DESIGN AND ANALYSIS OF MINE SHAFT AND ADIT FOR LIMESTONE QUARRY OF SIAM CITY CEMENT PUBLIC COMPANY LIMITED

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การออกแบบและวิเคราะห์ปล่องเหมืองและอุโมงค์สำหรับเหมืองหินปูนของ บริษัท ปูนซีเมนต์นครหลวง จำกัด (มหาชน)

นายอดิศักดิ์ บุญบาตร

วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิศวกรรมศาสตรมหาบัณฑิต

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

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อดิศักดิ์ บุญบาตร : การออกแบบและวิเคราะห์ปล่องเหมืองและอุโมงค์สำหรับเหมืองหินปูนของบริษัท ปูนซีเมนต์นครหลวง จำกัด (มหาชน) (DESIGN AND ANALYSIS OF MINE SHAFT AND ADIT FOR LIMESTONE QUARRY OF SIAM CITY CEMENT PUBLIC COMPANY LIMITED) อาจารย์ที่ปรึกษา : รองศาสตราจารย์ ดร.กิตติเทพ เฟื่องขจร, 120 หน้า.

จุดประสงค์ของการวิจัยนี้เพื่อวิเคราะห์เสถียรภาพและออกแบบค้ำยันสำหรับปล่องขนส่งและอุโมงค์สำหรับขนวัตถุดิบจากเหมืองหินปูนของบริษัท ปูนซีเมนต์นครหลวง จำกัด (มหาชน) จังหวัดสระบุรี ประเทศไทย เส้นผ่าศูนย์กลางปล่องขนส่งกว้าง 5 เมตร และมีความลึก 100 เมตร ส่วนอุโมงค์เป็นรูปเกือกม้ากว้าง 5 เมตร สูง 6 เมตรและความยาวของอุโมงค์ 450 เมตร โดยมีความลาดเอียง 3% หินตามแนวอุโมงค์ประกอบด้วยหินปูนอายุเพอร์เมียน

ในงานวิจัยนี้ได้ศึกษาเกี่ยวกับคุณลักษณะของมวลหิน การประเมินค่าตัวแปรที่เกี่ยวข้องกับมวลหิน การวิเคราะห์เสถียรภาพและการออกแบบระบบค้ำยันหินรอบอุโมงค์และปล่องขนส่ง มวลหินตลอดความยาวของอุโมงค์และปล่องขนส่งได้ถูกจำแนกโดยใช้ระบบการจำแนกมวลหินเชิงประสบการณ์ซึ่งได้รวมไปถึง ระบบการจำแนกหินด้วยระบบการให้คะแนน (RMR) ระบบดัชนีคุณภาพมวลหินในอุโมงค์ของ NGI (Q-system) ระบบดัชนีมวลหิน (RMi) และดัชนีความแข็งธรณีวิทยา (GSI) ค่าสัมประสิทธิ์ความยืดหยุ่นของมวลหินและตัวแปรของ Hook-Brown แนวทางการออกแบบค้ำยันที่ใช้กันอยู่ทั่วไปจะถูกนำมาใช้ประกอบกับผลการจำแนกคุณลักษณะของมวลหินในสถานที่จริง การจำลองด้วยแบบจำลองตัวเลข (ใช้โปรแกรม UDEC) ถูกนำมาใช้เพื่อประเมินเกี่ยวกับเสถียรภาพของอุโมงค์ทั้งที่มีและไม่มีระบบค้ำยัน เกณฑ์การวิบัติของมวลหินที่เสนอโดย Hoek และ Brown ถูกนำมาใช้ประเมินบริเวณที่มีการเคลื่อนตัวสูงสุดของหินรอบอุโมงค์และปล่องขนส่ง ระบบค้ำยันที่แนะนำให้ใช้โดยวิธีเชิงประสบการณ์ถูกใช้ในการศึกษาด้วยการจำลองเชิงตัวเลข ซึ่งประกอบด้วยหมุดยึดหิน โครงเหล็กค้ำยันและคอนกรีตพ่นร่วมกันตาข่ายลวด คุณสมบัติของส่วนที่เป็นค้ำยัน เช่น ความยาวของหมุดยึดหิน ระบบการติดตั้งหมุดยึดหิน ชนิดและระยะห่างระหว่างโครงเหล็กค้ำยัน ความหนาของคอนกรีตพ่นได้ใช้ให้เหมือนกับที่ระบุไว้ในวิธีเชิงประสบการณ์ ก่อนการติดตั้งระบบค้ำยัน พื้นที่ที่เกิดการเคลื่อนตัวสูงสุดของหินรอบอุโมงค์และปล่องขนส่งได้ให้ความสำคัญ ผลการศึกษาระบุว่าก่อนการค้ำยันมีบางปัญหาเกิดขึ้นเกี่ยวกับเสถียรภาพของอุโมงค์และปล่องขนส่ง กล่าวคือหลังจากที่ติดตั้งระบบค้ำยัน พื้นที่ที่เกิดการเคลื่อนตัวสูงสุดของอุโมงค์และปล่องขนส่งได้ลดลง สิ่งนี้บ่งชี้ว่าระบบการค้ำยันมีความเหมาะสมที่ให้ให้เกิดเสถียรภาพของอุโมงค์และปล่องขนส่ง สภาวะที่ดีที่สุดระหว่างผลการออกแบบค้ำยันด้วยวิธีเชิงประสบการณ์และวิธีทางด้านระเบียบวิธีเชิงตัวเลขนั้นได้กำหนดเพื่อใช้ออกแบบการค้ำยันจริงสำหรับอุโมงค์และปล่องขนส่งวัตถุดิบจากเหมืองหินปูนของบริษัท ปูนซีเมนต์นครหลวง จำกัด (มหาชน).

สาขาวิชา เทคโนโลยีธรณี	ลายมือชื่อนักศึกษา_______________
ปีการศึกษา 2555	ลายมือชื่ออาจารย์ที่ปรึกษา_______________

ADISAK BOONBATR : DESIGN AND ANALYSIS OF MINE SHAFT AND ADIT FOR LIMESTONE QUARRY OF SIAM CITY CEMENT PUBLIC COMPANY LIMITED. THESIS ADVISOR : ASSOC. PROF. KITTITEP FUENKAJORN, Ph.D., P.E., 120 PP.

