 2007). Most of the main Rakhine-Chin-Naga Hill ranges are a thick monotonous series of apparently unfossiliferous marine flysch-type sediments including slaty shales, phyllite, slates and poorly-graded calcareous sandstones.

Rock sequence in the project area, known as the Chin Flysch, is folded tightly, and even isoclinally. Quartz and calcite veins are very common. The sequence is predominantly argillaceous (slate), consisting mainly of yellowish grey to dark grey, hard, locally ferruginous and calcareous. Slate, the major rock unit, is fine grained metamorphic rock derived mostly from shale. Approximately 20 to 25% of the sequence consists of hard bedded greywackes sandstone, usually find-grained and locally micaceous and calcareous.

3.3 Engineering Geology

Geological data collection is carried out to classify the rock mass as accurately as possible. It is necessary that every effort should be made to obtain complete characteristics of the rock mass at an early stage of the project. The given sufficient warning of a potential problem, the engineer can usually provide a solution by supporting or reinforcing the rock mass around the opening or by providing drainage or diverting accumulations of ground water. Therefore, the integrated geological data can assist in the design solution and anticipate any unfavorable geological condition, which can give rise to problem during the excavation of the openings.

Engineering geological descriptions of the rock masses are based on the procedures suggested by ISRM (1981). In this study, special emphasis is given to the characteristics of the discontinuities and to the degree of weathering, each of which has an influence on the engineering properties of rock mass. Folds and joints are the most dominant structural discontinuities observed in the study area. The major trend of the joint is 320/60 (strike/dip) and the tunnel direction is N60E (Win and Aung, 2007). The strike of the joints and tunnel axis are nearly perpendicular. The area shows varying weathering grades of rock. Near ground surface, rocks are highly to completely weathered extending to a depth of 20 – 30 m. The geological cross-section along the tunnel alignment is drawn based on the borehole results, surface exposures and time domain electromagnetic survey (TDEM) map. The tunnel alignment is divided into three distinct zones of rock mass : zone 1 : moderately to highly weathered slate; zone 2 : slightly to moderately weathered slate; and zone 3 : slightly weathered slate with alternation of greywacke sandstone band (Figure 3.2). Each of which has different engineering geological properties and lithologic types. The RQD values are defined from borehole data based on the equation proposed by Deere (1989) (equation 2.7 in chapter 2).

Zone 1 is slate, which is moderately to highly weathered. The color is yellowish grey to grey. Average RQD for this zone is 16% and average joint spacing is 0.3 m. The persistence of joints varies 1-3 m. Joints surfaces are generally slightly rough and undulating. Discontinuities aperture is less than 0.1 mm and mostly filled by soft filling material with clay content. The groundwater condition is wet. This zone is 46.39% of the total length of diversion tunnel.

Zone 2 is slate, which is slightly to moderately weathered. They are light grey to grey in color. Average RQD for this zone is 49.86% and average joint spacing is 0.5 m. The persistence of joints varies 1-3 m. Joints surfaces are generally slightly rough and undulating. Discontinuities aperture is less than 0.1 mm. Quartz and calcite veins are observed. The groundwater condition is wet. This zone is 27.26% of the total length of diversion tunnel.

Zone 3 is slate, alternation with sandstone band (greywacke), slightly weathered, hard and compacted. Greywacke sandstones are hard, dark grey in color and fine grained. Most of rock units in this zone are dark grey in color. Average RQD for this zone is 60.80% and average joint spacing is 0.6 m. The persistence of joints is 1-3 m. Joints surfaces are generally slightly rough and undulating. Apertures in discontinuities are less than 0.1 mm. Quartz and calcite veins are observed and some of joints are closed and tight. The groundwater condition is wet. This zone is 26.35% of the total length of diversion tunnel.

Total 21 different sections are classified based on their locally input variables in terms of engineering geological and geotechnical parameters and the induced overburden stress. A section along the tunnel alignment is given in Figure 3.2.

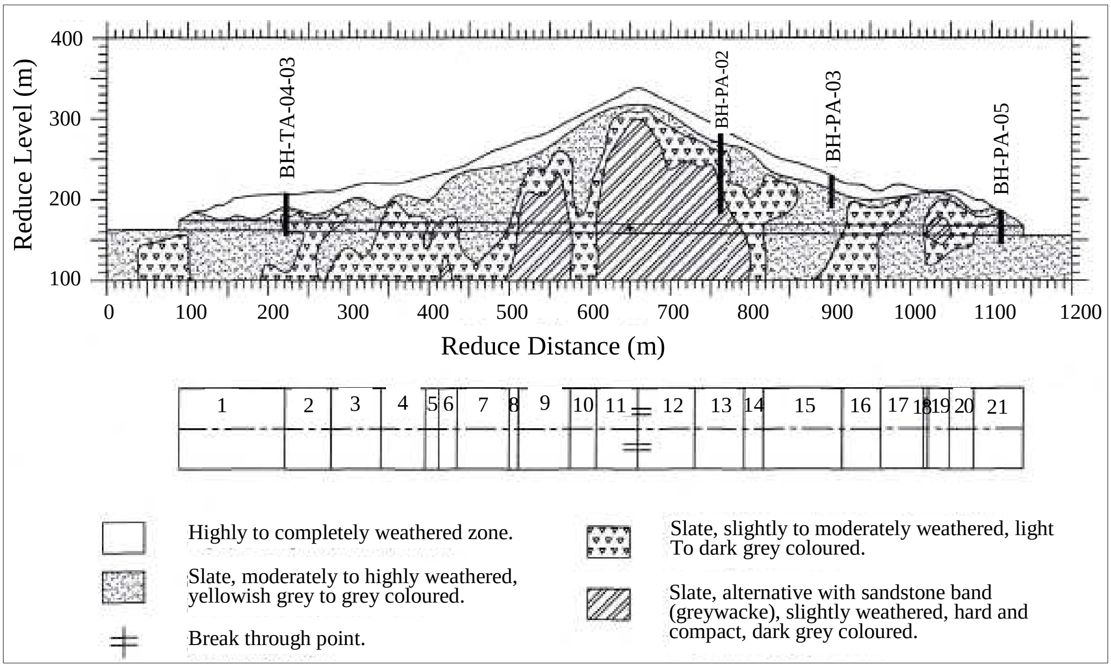

Figure 3.2 Geological cross-section of the Manipura diversion tunnel alignment.

3.4 Laboratory Testing

Laboratory experiments were carried out to determine the physical and mechanical properties of intact rock including unit weight, Young's modulus, Poisson's ratio and uniaxial compressive strength. Rock core samples were collected from vertical and horizontal borehole drilling investigations. All laboratory tests were conducted in accordance with the relevant ASTM standard (ASTM D 7012-07) and the ISRM suggested methods (ISRM, 1981). Test results are presented in Table 3.1.

Table 3.1 Physical and mechanical properties of intact rocks.

No.	Parameters, symbol, unit	Zone 1	Zone 2	Zone 3
1	Uniaxial compressive strength, σ_c, MPa	16.71	26.22	41.95
2	Unit weight, γ, kN/m^3	26.59	27.76	28.94
3	Young's modulus, E_i, GPa	10.34	21.04	22.83
4	Poisson's ratio, υ	0.35	0.34	0.30
5	Hoek and Brown parameter, m_i	7	7	7
6	Hoek and Brown parameter, s_i	1	1	1

CHAPTER IV

ROCK MASS CHARACTERIZATIONS

4.1 Introduction

This chapter describes the characterizations of rock mass around the proposed tunnels by using rock mass classification systems and comparison of the rock mass classification results. Rock mass classification schemes have been developing for over 100 years since Ritter (1879) attempted to formalize an empirical approach to tunnel design, in particular for determining support requirements. Rock mass classification systems evaluate the quality and expected behavior of rock masses based on the most important parameters that influence the rock mass quality. Therefore, the rock mass characterization has been performed to access the rock mass quality in accordance with the existing engineering rock mass classification systems.

Rock masses along the tunnel alignment are classified by four individual rock mass classification systems including rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi) and geological strength index (GSI). The required input parameters and engineering geological properties for the rock mass classification systems are described in chapter 3.

4.2 Rock Mass Rating System (RMR)

The rock mass rating system was initially developed by Bieniawski (1973), otherwise known as geomechanics classification system. It was modified over the years as more case histories, became available and to conform to international standards and procedures (Bieniawski, 1979). In this research, the 1989 version of the classification table has been used. Uniaxial compressive strength of intact rock (UCS), rock quality designation (RQD), discontinuity spacing, discontinuity conditions, ground water conditions and discontinuity orientation are the utilized parameters of rock mass rating system. Based on rock mass rating system, the rating value and class of rock mass along the diversion tunnel alignment are shown in Table 4.1.

The results show that zone 1 is poor and, zones 2 and 3 are fair. The RMR rating values for sections 1, 3, 5, and 7 of zone 1 are 36, and sections 15, 17, and 21 of zone 1 are 31. This is due the effects of discontinuity orientation in the tunnel. The two tunnels have been driven from both sides (inlet and outlet). The break through point is between the rock mass sections 11 and 12. The major trend of the joint is 320/60 (strike/dip) and the tunnel direction is N60E (Win and Aung, 2007). The strike of the joints and tunnel axis are nearly perpendicular. The tunnels driven from the inlet are under very favorable condition and driven from the outlet are under favorable condition. Therefore, zones 2 and 3 also have RMR rating values of 48 and 43, and 52 and 47, respectively.

Table 4.1 Rock mass rating value and class of rock mass along the Manipura diversion tunnel alignment.

No.	Zone	Section	RMR rating value	RMR Class	Description
1	1	1	36	IV	Poor
2		3			
3		5			
4		7			
5		15	31		
6		17			
7		21			
8	2	2	48	III	Fair
9		4			
10		6			
11		8			
12		10			
13		14	43		
14		16			
15		18			
16		20			
17	3	9	52	III	Fair
18		11			
19		12	47		
20		13			
21		19			

4.3 NGI Tunneling Quality Index (Q system)

The Q system proposed by Barton et al. (1974) is a numerical description of the rock mass quality with respect to the tunnel stability and consists of six parameters, which are estimated from geological mapping, in-situ measurements and drilled core loggings. These six parameters are (1) rock quality designation (RQD), (2) joint set number (J_n), (3) joint roughness number (J_r), (4) joint alternation number (J_a), (5) joint water reduction number (J_w) and (6) stress reduction factor (SRF). The numerical value of Q index is defined by a function of these six parameters (equation 2.1 in chapter 2). Geol et al. (1995) suggested the parameter (Q_N) for stress free form

Q. In order to calculate Q_N, SRF is taken 1 (equation 2.2 in chapter 2). In 2002, Barton improved the value of Q to Q_c (normalization of Q value). The Q index value and class of rock mass classified by Q system are presented in Table 4.2.

Table 4.2 Q index value and class of rock mass along the Manipura diversion tunnel alignment together with Q_N value and Q_c value.

<table>
<tr><th>No.</th><th>Zone</th><th>Section</th><th>Q index value</th><th>Q Class</th><th>Description</th><th>Q_N</th><th>Q_c</th></tr>
<tr><td>1</td><td rowspan="7">1</td><td>1</td><td rowspan="2">0.2667</td><td rowspan="7">E</td><td rowspan="7">Very Poor</td><td rowspan="7">1.3333</td><td rowspan="2">0.0446</td></tr>
<tr><td>2</td><td>3</td></tr>
<tr><td>3</td><td>5</td><td rowspan="4">0.5333</td><td rowspan="4">0.0891</td></tr>
<tr><td>4</td><td>7</td></tr>
<tr><td>5</td><td>15</td></tr>
<tr><td>6</td><td>17</td></tr>
<tr><td>7</td><td>21</td><td>0.2667</td><td>0.0446</td></tr>
<tr><td>8</td><td rowspan="9">2</td><td>2</td><td>1.2465</td><td rowspan="9">D</td><td rowspan="9">Poor</td><td rowspan="9">6.2325</td><td>0.3268</td></tr>
<tr><td>9</td><td>4</td><td rowspan="7">2.4930</td><td rowspan="7">0.6537</td></tr>
<tr><td>10</td><td>6</td></tr>
<tr><td>11</td><td>8</td></tr>
<tr><td>12</td><td>10</td></tr>
<tr><td>13</td><td>14</td></tr>
<tr><td>14</td><td>16</td></tr>
<tr><td>15</td><td>18</td></tr>
<tr><td>16</td><td>20</td><td>1.2465</td><td>0.3268</td></tr>
<tr><td>17</td><td rowspan="5">3</td><td>9</td><td rowspan="5">3.0400</td><td rowspan="5">D</td><td rowspan="5">Poor</td><td rowspan="5">7.6000</td><td rowspan="5">1.2643</td></tr>
<tr><td>18</td><td>11</td></tr>
<tr><td>19</td><td>12</td></tr>
<tr><td>20</td><td>13</td></tr>
<tr><td>21</td><td>19</td></tr>
</table>

The results indicate that zone 1 is classified as very poor quality and, zones 2 and 3 are poor quality. The Q index values for sections 1, 3, and 21 of zone 1 are 0.2667, sections 5, 7, 15, and 17 of zone 1 are 0.5333, sections 2 and 20 of zone 2 are 1.2465, sections 4, 6, 8, 10, 14, 16, and 18 of zone 2 are 2.4930, and sections 9, 11, 12, 13, and 19 of zone 3 are 3.0400. Even though sections are in the same zone, the Q index values are different. This is due to the value of stress reduction factor (SRF). Stress reduction factor varies according to depth of excavation.

4.4 Rock Mass Index (RMi)

Palmstrom (1995) proposed rock mass index (RMi) for general characterization and it has been developed over the years. For the jointed rock, RMi is defined as the multiplication of the uniaxial compressive strength of intact rock (σ_c) and the reducing effect of joint penetrating (JP) of rock mass (equation 2.4 in chapter 2). JP is the jointing parameter combined by the empirical relations jC (joint conditions) and V_b (block volume) as shown in equation 2.5 in chapter 2. Block volume (V_b) was estimated by the following equation proposed by Palmstrom (1995):

$$V_b = \beta \times J_v^{-3} \qquad (4.1)$$

where J_v is the volumetric joint count and β is the block shape factor.

Equations 2.4 through 2.6 in chapter 2 can be used to estimate the RMi value of the rock mass. The RMi requires more calculations than the RMR and the Q system, but the spreadsheets have been developed. The RMi-calc., version 2 and RMi support, version 3.1 have been used in this research. The RMi index value and class of the rock mass along the Manipura diversion tunnel alignment are described in Table 4.3.

The index values of zones 1, 2, and 3 are 0.2858, 0.9310, and 2.5153, respectively. Under the evaluated RMi index values, zones 1 and 2 are related to low quality and zone 3 is medium quality.

Table 4.3 RMi index value and class of rock mass along the Manipura diversion tunnel alignment.

No.	Zone	Section	RMi index value	Description
1	1	1	0.2858	Low
2		3		
3		5		
4		7		
5		15		
6		17		
7		21		
8	2	2	0.9310	Low
9		4		
10		6		
11		8		
12		10		
13		14		
14		16		
15		18		
16		20		
17	3	9	2.5153	Medium
18		11		
19		12		
20		13		
21		19		

4.5 Geological Strength Index (GSI)

The geological strength index (GSI) was proposed by Hoek et al. (1995). It has been developed in engineering rock mechanics to meet the need for reliable input data, particularly those related to rock mass properties required as inputs into numerical analysis or closed form solutions for designing tunnels, slopes or foundations in rock. The GSI is based on the appearance of rock mass and its structure (e.g very good, good) and the structure of the rock mass (eg. blocky, disturbed and disintegrated). Sonmez and Ulusay (1999) proposed two terms namely, structural rating (SR) and surface condition rating (SCR). Structural rating (SR) is based on volumetric joint count (J_v) and surface condition rating (SCR) is estimated from the input parameters including roughness, weathering and infilling of discontinuities.

The modified quantitative GSI table (Sonmez, 2001) is used in this research. The GSI index value and class of rock mass along the Manipura diversion tunnel alignment are shown in Table 4.4. Based on the modified quantitative GSI table developed by Sonmez (2001), the GSI values of zones 1, 2, and 3 are 28, 38, and 44, respectively. As a result, zone 1 is poor, zone 2 is fair, and zone 3 is good quality.

Table 4.4 GSI index value and class of rock mass along the Manipura diversion tunnel alignment.

No.	Zone	Section	GSI index value	Description
1		1		
2		3		
3		5		
4	1	7	28	Poor
5		15		
6		17		
7		21		
8		2		
9		4		
10		6		
11		8		
12	2	10	38	Fair
13		14		
14		16		
15		18		
16		20		
17		9		
18		11		
19	3	12	44	Good
20		13		
21		19		

4.6 Comparison of the Rock Mass Classification Results from Four Different Rock Mass Classification Systems

Bieniawski (1989) modified the rock mass rating classification table. There are five categories of rock mass class : (1) very good rock, (2) good rock, (3) fair rock, (4) poor rock and (5) very poor rock. These rock mass classes are determined based on five parameters of rock mass rating system.

In NGI tunneling quality index (Q system), there are seven categories of rock mass class based on Q index value: (1) A : exceptionally good, extremely good and very good; (2) B : good; (3) C : fair; (4) D : poor; (5) E : very poor; (6) F : extremely poor; and (7) G : exceptionally poor. These rock mass classes are determined based on six parameters.

The rock mass index (RMi) categorizes three rock mass classes : (1) low; (2) medium; (3) high; based on the uniaxial compressive strength of intact rock and the reducing effect of joint penetrating of rock mass. The geological strength index (GSI) categorizes five rock mass classes based on surface condition rating (SCR). These five rock mass classes are the same to those of rock mass rating system.

In rock mass rating system, there is no input parameter for rock stresses but stresses up to 25 MPa are included in the estimated RMR value. The number of joint set is considered indirectly in rock mass rating classification system. The Q system considers being a function of only three parameters which are crude measures of block size, inter-block shear strength and active stress. The RMi system has similar input parameters to those of Q system. Jointing parameters are almost the same. The GSI system classifies the rock mass class based on the surface condition rating such as roughness rating, weathering rating and infilling rating. All systems consider the condition of discontinuities. The RMR and Q systems consider ground water condition which is indirectly considered in the RMi and GSI systems. The utilized parameters of the four different rock mass classification systems are varied. Therefore, they classify different rock mass classes in accordance with their utilized parameters. The rock mass classes along the Manipura diversion tunnel alignment classified by the four rock mass classification systems are summarized in Table 4.5. Based on the results, zone 1 is generally identified as poor rock, zone 2 is fair rock and zone 3 is good rock.

Table 4.5 Summary of the rock mass classes from different rock mass classification systems.

<table>
<tr><th>No.</th><th>Zone</th><th>Section</th><th>RMR Class</th><th>Q Class</th><th>RMi Class</th><th>GSI Class</th></tr>
<tr><td>1</td><td rowspan="7">1</td><td>1</td><td rowspan="7">Poor</td><td rowspan="7">Very Poor</td><td rowspan="7">Low</td><td rowspan="7">Poor</td></tr>
<tr><td>2</td><td>3</td></tr>
<tr><td>3</td><td>5</td></tr>
<tr><td>4</td><td>7</td></tr>
<tr><td>5</td><td>15</td></tr>
<tr><td>6</td><td>17</td></tr>
<tr><td>7</td><td>21</td></tr>
<tr><td>8</td><td rowspan="9">2</td><td>2</td><td rowspan="9">Fair</td><td>Very Poor</td><td rowspan="9">Low</td><td rowspan="9">Fair</td></tr>
<tr><td>9</td><td>4</td><td rowspan="7">Poor</td></tr>
<tr><td>10</td><td>6</td></tr>
<tr><td>11</td><td>8</td></tr>
<tr><td>12</td><td>10</td></tr>
<tr><td>13</td><td>14</td></tr>
<tr><td>14</td><td>16</td></tr>
<tr><td>15</td><td>18</td></tr>
<tr><td>16</td><td>20</td><td>Very Poor</td></tr>
<tr><td>17</td><td rowspan="5">3</td><td>9</td><td rowspan="5">Fair</td><td rowspan="5">Poor</td><td rowspan="5">Medium</td><td rowspan="5">Good</td></tr>
<tr><td>18</td><td>11</td></tr>
<tr><td>19</td><td>12</td></tr>
<tr><td>20</td><td>13</td></tr>
<tr><td>21</td><td>19</td></tr>
</table>

CHAPTER V

GEOTECHNICAL ROCK MASS PARAMETER ESTIMATION AND STABILITY ANALYSIS

5.1 Introduction

This chapter describes the estimation of geotechnical rock mass parameters and stability analysis. The geotechnical rock mass parameters are evaluated by empirical equations which are developed by many researchers based on the rock mass classification systems. The stability of tunnels is evaluated in terms of stand-up time, estimation of maximum unsupported span and factor of safety for all sections of tunnel alignment.

5.2 Geotechnical Rock Mass Parameter Estimation

Rock mass properties such as Hoek and Brown constants, deformation modulus of rock mass and strength of rock mass are important parameters for the stability analysis and support design of tunnel. Reliable input parameters to finite element method can produce meaningful calculations and feasible support design. Field tests to determine some parameters directly are time consuming and expensive. Consequently, several authors have proposed empirical relationships for estimating the value of isotropic rock mass parameters based on empirical rock mass classification schemes.

5.2.1 Rock Mass Deformation Modulus

In many designs for the reliable support system of a tunnel, the deformations of the rock mass surrounding the tunnel are important and a numerical analysis of these deformations requires an estimate of the rock mass deformation modulus. In-situ determination of the deformation modulus of rock mass is costly and often very difficult. Thus, empirical methods are generally used in estimating of rock mass deformation modulus. Based on the RMR rating value, many researchers have proposed different empirical equations to calculate the rock mass deformation modulus. The following describes some equations:

Bieniawski (1978) has defined E_m as:

$$E_m = 2RMR-100 \text{ (GPa)} \qquad \text{For RMR} > 50 \qquad (5.1)$$

Serafim and Pereira (1983) have proposed:

$$E_m = 10^{\left(\frac{RMR-10}{40}\right)} \text{ (GPa)} \qquad \text{For RMR} < 50 \qquad (5.2)$$

Read, Richards, and Perrin (1999) has proposed the following equation:

$$E_m = 0.1\left(\frac{RMR}{10}\right)^3 \text{ (GPa)} \qquad (5.3)$$

where E_m is the deformation modulus of the rock mass.

Based on the NGI tunneling quality index (Q system), many researchers proposed several equations to estimate rock mass deformation modulus. Simple equations have been presented from the Q system as follows:

Grimstad and Barton (1993) have proposed the equation for Q > 1:

$$E_m = 25 \log Q \qquad \text{(GPa)} \tag{5.4}$$

E_m was expressed as below by Barton (2002):

$$E_m = 10 Q_c^{1/3} = 10 \left(Q \times \frac{\sigma_c}{100}\right)^{1/3} \qquad \text{(GPa)} \tag{5.5}$$

where Q_c is the normalization of Q-value and σ_c is uniaxial compressive strength of intact rock.