ADIT/SHAFT/DESIGN/STABILITY/SUPPORT

The objective of this study is to perform stability analysis and support design for portal, shaft and adit to access the limestone quarry of Siam City Cement Public Company Limited (SCCC), Saraburi province, Thailand. The shaft has circular shape, 5 m diameter and 100 m depth. The adit has horseshoe shape, 5 m wide, 6 m high and 450 m long with inclination about 3%. The bedrocks along the adit alignment are carbonates and siliciclastics of Permo-Carboniferous age. The study involves rock mass characterizations, evaluation of rock mass parameters, stability analysis and support design for the rock mass around the shaft and adit. They are classified by using 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 determine the in-situ rock mass strength, deformation modulus of rock mass and Hoek-Brown parameters. Traditional guidelines for the rock support have been used based on the results of the site characterizations. The numerical models are developed for using with the Universal Distinct Element Code (UDEC) to determine the displacements around the opening to evaluate the performance of the support system recommended by the empirical methods. The support systems include rock bolts, steel rib and shotcrete with wire mesh. The properties of support components, such as bolts length, spacing of steel rib, bolts patterns and thickness of shotcrete, are similar to those proposed by the empirical methods. Before support installation, relatively large displacements are observed. The results indicate that there would be some stability problems for the shaft and adit. After support installation, the maximum displacements are decreased. This indicates that the applied support systems are adequate to obtain the shaft and adit stability. Optimization between the empirical and numerical results is made to obtain the suitable support design for the shaft and adit to access the limestone quarry of the SCCC.

School of Geotechnology Student's Signature____________________

Academic Year 2012 Advisor's Signature____________________

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Adisak Boonbatr

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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

SYMBOLS AND ABBREVIATIONS (Continued)

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
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

SYMBOLS AND ABBREVIATIONS (Continued)

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
W	=	Width of opening
β	=	Block shape factor

SYMBOLS AND ABBREVIATIONS (Continued)

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 Siam City Cement Public Company Limited (SCCC) limestone quarry is located 129 km north of Bangkok, in the Saraburi province, along the Highway number 2 (Figure 1.1).

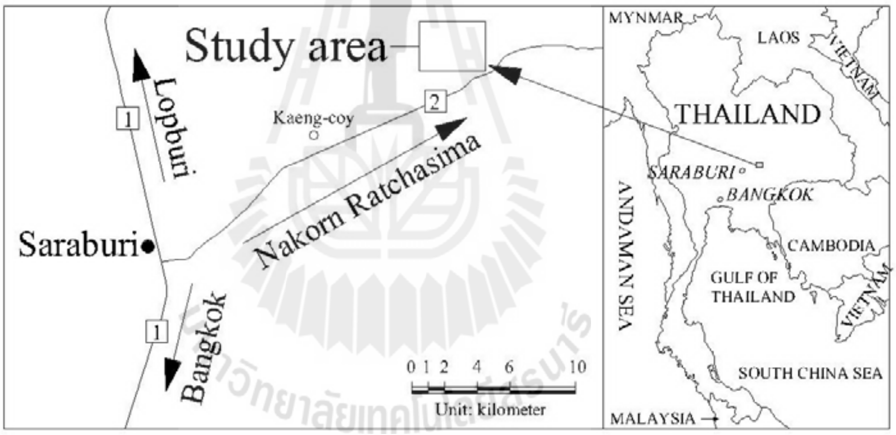


Figure 1.1 Location map of the project area (Scale 1:5000).

The planned haul road is 3 km uphill, the haul distance 1 km. It consists of a portal, adit and shaft cut directly to the limestone pre-blending pile to reduce haul age cost. The project consists of portal, shaft with depth of about 100 m. The adit is 450 m long (Figure 1.2). The portal is installed in the well-bedded limestone (WB). The shaft is installed in the spatic limestone (SP), massive limestone in the mid of the limestone quarry pit. The adit is N8E thought WB, thrust fault zone (F) and SP. The geotechnical evaluation of the shaft, portal and adit is relied on the exploratory data, field observations and laboratory test.

These data consists of field investigation, laboratory determination of material properties, geological map, topographic map and outcrop surface map along the adit axis.

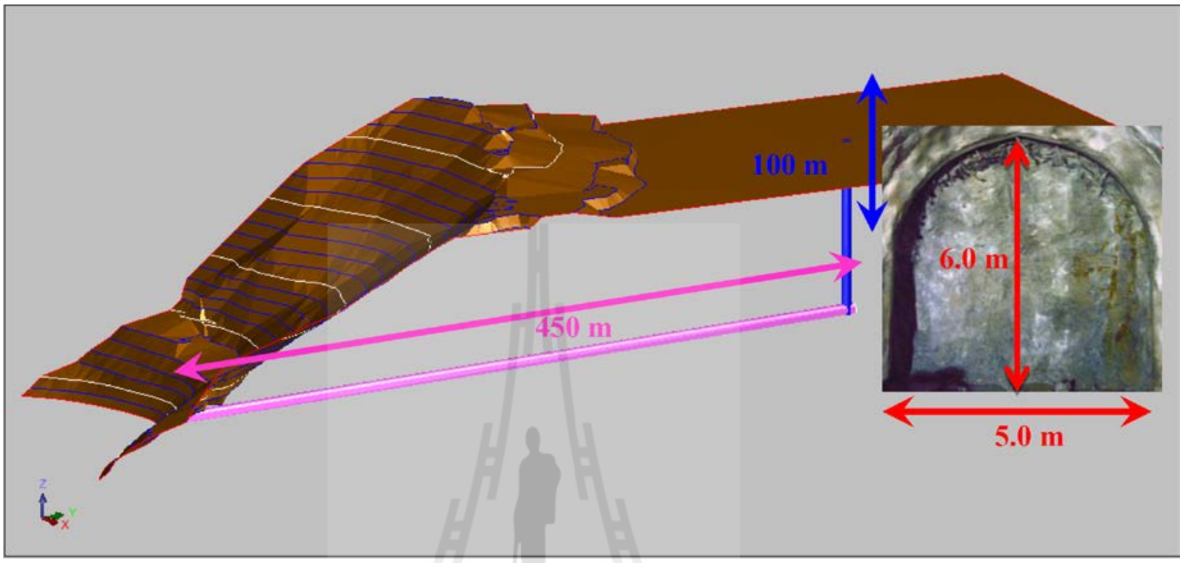


Figure 1.2 3D view of the shaft 100 m depth and the adit long 450 m.