From rock mass index (RMi), Palmstrom (1995) proposed the equation for RMi > 0.1,

$$E_m = 5.6 \, RMi^{0.375} \qquad \text{(GPa)} \tag{5.6}$$

Using the geological strength index (GSI), provided the uniaxial compressive strength of intact rock is known the rock mass deformation modulus E_m for $\sigma_c \leq 100$ MPa is estimated in GPa from the following equation (Hoek, Carranza-Torres, and Corkum, 2002).

$$E_m \text{ (GPa)} = \left(1 - \frac{D}{2}\right) \sqrt{\frac{\sigma_c}{100}} \times 10^{\left(\frac{GSI-10}{40}\right)} \tag{5.7}$$

For σ_c > 100 MPa, use equitation 5.8.

$$E_m \text{ (GPa)} = (1- \frac{D}{2}) \times 10^{\left(\frac{GSI-10}{40}\right)} \qquad (5.8)$$

The original equation proposed by Hoek and Brown has been modified, by the inclusion of the factor D, to allow for the effects of blast damage and stress relaxation. In the case of Manipura diversion tunnel constructions, control blasting method is used. Therefore, the value of D is zero. The results of the deformation modulus of rock mass for all sections of the Manipura diversion tunnels calculated from above mentioned empirical equations are presented in Table 5.1.

Table 5.1 Calculated deformation modulus of rock mass (E_m) for all sections of the Manipura diversion tunnels.

<table>
<tr><th rowspan="2">No.</th><th rowspan="2">Zone</th><th rowspan="2">Section</th><th colspan="3">From RMR</th><th colspan="2">From Q</th><th>From RMi</th><th>From GSI</th><th rowspan="2">Avg.</th><th rowspan="2">SD</th></tr>
<tr><th>Eq. (5.1)</th><th>Eq. (5.2)</th><th>Eq. (5.3)</th><th>Eq. (5.4)</th><th>Eq. (5.5)</th><th>Eq. (5.6)</th><th>Eq. (5.7)</th></tr>
<tr><td>1</td><td rowspan="7">1</td><td>1</td><td rowspan="7"></td><td rowspan="4">4.47</td><td rowspan="4">4.67</td><td rowspan="7"></td><td rowspan="2">3.55</td><td rowspan="7">3.50</td><td rowspan="7">1.15</td><td rowspan="2">3.47</td><td rowspan="2">1.40</td></tr>
<tr><td>2</td><td>3</td></tr>
<tr><td>3</td><td>5</td><td rowspan="4">4.47</td><td rowspan="2">3.65</td><td rowspan="2">1.47</td></tr>
<tr><td>4</td><td>7</td></tr>
<tr><td>5</td><td>15</td><td rowspan="3">3.35</td><td rowspan="3">2.98</td><td rowspan="2">3.09</td><td rowspan="2">1.21</td></tr>
<tr><td>6</td><td>17</td></tr>
<tr><td>7</td><td>21</td><td>3.55</td><td>2.91</td><td>1.01</td></tr>
<tr><td>8</td><td rowspan="9">2</td><td>2</td><td rowspan="9"></td><td rowspan="5">8.91</td><td rowspan="5">11.06</td><td>2.39</td><td>6.89</td><td rowspan="9">5.45</td><td rowspan="9">2.57</td><td>6.21</td><td>3.46</td></tr>
<tr><td>9</td><td>4</td><td rowspan="7">9.92</td><td rowspan="7">8.68</td><td rowspan="4">7.76</td><td rowspan="4">3.16</td></tr>
<tr><td>10</td><td>6</td></tr>
<tr><td>11</td><td>8</td></tr>
<tr><td>12</td><td>10</td></tr>
<tr><td>13</td><td>14</td><td rowspan="4">6.68</td><td rowspan="4">7.95</td><td rowspan="3">6.87</td><td rowspan="3">2.62</td></tr>
<tr><td>14</td><td>16</td></tr>
<tr><td>15</td><td>18</td></tr>
<tr><td>16</td><td>20</td><td>2.39</td><td>6.89</td><td>5.32</td><td>2.34</td></tr>
<tr><td>17</td><td rowspan="5">3</td><td>9</td><td rowspan="2">4.00</td><td rowspan="2"></td><td rowspan="2">14.06</td><td rowspan="5">12.07</td><td rowspan="5">10.81</td><td rowspan="5">7.91</td><td rowspan="5">4.59</td><td rowspan="2">8.91</td><td rowspan="2">4.10</td></tr>
<tr><td>18</td><td>11</td></tr>
<tr><td>19</td><td>12</td><td rowspan="3"></td><td rowspan="3">8.41</td><td rowspan="3">10.38</td><td rowspan="3">9.03</td><td rowspan="3">2.67</td></tr>
<tr><td>20</td><td>13</td></tr>
<tr><td>21</td><td>19</td></tr>
</table>

5.2.2 Hoek and Brown Parameters

The Hoek and Brown failure criterion for rock masses is widely accepted and has been applied in a large number of projects around the world. Hoek and Brown failure criterion for rock masses uses 'm_j' and 's_j' constants. Some empirical equations based on the empirical methods are used to calculate those constants as follows:

Hoek and Brown (1988) proposed a set of relations between the RMR and the parameters ‘m_j’ and ‘s_j’.

For disturbed rock mass,

$$m_j = m_i \exp\left(\frac{RMR - 100}{14}\right) \tag{5.9}$$

$$s_j = \exp\left(\frac{RMR - 100}{6}\right) \tag{5.10}$$

For undisturbed rock mass,

$$m_j = m_i \exp\left(\frac{RMR - 100}{28}\right) \tag{5.11}$$

$$s_j = \exp\left(\frac{RMR - 100}{9}\right) \tag{5.12}$$

Singh, Viladkar, Samadhiya, and Mehrota (1997) has described the following approximations to calculate m_j and s_j constants for tunnels:

$$\frac{m_j}{m_i} = 0.135\, Q_N^{1/3} \tag{5.13}$$

$$s_j = 0.002\, Q_N \tag{5.14}$$

where, Q_N is the stress free from Q, shown in equation 2.2 in chapter 2.

Palmstorm (1995) offered a method to calculate the Hoek and Brown constants 'm_j' and 's_j' as follow:

$$m_j = m_i \, JP^{0.64} \tag{5.15}$$

$$m_j = m_i \, JP^{0.857} \tag{5.16}$$

$$s_j = JP^{2.0} \tag{5.17}$$

where JP is the jointing parameter combines by empirical relations JC (joint conditions) and V_b (block volume) as described in equation 2.5 in chapter 2.

Hoek et al. (2002) expressed as m_j, a reduced value of material constant m_i and, s_j. They are constants for the rock mass given by the following relationships:

$$m_j = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{5.18}$$

$$s_j = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{5.19}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \tag{5.20}$$

where D is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in-situ rock masses to 1 for very disturbed rock masses. For the control blasting method, D value is 0. The calculated Hoek and Brown constants of rock mass, 'm_j' and 's_j', for all sections of Manipura diversion tunnels are shown in Tables 5.2 and 5.3, respectively, together with their average value and standard deviation.

Table 5.2 Calculated Hoek and Brown constant of rock mass (m_j) for all sections of the Manipura diversion tunnels.

<table>
<tr><th rowspan="2">No.</th><th rowspan="2">Zone</th><th rowspan="2">Section</th><th>From RMR</th><th>From Q</th><th>From RMi</th><th>From GSI</th><th rowspan="2">Avg.</th><th rowspan="2">SD</th></tr>
<tr><th>Eq. (5.11)</th><th>Eq. (5.13)</th><th>Eq. (5.15)</th><th>Eq. (5.18)</th></tr>
<tr><td>1</td><td rowspan="7">1</td><td>1</td><td rowspan="4">0.7119</td><td rowspan="7">1.0401</td><td rowspan="7">0.5180</td><td rowspan="7">0.5350</td><td rowspan="4">0.7013</td><td rowspan="4">0.2423</td></tr>
<tr><td>2</td><td>3</td></tr>
<tr><td>3</td><td>5</td></tr>
<tr><td>4</td><td>7</td></tr>
<tr><td>5</td><td>15</td><td rowspan="3">0.5955</td><td rowspan="3">0.6722</td><td rowspan="3">0.2475</td></tr>
<tr><td>6</td><td>17</td></tr>
<tr><td>7</td><td>21</td></tr>
<tr><td>8</td><td rowspan="9">2</td><td>2</td><td rowspan="5">1.0928</td><td rowspan="9">1.7391</td><td rowspan="9">0.8266</td><td rowspan="9">0.7646</td><td rowspan="5">1.1058</td><td rowspan="5">0.4456</td></tr>
<tr><td>9</td><td>4</td></tr>
<tr><td>10</td><td>6</td></tr>
<tr><td>11</td><td>8</td></tr>
<tr><td>12</td><td>10</td></tr>
<tr><td>13</td><td>14</td><td rowspan="4">0.9141</td><td rowspan="4">1.0611</td><td rowspan="4">0.4561</td></tr>
<tr><td>14</td><td>16</td></tr>
<tr><td>15</td><td>18</td></tr>
<tr><td>16</td><td>20</td></tr>
<tr><td>17</td><td rowspan="5">3</td><td>9</td><td rowspan="2">1.2606</td><td rowspan="5">1.8580</td><td rowspan="5">1.1559</td><td rowspan="5">0.9473</td><td rowspan="2">1.3055</td><td rowspan="2">0.3907</td></tr>
<tr><td>18</td><td>11</td></tr>
<tr><td>19</td><td>12</td><td rowspan="3">1.0545</td><td rowspan="3">1.2539</td><td rowspan="3">0.4116</td></tr>
<tr><td>20</td><td>13</td></tr>
<tr><td>21</td><td>19</td></tr>
</table>

Table 5.3 Calculated Hoek and Brown constant (s_j) for all sections of the Manipura diversion tunnels.

<table>
<tr><th rowspan="2">No.</th><th rowspan="2">Zone</th><th rowspan="2">Section</th><th>From RMR</th><th>From Q</th><th>From RMi</th><th>From GSI</th><th rowspan="2">Avg.</th><th rowspan="2">SD</th></tr>
<tr><th>Eq. (5.12)</th><th>Eq. (5.14)</th><th>Eq. (5.17)</th><th>Eq. (5.19)</th></tr>
<tr><td>1</td><td rowspan="7">1</td><td>1</td><td rowspan="4">0.0008</td><td rowspan="7">0.0027</td><td rowspan="7">0.0003</td><td rowspan="7">0.0003</td><td rowspan="4">0.0010</td><td rowspan="4">0.0011</td></tr>
<tr><td>2</td><td>3</td></tr>
<tr><td>3</td><td>5</td></tr>
<tr><td>4</td><td>7</td></tr>
<tr><td>5</td><td>15</td><td rowspan="3">0.0005</td><td rowspan="3">0.0009</td><td rowspan="3">0.0012</td></tr>
<tr><td>6</td><td>17</td></tr>
<tr><td>7</td><td>21</td></tr>
<tr><td>8</td><td rowspan="9">2</td><td>2</td><td rowspan="5">0.0031</td><td rowspan="9">0.0125</td><td rowspan="9">0.0013</td><td rowspan="9">0.0010</td><td rowspan="5">0.0045</td><td rowspan="5">0.0054</td></tr>
<tr><td>9</td><td>4</td></tr>
<tr><td>10</td><td>6</td></tr>
<tr><td>11</td><td>8</td></tr>
<tr><td>12</td><td>10</td></tr>
<tr><td>13</td><td>14</td><td rowspan="4">0.0018</td><td rowspan="4">0.0041</td><td rowspan="4">0.0056</td></tr>
<tr><td>14</td><td>16</td></tr>
<tr><td>15</td><td>18</td></tr>
<tr><td>16</td><td>20</td></tr>
<tr><td>17</td><td rowspan="5">3</td><td>9</td><td rowspan="2">0.0048</td><td rowspan="5">0.0152</td><td rowspan="5">0.0036</td><td rowspan="5">0.0020</td><td rowspan="2">0.0064</td><td rowspan="2">0.0060</td></tr>
<tr><td>18</td><td>11</td></tr>
<tr><td>19</td><td>12</td><td rowspan="3">0.0028</td><td rowspan="3">0.0059</td><td rowspan="3">0.0062</td></tr>
<tr><td>20</td><td>13</td></tr>
<tr><td>21</td><td>19</td></tr>
</table>

5.2.3 Rock Mass Strength

The rock mass strength is one of the important parameters for the design of all types of underground excavation and stability analysis. A frequently applied approach for estimation of the rock mass strength is through an empirical failure criterion, often in conjunction with rock mass classification systems. Many researchers have proposed several empirical equations to calculate the strength of rock mass (σ_{cm}) based on rock mass classification systems as follows:

Ramamurthy (1986) proposed the following equation based on the RMR rating value:

$$\sigma_{cm} = \sigma_c \exp\left(\frac{\mathrm{RMR} - 100}{18.75}\right) \tag{5.21}$$

Goel (1994) suggested the following equation based on Q_N:

$$\sigma_{cm} = \left(\frac{5.5\gamma {Q_N}^{1/3}}{\sigma_c\ B^{0.1}}\right) \tag{5.22}$$

where Q_N is the stress free from Q (equation 2.2 in chapter 2), γ is the unit weight of rock mass (t/m^3), σ_c is the uniaxial compressive strength of intact rock (MPa) and B is the width of tunnel (m).

The main principle in the development of RMi has been focusing on the effects of the defects in a rock mass in reducing the strength of the intact rock. As it meant to express the compressive strength of the rock mass, it can be defined as (Palmstrom, 1995):

$$\sigma_{cm} = RMi = \sigma_c \, JP \tag{5.23}$$

where σ_c is the uniaxial compressive strength of intact rock and JP is the jointing parameter.

In order to apply the Hoek and Brown criterion for estimating the strength of rock masses, three properties of the rock mass have to be estimated. These are the uniaxial compressive strength of the intact rock (σ_c), the value of the Hoek and Brown constant (m_i) for the intact rock and the value of GSI for the rock mass. RocData software version 3.0 (2004) is used in this research to estimate the uniaxial compressive strength of rock mass by using geological strength index (GSI). The calculated uniaxial compressive strength of rock mass for all sections of Manipura diversion tunnels is presented in Table 5.4.

To overcome the characteristic limitation of the equations, several equations proposed by many researchers have been used to estimate the rock mass parameters along the Manipura diversion tunnel alignment. The average value is used as input parameter for numerical simulation and stability analysis.

Table 5.4 Calculated uniaxial compressive strength of rock mass (σ_{cm}) for all sections of the Manipura diversion tunnels.

<table>
<tr><th rowspan="2">No.</th><th rowspan="2">Zone</th><th rowspan="2">Section</th><th>From RMR</th><th>From Q</th><th>From RMi</th><th>From GSI</th><th rowspan="2">Avg.</th><th rowspan="2">SD</th></tr>
<tr><th>Eq. (5.21)</th><th>Eq. (5.22)</th><th>Eq. (5.23)</th><th>RocData Version 3.0</th></tr>
<tr><td>1</td><td rowspan="7">1</td><td>1</td><td rowspan="4">0.5503</td><td rowspan="7">0.7536</td><td rowspan="7">0.2858</td><td rowspan="7">0.2490</td><td rowspan="4">0.4597</td><td rowspan="4">0.2375</td></tr>
<tr><td>2</td><td>3</td></tr>
<tr><td>3</td><td>5</td></tr>
<tr><td>4</td><td>7</td></tr>
<tr><td>5</td><td>15</td><td rowspan="3">0.4215</td><td rowspan="3">0.4275</td><td rowspan="3">0.2297</td></tr>
<tr><td>6</td><td>17</td></tr>
<tr><td>7</td><td>21</td></tr>
<tr><td>8</td><td rowspan="9">2</td><td>2</td><td rowspan="5">1.6375</td><td rowspan="9">0.8386</td><td rowspan="9">0.9310</td><td rowspan="9">0.7650</td><td rowspan="5">1.0430</td><td rowspan="5">0.4021</td></tr>
<tr><td>9</td><td>4</td></tr>
<tr><td>10</td><td>6</td></tr>
<tr><td>11</td><td>8</td></tr>
<tr><td>12</td><td>10</td></tr>
<tr><td>13</td><td>14</td><td rowspan="4">1.2542</td><td rowspan="4">0.9472</td><td rowspan="4">0.2156</td></tr>
<tr><td>14</td><td>16</td></tr>
<tr><td>15</td><td>18</td></tr>
<tr><td>16</td><td>20</td></tr>
<tr><td>17</td><td rowspan="5">3</td><td>9</td><td rowspan="2">3.2429</td><td rowspan="5">0.5837</td><td rowspan="5">2.5153</td><td rowspan="5">1.7710</td><td rowspan="2">2.0282</td><td rowspan="2">1.1351</td></tr>
<tr><td>18</td><td>11</td></tr>
<tr><td>19</td><td>12</td><td rowspan="3">2.4839</td><td rowspan="3">1.8385</td><td rowspan="3">0.9044</td></tr>
<tr><td>20</td><td>13</td></tr>
<tr><td>21</td><td>19</td></tr>
</table>

5.3 Stability Analysis

The classical approach used in the design of engineering structures is to consider the relationship between the capacity C (strength or resisting force) of the element and the demand D (stress or disturbing force). The factor of safety of the structure is defined as FS = C/D and failure is assumed to occur when FS is less than 1. In the case of underground excavation, the in-situ stress is required to analyze for stability. The stand-up time and estimation of maximum unsupported span are also some of the important issues for the safety of the underground excavation.

5.3.1 Stand-up Time and Maximum Unsupported Span

The stand-up time of the rock mass and the evaluation of maximum unsupported span are important for the tunneling sequence and safety for the tunnel construction. Bieniawski (1976) proposed the relationship between the stand-up time of an unsupported underground excavation span and the CSIR geomechanics classification, rock mass rating system (RMR). The chart is useful to estimate the stand-up time of the rock mass and maximum unsupported span. This may lead to provide effective planning of the excavation and supporting sequences for the tunnel construction.

Based on the NGI tunneling quality index (Q system), Barton et al. (1974) defined an additional quantity, the equivalent dimension (D_e), to evaluate the maximum unsupported span and support requirements for a particular dimension of underground excavation. The equivalent dimension (D_e) is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the excavation support ratio (ESR) (equation 2.3 in chapter 2). For the diversion tunnel, ESR value is 1.6. In order to estimate the maximum unsupported span of underground excavation, the relationship between the maximum equivalent dimension (D_e) of an unsupported underground excavation and the NGI tunneling quality index (Q system) was proposed by Barton et al. (1974). The estimated maximum unsupported span and stand-up time of the rock mass for all sections of Manipura diversion tunnels are shown in Table 5.5.

Table 5.5 Estimated maximum unsupported span and stand-up time of the rock mass for all sections of the Manipura diversion tunnels.

<table>
<tr><th rowspan="2">No.</th><th rowspan="2">Zone</th><th rowspan="2">Section</th><th colspan="3">From RMR</th><th colspan="2">From Q</th></tr>
<tr><th>Value</th><th>Max. unsupported span</th><th>Stand-up time</th><th>Value</th><th>Max. unsupported span</th></tr>
<tr><td>1</td><td rowspan="7">1</td><td>1</td><td rowspan="4">36</td><td rowspan="4">1.40 m</td><td rowspan="4">1 day</td><td rowspan="2">0.2667</td><td rowspan="2">2.40 m</td></tr>
<tr><td>2</td><td>3</td></tr>
<tr><td>3</td><td>5</td><td rowspan="4">0.5333</td><td rowspan="4">2.80 m</td></tr>
<tr><td>4</td><td>7</td></tr>
<tr><td>5</td><td>15</td><td rowspan="3">31</td><td rowspan="3">1.20 m</td><td rowspan="3">16 hrs</td></tr>
<tr><td>6</td><td>17</td></tr>
<tr><td>7</td><td>21</td><td>0.2667</td><td>2.40 m</td></tr>
<tr><td>8</td><td rowspan="9">2</td><td>2</td><td rowspan="5">48</td><td rowspan="5">1.75 m</td><td rowspan="5">8 days</td><td>1.2465</td><td>4.00 m</td></tr>
<tr><td>9</td><td>4</td><td rowspan="7">2.4930</td><td rowspan="7">4.80 m</td></tr>
<tr><td>10</td><td>6</td></tr>
<tr><td>11</td><td>8</td></tr>
<tr><td>12</td><td>10</td></tr>
<tr><td>13</td><td>14</td><td rowspan="4">43</td><td rowspan="4">1.60 m</td><td rowspan="4">4 days</td></tr>
<tr><td>14</td><td>16</td></tr>
<tr><td>15</td><td>18</td></tr>
<tr><td>16</td><td>20</td><td>1.2465</td><td>4.00 m</td></tr>
<tr><td>17</td><td rowspan="5">3</td><td>9</td><td rowspan="2">52</td><td rowspan="2">1.80 m</td><td rowspan="2">14 days</td><td rowspan="5">3.0400</td><td rowspan="5">5.60 m</td></tr>
<tr><td>18</td><td>11</td></tr>
<tr><td>19</td><td>12</td><td rowspan="3">47</td><td rowspan="3">1.75 m</td><td rowspan="3">7 days</td></tr>
<tr><td>20</td><td>13</td></tr>
<tr><td>21</td><td>19</td></tr>
</table>

5.3.2 In-situ Stress Analysis and Safety Factor

The stresses naturally exist in the rock mass related to the weight of the overlying strata and the geological history of the rock mass. When an underground excavation is made in the rock, these stresses are disturbed and new stresses are re-distributed in the rock in the immediate vicinity of the underground opening. In that case, failure of the rock adjacent to the excavation boundary can lead to instability. Therefore, the estimation of in-situ stress at the boundary of the underground opening is required to control the instability problem.

The condition that the only stresses, which can exist at the boundary of an excavation, are the stresses tangential to the boundary holds true for all excavation shapes which are free of internal loading. The tangential stress at the boundary of the underground opening can be estimated by the following equations proposed by Hoek and Brown (1990):

For the tangential stress at roof,

$$\sigma_{\theta roof} = (A \times k - 1)\, \sigma_v \qquad (5.24)$$

For the tangential stress at side wall,

$$\sigma_{\theta wall} = (B - 1)\, \sigma_v \qquad (5.25)$$

where σ_v is the vertical stress, k is the stress ratio (σ_h/σ_v) and, A and B are the constants. In the case of Manipura diversion tunnel, modified horseshoe shape, A is 3.1 and B is 2.7. The horizontal stress is difficult to estimate. It is known that they are variable at shallow depth, tending to a hydrostatic state in deep environment. The magnitude of horizontal stress is usually more than vertical stress at shallow depths (less than 500 m) whereas they trend to a hydrostatic state at depth of about 1000 m below the surface (Hoek and Brown, 1990). In this research, the ratio of horizontal stress to vertical stress (k) is assumed to be 1 as suggested by Hoek (2003). The vertical stress (σ_v) is directly proportional to the unit weight (γ) of overlying rock load and the height of the overburden (H). The vertical stresses for all sections of Manipura diversion tunnels are calculated by the following equation:

$$\sigma_v = \gamma H \tag{5.26}$$

After estimating the overall stresses for all sections of Manipura diversion tunnels, these results are summarized for the calculation of safety factor as shown in Table 5.6.

The safety factor is taken as the ratio between the rock mass strength and the stress around the underground opening. The calculated values of rock mass strength for all sections of the Manipura diversion tunnels are described in Table 4.7 and the average values of rock mass strength are used to calculate the factor of safety. To maintain stability, the acceptable factor of safety should be greater than 1.

The results show that all of the sections are under failure condition and they need to be supported to increase safety factor.

Table 5.6 Calculated induced stress and factor of safety for all sections of the Manipura diversion tunnels.

No.	Zone	Section	H (m)	σ_v (MPa)	$\sigma_{\theta roof}$ (MPa)	$\sigma_{\theta wall}$ (MPa)	σ_{cm} (MPa)	FS (Roof)	FS (wall)
1	1	1	39.88	1.0608	2.2277	1.8034	0.4597	0.21	0.25
2		3	53.23	1.4159	2.9734	2.4071	0.4597	0.15	0.19
3		5	66.97	1.7814	3.7409	3.0284	0.4597	0.12	0.15
4		7	101.33	2.6954	5.6603	4.5821	0.4597	0.08	0.10
5		15	97.84	2.6025	5.4653	4.4243	0.4275	0.08	0.10
6		17	48.31	1.2850	2.6986	2.1846	0.4275	0.16	0.20
7		21	38.51	1.0244	2.1512	1.7414	0.4275	0.20	0.25
8	2	2	42.79	1.1896	2.4981	2.0223	1.0430	0.42	0.52
9		4	62.63	1.7411	3.6563	2.9599	1.0430	0.29	0.35
10		6	72.46	2.0144	4.2302	3.4245	1.0430	0.25	0.30
11		8	106.12	2.9501	6.1953	5.0152	1.0430	0.17	0.21
12		10	149.16	4.1466	8.7080	7.0493	1.0430	0.12	0.15
13		14	109.52	3.0447	6.3938	5.1759	0.9472	0.15	0.18
14		16	58.56	1.6280	3.4187	2.7675	0.9472	0.28	0.34
15		18	48.31	1.3430	2.8203	2.2831	0.9472	0.34	0.41
16		20	48.86	1.3583	2.8524	2.3091	0.9472	0.33	0.41
17	3	9	132.67	3.8342	8.0517	6.5181	2.0282	0.25	0.31
18		11	173.4	5.0113	10.5236	8.5191	2.0282	0.19	0.24
19		12	173.4	5.0113	10.5236	8.5191	1.8385	0.17	0.22
20		13	140.85	4.0706	8.5482	6.9200	1.8385	0.22	0.27
21		19	48.86	1.4121	2.9653	2.4005	1.8385	0.62	0.77

CHAPTER VI

SUPPORT DESIGN

6.1 Introduction

This chapter describes the estimation of support capacity and the design of support systems for the Manipura diversion tunnels by using empirical methods and numerical method. The performance of support elements, such as rock bolt and shotcrete, is analyzed by numerical modeling. The design results are compared with those obtained from empirical methods and numerical method.

6.2 Support Capacity Estimation

The prediction of support capacity is one of the important tasks for the assessment of the reliable support systems for underground openings. Several relations based on rock mass classification systems are used to estimate the required support capacity for all sections of the Manipura diversion tunnels.

Bieniawski (1974) proposed the following equation to estimate the support pressure (P_{roof}) based on rock mass rating system (RMR):

$$P_{roof} = \left(\frac{100 - RMR}{100}\right)W\gamma \qquad (6.1)$$

where P_{roof} is the support pressure (kN/m^2), W is the width of opening (m) and γ is the unit weight of overburden (kN/m^3).

Another approach, proposed by Barton *et al*.(1974), is based on NGI tunneling quality index value (Q value) as follow:

$$P_{roof} = \frac{200}{J_r} Q^{1/3} \tag{6.2}$$

where P_{roof} is the roof support pressure (kN/m^2) and J_r is the discontinuity roughness.

The support pressure is calculated by these two equations for all sections of Manipura diversion tunnels. The results are given in Table 6.1.

Table 6.1 Calculated support pressure for all sections of the Manipura diversion tunnels.

<table>
<tr><th rowspan="2">No.</th><th rowspan="2">Zone</th><th rowspan="2">Section</th><th>From RMR</th><th>From Q</th></tr>
<tr><th>Eq. (6.1), (MPa)</th><th>Eq. (6.2), (MPa)</th></tr>
<tr><td>1</td><td rowspan="7">1</td><td>1</td><td rowspan="4">0.2042</td><td rowspan="2">0.0429</td></tr>
<tr><td>2</td><td>3</td></tr>
<tr><td>3</td><td>5</td><td rowspan="4">0.0541</td></tr>
<tr><td>4</td><td>7</td></tr>
<tr><td>5</td><td>15</td><td rowspan="3">0.2201</td></tr>
<tr><td>6</td><td>17</td></tr>
<tr><td>7</td><td>21</td><td>0.0429</td></tr>
<tr><td>8</td><td rowspan="9">2</td><td>2</td><td rowspan="5">0.1732</td><td>0.0717</td></tr>
<tr><td>9</td><td>4</td><td rowspan="7">0.0904</td></tr>
<tr><td>10</td><td>6</td></tr>
<tr><td>11</td><td>8</td></tr>
<tr><td>12</td><td>10</td></tr>
<tr><td>13</td><td>14</td><td rowspan="4">0.1899</td></tr>
<tr><td>14</td><td>16</td></tr>
<tr><td>15</td><td>18</td></tr>
<tr><td>16</td><td>20</td><td>0.0717</td></tr>
<tr><td>17</td><td rowspan="5">3</td><td>9</td><td rowspan="2">0.1667</td><td rowspan="5">0.0966</td></tr>
<tr><td>18</td><td>11</td></tr>
<tr><td>19</td><td>12</td><td rowspan="3">0.1841</td></tr>
<tr><td>20</td><td>13</td></tr>
<tr><td>21</td><td>19</td></tr>
</table>

The results show that the support pressure obtained from the RMR system is greater than that obtained from the Q system. The width of the tunnel and the unit weight of overburden, which are not directly considered in Q system (equation 6.2), are considered in RMR relationship (equation 6.1). The material strength is also considered in RMR system. Therefore, the support capacity value estimated by RMR relationship is considered more realistic.

6.3 Support Design using Empirical Methods

Empirical methods are based on rock mass classification systems: rock mass rating system (RMR), NGI tunneling quality index (Q system) and rock mass index (RMi). All these systems have quantitative estimation of the rock mass quality linked with empirical design rules to estimate adequate rock support measures such as rock bolt, shotcrete and steel set.

6.3.1 Rock Mass Rating System (RMR)

The rock mass rating system (RMR), proposed by Bieniawski (1989), provides guidelines for the selection of rock reinforcement for tunnels. The method of excavation is provided based on the rock mass rating value.

The results suggest the two excavation methods : (1) top heading and bench cut excavation method (1.0-1.5 m advance in top heading, install support concurrently with excavation, 10 m from face, for zone 1) and (2) top heading and bench cut excavation method (1.5-3 m advance in top heading, commence support after each blast, complete support 10 m from face, for zone 2 and 3 of the tunnel). For zone 1, the RMR support table suggests that light to medium steel sets with spacing of 1.5 m should be installed where required. The suggested support systems assessed based on rock mass rating system (RMR) for each zone are presented in Table 6.2.

Table 6.2 Recommended support systems based on rock mass rating system (RMR).

No.	Zone	Section	RMR value	Rock bolt (20 mm diameter fully grouted)	Shotcrete	Steel Sets
1	1	1	36	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh	100-150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required.
2		3				
3		5				
4		7				
5		15	31			
6		17				
7		21				
8	2	2	48	Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown	50-100 mm in crown and 30 mm in sides	None
9		4				
10		6				
11		8				
12		10				
13		14	43			
14		16				
15		18				
16		20				
17	3	9	52	Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown	50-100 mm in crown and 30 mm in sides	None
18		11				
19		12	47			
20		13				
21		19				

6.3.2 NGI Tunneling Quality Index (Q system)

The NGI tunneling quality index (Q system) is related to tunnel support requirements by defining the equivalent dimensions of the excavation (D_e). The equivalent dimension is a function of both the size and the purpose of excavation as described in equation 2.3 in chapter 2. The relationship between the index Q and the equivalent dimension of an excavation determines the appropriate support measures. The support elements include rock bolt and fibre reinforced shotcrete. The summary of the support measures for all sections of the Manipura diversion tunnels based on Q system is given in Table 6.3.

Table 6.3 Recommended support systems based on NGI tunneling quality index (Q system).

No.	Zone	Section	Q index value	Rock bolt	Shotcrete
1	1	1	0.2667	Bolts 2.4-3 m long, spaced 1.3-1.5 m	Fibre reinforced shotcrete 90-120 mm
2		3			
3		5	0.5333	Bolts 2.4-3 m long, spaced 1.5-1.7 m	Fibre reinforced shotcrete 90-120 mm
4		7			
5		15			
6		17			
7		21	0.2667	Bolts 2.4-3 m long, spaced 1.3-1.5 m	Fibre reinforced shotcrete 90-120 mm
8	2	2	1.2465	Bolts 2.4-3 m long, spaced 1.7-2.1 m	Fibre reinforced shotcrete 50-90 mm
9		4	2.4930	Bolts 2.4-3 m long, spaced 1.7-2.1 m	Unreinforced shotcrete 40-100 mm
10		6			
11		8			
12		10			
13		14			
14		16			
15		18			
16		20	1.2465	Bolts 2.4-3 m long, spaced 1.7-2.1 m	Fibre reinforced shotcrete 50-90 mm
17	3	9	3.0400	Bolts 2.4-3 m long, spaced 1.7-2.1 m	Unreinforced shotcrete 40-100 mm
18		11			
19		12			
20		13			
21		19			

6.3.3 Rock Mass Index (RMi)

The rock mass index (RMi) provides two types of support chart, for discontinuous ground (jointed) and continuous ground (overstressed). For jointed rock (discontinuous ground), the relationship between the ground condition factor (Gc) and the size ratio (Sr) determines the appropriate support measures. For the continuous ground (overstressed), the required support is found in special support chart using the competency factor (Cg).

In this study, the RMi support spreadsheet, version 3.1 (Palmstrom, 2001) is used to get direct assessment of support types for all sections of the Manipura diversion tunnels. The support measures evaluated based on rock mass index (RMi) are summarized in Table 6.3. The suggested support types based on rock mass index (RMi) include rock bolts and fibre reinforced shotcrete.

Table 6.4 Recommended support systems based on rock mass index (RMi).

No.	Zone	Section	RMi index value	Rock bolt	Shotcrete
1	1	1	0.2858	Special bolting for roof and 1×1 m rock bolt for wall	Concrete lining or special design shotcrete for roof and 150-250 mm thickness fibre reinforced shotcrete for wall.
2		3			
3		5			
4		7			
5		15			
6		17			
7		21			
8	2	2	0.9310	Rock bolt 1×1 m for roof and 1.25×1.25 m for wall	Fibre reinforced shotcrete thickness 150-250 mm for roof and 100-150 mm for wall.
9		4			
10		6			
11		8			
12		10			
13		14			
14		16			
15		18			
16		20			
17	3	9	2.5153	Rock bolt 1.25×1.25 m for roof and 1.5×1.5 m for wall	Fibre reinforced shotcrete thickness 100-150 mm for roof and 70-100 mm for wall.
18		11			
19		12			
20		13			
21		19			

6.4 Support Design using Numerical Method

Phase^2 version 6.0, a finite element program developed by Rocscience (2007), has been used for calculating stresses, deformations and developed plastic zone around the tunnels and to evaluate the performance of support systems. Support elements used are composed of rock bolts and shotcrete. The properties of the support elements (length, pattern of bolts and thickness of shotcrete) are similar to those proposed by the empirical methods. Hoek and Brown failure criterion is used to estimate yielded elements and plastic zone of rock masses in the vicinity of tunnel. Plastic post-failure strength parameters are used in this analysis. The residual parameters are assumed as half of the peak strength parameters. In situ stress for the finite element models is assumed as hydrostatic and automatic mesh around the tunnel is generated. For this study, boundary conditions are defined as restrained X for both sides boundary, restrained Y for the lower boundary, and free surface for the upper boundary. The finite element mesh and boundary conditions for the analysis of sections 7, 15, 10, 14 and 12 are shown in Figures 6.1 through 6.5.

For the numerical simulations, five sections, sections 7 and 15 for the representative of zone 1, sections 10 and 14 for the representative of zone 2 and section 12 for the representative of zone 3, are selected. This is because section 7 is under the highest stress, 5.66 MPa, and section 15 is the lowest rock mass strength, 0.43 MPa, among the sections of zone 1. Similarly, section 10 is under the highest stress, 8.71 MPa, and section 14 has the lowest rock mass strength, 0.95 MPa, of zone 2. In zone 3, section 12 is under the highest stress, 10.52 MPa, and has the lowest rock mass strength, 1.84 MPa. Two kinds of models, unsupported and supported are simulated for each section of tunnel. The rock mass parameters calculated by empirical methods, described in chapter 5, are used as input parameters in numerical simulations.

For section 7 of zone 1, Figure 6.6 shows the strength factor contour, the radius of plastic zone and the number of yielded finite elements around the tunnel with and without support installation. Before support installation, the radius of plastic zone is 14.15 m and 839 yielded finite elements are observed. After support installation, the radius of plastic zone is reduced to 8.61 m and yielded finite elements are also decreased to 432 numbers. Figure 6.11 shows the displacement contour, displacement vectors and maximum total displacement of the tunnel with and without support installation. Before support installation, the maximum total displacement is 20.75 mm and after support installation, the maximum total displacement is reduced to 7.33 mm.

For section 15 of zone 1, as shown in Figure 6.7, the radius of plastic zone is 13.80 m and the yielded finite elements are 748 in the unsupported case. In the supported case, the radius of plastic zone is reduced to 8.34 m and the yielded finite elements are reduced to 335. As shown in Figure 6.12, maximum total displacement is 23.64 mm without support installation and it is reduced to 7.84 mm in the supported case.

For section 10 of zone 2, as shown in Figure 6.8, the radius of plastic zone is 9.75 m and the yielded finite elements are 652 in the unsupported case. In the supported case, the radius of plastic zone is reduced to 8.15 m and the yielded finite elements are reduced to 487. The maximum total displacement is 9.08 mm without support installation and it is reduced to 5.88 mm after support installation as shown in Figure 6.13.

For section 14 of zone 2, as shown in Figure 6.9, the radius of plastic zone is 9.00 m and the yielded finite elements are 614 in the unsupported case. In the supported case, the radius of plastic zone is reduced to 7.87 m and the yielded finite elements are reduced to 389. The maximum total displacement is 6.69 mm without support installation and is reduced to 4.72 mm after support installation as shown in Figure 6.14.

For section 12 of zone 3, as shown in Figure 6.10, the radius of plastic zone is 8.25 m and the yielded finite elements are 549 in the unsupported case. In the supported case, the radius of plastic zone is reduced to 7.55 m and the yielded finite elements are reduced to 370. The maximum total displacement is 7.31 mm without support installation and is reduced to 5.57 mm after support installation as shown in Figure 6.15.

Even though the maximum total displacements are very small in all numerical modeling results, the extent of plastic zone and yielded elements suggest that there would be some stability problems for tunnels. $Phase^2$ is a small strain finite element program and thus it cannot accommodate the very large strains (Basarir, Ozsan, and Karakus, 2005). In the case of Manipura diversion tunnel support design, it is more important to consider the extent of plastic zone and yielded elements rather than the magnitude of displacement. According to plasticity theory, a plastic zone occurs around a tunnel after excavation when induced stresses exceed the rock mass strength. After support installation, both the number of yielded elements and the extent of plastic zone are decreased as shown in Figures 6.6 through 6.10. Maximum total displacement is reduced as well in the supported cases as presented in Figures 6.11

through 6.15. The results indicate that the applied support systems are adequate to obtain tunnel stability.

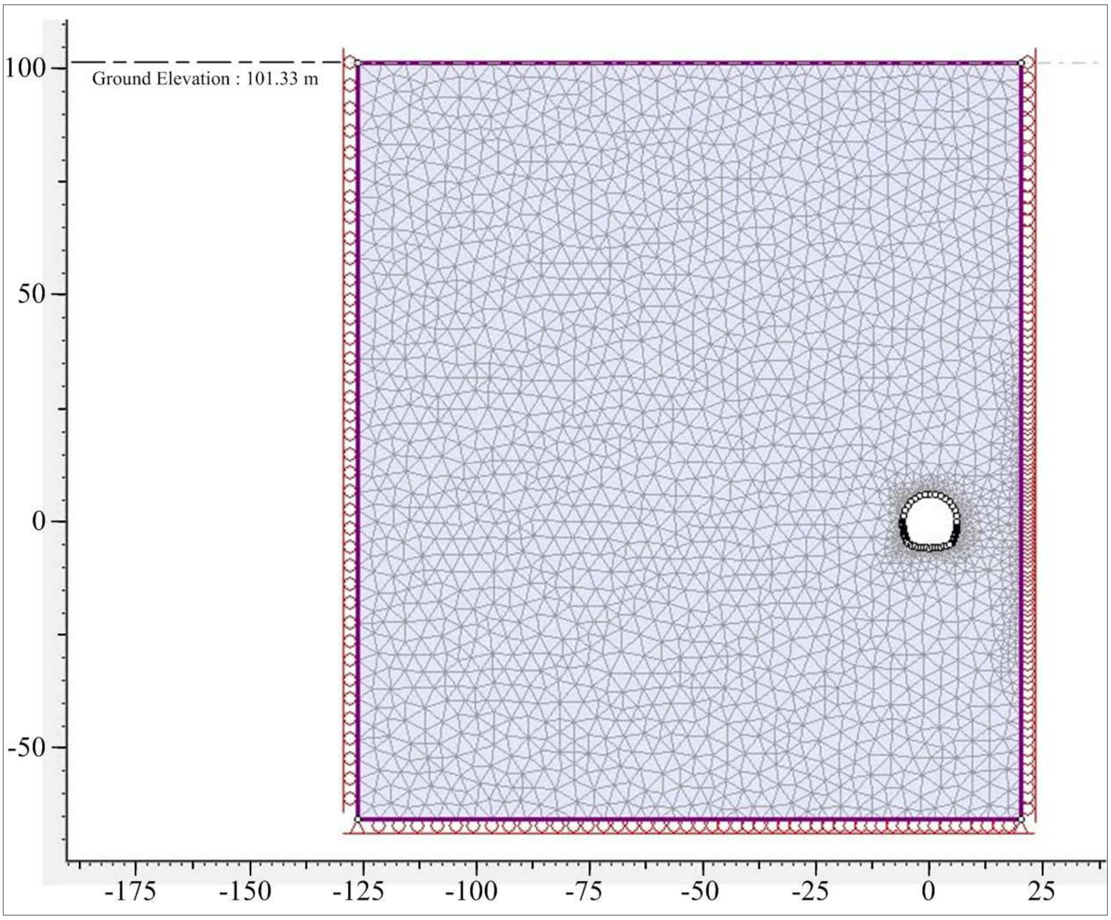

Figure 6.1 Finite element mesh and boundary conditions for the analysis of section 7 of zone 1.

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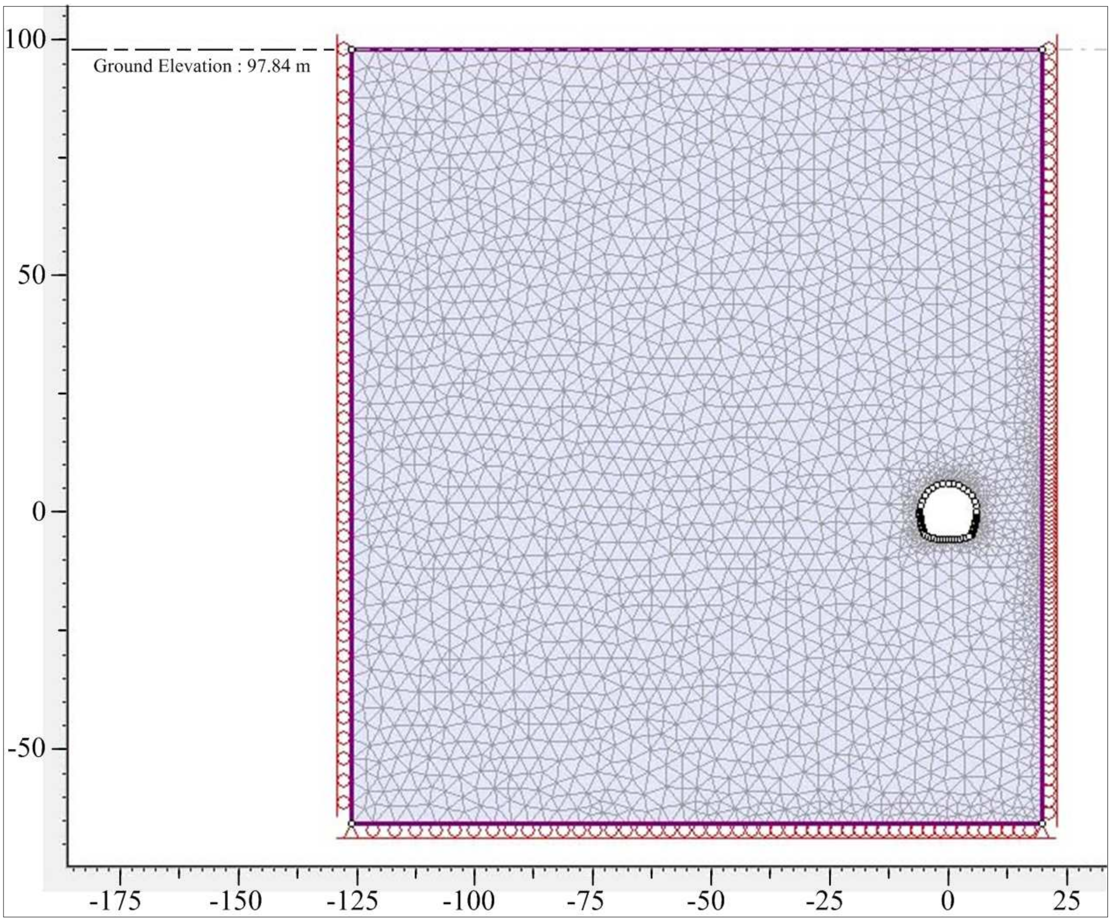

Figure 6.2 Finite element mesh and boundary conditions for the analysis of section 15 of zone 1.