Rock mass classification systems are a useful tool for the preliminary design stage of a project. To classify the rock mass quality, rock mass classification systems, such as rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi), and geological strength index (GSI) are utilized. Their rating values are used to estimate tunnel support systems and to evaluate the rock mass parameters. These empirical methods have been originally obtained from many tunneling case studies. They have been applied to many construction tunnel designs. However, these empirical methods cannot adequately calculate stress redistributions, support performance and deformations around a tunnel. Therefore, 2D finite element software, such as UDEC, will be used for the numerical simulations. The rock mass parameters evaluated by empirical equations are utilized as input data for numerical modeling (using UDEC). The comparison will be made the results obtained from empirical methods with numerical method to assess the support systems

1.2 Research objectives

The objective of this study is to perform stability analysis and support design for portal, shaft and adit to access the limestone quarry. The proposed study involves performing a design methodology of the portal, shaft and adit and comparing the support design results obtained from the empirical methods with the numerical method. The review focuses on the rock mass classification method determination of input parameters, support design and, stress analysis and support design by using the numerical method, UDEC. The rock mass along the SCCC limestone quarry portal, adit and shaft are classified by using the empirical methods such as rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi), and geological strength index (GSI). The rating values are used to evaluate the stability and support design of the portal, adit and shaft. The support systems are also analyzed by using numerical method, UDEC. The feasible support designs can be accessed by comparing the result 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, support design (empirical methods and numerical method), comparisons, 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, evaluation of rock mass parameters, stability analysis and support estimation of underground excavation, numerical modeling. The sources of information are from journals, technical reports and conference papers. A summary of the literature review is given in the thesis.

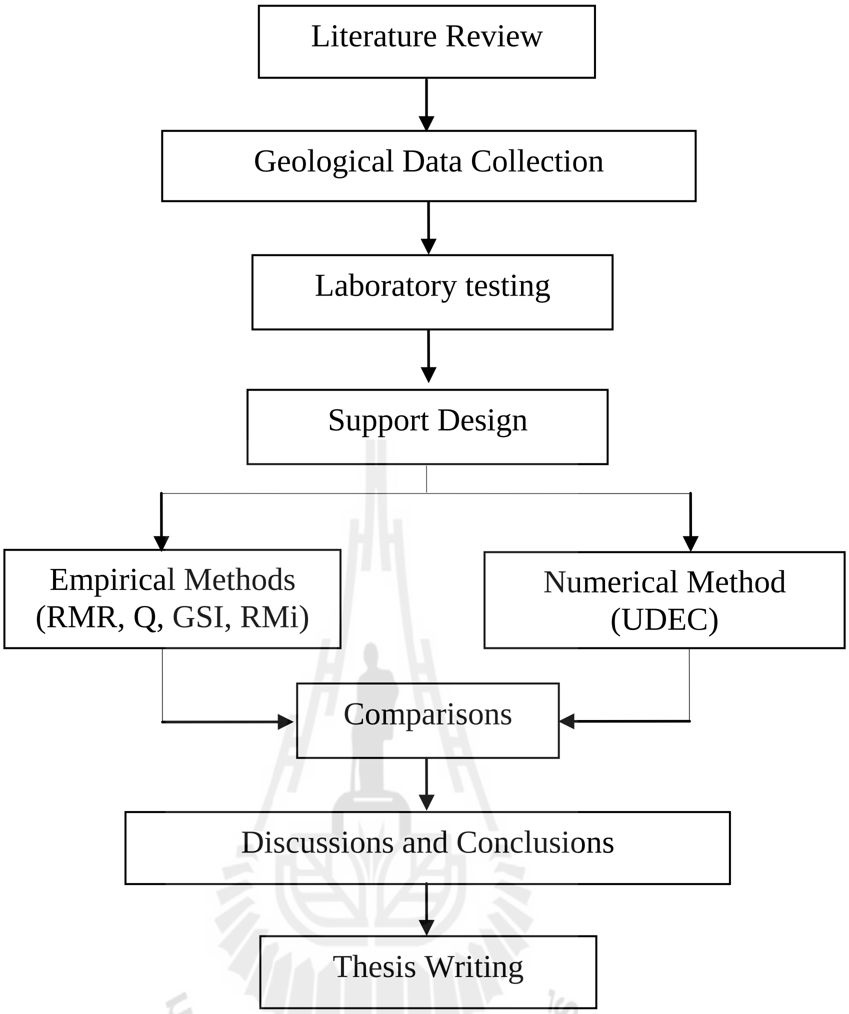


Figure 1.3 Research methodology

1.3.2 Geological data collection

The geotechnical evaluation of the SCCC quarry is relied on the exploratory data, field observations and laboratory test results. For this task a stations have been selected to represent the rock conditions along adit axis.

1.3.3 Rock mass characterizations

The rock mass along the tunnel alignments are classified by using the rock mass classification systems such as 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 design of the tunnels.

1.3.4 Support design

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

The performances of the support elements suggested from empirical methods are analyzed by numerical methods. A series of numerical simulations are performed to assess the stability conditions of the tunnels with and without support system. Optimization between the empirical and numerical results is made to obtain the suitable support design for the tunnels.

1.3.5 Comparisons

Results obtained from empirical methods are compared with the support system from the numerical method.