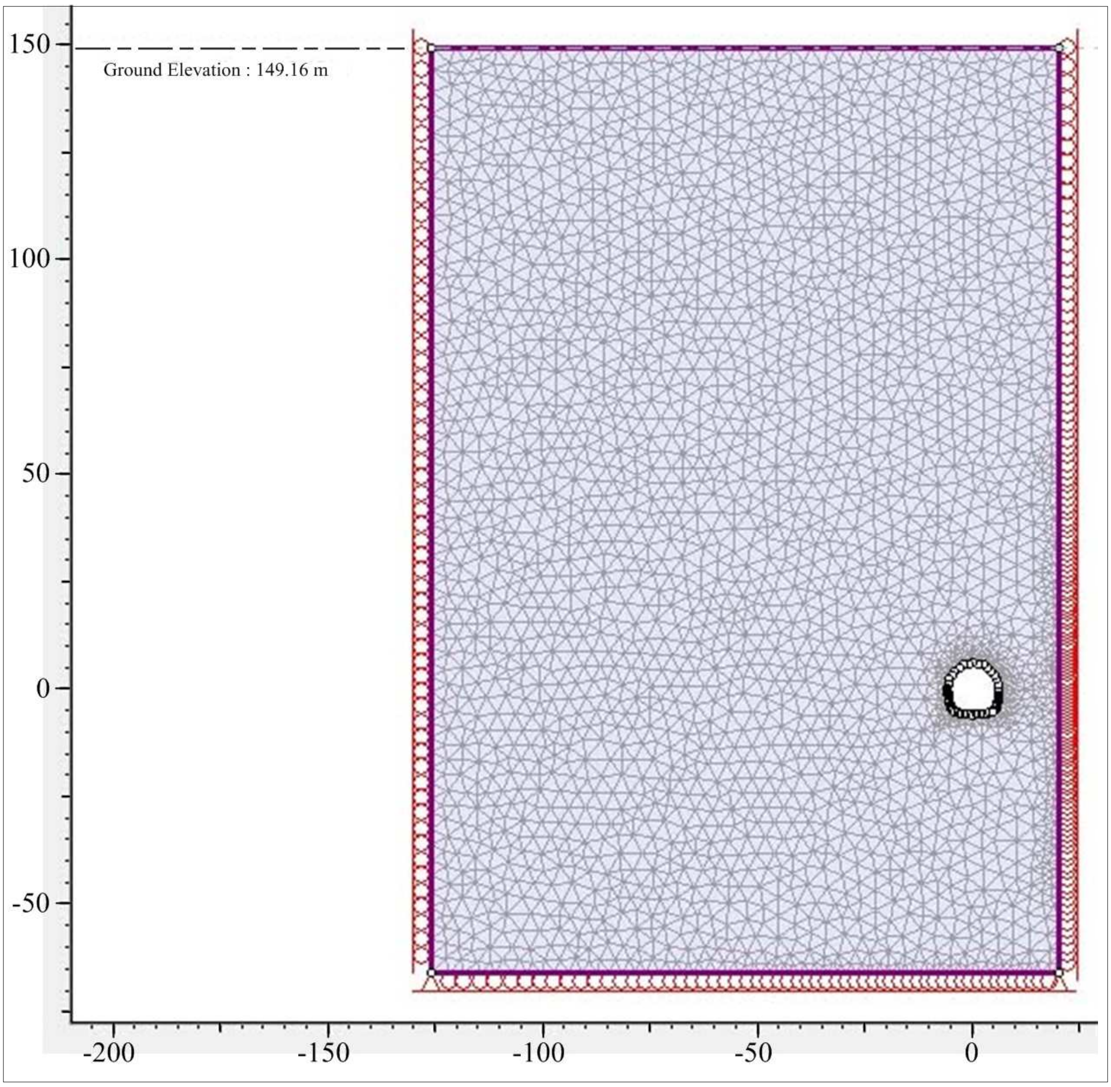

Figure 6.3 Finite element mesh and boundary conditions for the analysis of section 10 of zone 2.

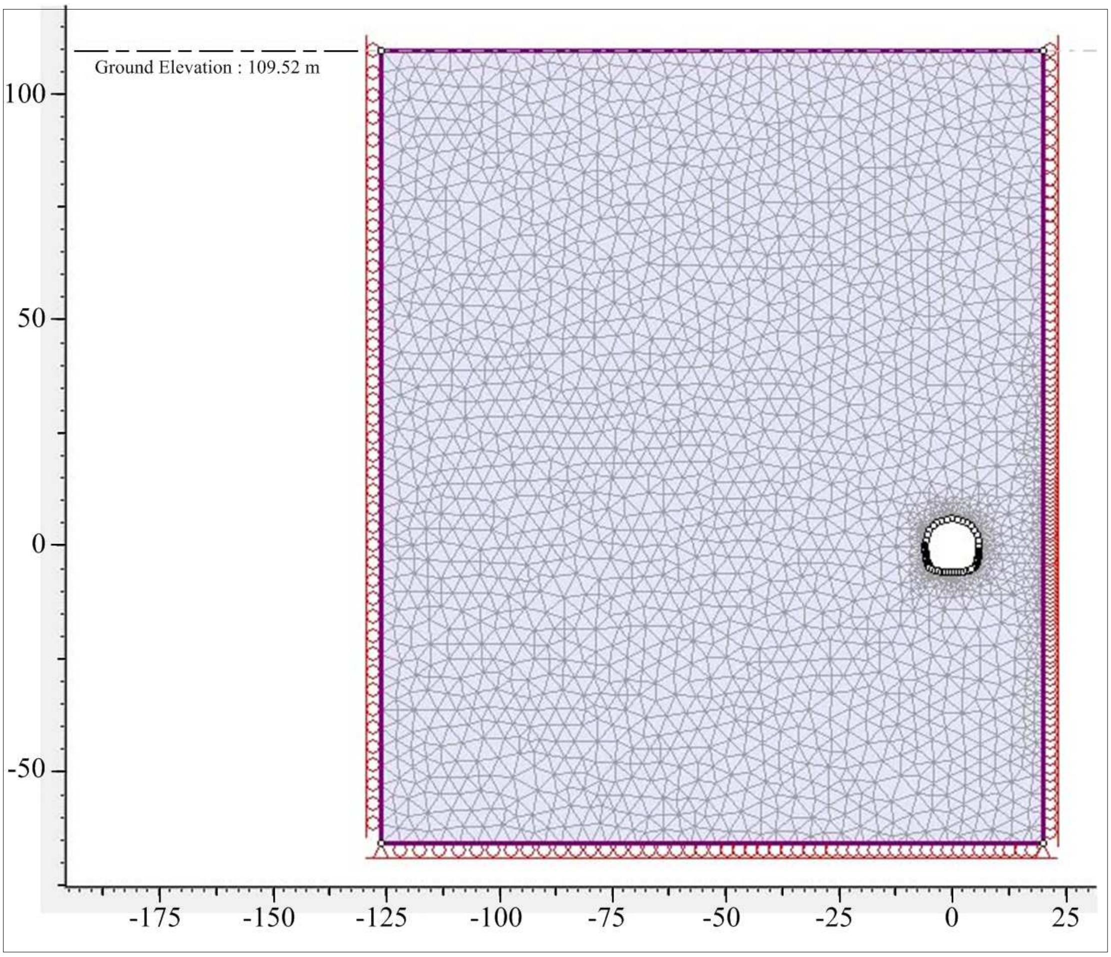

Figure 6.4 Finite element mesh and boundary conditions for the analysis of section 14 of zone 2.

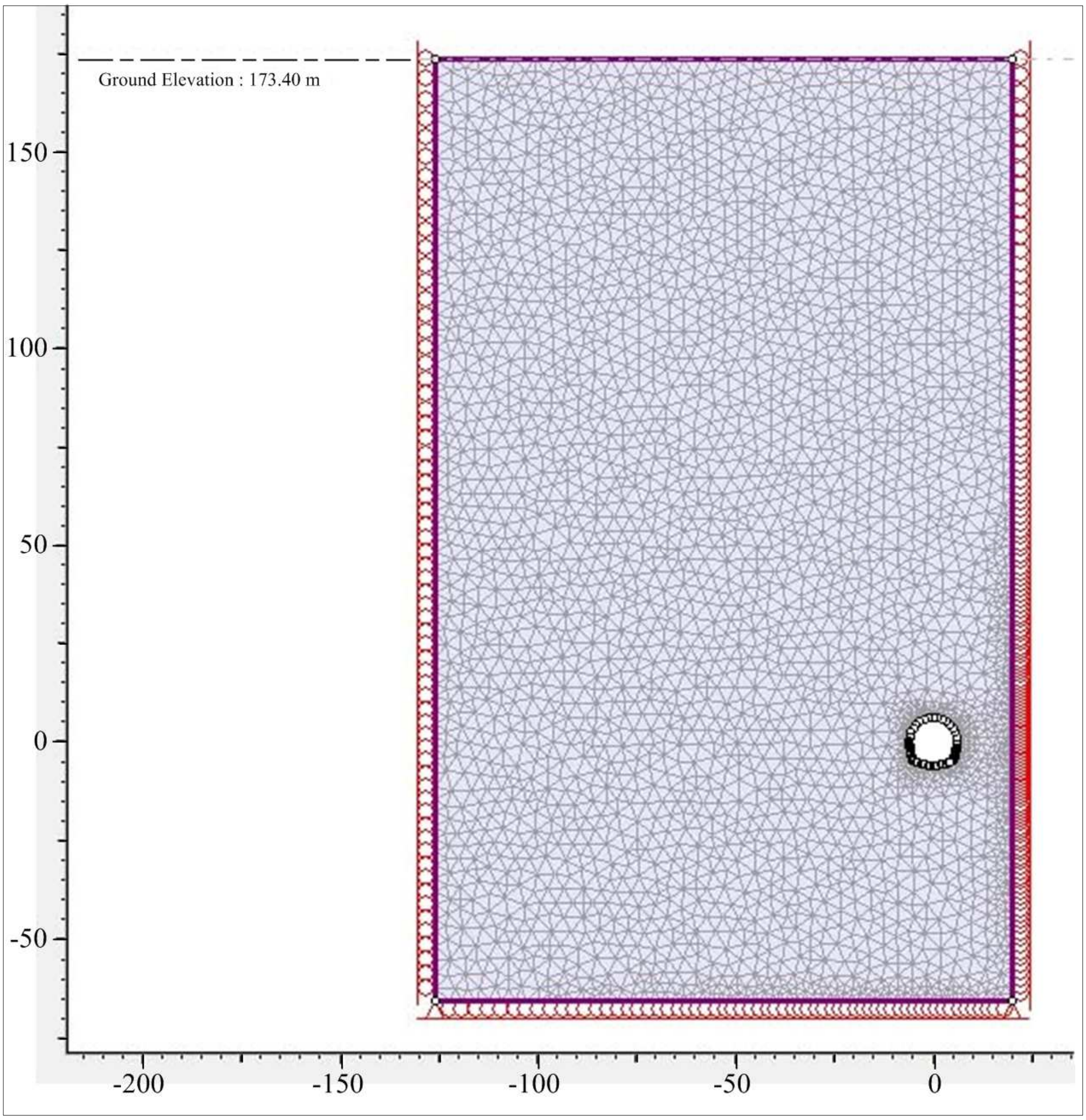

Figure 6.5 Finite element mesh and boundary conditions for the analysis of section 12 of zone 3.

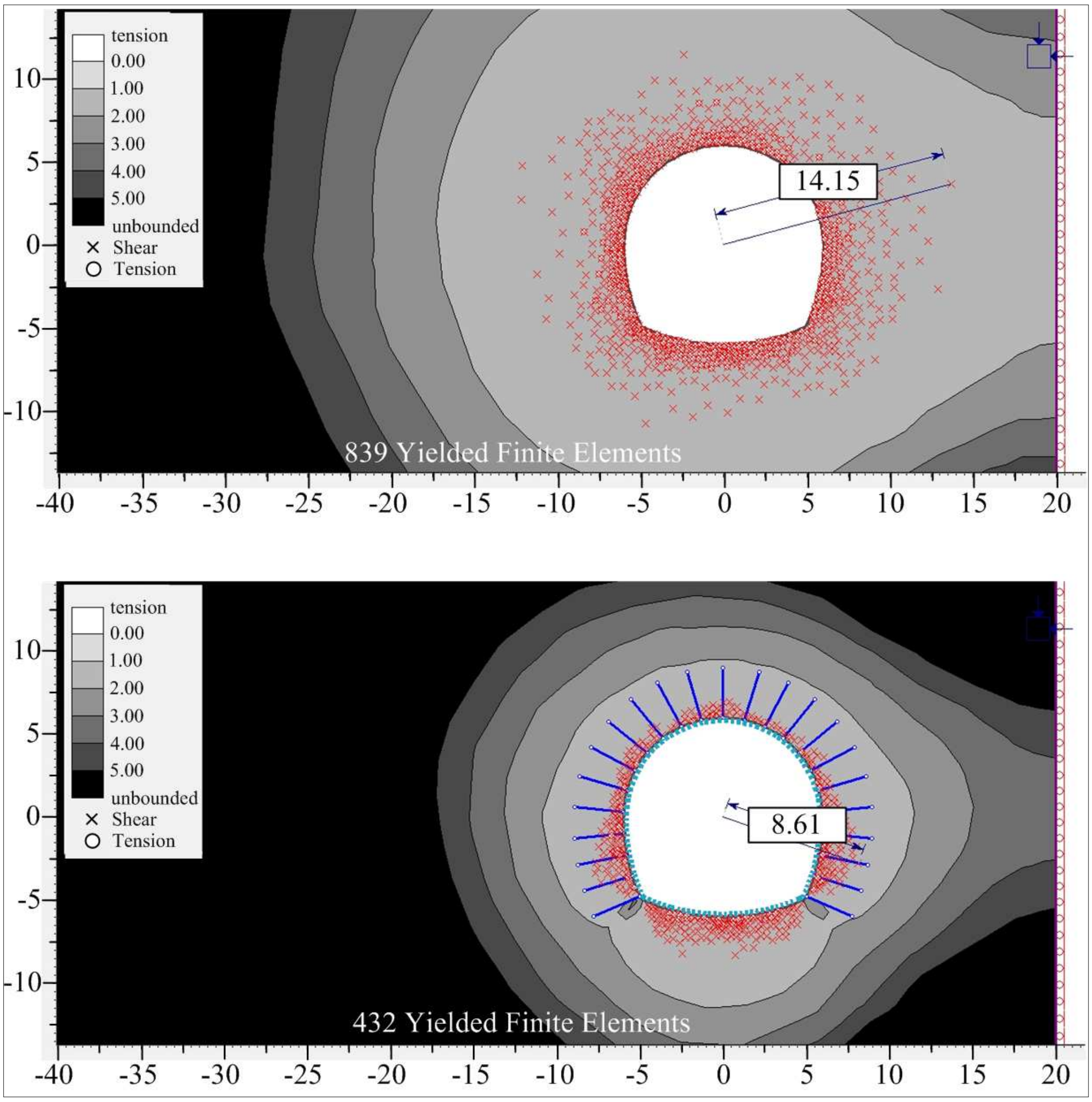

Figure 6.6 Strength factor contour and radius of plastic zone of section 7 of zone 1 before and after support installation.

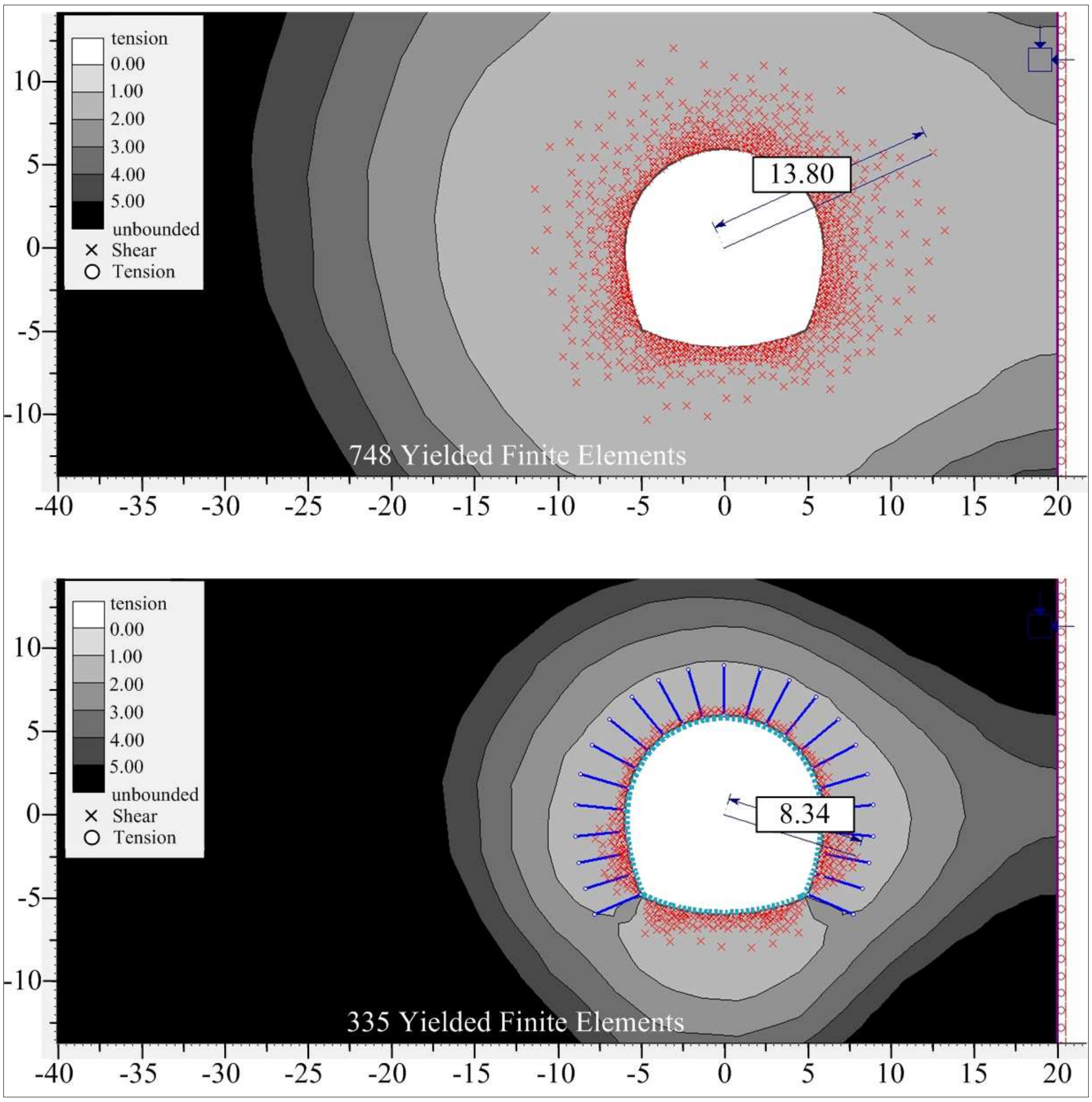

Figure 6.7 Strength factor contour and radius of plastic zone of section 15 of zone 1 before and after support installation.

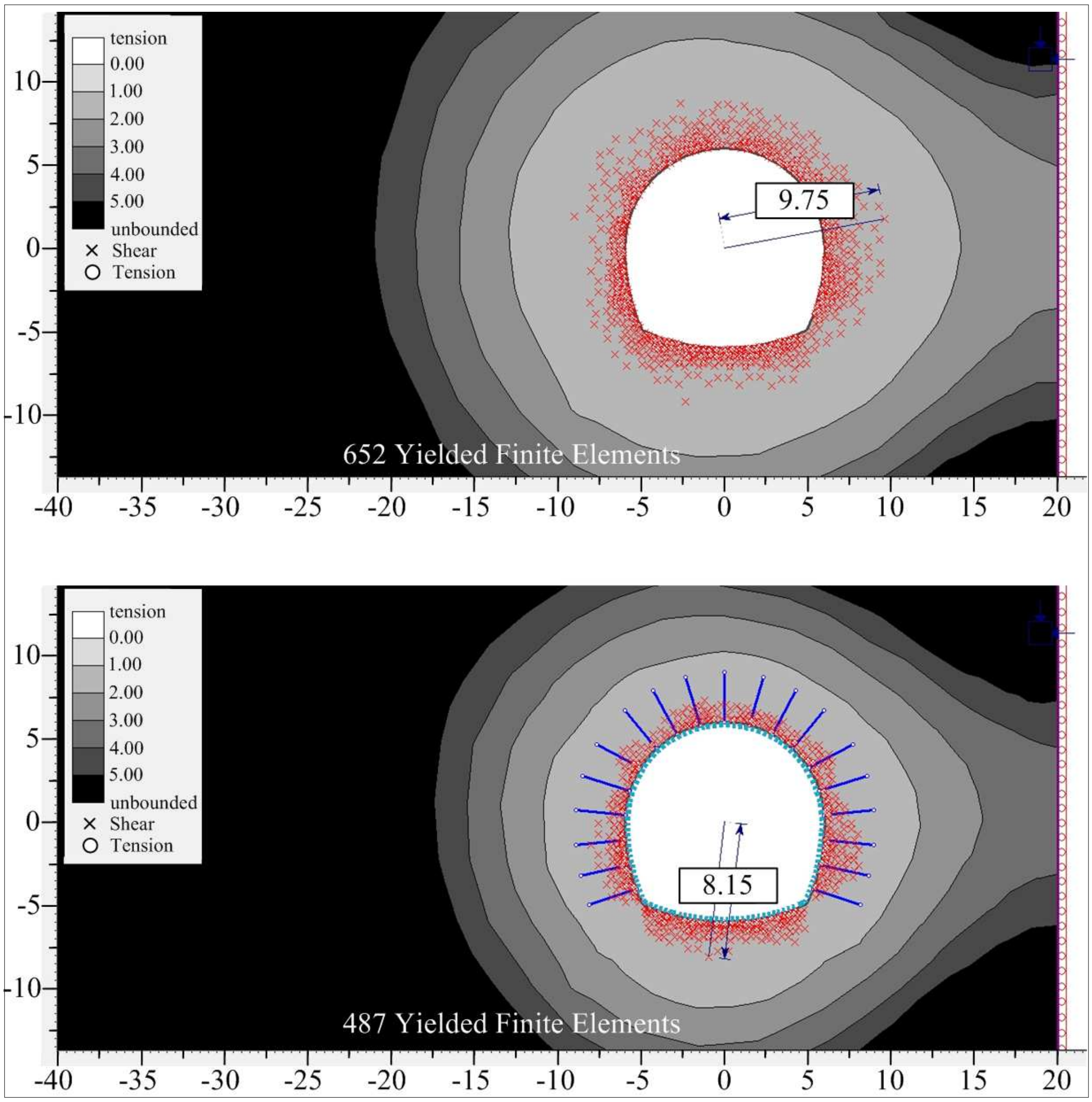

Figure 6.8 Strength factor contour and radius of plastic zone of section 10 of zone 2 before and after support installation.

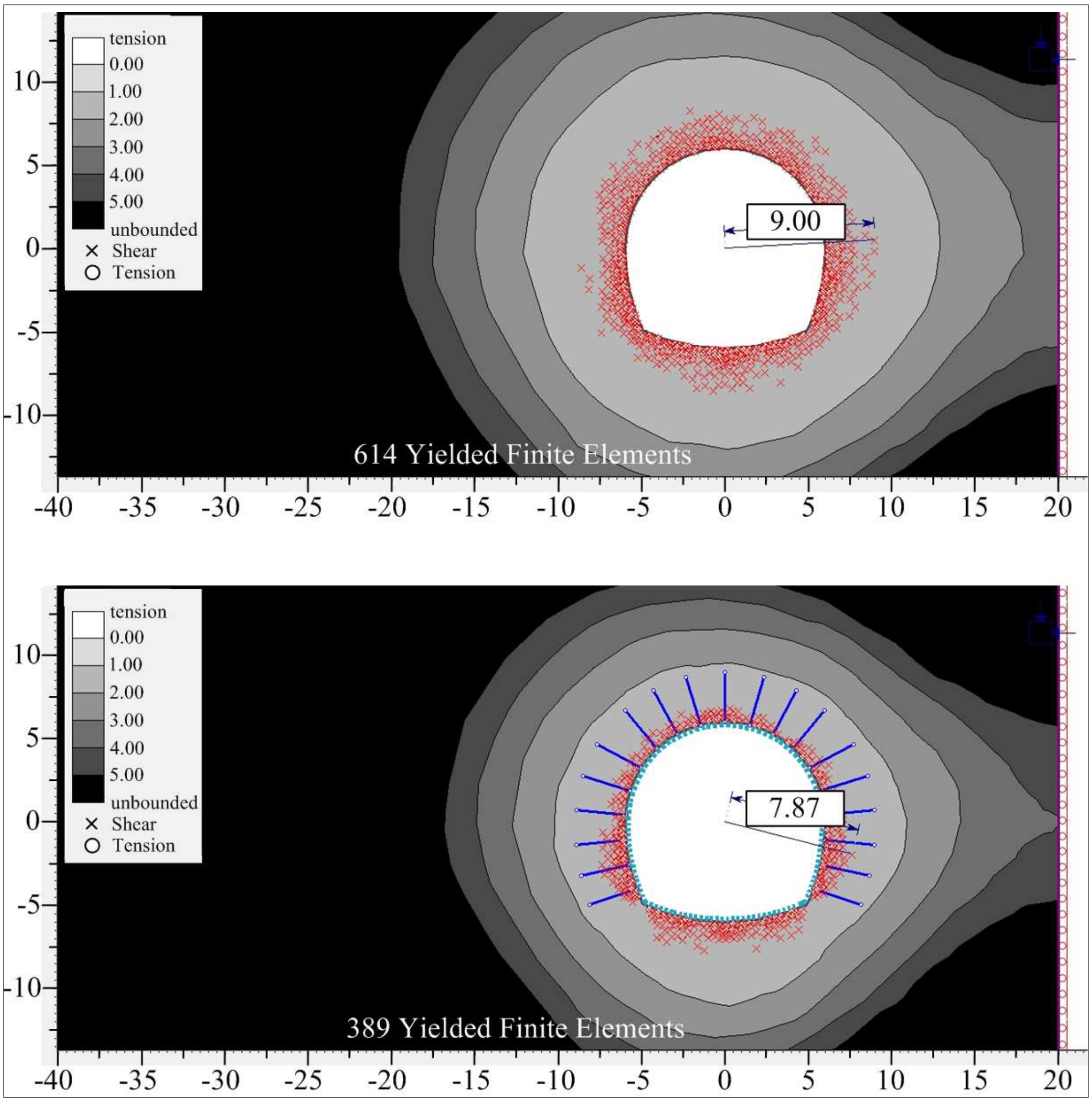

Figure 6.9 Strength factor contour and radius of plastic zone of section 14 of zone 2 before and after support installation.

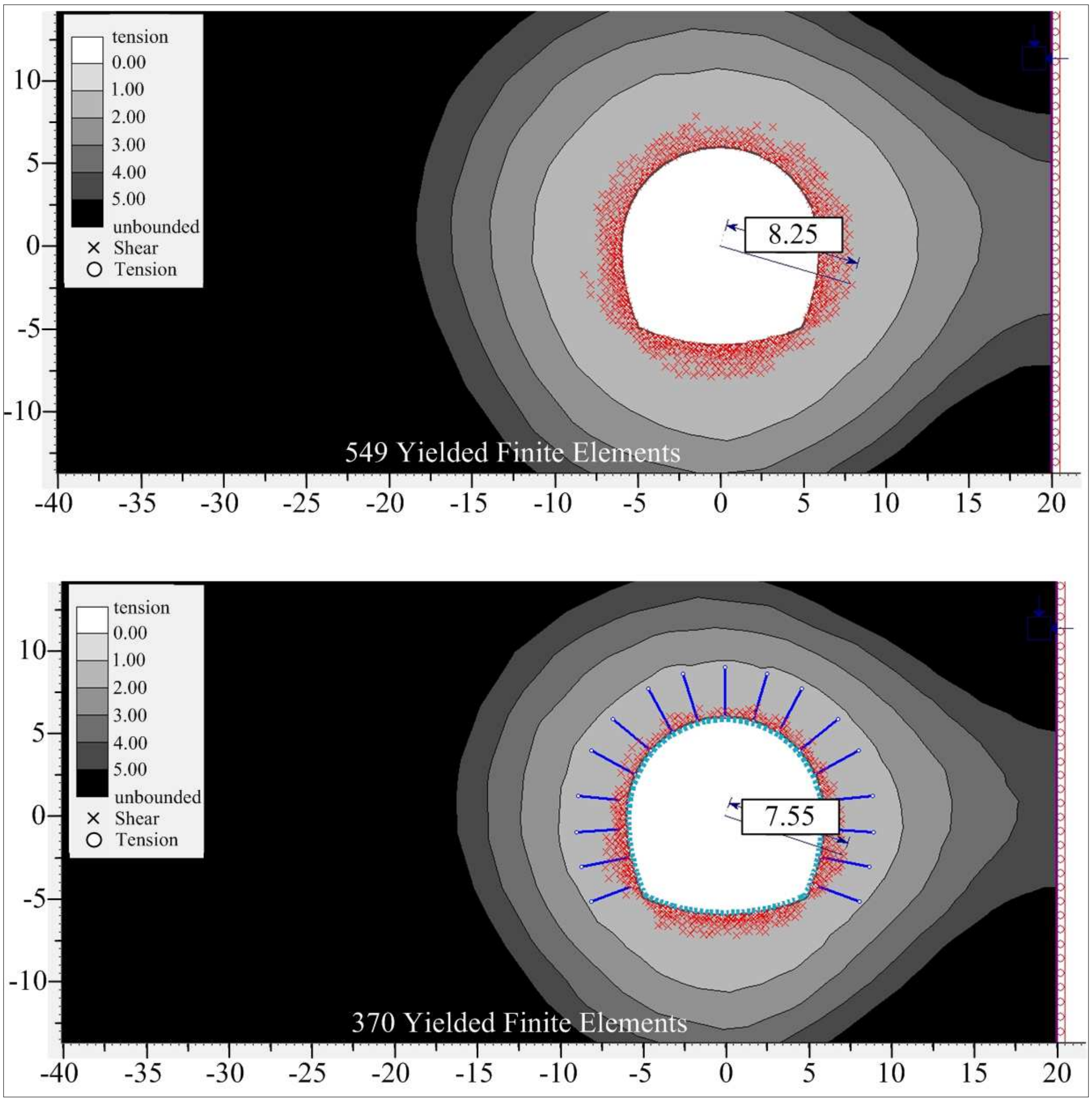

Figure 6.10 Strength factor contour and radius of plastic zone of section 12 of zone 3 before and after support installation.

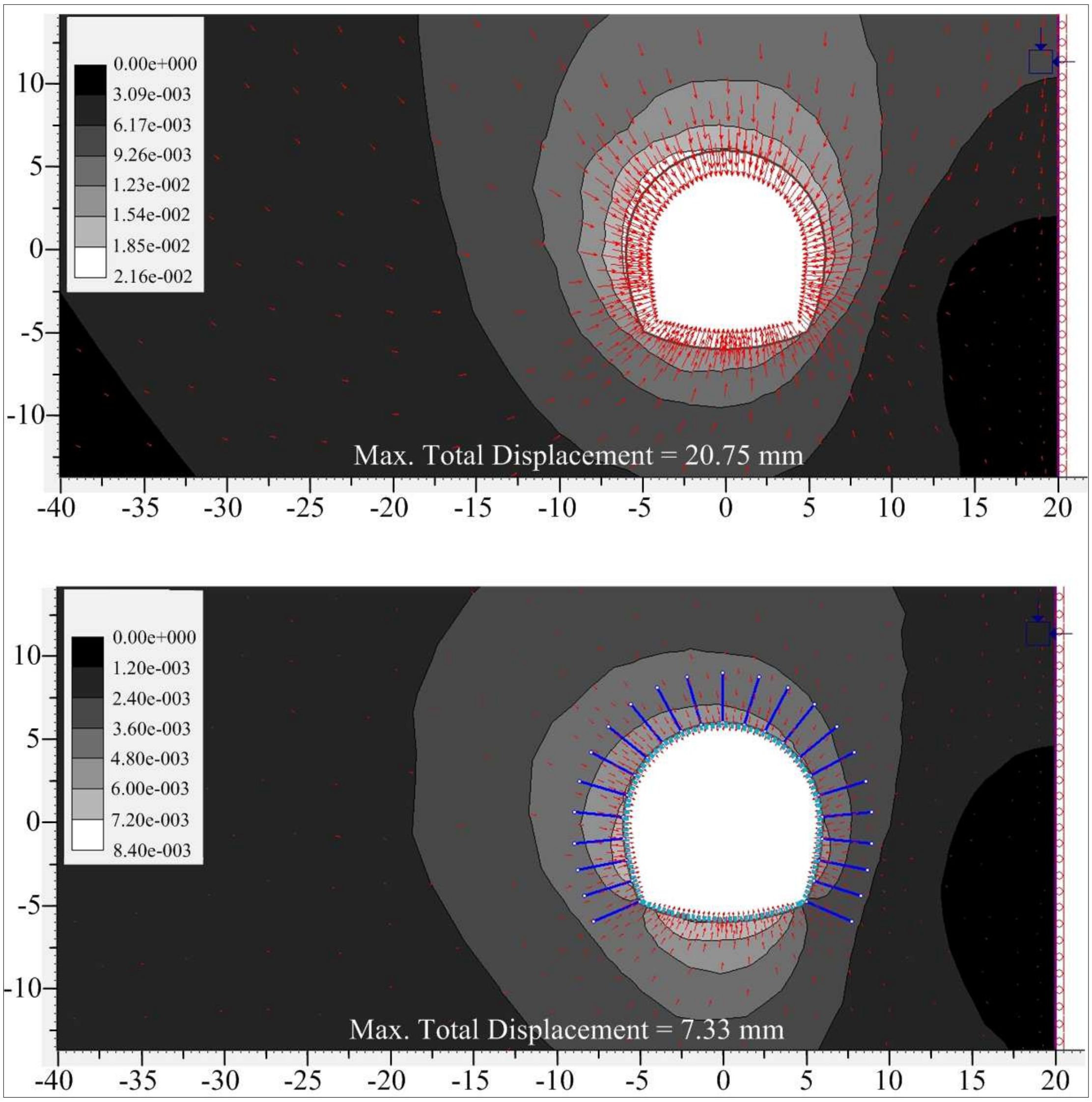

Figure 6.11 Displacement contour and maximum total displacement of section 7 of zone 1 before and after support installation.

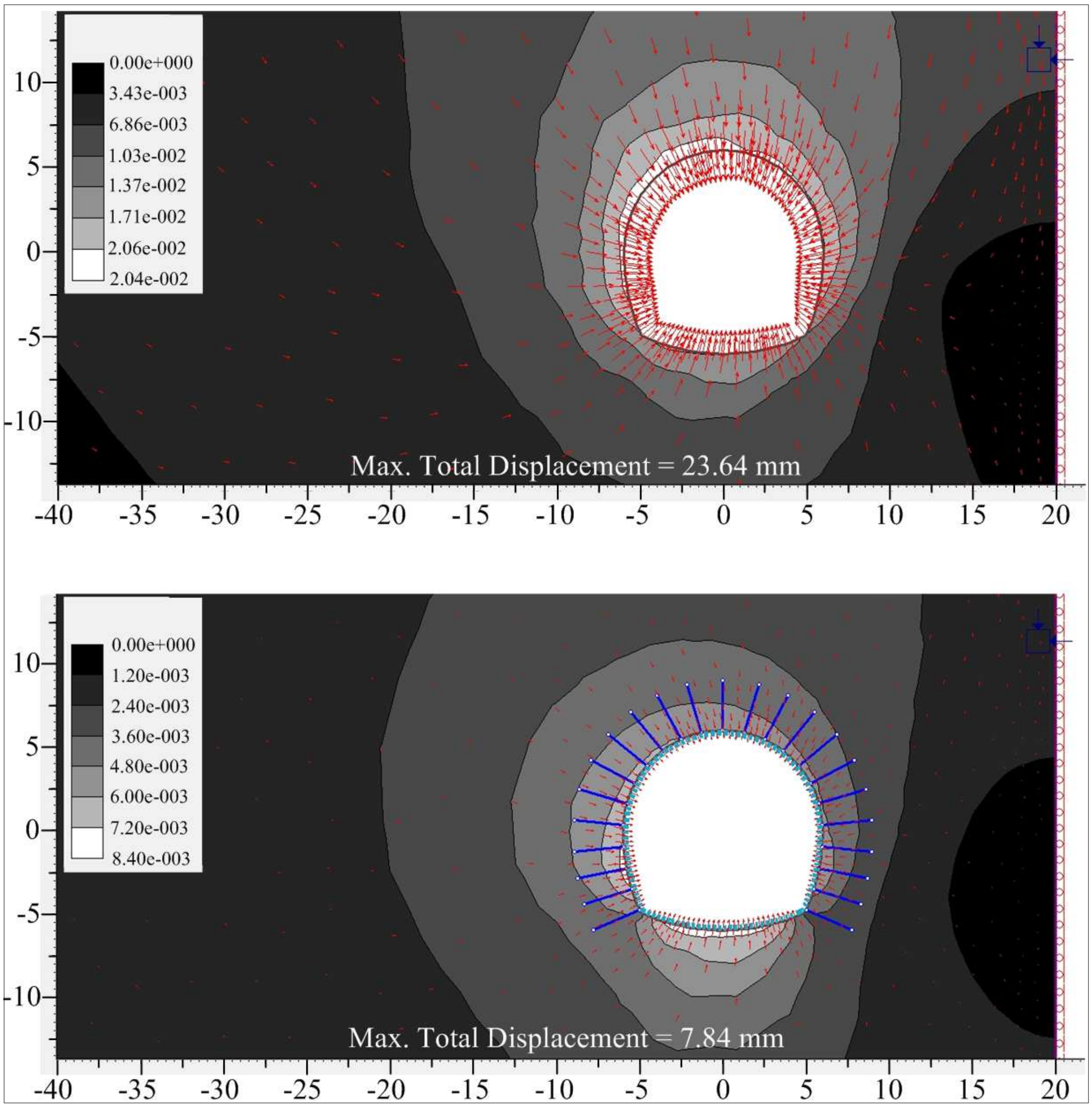

Figure 6.12 Displacement contour and maximum total displacement of section 15 of zone 1 before and after support installation.

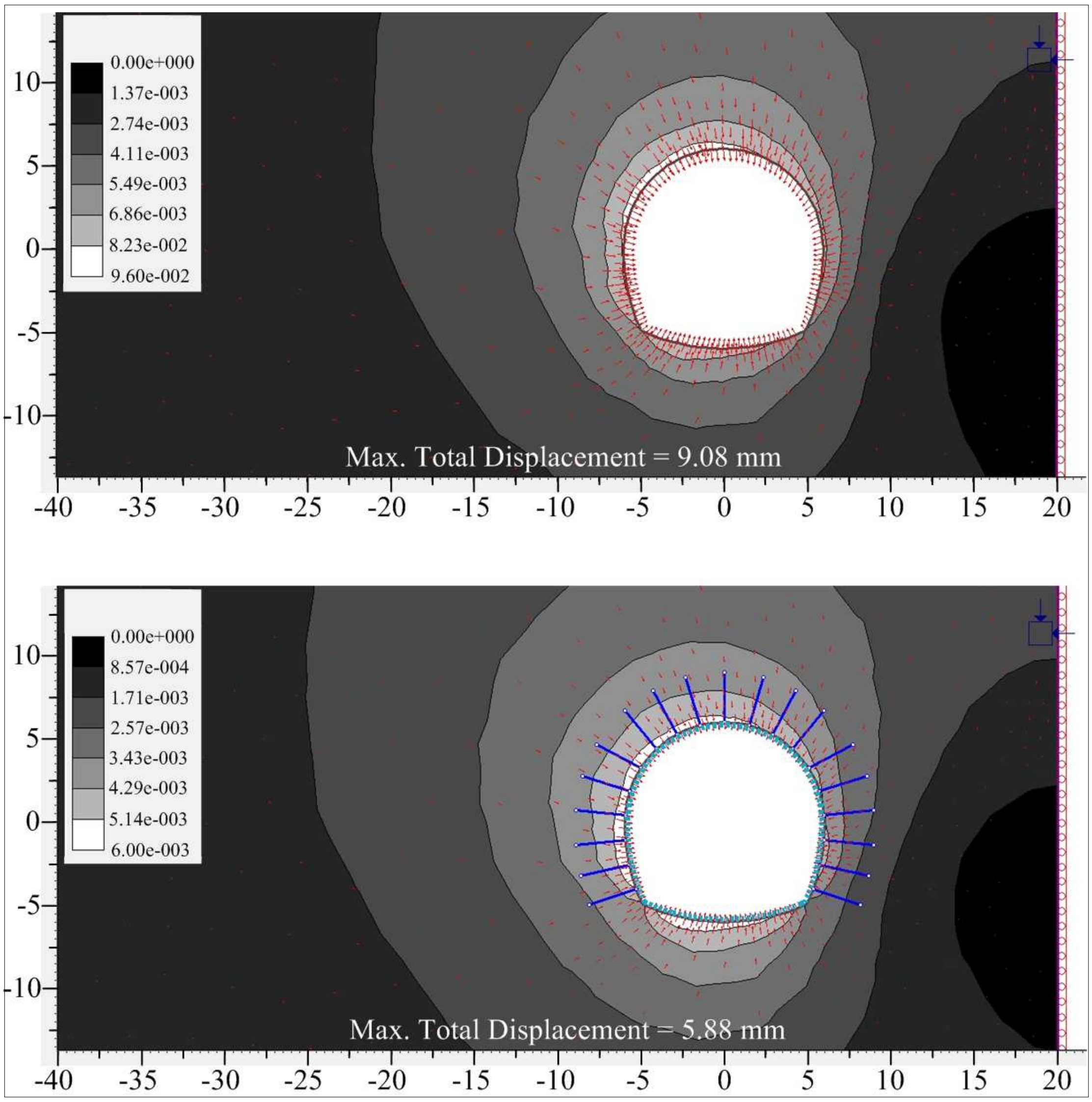

Figure 6.13 Displacement contour and maximum total displacement of section 10 of zone 2 before and after support installation.

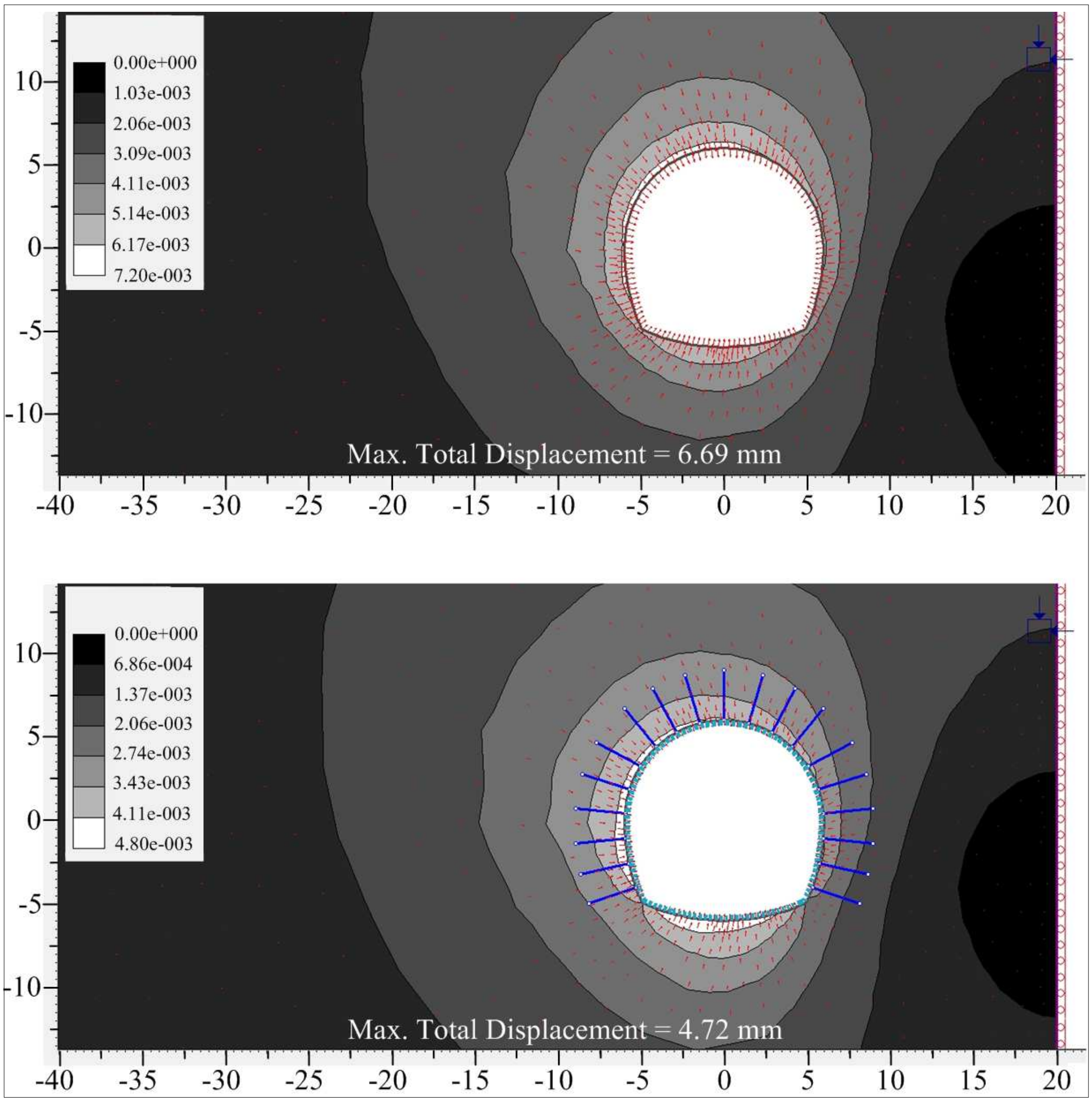

Figure 6.14 Displacement contour and maximum total displacement of section 14 of zone 2 before and after support installation.

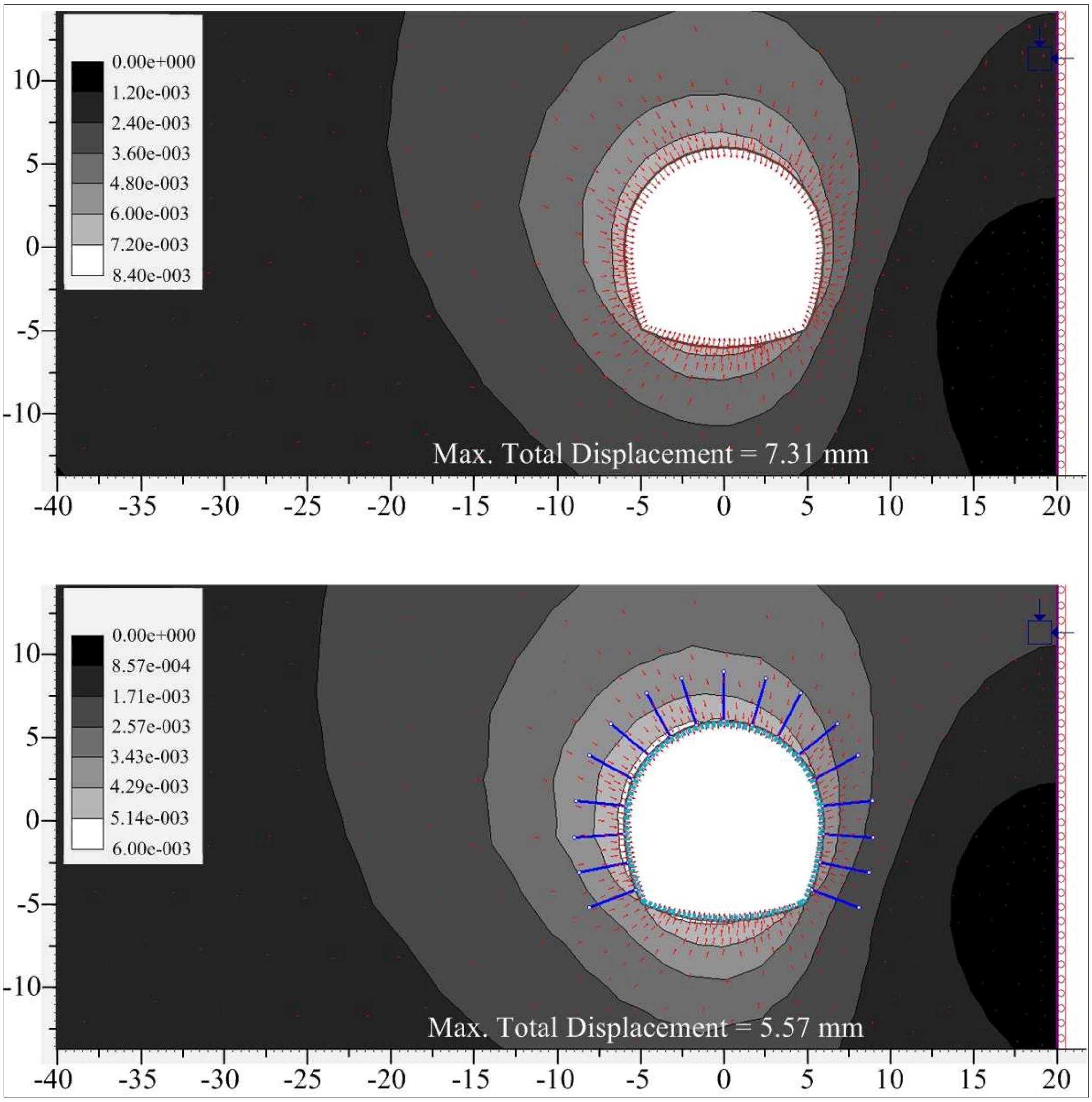

Figure 6.15 Displacement contour and maximum total displacement of section 12 of zone 3 before and after support installation.