1.3.6 Discussions, conclusions and thesis writing

The research results will be concluded and provided the proposed support systems for the 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 SCCC quarry portal, shaft and adit is conducted. Project area is SCCC limestone quarry, final pit wall side of cement plant 3 only. The shape of the SCCC quarry adit is horseshoes shape. The shape of the SCCC quarry shaft is circular. The portal, shaft and adit have been constructed by using drill-and-blast technique. Excavation sequence will not be considered. The geological investigation of the SCCC quarry portal, shaft and adit 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 results of the literature review.

Chapter III describes the geological data collection. **Chapter IV** presents the characterizations of rock mass class 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 design for the portal, shaft and adit. Estimating the feasible support design of the portal, shaft and adit are divided into 3 tests, including 1) support design by empirical methods 2) support design by numerical method (using UDEC), 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.

CHAPTER II

LITERATURE REVIEW

2.1 Introduction

This chapter summarizes the results of literature review carried out to perform an understanding of stability analysis and support design of portal, adit and vertical shaft. Topics relevant to this study involve rock mass classification systems, such as rock mass rating (RMR), NGI tunneling quality index (Q system), geological strength index (GSI), rock mass index (RMi), numerical modeling (UDEC) 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. In any analysis of rock mass behavior that includes deformation modulus is an important input parameter. Field tests to determine this parameter directly are time consuming, expensive and the reliability of the results of these tests is sometimes questionable. Consequently, several authors have proposed empirical relationships for estimating the value of an isotropic rock mass deformation modulus based on empirical rock mass classification schemes (Hoek and Diederichs, 2005). The four methods of quantitative rock mass classifications (RMR, Q, RMi and GSI) will be applied.

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 must be made adjustment for the discontinuity orientation. Bieniawski (1989) has provided guidelines for the selection of rock support for horseshoe shaped tunnels excavated by the drill-and-blast technique, shown in Table 2.1.

In many designing the primary support and final lining for 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. Based on the RMR rating value, many researchers have proposed different empirical equations to calculate the rock mass deformation modulus as follows:

Bieniawski (1978) has defined E_{mass} as:

$$E_{mass} = 2RMR-100 \text{ (GPa)} \qquad \text{For RMR} > 50 \tag{2.1}$$

Serafim and Pereira (1983) have proposed:

$$E_{mass} = 10^{\left(\frac{RMR-10}{40}\right)} \text{ (GPa)} \qquad \text{For RMR} < 50 \tag{2.2}$$

Read et al. (1999) has proposed the following equation:

$$E_{mass} = 0.1\left(\frac{RMR}{10}\right)^3 \text{ (GPa)} \tag{2.3}$$

where E_{mass} is the deformation modulus of the rock mass.

Table 2.1: Guidelines for excavation and support of 10 m span rock tunnels in accordance with the RMR system (After Bieniawski, 1989).

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock *RMR*: 81-100	Full face, 3 m advance.	Generally no support required except spot bolting.		
II - Good rock *RMR*: 61-80	Full face , 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock *RMR*: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	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.
IV - Poor rock *RMR*: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	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.
V – Very poor rock *RMR*: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

2.2.2 NGI tunneling quality index (Q system)

The Q system of rock mass classification was developed in Norway by Barton et al. (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) RQD
2) Number of joint sets

3) Roughness of the most unfavorable joint or discontinuity

4) Degree of alteration or filling along the weakest joint.

5) Water inflow.

6) Stress condition

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

$$Q = \frac{RQD}{J_n} \cdot \frac{J_v}{J_a} \cdot \frac{J_w}{SRF} \tag{2.4}$$

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 = 1000 on a logarithmic rock mass quality scale.

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

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio, ESR}} \tag{2.5}$$

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, shown in Figure 2.1.

The Q-values and support in Figure 2.1 are related to the total amount of support (temporary and permanent) in the roof. The diagram is based on numerous tunnel support cases. Wall support can also be found by applying the wall height and the following adjustments to Q:

For $Q > 10$ use $Q_{wall} = 5Q$ (2.6)

For $0.1 < Q < 10$ use $Q_{wall} = 2.5Q$ (2.7)

For $Q < 0.1$ use $Q_{wall} = Q$ (2.8)

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 (Palmström and Broch, 2006).

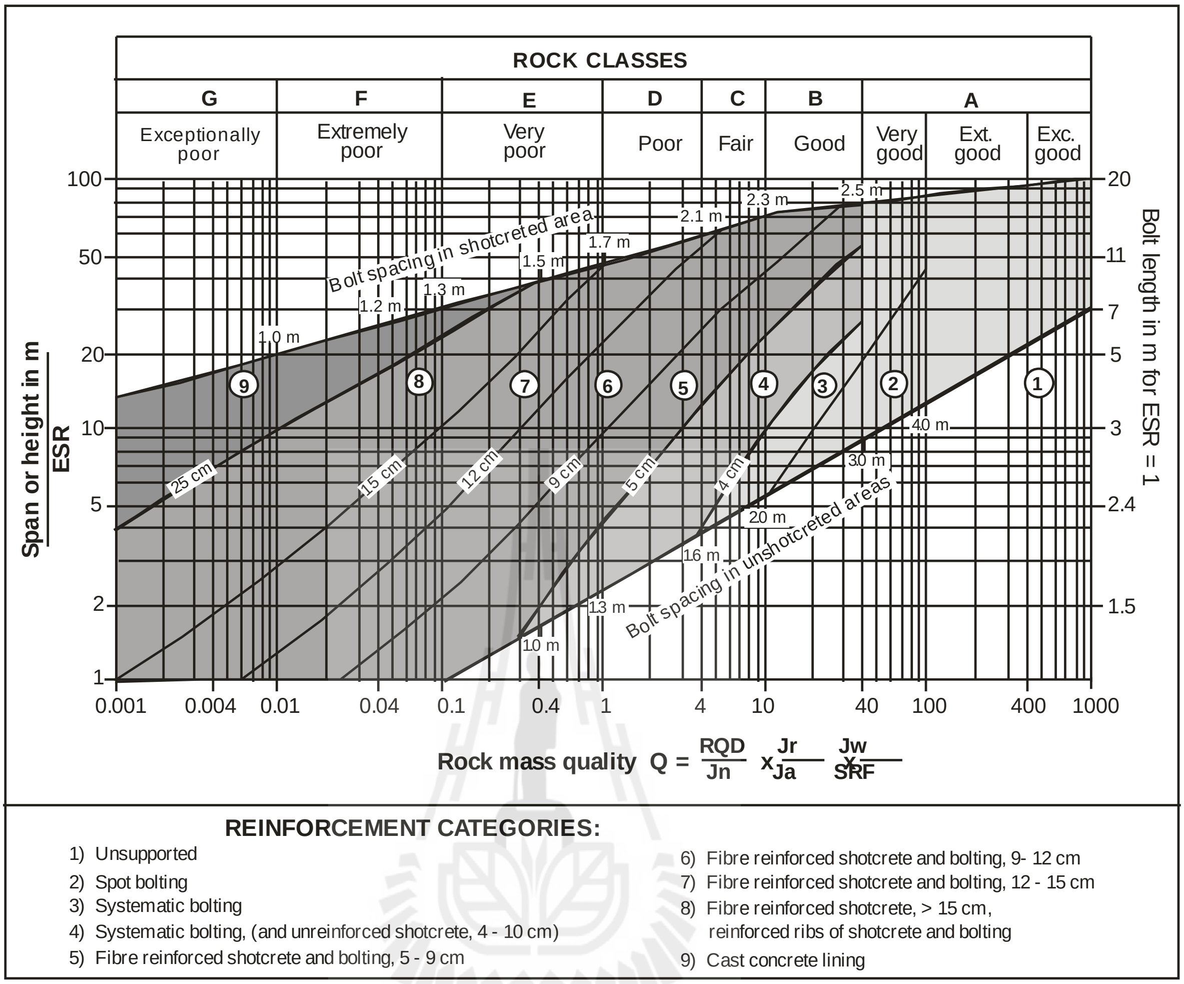


Figure 2.1: Estimated support categories based on the tunneling quality index Q (After Grimstad and Barton, 1993, reproduced from Palmström and Broch, 2006).

Quantitative classification systems are used to estimate the deformation modulus of rock masses, E_m. Simple equations have been presented from the Q-system as follow:

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

$$E_m = 25 \log Q \qquad \text{(GPa)} \qquad (2.9)$$

E_m was expressed as below by Barton (2002).

$$E_m = 10\, Q_c^{\,1/3} = 10\,(Q \times \frac{\sigma_c}{100})^{1/3} \qquad (2.10)$$

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

2.2.3 Rock mass index (RMi)

The rock mass index (RMi) was first presented by Palmström 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 Palmström (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,

$$RMi = \sigma_c \times JP \qquad (2.11)$$

where σ_c is uniaxial compressive strength of the 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:

$$JP = 0.2\,\sqrt{JC}\;V_b^{\,D} \qquad (D = 0.37\,JC^{\,-0.2}) \qquad (2.12)$$

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

For massive rock,

$$RMi = \sigma_c \times f_\sigma \text{ (applied for cases where } f_\sigma > JP) \tag{2.13}$$

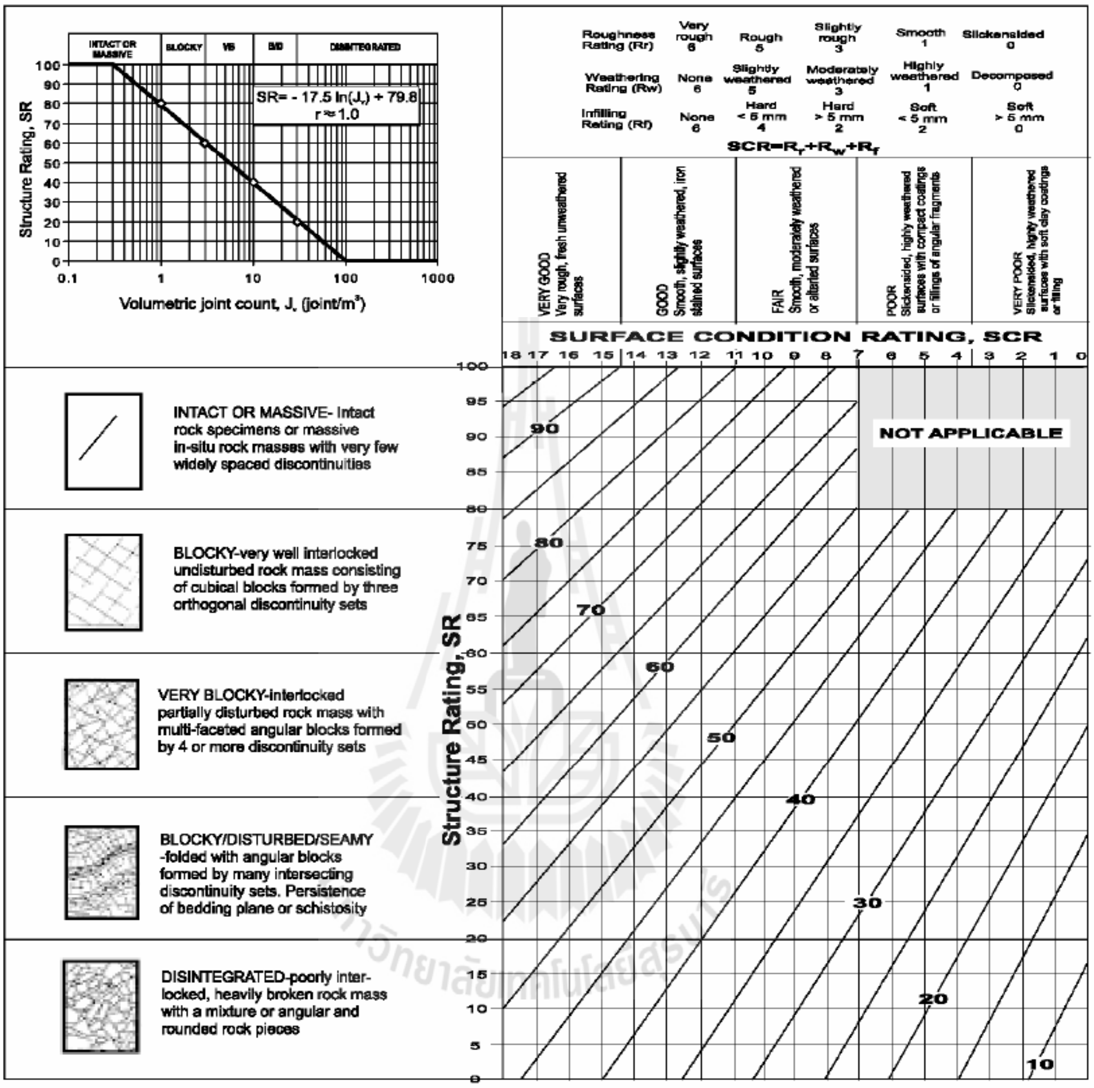
where f_σ is called the massivity parameter, given as $f_\sigma = \sigma_c\ (0.05/D_b)^{0.2}$ (D_b = block diameter). In most cases, $f_\sigma \approx 0.5$.