6.5 Comparisons

The estimation of reliable support system is one of the most difficult tasks in rock engineering. Several systems have been developed to estimate the support system. In the case of Manipura diversion tunnel constructions, the empirical methods and numerical method are used to assess the reliable support system and the comparison is made between the results obtained from empirical methods and numerical method.

For zone 1, the rock mass rating system provides the support system (systematic rock bolts 4-5 m long, 1-1.5 m spacing, 100-150 mm thickness shotcrete and light to medium steel ribs spaced 1.5 m where required). The NGI tunneling quality index (Q system) provides systematic rock bolts 2.4-3 m long, 1.3-1.5 m spacing and 90-120 mm thickness Fibre reinforced shotcrete. The rock mass index (RMi) system provides special bolting for roof and systematic rock bolts 1 m spacing, and concrete lining or special design shotcrete for roof and 150-250 mm thickness fibre reinforced shotcrete for wall. The numerical method provides systematic rock bolts 3 m long, 1.3 m spacing, and 200 mm thickness shotcrete.

For zone 2, the rock mass rating system recommends the support system; the systematic rock bolts 4-5 m long, 1.5-2 m spacing and 50-100 mm thickness shotcrete. The NGI tunneling quality index (Q system) recommends providing systematic rock bolts 2.4-3 m long, 1.7-2.1 m spacing and 50-90 mm thickness fibre reinforced shotcrete. The rock mass index (RMi) system recommends providing 1 m spacing rock bolts for roof and 1.25 m spacing for wall, and 150-250 mm thickness fibre reinforced shotcrete for roof and 100-150 mm thickness fibre reinforced shotcrete for wall. The numerical method recommends providing systematic rock bolts 3 m long, 1.5 m spacing, and 150 mm thickness shotcrete.

For zone 3, the rock mass rating system suggests the support system; the systematic rock bolts 4 m long, 1.5-2 m spacing and 50-100 mm thickness shotcrete. The NGI tunneling quality index (Q system) suggests providing the systematic rock bolts 2.4-3 m long, 1.7-2.1 m spacing and 40-100 mm thickness unreinforced shotcrete. The rock mass index (RMi) system suggests providing 1.25 m spacing rock bolts for roof and 1.5 m spacing for wall, and 100-150 mm fibre reinforced shotcrete for roof and 70-100 mm thickness fibre reinforced shotcrete for wall. The numerical method provides systematic rock bolts 3 m long, 1.5 m spacing, and 120 mm thickness shotcrete.

The rock mass rating system suggests longer rock bolt than do numerical method and other empirical methods. The thickness of shotcrete is similar with numerical method. The support systems suggested by NGI tunneling quality index (Q system) has thinner shotcrete thickness than does numerical method. The rock mass index (RMi) suggests overestimate support systems than numerical method and other empirical methods. Kaiser and Gale (1985) indicated that the Q system gave a better forecast of support quantities. The results from the rock mass rating system and NGI tunneling quality index (Q system) reasonably agree with numerical method. Very small discrepancies remain.

CHAPTER VII

DISCUSSIONS, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE STUDIES

7.1 Discussions

In this study, empirical methods are applied along with numerical method to assess reliable support systems for rock zones around the Manipura twin tunnels. The RMR system considers the orientation of discontinuities and material strength, which are not directly included in the Q system. However, the Q system considers stress and the joint set number of rock mass, which are only indirectly considered in the RMR system. Both systems include condition of discontinuities and ground water. The largest difference between the RMR and Q systems is the lack of stress parameters in the RMR system. The RMi system has similar input parameters to those of Q system. The RMi system applies best to massive, jointed and crushed rock masses where the joints in the various sets have similar properties. The GSI system is based on the visual impression of the rock structure, in terms of blockiness, and the surface condition of the discontinuities indicated by joint roughness and alternation. All empirical methods have their characteristic limitations to achieve their objectives, therefore, to overcome these limitations, the rock mass strength parameters along the tunnel alignment are estimated by four different empirical methods and their average values are used as input parameters for finite element analysis.

For the rock support systems of the Manipura diversion tunnels, the results indicate that the numerical method suggests more shotcrete thickness than do the empirical methods. The empirical methods suggest longer rock bolt than does the numerical method. This may be because the numerical method considers the overburden as continuous medium and the empirical methods considers the overburden as discontinuous medium.

The comparison is made between the support systems obtained from empirical methods and the results obtained from numerical method for every zone. After several trials of the finite element program are carried out based on the support systems suggested by empirical methods, the final reasonable estimate of tunnel support systems are determined as shown in Table 7.1.

In addition, the excavation methods which are top heading and bench cut excavation method (1.0-1.7 m advance in top heading, install support concurrently with excavation and 10 m from face) is recommended. Concrete lining or special design shotcrete is suggested to support at the corners of the tunnel floor to prevent high stress concentration caused by corner effect.

Table 7.1 Final recommended support systems for the Manipura diversion tunnels.

No.	Zone	Section	Fully grouted rock bolt (20 mm diameter)	Shotcrete
1	1	1	Length = 3 m Spacing = 1.3 x 1.3 m	Thickness = 200 mm with wire mesh
2		3		
3		5		
4		7		
5		15		
6		17		
7		21		
8	2	2	Length = 3 m Spacing = 1.5 x 1.5 m	Thickness = 150 mm with wire mesh
9		4		
10		6		
11		8		
12		10		
13		14		
14		16		
15		18		
16		20		
17	3	9	Length = 3 m Spacing = 1.5 x 1.5 m	Thickness = 120 mm with wire mesh
18		11		
19		12		
20		13		
21		19		

7.2 Conclusions

Rock masses along the Manipura tunnel alignment are characterized by means of rock mass classification systems based on the vertical borehole data, field investigations, engineering geological observations and laboratory test results. According to the results acquired from the rock mass characterizations and stability analysis, there are some stability problems in each zone. The empirical methods, rock mass classification systems, are also employed to estimate support requirements and required support capacities for the diversion tunnels. Five numerical models are constructed by using finite element software, Phase^2 code, to determine the induced stresses, deformations developed around the tunnel and evaluate the performance of the support system recommended by the empirical methods.

The strength parameters required for finite element analysis are estimated from the rock mass classification systems including rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi) and geological strength index (GSI). Used support elements are rock bolts and shotcrete as proposed by the empirical methods. The properties of support elements including length, pattern of bolts and the thickness of shotcrete are similar to those proposed by RMR system and Q system. Several iterations of the finite element program are performed to assess the more appropriate support elements. It leads to the final reasonable estimate of tunnel support systems. When the recommended support systems have been applied, the number of yielded elements and displacements are reduced significantly in numerical analysis. These results indicate that the recommended applied support systems are adequate to obtain tunnel stability. It also proves that the empirical methods reasonably agree with numerical method.

In many tunnel support designs, empirical methods are widely used due to their simplicity, however, they fail to predict interaction between the surrounding rock mass and supporting system.

Based on the result findings, it can be postulated that empirical methods should be applied together with numerical method for the safe tunnel support design. A great deal of judgment may be needed in the application of all kinds of rock mass classification systems in support design.

7.3 Recommendations for Future Studies

Hoek and Brown failure criterion has been used in this research. This failure criterion is widely accepted and has been used in a large number of projects around the world. In addition, the use of the Mohr-Coulomb criterion with strength parameters (cohesion and friction angle) is desirable to assess the effects of discontinuity conditions. The friction angle of the rock mass can be interpreted as the friction resistance along pre-existing discontinuities and asperities on these discontinuities (overriding of asperities). The cohesion can be thought of as the shear resistance of intact rock bridges in the rock mass, or the shear resistance of asperities on a discontinuity surface (shear through asperities). Therefore, studying the application of Mohr-Coulomb criterion for the estimation of underground support systems should be conducted. Moreover, the validity of the proposed support systems, obtained from combination of empirical and numerical method, should be checked by comparing predictions of the rock mass quality with actual measurements carried out during construction.

REFERENCES

ASTM D 7012-07. Compressive strength and elastic moduli of intact rock core specimens under varying states of stress and temperatures **In Annual Book of ASTM Standards**. Philadelphia, PA: 04.08. West Conshohocken: American Society for Testing and Materials.

Barton, N. (2002). Some new Q-value correlations to assist in site characterization and tunnel design. **International Journal of Rock Mechanics and Mining Sciences** 39: 185-216.

Barton, N., Lien, R., and Lunde, J. (1974). Engineering classification of rock masses for the design of tunnel support. **Rock Mechanics** 6: 183-236.

Basarir, H., Ozsan, A., and Karakus, M. (2005). Analysis of support requirements for a shallow diversion tunnel at Guledar dam site, Turkey. **Engineering Geology** 81: 131-145.

Bieniawski, Z. T. (1973). Engineering classification of jointed rock masses. **Transactions, South African Institution of civil Engineer** 15: 335-344.

Bieniawski, Z. T. (1974). **Advances in rock mechanics 2**. Washington, D.C.: National Academy of Science.

Bieniawski, Z. T. (1976). Rock mass classification in rock engineering. **In Proceeding of the Symposium on Exploration for Rock Engineering** (pp 97-106). Johannesburg, South Africa: Geotechnical Division of the South African Institution of Civil Engineers and the South African National Group on Rock Mechanics.

Bieniawski, Z. T. (1978). Determining rock mass deformability. Experience from case histories. **International Journal of Rock Mechanics and Mining Science & Geomechanics Abstracts** 15: 237-147.

Bieniawski, Z. T. (1979). The geomechanics classification in rock engineering application. **In Proceeding of the 4th International Congress on Rock Mechanics** (pp 41-48). Montreux, Switzerland: Plenum.

Bieniawski, Z. T. (1983). The geomechanics classification (RMR System) in design applications to underground excavations. **In Proceeding of the International Symposium on Engineering Geology and Underground Construction** (pp 33-47). Lisbon, Portugal: LNEC.

Bieniawski, Z. T. (1986). Rock mass classification in rock engineering. In Z. T. Bieniawski and A. A. Balkema (Eds.). **Exploration for rock engineering** (pp. 97-106). Johannesburg, South Africa.

Bieniawski, Z. T. (1989). **Engineering rock mass classifications: A complete manual for engneers and geologists in mining, civil, and petroleum engineering**. New York, NY: John Wiley & Sons.

Deere, D. U. (1989). Rock Quality Designation (RQD) after twenty years. **In U.S. Armey Corps of Engineers Contract Report GL-89-1** (pp. 67). Vicksburg, MS: Waterways Experiment Station.

Deere, D. U., Hendron, A. J., Patton, F. D., and Cording, E. J. (1967). Design of surface and near surface construction in rock. **In Proceeding of the 8th US Symposium on Rock Mechanics** (pp 237-302). New York, NY: AIME.

Genis, M., Basarir, H., Ozarslan, A., Bilir, E., and Balaban, E. (2007). Engineering geological appraisal of the rock masses and preliminary support design. **Engineering Geology** 92: 14-26.

Goel, R. K. (1994). **Correlations for predicting support pressures and closures in tunnels**. PhD Thesis, Nagpur University, Nagpur, India.

Goel, R. K., Jethwa, J. L., and Paithankar, A. G. (1995). Indian experiences with Q and RMR systems. **Tunnelling and Underground Space Technology** 10: 97-109.

Grimstad, E. and Barton, N. (1993). Updating of the Q-system for NMT. **In Proceeding of the International Symposium on Sprayed Concrete** (pp 46-66). Fagernes, Oslo: Norwegian Concrete Association.

Hoek, E. (2003). **Numerical modeling for shallow tunnels in weak rocks** [On-line]. Available: http://www.rocscience.com.

Hoek, E. and Brown, E. T. (1988). The Hoek-Brown failure criterion, a 1988 update. **In Proceeding of the 15th Canadian Rock Mechanics Symposium** (pp 31-48). Toronto, Canada: University of Toronto.

Hoek, E. and Brown, E. T. (1990). **Underground excavations in rock**. London, UK: Institution of Mining and Metallurgy.

Hoek, E. and Brown, E. T. (1997). Practical estimates or rock mass strength. **International Journal of Rock Mechanics and Mining Science & Geomechanics Abstracts** 34: 1165-1186.

Hoek, E., Carranza-Torres, C., and Corkum, B. (2002). Hoek-Brown failure criterion. **In Proceeding of the 5th North American Rock Mechanics Symposium and the 17th Tunnelling Association of Canada Conference** (pp 267-273). Toronto, Ontario: University of Toronto.

Hoek, E. and Diederichs, M. S. (2005). Empirical estimation of rock mass modulus. **International Journal of Rock Mechanics and Mining Sciences** 43: 203-215.

Hoek, E., Kaiser, P. K., and Bawden, W. F. (1995). **Support of underground excavations in hard rock**. Rotterdam, The Netherlands: Balkema, Rotterdam.

Hoek, E., Marinos, P., and Benissi, M. (1998). Applicability of the geological strength index (GSI), Classification for very weak and sheared rock masses. The case of the Athens Schist Formation. **Bulletin of Engineering Geology and the Environment** 57: 151-160.

ISRM (1981). Rock characterization, testing and monitoring. In E. T. Brown (Ed.), **ISRM Suggested methods** (pp. 211). New York, NY: Pergamon.

Kaiser, T. K. and Gale, A. D. (1985). **Evaluation of cost and emprical support design at B.C. rail tumbler ridge tunnels Canadian tunneling**. New York, NY: Tunneling Association of Canada, Wiley.

Mahtab, M. A. and Grasso, P. (1992). **Geomechanics principles in the design of tunnels and caverns in rocks**. New York, NY: Elsevier.

Marinos, P. and Hoek, E. (2000). GSI: A geologically friendly tool for rock mass strength estimation. **In Proceeding of the International Conference on Geotechnical and Geological Engineering** (pp 1422-1442). Melbourne: ICMS.

Marinos, P. and Hoek, E. (2001). Estimating the geotechnical properties of heterogeneous rock masses such as flysch. **Bulletin of the Engineering Geology & the Environment** 60: 85-92.

Oo, Z. S. (2003). **Systematic approach to geotechnical investigation of `Ye Nwe diversion tunnel**. M. Eng. Thesis, Yangon Technological University, Yangon, Myanmar.

Ozsan, A. and Basarir, H. (2002). Support capacity estimation of a diversion tunnel in weak rock. **Engineering Geology** 68: 319-331.

Palmstrom, A. (1982). The volumetric joint count - a useful and simple measure of the degree of rock jointing. **In Proceeding of the 4th International Congress, International Association of Engineering Geology** (pp 221-228). Dehli: Indian Society of Engineering Geology.

Palmstrom, A. (1995). **RMi - A rock mass characterization system for rock engineering purposes**. Ph.D. Thesis, University of Oslo, Norway.

Palmstrom, A. (2001). **RMi-calc., version 2 and RMi support, version 3.1** [On-line]. Available: http://www.rockmass.net.

Palmstrom, A. (2009). **Combining the RMR, Q, and RMi classification systems** [On-line]. Available: http://www.rockmass.net.

Palmstrom, A. and Broch, E. (2006). Use and misuse of rock mass classification systems with particular reference to the Q-system. **International Journal of Tunnels and Underground Space Technology** 21: 575-593.

Ramamurthy, T. (1986). Stability of rock mass. **the 8th Annual Lecture, Indian Geotech Journal** 12: 1-74.

Rasouli, M. (2009). Engineering geological studies of the diversion tunnel focusing on stabilization analysis and support design, Iran. **Engineering Geology** 108: 208-224.

Read, S. A. L., Richards, L. R., and Perrin, N. D. (1999). Applicability of the Hoek-Brown failure criterion to New Zealand greywacke rocks. **In Proceeding of the 9th International Society for Rock Mechanics Congress** (pp 655-660). Paris, France: Pergamon.

Ritter, W. (1879). **Die Statik der Tunnelgewölbe**. Berlin: Springer.

Rocscience (2004). **RocData User's Guide** [On-line]. Available: http://www.roccscience.com.

Rocscience (2007). **Phase^2 User's Guide** [On-line]. Available: http://www.rocscience.com.

Serafim, J. L. and Pereira, J. P. (1983). Considerations of the geomechanics classification of Bieniawski. **In Proceeding of the International Symposium Engineering Geology and Underground Construction** (pp 1133-1142). Rotterdam, The Netherlands: Balkema, Rotterdam.

Singh, B., Viladkar, M. N., Samadhiya, N. K., and Mehrota, V. K. (1997). Rock mass strength parameters mobilized in tunnels. **Tunneling and Underground Space Technology** 12: 47-54.

Sonmez, H. (2001). **Investigation of the applicability of the Hoek-Brown criteria to the failure behaviour of the fissured clays**. Ph.D. Thesis, Hacettepe University, Ankara, Turkey.

Sonmez, H. and Ulusay, R. (1999). Modifications to the Geological Strength Index (GSI) and their applicability to stability of slopes. **International Journal of Rock Mechanics and Mining Sciences** 36: 743-760.

Win, H. A. K. and Aung, K. (2007). **Engineering Geology Office 2 Report**. Myanmar: Irrigation Department.

APPENDIX A

TECHNICAL PUBLICATIONS

PUBLICATIONS

Soe, B. and Fuenkajorn, K. (2010). **Analysis of support design for Manipura diversion tunnels in Myanmar.** In Proceeding of EIT-JSCE Joint Symposium on Engineering for Geo-Hazards: Earthquakes and Landslides - for Surface and Subsurface Structures, Bangkok, Thailand, 6-8 September, 2010.

Soe, B. and Fuenkajorn, K. (2010). **Stability analysis and support design of Manipura diversion tunnels in Myanmar.** In Proceeding of the ISRM International Symposium 2010 and 6th Asian Rock Mechanics Symposium, New Delhi, India, 23-28 October, 2010.

Analysis of support design for Manipura diversion tunnels in Myanmar

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Keywords: tunnel, analysis, design, support, Manipura

ABSTRACT: This paper presents the analysis of support design for two parallel diversion tunnels of the Manipura multi-purposed dam project in Myanmar. Rock masses along the diversion tunnel alignment are classified by rock mass classification systems including rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi), and geological strength index (GSI). The relationship between the rock mass quality and rock support measures in accordance with existing rock mass classification systems is used to estimate the support system for the tunnels. A series of numerical simulations (using Phase^2 code) is performed to assess the stability conditions of the tunnels with and without the support systems. Mohr-Coulomb failure criterion is used in numerical analysis. The required input parameters for numerical modeling (Phase^2 code) are defined from empirical relations. Several trials of the finite element analysis are performed to optimize the results obtained from empirical methods and numerical method. After installation of the recommended support system, the simulated extent of yielded zone and the radius of plastic zone significantly decrease. The use of empirical methods together with numerical method can provide the reasonable support systems for the underground openings.

1 INTRODUCTION

The Manipura multi-purposed dam project will provide the irrigated water and hydro-electricity to the local area. The project is located on the Manipura River, 52 km from the Kalay Town, Kalay Township, Sagaing Division in Myanmar (Figure 1). The two parallel tunnels are 1050 m long with 12 m wide modified horseshoe shape. They are driven underneath the south bank mountain to divert the water whilst a massive rock-fill dam is built. Failure to do so will result in an inadequate basis for the design and could be very costly when unexpected problems are sometimes encountered at a later stage in the project. Based on the geological investigations, some stability problems are expected in Manipura diversion tunnel constructions.

Rock masses along the diversion tunnel alignment are classified by rock mass classification systems including rock mass rating (RMR), NGI tunneling quality index (Q system), rock mass index (RMi) and geological strength index (GSI). Their rating values are used to determine tunnel support systems and to estimate the rock mass parameters. The finite element program, Phase^2 version 6.0 developed by Rocscience (2007), is used to evaluate deformations, developed plastic zone around the tunnels and the performance of support system. The rock mass parameters estimated by empirical relations are used as input data for the numerical modeling. Optimization between the empirical and numerical results is made to obtain the suitable support design for the Manipura diversion tunnels.

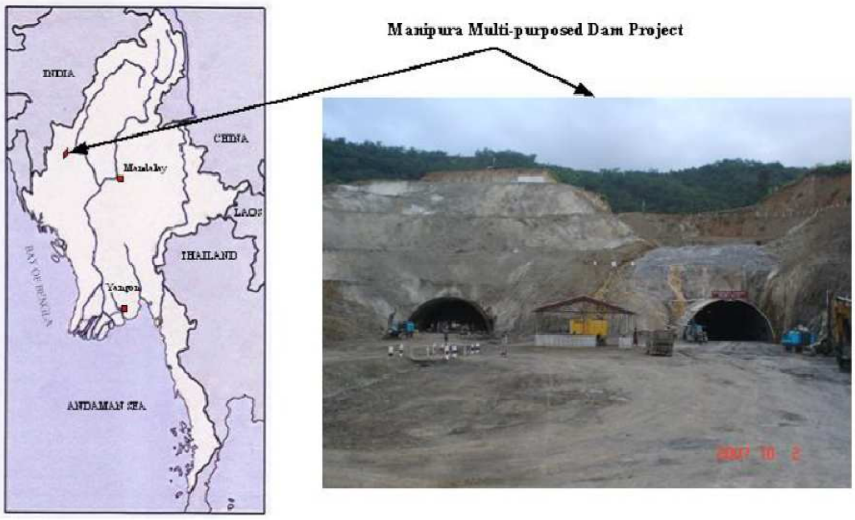

Figure 1. Location of the Manipura multi-purposed dam project.