The RMi requires more calculations than the RMR and the Q system, but the spreadsheets have been developed (see www.rockmass.net) 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 will be used.

2.2.4 Geological strength index (GSI)

The geological strength index (GSI) 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 description the value of GSI is estimated from the contours.

Table 2.2: The modified quantitative GSI system (Sonmez and Ulusay, 1999)



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), shown in Table 2.2.

Table 2.3: Field estimates of uniaxial compressive strength of intact rock (Marinos and Hoek, 2000)

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

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

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 are reproduced in Tables 2.3 and 2.4.

Table 2.4: Values of the constant m_i for intact rock (Marinos and Hoek, 2000)

Rock type	Class	Group	Texture: Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates * Breccias *	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6	Gneiss 28 ± 5	
	Foliated**			Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

* Conglomerates and breccias may present a wide range of m_i values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone, to values used for fine grained sediments (even under 10).

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

Using the GSI system, provided the UCS value is known the rock mass deformation modulus E_m for $\sigma_{ci} \leq 100$ MPa is estimated in GPa from the following equation (Hoek et al, 2002).

$$E_m \text{ (GPa)} = (1-\frac{D}{2})\sqrt{\frac{\sigma_{ci}}{100}} \times 10^{\left(\frac{GSI-10}{40}\right)} \quad (2.14)$$

For $\sigma_{ci} > 100$ MPa, use equitation 15.

$$E_m \text{ (GPa)} = (1- \frac{D}{2}) \times 10^{\left(\frac{GSI-10}{40}\right)} \quad (2.15)$$

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.

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 et al. (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 (After Deere, 1989). Hence:

$$\text{RQD (\%)} = 100 \times \frac{\text{Length of core in pieces} \geq 100 \text{ mm}}{\text{Length of borehole}} \quad (2.16)$$

Palmström (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:

$$\text{RQD} = 115 - 3.3J_v \quad (2.17)$$

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 adit, portal and vertical shaft, Universal Distinct Element Code (UDEC) will be used. UDEC is a numerical modeling code for advanced geotechnical analysis of rock, and structural support in two dimensions. UDEC simulates the response of discontinuous media (such as jointed rock) that is subject to either static or dynamic loading. UDEC is a discontinuum code that simulates either the quasi-static or dynamic response to loading of rock media containing multiple, intersecting joint structures. Because it is not limited to a particular type of problem or initial condition, UDEC may be applied to any case where an understanding of the two-dimensional response of such structures is needed. UDEC provides rigid or deformable blocks, multiple material models, full dynamic capability, and high resolution graphics to expedite the modeling process. Solution parameters may be specified by the user, maximizing the user's control over the duration, extent, and efficiency of the model run. Additional control and customization are available to the user through UDEC powerful built-in programming.

2.5 Review of papers

Basarir, et al. (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 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 is 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. However, the validity of the proposed support system, obtained form combination of empirical and numerical modeling should be verified by comparing predictions with actual measurements during construction.

Kockar and Akgun (2003) presented a methodology for tunnel and support design in mixed limestone, schist and phyllite conditions. Detailed geological and geotechnical field investigations in the project area encompassed geological mapping and geological cross-section preparation from boring data, selection of representative rock core samples for geo-mechanics laboratory testing, determination of rock material and rock mass characteristics, determination of RQD from boring data, and determination of discontinuity characteristics through scan-line survey. Laboratory tests were performed to determine the geo-mechanical parameters of good quality rock masses (i.e., regularly jointed, recrystallized limestone). For poor quality rock masses (i.e., phyllite, calc schist, pelitic schist and intercalation of these lithologies), the Hoek–Brown criterion was used to obtain the relevant geo-mechanical parameters since it was almost impossible to recover representative core samples for laboratory testing.

The tunnel grounds were classified according to the Q-system, RMR method and NATM. Empirical tunnel support types and categories were selected for each of the three classification systems. The shear strength parameters and geo-mechanical properties of the rock masses at each borehole location were obtained by using the geological strength index (GSI). Back analysis was performed on a failed rock slope to perform a check on the validity of the shear strength parameters obtained by the GSI method.

The tunnel grounds were divided into sections according to their rock mass classes. By using the appropriate geotechnical parameters, deformations and stress concentrations around each tunnel section were investigated and the interactions of the empirical support systems with the rock masses were analyzed by using the $Phase^2$ finite element software. The regularly jointed rock masses were modeled to be anisotropic, whereas irregularly jointed, highly foliated and very deformable soil-like lithologies were modeled to be isotropic.

In order to decide on the most suitable geometry and determine the stability of the portal, side or cut slope sections, slope stability analyses were performed. Initially, kinematics analyses were performed for the regularly bedded rock masses. Later, limit equilibrium analyses were performed for the kinematically failed rock slopes incorporating the effect of water pressure. Slope stability analyses of irregularly jointed, highly foliated and laminated weak lithologies were analyzed and compared by two different softwares

(Slope/W and PLAXIS 7.2). Following the slope stability analyses, recommendations were made regarding the required support systems or appropriate slope remediation measures.

Bararia and Ozsan (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 (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.

Ghafoori, et al. (2006) suggested that the rock mass classifications (RMR, Q-system, and GSI) were combined with two numerical models to investigate the overall stability of the excavation and to predict the deformation behavior of the the Kallat tunnel in the north east of Iran. Two models based, respectively, on a Finite Element Code (PHASES) and on a Distinct Element Code (UDEC) were defined. The applicability and validity of the proposed procedure has been checked by comparing the predictions with actual observations. It was found that the actual deformations are reasonably close to those predicted through the Distinct Element method. Detailed engineering geological characterization and performance observations were carried out at the site of the Kallat tunnel. The study area consists of calcareous sandstone, limestone, and marl overlain by a thick sequence of limestone. The studies include discontinuity measurements and laboratory testing to determine the geomechanical properties of the rocks for the tunnel site as well as the surrounding area. The strength and modulus of elasticity of rock masses were determined using the Hoek-Brown empirical strength criterion.

Numerical modeling studies (FEM and DEM) based on mapped field data and laboratory data, have used to evaluate the performance of rock mass prior to the tunnel construction. These predictive studies have been then compared with field observation. The DEM and the FEM were applied to the same section of the rock masses to compare their applicability. The DEM model rather than the FEM model proved to generate more realistic results because the DEM simulates the non-linear behavior of the multiple joint sets which control the mechanism of failure since the multiple joint sets in the FEM cannot be simulated.

CHAPTER III

GEOLOGICAL DATA COLLECTION

3.1 Introduction

The important phase of investigation for the designs of portal, adit, and vertical shaft is the careful exploration of local geological conditions. It is a prerequisite for the successful and economic design of engineering structures and underground excavations. Accordingly, a site investigation should attempt to foresee and provide against difficulties that may arise during construction because of ground and/or other local conditions. Investigations should not cease once the construction begins. It is essential that the predictions of the ground conditions that constitute the basic design assumption can be checked as the construction proceeds and the designs modify accordingly if conditions revealed to be different from prediction. In the case of the Siam City Cement Public Company Limited underground opening access in the limestone quarry, when the tunnels have to pass thought in the critical area. It is 100 m depth from the ground surface and long 450 m. Four vertical boreholes are drilled along the adit alignment at the depth of 390 m, and geological investigations have been performed. The engineering geology of the study area is recorded in the field and is used to define the characteristics of the rock mass. The geotechnical evaluation of the project is relied on the exploratory data, field observations and laboratory test results.

3.2 Geology

The rocks in the project area are mainly carbonates and siliciclastics. The different rock types reflect a marine environment of the sedimentation. The fossils that can be found show a Permo-Carboniferous age. The general trend of the geologic structures lies in the northwest-southeast direction (Figure 3.1). Adjacent to the thrust zone on the hanging wall is a thin bed, 30 to 40 m thick of dark silicified shale, which can be easily detected. Bedded limestone lies on the top of the shale. On the footwall, limestone adjacent to the thrust zone sometimes shows heavily fracturing. Away from the thrust zone, spatic limestone mass shows well-defined discontinuities (bedding plane and joints), Figure 3.2.

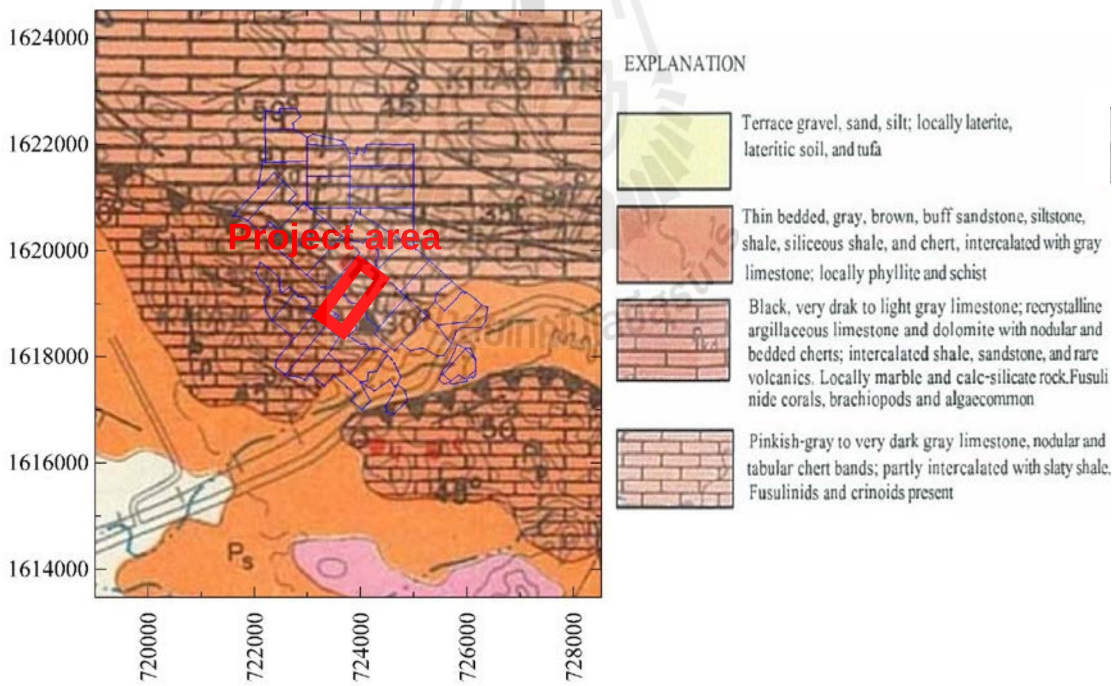


Figure 3.1 Regional geology map of the project area.