2 GEOLOGY AND ENGINEERING GEOLOGY OF THE PROJECT AREA

Geological data collection is carried out to classify the rock mass in the project area. The integrated data can assist in the design solution and anticipate any unfavorable geological condition, which can give rise to problem during the excavation of the openings. The tunnels are in the Chin Flysch of Tertiary as a part of the Rakhine (Arakan)-Chin-Naga Hill ranges (Win and Aung, 2007). Most of the main Rakhine-Chin-Naga Hill ranges are thick monotonous series of unfossiliferous marine flysch-type sediments including slaty shales, phyllite, slates and poorly-graded calcareous sandstones. Quartz and calcite veins are common. The sequence is predominantly argillaceous (slate), consisting mainly of yellowish grey to dark grey, hard, locally ferruginous and calcareous. Slate, the major rock unit, is fine grained metamorphic rock derived mostly from shale. Approximately 20-25% of the rock sequence consists of hard bedded greywackes sandstone, usually find-grained and locally micaceous and calcareous.

Engineering geological descriptions of the rock masses are based on the procedures suggested by ISRM (1981). The area shows varying weathering grades of rock. The tunnel alignment is divided into three distinct zones of rock mass: zone 1: moderately to highly weathered slate; zone 2: slightly to moderately weathered slate; and zone 3: slightly weathered slate with alternation of greywacke sandstone band. Each of which has different engineering geological properties and lithologic types. The major trend of the joint is 320/60 (strike/dip) and the tunnel direction is N60E (Win and Aung, 2007). The strike of the joints and tunnel axis are nearly perpendicular. According to the rock mass rating system (RMR), the tunnels driven from the inlet are under very favorable condition and driven from the outlet are under favorable condition. Total 21 different sections are classified based on their locally input variables in terms of engineering geological and geotechnical parameters and the induced overburden stress. A section along the tunnel alignment is given in Figure 2.

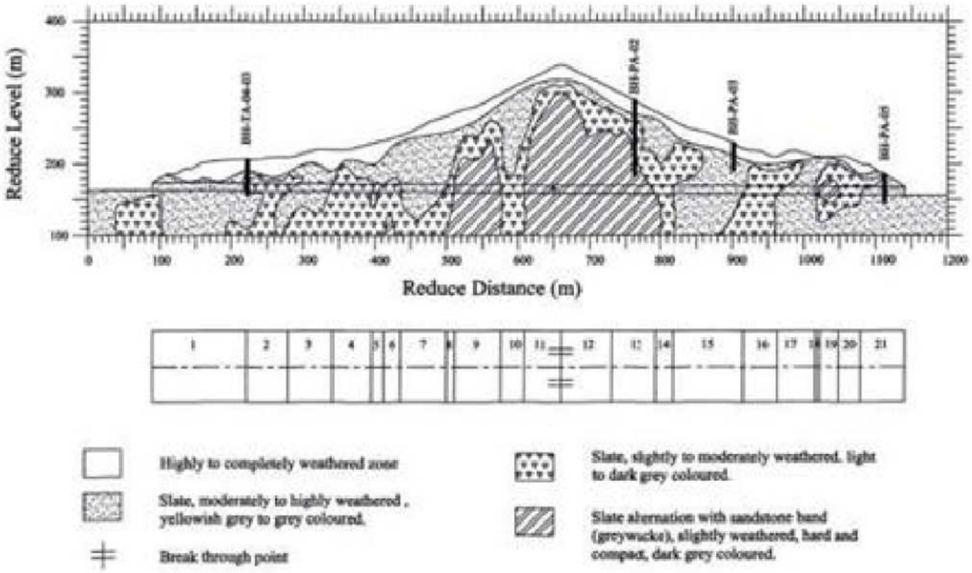

Figure 2. Geological cross-section along the Manipura diversion tunnel alignment.

3 LABORATORY TESTING

Laboratory experiments were carried out to determine the physical and mechanical properties of the intact rock. Rock core samples were collected from vertical and horizontal borehole drilling investigations. All laboratory tests were conducted in accordance with the relevant ASTM standards and the ISRM suggested methods. Uniaxial compressive strength of intact rock for zones 1, 2 and 3 are 16.71, 26.22 and 41.95 MPa, respectively. Unit weight of zone 1 is 26.59 kN/m^3, zone 2 is 27.76 kN/m^3 and zone 3 is 28.94 kN/m^3. Young's modulus of zone 1 is 10.34 GPa, zone 2 is 21.04 GPa and zone 3 is 22.83 GPa. Poisson's ratio of zones 1, 2 and 3 are 0.35, 0.34 and 0.30, respectively. Hoek and Brown constant (m_i) of intact rock of zones 1, 2 and 3 are 7.

4 ROCK MASS CHARACTERIZATIONS

Four individual rock mass classification systems including RMR (Bieniswski, 1973), Q system (Barton, Lien, and Lunde, 1974), RMi (Palmstrom, 1995) and GSI (Hoek et al., 1995), are currently applied to classify the rock mass quality along the tunnel alignment. The classification table proposed by Bieniawski (1989) and the modified quantitative GSI table proposed by Sonmez (2001) are used in this study. For the estimation of RMi value and RMi support design, the spread sheet softwares (RMi-calc., version 2 and RMi support, version 3.1) are applied. The rock mass classes for every zone based on these four classification systems are summarized in Table 1.

Table 1. Summary of the rock mass classes and their values.

Zone	RMR		Q		RMi		GSI	
	Value	Class	Value	Class	Value	Class	Value	Class
1	31	Poor rock	0.5333	Very Poor	0.2858	Low	28	Poor
2	43	Fair rock	2.4930	Poor	0.9310	Low	38	Fair
3	47	Fair rock	3.0400	Poor	2.5153	Medium	44	Good

5. GEOTECHNICAL ROCK MASS PARAMETERS ESTIMATION

Rock mass properties such as deformation modulus of the rock mass, Mohr-Coulomb parameters and rock mass strength are utilized parameters for the stability analysis and support design of tunnel. Field tests to determine some parameters directly are time consuming and expensive. Some empirical relations used here to calculate rock mass parameters are given in Tables 2 through 4. The calculated values are presented in Tables 5 through 8.

Table 2. Empirical relations for estimating deformation modulus of rock mass (E_m) in GPa.

Eq. no.	Equations		References
(1)	$E_m = 2RMR-100$	For RMR > 50	Bieniawski (1978)
(2)	$E_m = 10^{\left(\frac{RMR-10}{40}\right)}$	For RMR < 50	Serafim and Pereira (1983)
(3)	$E_m = 0.1\left(\frac{RMR}{10}\right)^3$		Read et al. (1999)
(4)	$E_m = 25 \log Q$	For Q > 1	Grimstad and Barton (1993)
(5)	$E_m = 10Q_c^{1/3} = 10\,(Q\times\frac{\sigma_c}{100})^{1/3}$		Barton (2002)
(6)	$E_m = 5.6\ RMi^{0.375}$	For RMi > 0.1	Palmstrom (1995)
(7)	$E_m = (1-\frac{D}{2})\sqrt{\frac{\sigma_c}{100}}\times 10^{\left(\frac{GSI-10}{40}\right)}$	For $\sigma_c \le 100$ MPa	Hoek et al. (2002)
(8)	$E_m = (1-\frac{D}{2})\times 10^{\left(\frac{GSI-10}{40}\right)}$	For $\sigma_c > 100$ MPa	Hoek et al. (2002)

Table 3. Empirical relations for estimating Mohr-Coulomb parameters (c_j and φ_j).

Eq. no.	Equations		References
(12)	$c_j = \left(\frac{RQD}{J_n}\right)\left(\frac{1}{SRF}\right)\left(\frac{\sigma_c}{100}\right)$	(MPa)	Barton (2002)
(13)	$\varphi_j^{\circ} = \tan^{-1}\left(\frac{J_r\,.\,J_w}{J_a}\right)$		Barton (2002)

Table 4. Empirical relations for estimating rock mass strength (σ_{cm}) in MPa.

Eq. no.	Equations		References
(9)	$\sigma_{cm} = \sigma_c \exp\left(\frac{RMR-100}{18.75}\right)$		Ramamurthy (1986)
(10)	$\sigma_{cm} = \left(\frac{5.5.\gamma.Q_N}{\sigma_c\ B^{0.1}}\right)^{1/3}$	B = Width of the tunnel	Goel (1994)
(11)	$\sigma_{cm} = RMi = \sigma_c\ JP$	JP = Jointing parameter	Palmstrom (1995)

Table 5. Calculated values of rock mass deformation modulus (E_m) in GPa.

Zone	From RMR		From Q		From RMi	From GSI	Avg.	SD
	Eq. 2	Eq. 3	Eq. 4	Eq. 5	Eq. 6	Eq. 7		
1	3.3497	2.9791		4.4664	3.5012	1.1521	3.0897	1.2144
2	6.6834	7.9507	9.9181	8.6788	5.4519	2.5664	6.8749	2.6191
3	8.4140	10.3823	12.0718	10.8131	7.9142	4.5853	9.0301	2.6698

Table 6. Mohr-Coulomb parameter (c_j) in MPa.

Zone	From RMR	From Q (Eq. 12)	From GSI (RocData 3.0)	Avg.	SD
1	0.1526	0.0891	0.2170	0.1529	0.0640
2	0.2158	0.4358	0.3640	0.3385	0.1122
3	0.2368	0.8502	0.6820	0.5897	0.3170

Table 7. Mohr-Coulomb parameter (φ_j°).

Zone	From RMR	From Q (Eq. 13)	From GSI (RocData 3.0)	Avg.	SD
1	20.26	45.00	30.51	31.92	12.43
2	26.58	56.31	36.15	39.68	15.18
3	28.68	56.31	37.76	40.92	14.08

Table 8. Rock mass strength (σ_{cm}) in MPa.

Zone	Eq. 9	Eq. 10	Eq. 11	(RocData3.0)	Avg.	SD
1	0.4215	0.7536	0.2858	0.2490	0.4275	0.2297
2	1.2542	0.8386	0.9310	0.7650	0.9472	0.2156
3	2.4839	0.5837	2.5153	1.7710	1.8385	0.9044

6. SUPPORT DESIGN USING EMPIRICAL APPROACHES

Traditional guidelines for the rock support are used based on the results of the site characterizations including rock mass rating system (RMR), NGI tunneling quality index (Q system) and rock mass index (RMi). All these systems have the relationship between the rock mass quality and the empirical design rules to estimate adequate rock support measures such as rock bolt, shotcrete and steel set. The results are summarized in Table 9.

The RMR system considers the orientation of discontinuities and material strength, which are not directly included in the Q system. However, the Q system considers stress and the joint set number of rock mass, which are only indirectly considered in the RMR system. Both systems include condition of discontinuities and ground water. The largest difference between the RMR and Q systems is the lack of stress parameters in the RMR system. The RMi system has similar input parameters to those of Q system. Ground water condition is indirectly considered in RMi system. All empirical methods have their characteristic limitations to achieve their objectives. Support design by empirical methods alone is conservative because these empirical methods cannot adequately calculate stress distributions, support performance and deformational behavior around a tunnel.

Table 9. Suggested support systems based on empirical approaches.

Zone	From RMR	From Q	From RMi
1	BL: 4-5 m, BS: 1-1.5 m CS(W): 100-150 mm WS(W): 100 mm SR: 1.5 m spaced where required	BL: 2.4-3 m BS: 1.5-1.7 m CS(W): 90-120 mm	special bolting, concrete lining or special design shotcrete for roof and 150-250 mm thickness fibre reinforced shotcrete for wall
2	BL: 4 m, BS: 1.5-2 m CS(W): 50-100 mm WS(W): 30 mm	BL: 2.4-3 m BS: 1.7-2.1 m CS: 40-100 mm	CBS: 1 × 1 m, WBS: 1.25 × 1.25 m CS(W) : 150-250 mm WS(W): 100-150 mm
3	BL: 4 m, BS: 1.5 × 2 m CS(W): 50-100 mm WS(W): 30 mm	BL: 2.4-3 m CBS: 1.7-2.1 m CS: 40-100 mm	CBS: 1.25 × 1.25 m WBS: 1.5 × 1.5 m CS(W) : 100-150 mm WS(W): 70-100 mm

BL: bolt length for crown and wall, BS: systematic bolt spacing for crown and wall, CS: shotcrete thickness for crown
WS: shotcrete thickness for wall, CBS: systematic bolt spacing for crown, (W): shotcrete with wire mesh
WBS: systematic bolt spacing for wall, SR: steel rib.

7. SUPPORT DESIGN USING NUMERICAL METHOD

A series of numerical simulations is performed by using a finite element program, Phase^2 version 6.0. Mohr-Coulomb failure criterion is used in finite element analysis. In situ stress is assumed as hydrostatic. Automatic mesh around the tunnel is generated. Support elements used are composed of rock bolts and shotcrete as proposed by the empirical methods. In this analysis, boundary conditions are defined as restrained X for both sides boundary, restrained Y for the lower boundary and free surface for the upper boundary. The strength factor contours, yielded finite elements, radius of plastic zone, maximum total displacement and displacement vectors are described in Figure 3. The results of the supported and unsupported cases are summarized in Table 10.

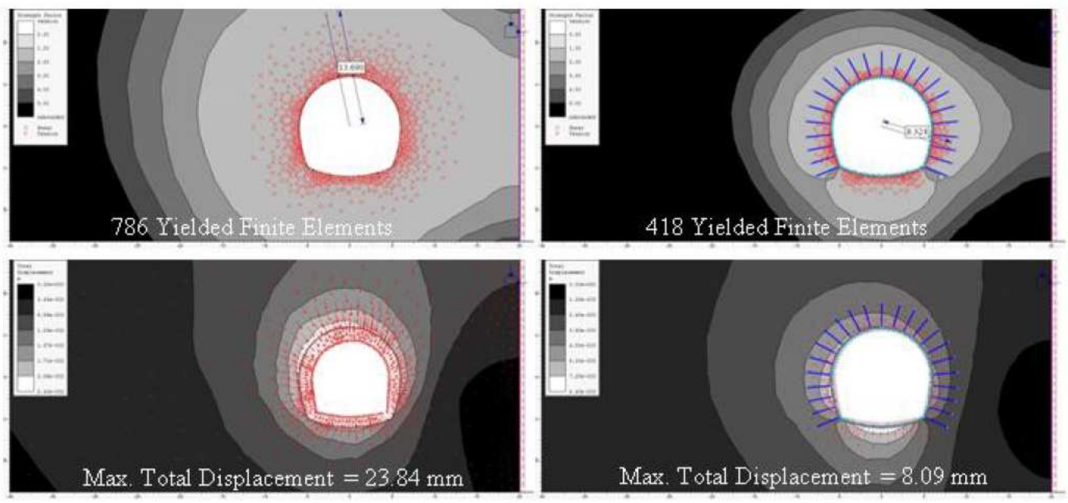

Figure 3. Yielded finite elements, radius of plastic zone, strength factor contour, maximum displacement and displacement vectors for unsupported and supported case of zone 1.

The small amount of total displacement is observed in each zone. The extent of plastic zone and yielded elements suggest that there would be some stability problems for tunnels. Phase^2 is a small strain finite element program and thus it cannot accommodate the very large strains. In the case of Manipura diversion tunnel support design, it is more important to consider the extent of plastic zone and yielded elements rather than the magnitude of displacement. After support installation, both the yielding zone and the radius of plastic zone are decreased as shown in Figure 3. Maximum total displacement is also reduced for the supported cases. This suggests that the applied support systems are adequate to obtain tunnel stability.

Table 10. Yielded finite elements, radius of plastic zone and maximum total displacement for unsupported and supported cases.

Zone	Yielded finite elements (no)		Radius of plastic zone (m)		Maximum total displacement (mm)	
	unsupported	supported	unsupported	supported	unsupported	supported
1	786	418	13.690	8.324	23.84	8.09
2	626	440	9.001	7.855	6.32	4.69
3	580	444	8.393	7.837	7.12	5.59

8. DISCUSSIONS

The results indicate that the numerical method suggests thicker shotcrete and shorter bolt than do the empirical methods. This may be because the numerical method considers the overburden as continuous medium and the empirical methods considers as discontinuous medium. After several trials, a reasonable estimate of tunnel support systems is achieved as presented in Table 11.

Concrete lining or special design shotcrete is suggested to support at the corners of the tunnel floor to prevent high stress concentration caused by corner effect. The excavation method which is top heading and bench cut excavation method (1.0-1.7 m advance in top heading, install support concurrently with excavation, 10 m from face) is also recommended.

Table 11. Recommended support systems for Manipura diversion tunnels.

Zone	Rock bolt (20 mm diameter)	Shotcrete
1	Length = 3 m, Spacing = 1.3 x 1.3 m	Thickness = 200 mm with wire mesh
2	Length = 3 m, Spacing = 1.5 x 1.5 m	Thickness = 150 mm with wire mesh
3	Length = 3 m, Spacing = 1.5 x 1.5 m	Thickness = 120 mm with wire mesh

9. CONCLUSIONS

Rock mass characterization is performed by using rock mass classification systems. The required support measures are determined by using the traditional guidelines which are based on rock mass classification systems. Six numerical models are constructed by using Phase^2 code to determine the induced stress, deformations developed around the tunnel and to evaluate the performance of the support systems recommended by the empirical methods. Empirical relations are used to estimate the required rock mass parameters for finite element analysis. Used support elements are rock bolts and shotcrete as proposed by the empirical methods. Several trials of the finite element analysis are performed to assess the more appropriate support elements. It leads to the final reasonable estimate of tunnel support systems. When the recommended support systems are applied, the extent of yielded zone and displacement significantly reduce in numerical analysis. These results indicate that the recommended applied support systems are adequate to obtain tunnel stability. It also suggests that the empirical methods reasonably agree with the numerical method.

It can be postulated that a more reliable support design can be achieved by using a numerical method supported by the empirical methods. A great deal of judgment may be needed in the application of all kinds of rock mass classification systems in support design.

ACKNOWLEDGMENT

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REFERENCES

Barton, N. (2002). Some new Q-value correlations to assist in site characterization and tunnel design. *Int. J. of Rock Mech. and Min. Sci.* Vol. 39, pp 185-216.

Barton, N., Lien, R. and Lunde, J. (1974). Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics.* Vol. 6, pp. 183-236.

Bieniawski. Z. T. (1973). Engineering classification of jointed rock masses. *Transactions, South African Institution of Civil Engineer.* Vol. 15, pp. 335-344.

Bieniawski. Z. T. (1978). Determining rock mass deformability. Experience from case histories. *International Journal of Rock Mechanics and Mining Science & Geomechanics Abstracts.* Vol. 15, pp. 237-247.

Bieniawski. Z. T. (1989). *Engineering Rock Mass Classifications: A Complete Manual for Engneers and Geologists in Mining, Civil, and Petroleum Engineering.* John Wiley & Sons, New Yoke. 251 pp.

Goel, R.K. (1994). Correlations for Predicting Support Pressures and Closures in Tunnels. *PhD Thesis,* Nagpur University, Nagpur, India. 308 pp.

Grimstad, E. and Barton, N. (1993). Updating of the Q-system for NMT. *International Symposium on Sprayed Concrete.* Fagernes, Proceeding. pp. 46-66.

Hoek, E., Carranza-Torres, C. and Corkum, B. (2002). Hoek-Brown Failure criterion – 2002 edition. *Proc. NARMS-TAC Conference,* Toronto. Vol. 1, pp. 267-273.

Hoek, E., Kaiser, P. K. and Bawden, W. F. (1995). *Support of Underground Excavations in Hard Rock.* Balkema, Rotterdam. 215 pp.

ISRM (1981). Rock characterization, testing and monitoring. *In: Brown, E. T. (Ed), ISRM Suggested Methods.* Pergamon Press, New York. 211pp.

Palmstrom, A. (1995). RMi - A Rock Mass Characterization System for Rock Engineering Purposes. *Ph.D.Eng. Thesis,* University of Oslo, Norway.

Ramamurthy, T. (1986). Stability of rock mass, 8th Annual Lecture, *Indian Geotech. J.* Vol.12, pp. 1–74.

Read, S. A. L., Richards, L. R. and Perrin, N. D. (1999). Applicability of the Hoek–Brown failure criterion to New Zealand greywacke rocks. *Proceeding 9th International Society for Rock Mechanics Congress,* Paris. Vol. 2, pp. 655-660.

Rocscience, (2007). Phase^2 User's Guide. Rocscience Inc, Toronto, Ontario, Canada. http://www.rocscience.com.

Serafim, J. L. and Pereira, J. P. (1983). Considerations of the Geomechanics Classification of Bieniawski. *Proceedings International Symposium Engineering Geology and Underground Construction,* Balkema, Rotterdam. Vol. 1, pp. 1133-1142.

Sonmez, H. (2001). Investigation of the Applicability of the Hoek–Brown Criteria to the Failure Behavior of the Fissured Clays. *Ph.D. Thesis,* Hacettepe University. 215pp , unpublished, in Turkish.

Win, H. A. K. and Aung, K. (2007), *Engineering Geology Office 2 Report.* Irrigation Department, Myanmar, unpublished. 28 pp.

STABILITY ANALYSIS AND SUPPORT DESIGN OF MANIPURA DIVERSION TUNNELS IN MYANMAR

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ABSTRACT

This paper presents the stability analysis and support design for two parallel diversion tunnels of the Manipura multi-purposed dam project in Myanmar by using empirical approaches and numerical method. The geological evaluation of the Manipura diversion tunnels is relied on exploratory data, field observations and laboratory test results. Rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi), and geological strength index (GSI) are applied to assist in the support designs. Traditional guidelines for the rock support are used based on the results of the site characterizations. A series of numerical simulations (using Phase^2 code) is performed to assess the stability conditions of the tunnels with and without the support systems. The required input parameters for numerical modeling (Phase^2 code) are evaluated by empirical equations. After support installation, the extent of yielded zone and the radius of plastic zone significantly decrease as shown in the numerical results. Optimization between the empirical and numerical results is made to obtain the suitable support design for the tunnels. The results indicate that the use of empirical methods together with numerical method can provide the reasonable support systems for the underground openings.

1. INTRODUCTION

The Manipura multi-purposed dam project is located on the Manipura River, 52 km from the Kalay Town, Kalay Township, Sagaing Division in Myanmar (Figure 1). The project is a part of the Multi-purposed Government Funded Scheme and has been implemented under the supervision of Irrigation Department, Ministry of Agriculture and Irrigation, Myanmar. The dam will provide irrigated water for 20242 hectares and hydro-electricity to the local area. The two parallel tunnels are 1050 m long with 12 m wide modified horseshoe shape. They are driven underneath the south bank mountain to divert the water whilst a massive rock-fill dam is built. Within the confined space of a tunnel, it is difficult and dangerous to deal with stability or water problems which are sometimes encountered unexpectedly. In the case of Manipura diversion tunnel constructions, some tunnel stability problems are expected based on geological investigations.

Figure 1. Location of the Manipura multi-purposed dam project

To classify the rock mass quality, rock mass classification systems including rock mass rating system (RMR) [1], NGI tunneling quality index (Q system) [2], rock mass index (RMi) [3], and geological strength index (GSI) [4] are applied. Their rating values are used to determine tunnel support systems and to evaluate the rock mass parameters. These empirical methods have been originally obtained from many underground case studies and they have been applied to many construction designs. These empirical methods cannot however adequately calculate stress distributions, support performance and deformational behavior around a tunnel. Therefore, 2D finite element code, Phase^2 version 6.0, has been used. The rock mass parameters evaluated by empirical equations are used as input data for the numerical modeling. Several trials of the finite element analysis are performed based on the support systems proposed by empirical methods to optimize the results and to assess the reliable support systems.

2. GEOLOGY OF THE PROJECT AREA

The tunnels are in the Chin Flysch aged of Tertiary as a part of the Rakhine(Arakan)-Chin-Naga Hill ranges[5]. Most of the main Rakhine-Chin-Naga Hill ranges are thick monotonous series of apparently unfossiliferous marine flysch-type sediments including slaty shales, phyllite, slates and poorly-graded calcareous sandstones. Rock sequence in the project area, known as the Chin Flysch, is folded tightly, and even isoclinally. Quartz and calcite veins are very common. The sequence is predominantly argillaceous (slate), consisting mainly of yellowish grey to dark grey, hard, locally ferruginous and calcareous. Slate, the major rock unit, is fine grained metamorphic rock derived mostly from shale. Approcimately 20 to 25 % of the rock sequence consists of grey, hard, thin and thick bedded greywackes sandstone, usually find-grained and locally micaceous and calcareous.

3. ENGINEERING GEOLOGY

Geological data collection is carried out to classify the rock mass as accurately as possible. The integrated geological data collection can assist in the design solution and anticipate any serve geological condition, which can give rise to problem during the excavation of the opening. Engineering geological descriptions of the rock masses are based on the procedures suggested by ISRM [6]. The major trend of the joint is 320/60 (strike/dip) and the tunnel direction is N60E [5]. The strike of the joints and tunnel axis are nearly perpendicular. The area has the presence of varying weathering grades of rock. The geological cross-section map along the tunnel alignment is drawn based on the borehole results, surface exposures and time domain electromagnetic survey (TDEM) map. The tunnel alignment is divided into three different zones of rock mass: zone 1: moderately to highly weathered slate; zone 2: slightly to moderately weathered slate; and zone 3: slightly weathered slate with alternation of greywacke sandstone band. Each of which has different engineering geological properties and lithologic types. Total 21 different sections are classified based on their locally input variables in terms of engineering geological and geotechnical parameters and the induced overburden stress. The details sections along the tunnel alignment are described in Figure 2.

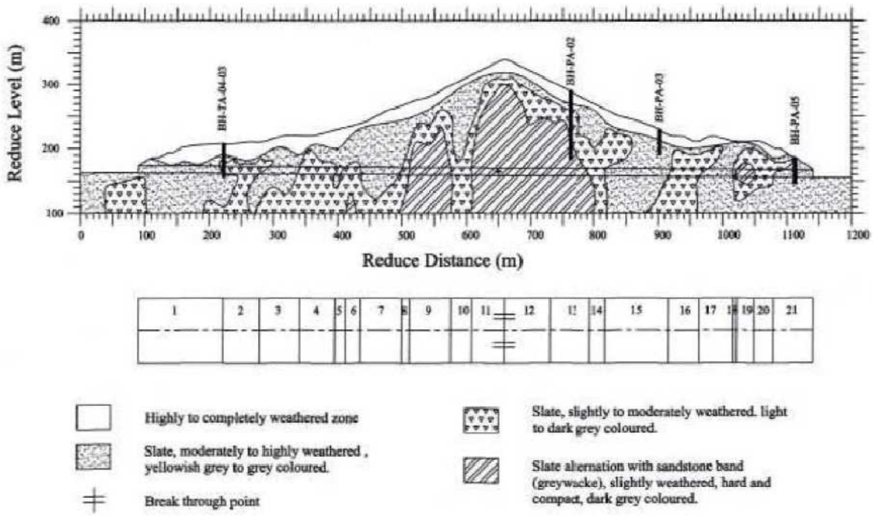

Figure 2. Geological cross-section of the Manipura diversion tunnel alignment

4. LABORATORY TESTING

Laboratory experiments were carried out to determine the physical and mechanical properties of intact rock. Rock core samples were collected from vertical and horizontal borehole drilling investigations. All laboratory tests were conducted in accordance with the ASTM [7] and the ISRM suggested methods. Uniaxial compressive strength of intact rock for zones 1, 2 and 3 are 16.71, 26.22 and 41.95 MPa, respectively. Unit weight of zone 1 is 26.59 kN/m^3, zone 2 is 27.76 kN/m^3 and zone 3 is 28.94 kN/m^3. Young's modulus of zone 1 is 10.34 GPa, zone 2 is 21.04 GPa and zone 3 is 22.83 GPa. Poisson ratio of zones1, 2 and 3 are 0.35, 0.34 and 0.30, respectively. Hoek and Brown constant (m_i) of intact rock of zones 1, 2 and 3 are 7.

5. ROCK MASS CHARACTERIZATIONS

The rock mass characterization has been performed to access the rock mass quality in accordance with the existing engineering rock mass classification systems. Four individual rock mass classification systems including RMR, Q, RMi and GSI, are currently applied. The classification table of Bieniawski [8] and the modified quantitative GSI table [9] are used in this study. For the estimation of RMi value and RMi support design, RMi-calc., version 2 and RMi support, version 3.1 [10] are applied. The rock mass classes for every zone based on these four rock mass classification systems are summarized in Table 1.

Table 1. Summary of the rock mass classes and their values

Zone	RMR		Q		RMi		GSI	
	Value	Class	Value	Class	Value	Class	Value	Class
1	31	Poor rock	0.5333	Very Poor	0.2858	Low	28	Poor
2	43	Fair rock	2.4930	Poor	0.9310	Low	38	Fair
3	47	Fair rock	3.0400	Poor	2.5153	Medium	44	Good

6. GEOTECHNICAL ROCK MASS PARAMETERS ESTIMATION AND STABALITY ANALYSIS

Rock mass properties such as Hoek and Brown constants, deformation modulus of the rock mass and rock mass strength are utilized parameters for the stability analysis and support design of tunnel. Field tests to determine some parameters directly are time consuming and expensive. Some empirical relations used here to calculate rock mass parameters are given in Tables 2, 3 and 4. The calculated values are presented in Tables 5, 6, 7 and 8.

Table 2. Empirical relations for estimating deformation modulus of rock mass (E_m).

Eq. no.	Equations			References
(1)	$E_m = 2RMR-100$	(GPa)	For RMR > 50	[11]
(2)	$E_m = 10^{\left(\frac{RMR-10}{40}\right)}$	(GPa)	For RMR < 50	[12]
(3)	$E_m = 0.1\left(\frac{RMR}{10}\right)^3$	(GPa)		[13]
(4)	$E_m = 25 \log Q$	(GPa)	For Q > 1	[14]
(5)	$E_m = 10Q_c^{1/3} = 10\,(Q\times\frac{\sigma_c}{100})^{1/3}$	(GPa)		[15]
(6)	$E_m = 5.6\ RMi^{0.375}$	(GPa)	For RMi > 0.1	[3]
(7)	$E_m = (1-\frac{D}{2})\sqrt{\frac{\sigma_c}{100}}\times 10^{\left(\frac{GSI-10}{40}\right)}$	(GPa)	For $\sigma_c \leq 100$ MPa	[16]
(8)	$E_m = (1-\frac{D}{2})\times 10^{\left(\frac{GSI-10}{40}\right)}$	(GPa)	For $\sigma_c > 100$ MPa	[16]

Table 3. Empirical relations for estimating rock mass strength (σ_{cm})

Eq. no.	Equations		References
(9)	$\sigma_{cm} = \sigma_c \exp\left(\frac{RMR - 100}{18.75}\right)$		[17]
(10)	$\sigma_{cm} = \left(\frac{5.5\gamma.5\ N^{1/3}}{\sigma_c\ B^{0.1}}\right)$	B = Width of the tunnel	[18]
(11)	$\sigma_{cm} = RMi = \sigma_c\ JP$	JP = Jointing parameter	[3]

Table 4. Empirical relations for estimating Hoek and Brown parameters (m_j and s_j)

Eq. no.	Equations		References
(13)	$m_j = m_i \exp\left(\frac{RMR - 100}{14}\right)$	For disturbed rock mass	[19]
(14)	$s_j = \exp\left(\frac{RMR - 100}{6}\right)$	For disturbed rock mass	[19]
(15)	$m_j = m_i \exp\left(\frac{RMR - 100}{28}\right)$	For undisturbed rock mass	[19]
(16)	$s_j = \exp\left(\frac{RMR - 100}{9}\right)$	For undisturbed rock mass	[19]
(17)	$\frac{m_j}{m_i} = 0.135\ Q_N^{1/3}$	$\left(Q_N = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot J_w\right)$	[20]
(18)	$s_j = 0.002\ Q_N$		[20]
(19)	$m_j = m_i\ JP^{0.64}$	For undisturbed rock mass	[3]
(20)	$m_j = m_i\ JP^{0.857}$	For disturbed rock mass	[3]
(21)	$s_j = JP^{2.0}$		[3]
(22)	$m_j = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right)$	D = Disturbrance factor	[16]
(23)	$s_j = \exp\left(\frac{GSI - 100}{9 - 3D}\right)$		[16]
(24)	$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$		[16]

Table 5. Calculated values of rock mass deformation modulus (E_m)

Zone	From RMR		From Q		From RMi	From GSI	Avg.	SD
	Eq. 2	Eq. 3	Eq. 4	Eq. 5	Eq. 6	Eq. 7		
1	3.3497	2.9791		4.4664	3.5012	1.1521	3.0897	1.2144
2	6.6834	7.9507	9.9181	8.6788	5.4519	2.5664	6.8749	2.6191
3	8.4140	10.3823	12.0718	10.8131	7.9142	4.5853	9.0301	2.6698

Table 6. Rock mass strength (σ_{cm})

Zone	Eq. 9	Eq. 10	Eq. 11	(RocData 3.0)[21]	Avg.	SD
1	0.4215	0.7536	0.2858	0.2490	0.4275	0.2297
2	1.2542	0.8386	0.9310	0.7650	0.9472	0.2156
3	2.4839	0.5837	2.5153	1.7710	1.8385	0.9044

Table 7. Hoek and Brown parameter (m_j)

Zone	FromRMR	From Q	From RMi	From GSI	Avg.	SD
	Eq. 14	Eq. 16	Eq. 18	Eq. 21		
1	0.5955	1.0401	0.5180	0.5350	0.6722	0.2475
2	0.9141	1.7391	0.8266	0.7646	1.0611	0.4561
3	1.0545	1.8580	1.1559	0.9473	1.2539	0.4116

Table 8. Hoek and Brown parameter (s_j)

Zone	From RMR	From Q	From RMi	From GSI	Avg.	SD
	Eq. 15	Eq. 17	Eq. 20	Eq. 22		
1	0.0005	0.0027	0.0003	0.0003	0.0009	0.0012
2	0.0018	0.0125	0.0013	0.0010	0.0041	0.0056
3	0.0028	0.0152	0.0036	0.0020	0.0059	0.0062

7. SUPPORT DESIGN USING EMPIRICAL APPROACHES

Empirical methods are based on rock mass classification systems including rock mass rating system (RMR), NGI tunneling quality index (Q system) and rock mass index (RMi). All these systems have quantitative estimation of the rock mass quality linked with empirical design rules to estimate adequate rock support measures such as rock bolt, shotcrete and steel set. The results are summarized in Table 9.

Table 9. Suggested support systems based on empirical approaches

Zone	From RMR	From Q	From RMi
1	BL: 4-5 m, BS: 1-1.5 m CS(W): 100-150 mm WS(W): 100 mm SR: 1.5 m spaced where required	BL: 2.4-3 m BS: 1.5-1.7 m CS(W): 90-120 mm	special bolting, concrete lining or special design shotcrete for roof and 150-250 mm thickness fibre reinforced shotcrete for wall
2	BL: 4 m, BS: 1.5-2 m CS(W): 50-100 mm WS(W): 30 mm	BL: 2.4-3 m BS: 1.7-2.1 m CS: 40-100 mm	CBS: 1 × 1 m WBS: 1.25 × 1.25 m CS(W) : 150-250 mm WS(W): 100-150 mm
3	BL: 4 m, BS: 1.5 × 2 m CS(W): 50-100 mm WS(W): 30 mm	BL: 2.4-3 m CBS: 1.7-2.1 m CS: 40-100 mm	CBS: 1.25 × 1.25 m WBS: 1.5 × 1.5 m CS(W) : 100-150 mm WS(W): 70-100 mm

BL: bolt length for crown and wall, BS: systematic bolt spacing for crown and wall, CS: shotcrete thickness for crown, WS: shotcrete thickness for wall, CBS: systematic bolt spacing for crown, (W): shotcrete with wire mesh, WBS: systematic bolt spacing for wall, SR: steel rib

8. SUPPORT DESIGN USING NUMERICAL METHOD

Phase^2 version 6.0, a finite element program developed by Rocscience [22], has been used to perform a series of numerical simulations. Support elements used are composed of rock bolts and shotcrete as proposed by the empirical methods. Hoek and Brown failure criterion is used to estimate yielding zone around the tunnels and the maximum displacement. Plastic post-failure strength parameters are used in this analysis. The residual parameters are assumed as half of the peak strength parameters. In situ stress for the finite element models is assumed as hydrostatic and automatic mesh around the tunnel is generated. For this study, boundary conditions are defined as restrained X for both sides boundary, restrained Y for the lower boundary and free surface for the upper boundary. The strength factor contours, yielded finite elements, radius of plastic zone, maximum total displacement and displacement vectors are described in Figure 2. The results of the supported and unsupported cases are summarized in Table 10.

Even though the maximum total displacements are very small in all numerical modeling results, the extent of plastic zone and yielded elements suggest that there would be some stability problems for tunnels. Phase^2 is a small strain finite element program and thus it cannot accommodate the very large strains [23]. In the case of Manipura diversion tunnel support design, it is more important to consider the extent of plastic zone and yielded elements rather than the magnitude of displacement. After support installation, both the yielding zone and the radius of plastic zone are decreased as shown in Figure 3. Maximum total displacement is also reduced for the supported cases. This suggests that the applied support systems are adequate to obtain tunnel stability.

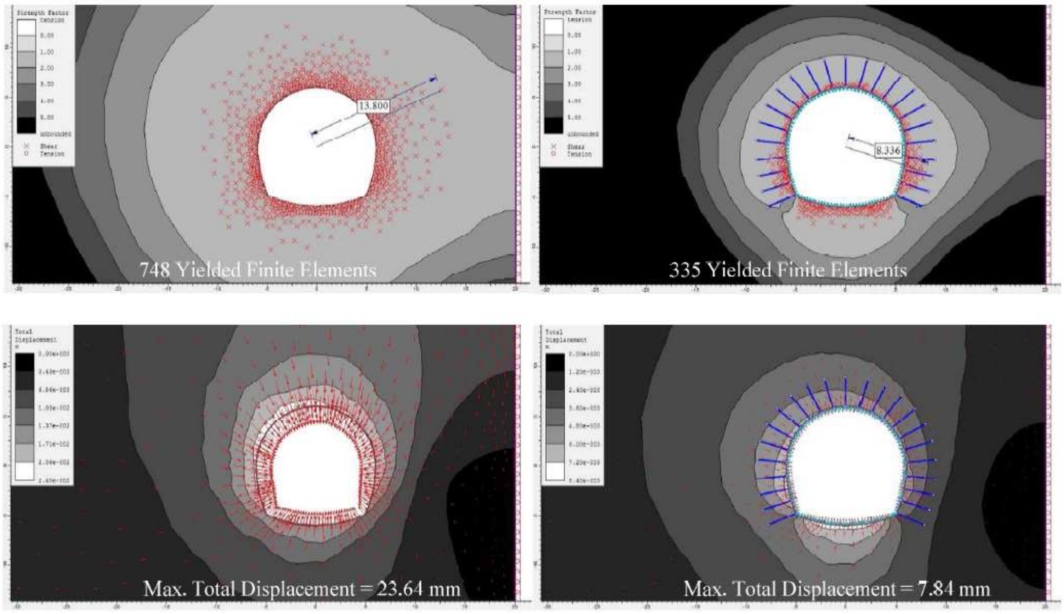

Figure 3. Yielded finite elements, radius of plastic zone, strength contour, maximum total displacement and displacement vectors for unsupported and supported case of zone 1.

Table 10. Yielded finite elements, radius of plastic zone and maximum total displacement for unsupported and supported cases.

Zone	Yielded finite elements (no)		Radius of plastic zone (m)		Maximum total displacement (mm)	
	unsupported	supported	unsupported	supported	unsupported	supported
1	748	335	13.800	8.336	23.64	7.84
2	614	389	9.001	7.865	6.69	4.72
3	549	370	8.249	7.552	7.31	5.57

9. DISCUSSIONS

Many empirical methods have been developed by several researchers and the utilized parameters are varied. All empirical methods have their characteristic limitations to achieve their objectives. As a result, to overcome these limitations, the rock mass parameters along the tunnel alignment were estimated by four different empirical methods and their average values were used as input parameters for finite element analysis.

The results indicate that the numerical method provides more shotcrete thickness than do empirical methods. The empirical methods suggest longer rock bolt than does the numerical method. This may be because the numerical method considers the overburden as continuous medium and the empirical methods considers as discontinuous medium. After several trials, the final reasonable estimate of tunnel support systems are achieved as presented in Table 11.

In addition, the excavation method which is top heading and bench cut excavation method (1.0-1.7 m advance in top heading, install support concurrently with excavation, 10 m from face) is recommended. Concrete lining or special design shotcrete is suggested to support at the corners of the tunnel floor to prevent high stress concentration caused by corner effect.

Table 11. Recommended support systems for Manipura diversion tunnels

Zone	Rock bolt	Shotcrete
1	Length = 3 m, Spacing = 1.3 x 1.3 m	Thickness = 200 mm with wire mesh
2	Length = 3 m, Spacing = 1.5 x 1.5 m	Thickness = 150 mm with wire mesh
3	Length = 3 m, Spacing = 1.5 x 1.5 m	Thickness = 120 mm with wire mesh

10. CONCLUSIONS

Rock masses along the Manipura twin tunnel alignment are characterized by means of rock mass classification systems based on the vertical borehole data, field investigations, engineering geological observations and laboratory test results. According to the results acquired from the rock mass characterizations and stability analysis, there are some stability problems in each zone. The empirical methods, rock mass classification systems, are employed to determine the required support systems for the Manipura diversion tunnels. Three numerical models are constructed by using finite element software, Phase^2 code, to determine the induced stresses, deformations developed around the tunnel and evaluate the performance of the support system recommended by the empirical methods.

The strength parameters required for finite element analysis are estimated from empirical relations based on the rock mass classification systems. Used support elements are rock bolts and shotcrete as proposed by the empirical methods. Several trials of the finite element analysis are performed to assess the more appropriate support elements. It leads to the final reasonable estimate of tunnel support systems. When the recommended support systems are applied, the extent of yielded zone and displacement significantly reduce in numerical analysis. These results indicate that the recommended applied support systems are adequate to obtain tunnel stability. It also suggests that the empirical methods reasonably agree with the numerical method.

Based on the result findings, it can be concluded that empirical methods should be applied along with numerical method for the safe tunnel support design. A great deal of judgment may be needed in the application of all kinds of rock mass classification systems in support design.

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BIOGRAPHHY

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REFERENCES

[1] Bieniawski. Z. T. (1973). Engineering classification of jointed rock masses. Transactions, South African Institution of Civil Engineer. Vol. 15, pp. 335-344.

[2] Barton, N., Lien, R. and Lunde, J. (1974). Engineering classification of rock masses for the design of tunnel support. Rock Mechanics. Vol. 6, pp. 183-236.

[3] Palmstrom, A. (1995). RMi - A Rock Mass Characterization System for Rock Engineering Purposes. Ph.D.Eng. Thesis, University of Oslo, Norway.

[4] Hoek, E., Kaiser, P. K. and Bawden, W. F. (1995). Support of Underground Excavations in Hard Rock. Balkema, Rotterdam. 215 pp.

[5] Win, H. A. K. and Aung, K. (2007), Engineering Geology Office 2 Report, Irrigation Department, Myanmar, unpublished. 28 pp.

[6] ISRM (1981). Rock characterization, testing and monitoring. In: Brown, E. T. (Ed), ISRM Suggested Methods. Pergamon Press, New York. 211pp.

[7] ASTM D 7012-07. Compressive strength and elastic moduli of intact rock core specimens under varying states of stress and temperatures. Annual Book of ASTM Standards. 04.08. West Conshohocken: American Society for Testing and Materials: Philadelphi.

[8] Bieniawski. Z. T. (1989). Engineering Rock Mass Classifications: A Complete Manual for Engneers and Geologists in Mining, Civil, and Petroleum Engineering. John Wiley & Sons, New Yoke. 251 pp.

[9] Sonmez, H. (2001). Investigation of the Applicability of the Hoek–Brown Criteria to the Failure Behavior of the Fissured Clays. Ph.D. Thesis, Hacettepe University. 215pp , unpublished, in Turkish.

[10] Palmstrom, A. (2001). RMi-calc., version 2 and RMi support, version 3.1. http://www.rockmass.net

[11] Bieniawski. Z. T. (1978). Determining rock mass deformability. Experience from case histories. International Journal of Rock Mechanics and Mining Science & Geomechanics Abstracts. Vol. 15, pp. 237-247.

[12] Serafim, J. L. and Pereira, J. P. (1983). Considerations of the Geomechanics Classification of Bieniawski. Proceedings International Symposium Engineering Geology and Underground Construction, Balkema, Rotterdam. Vol. 1, pp. 1133-1142.

[13] Read, S. A. L., Richards, L. R. and Perrin, N. D. (1999). Applicability of the Hoek–Brown failure criterion to New Zealand greywacke rocks. Proceeding 9th International Society for Rock Mechanics Congress, Paris. Vol. 2, pp. 655-660.

[14] Grimstad, E. and Barton, N. (1993). Updating of the Q-system for NMT. International Symposium on Sprayed Concrete. Fagernes, Proceeding. pp. 46-66.

[15] Barton, N. (2002). Some new Q-value correlations to assist in site characterization and tunnel design. International Journal of Rock Mechanics and Mining Sciences. Vol. 39, pp 185-216.

[16] Hoek, E., Carranza-Torres, C. and Corkum, B. (2002). Hoek-Brown Failure criterion – 2002 edition. Proc. NARMS-TAC Conference, Toronto. Vol. 1, pp. 267-273.

[17] Ramamurthy, T. (1986). Stability of rock mass, 8th Annual Lecture, Indian Geotech. J. Vol.12, pp. 1–74.

[18] Goel, R.K. (1994). Correlations for Predicting Support Pressures and Closures in Tunnels. PhD Thesis, Nagpur University, Nagpur, India. 308 pp.

[19] Hoek, E. and Brown, E. T. (1988). The Hoek-Brown Failure Criterion - a 1988 Update. Proceedings of the 15th Canadian Rock Mechanics Symposium, University of Toronto. Vol. 1, pp 31-48.

[20] Singh, B., Viladkar, M. N., Samadhiya, N. K. and Mehrota, V.K. (1997). Rock mass strength parameters mobilized in tunnels. Tunneling and Underground Space Technology. Vol 12, pp. 47–54.

[21] Rocscience, (2004). RocData User's Guide. Rocscience Inc, Toronto, Ontario, Canada. http://www.rocscience.com.

[22] Rocscience, (2007). Phase^2 User's Guide. Rocscience Inc, Toronto, Ontario, Canada. http://www.rocscience.com.

[23] Basarir, H., Ozsan, A. and Karakus, M. (2005). Analysis of support requirements for a shallow diversion tunnel at Guledar dam site, Turkey. Engineering Geology, Elsevier publication, Vol. 81, pp. 131-145.

BIOGRAPHY

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