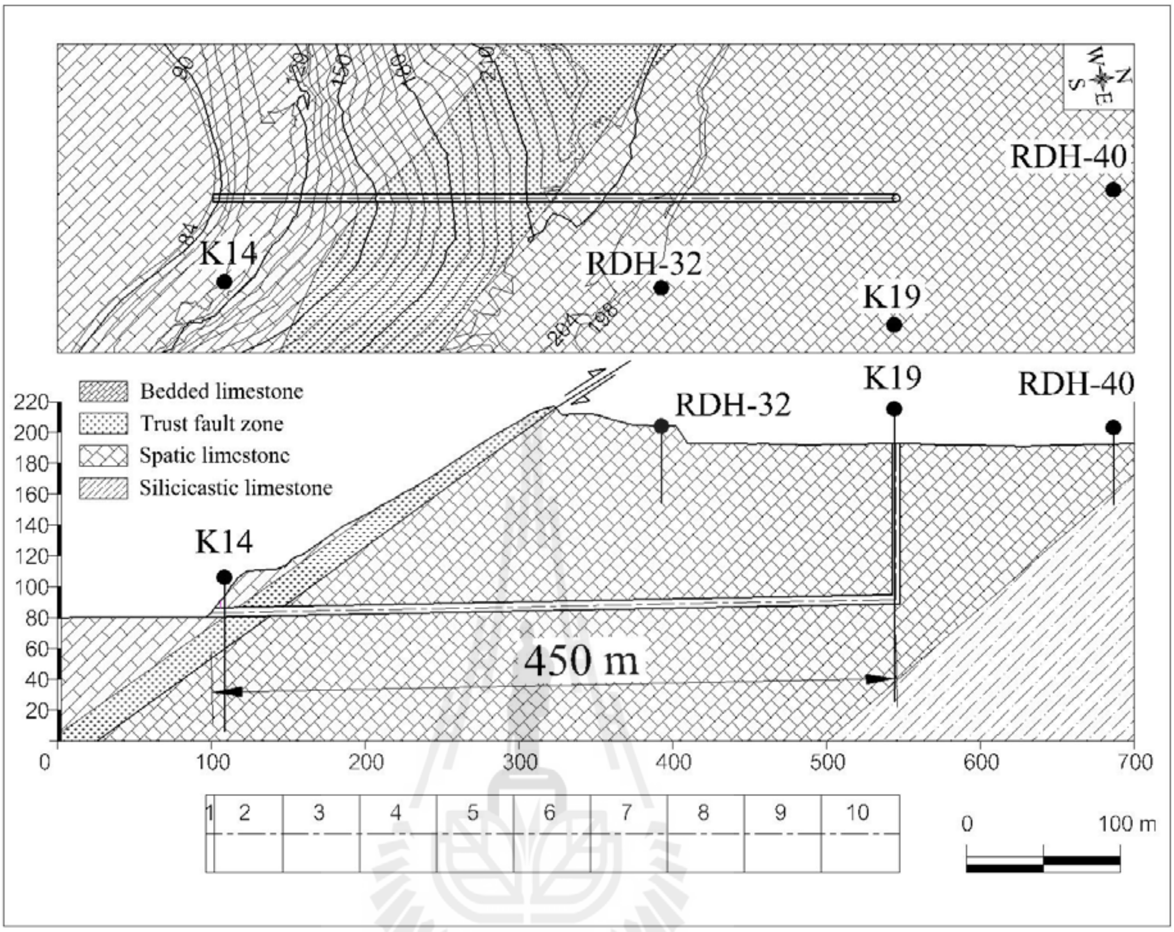


Figure 3.2 Details geological map and cross-section of project area.

3.3 Engineering Geology

Geological descriptions of the rock masses are based on the procedures suggested by Brown (1981). In this study, special emphasis is placed on the characteristics of the discontinuities and also to the degree of weathering. All of which have an influence on the engineering properties of rock mass. Thrust fault and joints are the most dominant structural discontinuities observed in the area. The thrust fault orientation is 130/60 (strike/dip) and the adit direction is N8E. The minor joints are varied in orientation. The adit axis is divided into three different zones of rock mass, bedded limestone, thrust fault zone and spatic limestone. Each of which has different engineering geological properties and lithologic types. A total 11 different sections are classified based on their locally input variables in terms of the geological and geotechnical parameters and the induced overburden stresses.

The bedded limestone in zone 1 will be cut face of adit entrance, Figure 3.3. Sequence crops out consisting of compact, dark grey micritic limestone (with some crinoidal biosparits), chert layers, calcareous siltstone and siltstone. Some layers are very hard and seem to be silicified. Clear dipping towards southwest with an angle of about 40 to 70 degrees. Apertures are 2.5–10 mm wide without material infilling. Average joint spacing ranges between 30 and 50 cm. Discontinuities surfaces are tight. The uniaxial compressive strength (UCS) is measured as 37.4±6.6 MPa. The rock quality designation (RQD) is 51%.

Figure 3.3 Rock mass of bedded limestone in zone 1.

The kakirite in zone 2 is thrust fault zone and about 10 to 30 m thick is observed. This unit consists mainly of heavily brecciated black shale and reddish and bright grey sandstone. Also some tectonised limestone occurred. The tectonic breccia crosses the limestone quarry from SE to NW and dips towards southwest with an angle of about 30 to 60 degrees. Apertures are 0.5–2.5 mm wide. Average joint spacing ranges between 5 and 20 cm. Discontinuity surfaces are slickenside with occasional calcite and clay infilling. The UCS is measured as 66.7±11.9 MPa. The RQD is 34%.

Figure 3.4 Rock mass of kakirite in zone 2.

The spatic limestone in zone 3 is light grey, partly pinkish-violet. The limestone is biosparites or biomiclasts. Some siliciclastic intercalations which cross bedding (sediment structures) have been observed. Sporadically well-rounded micritic extra-clasts are found. Thick massive bedding and homogeneous are typical. At the contact with the siliciclastic unit, dip of the layers towards southwest with an angle of about 40 to 60 degrees is observed. Apertures are 2.5–10 mm wide. Average joint spacing ranges between 50 and 150 cm. Discontinuity surfaces are tight. The UCS is measured as 59.6±13.8 MPa. The RQD is 59%.

Figure 3.5 Rock mass of spatic limestone in zone 3.

In total 216 discontinuities have been measured in the field. Discontinuity orientations are processed by computer software DIPS 5.1, based on equal-area stereographic projection and dominant discontinuity sets are distinguished. The determined dominant discontinuity sets are illustrated in Figure 3.6.

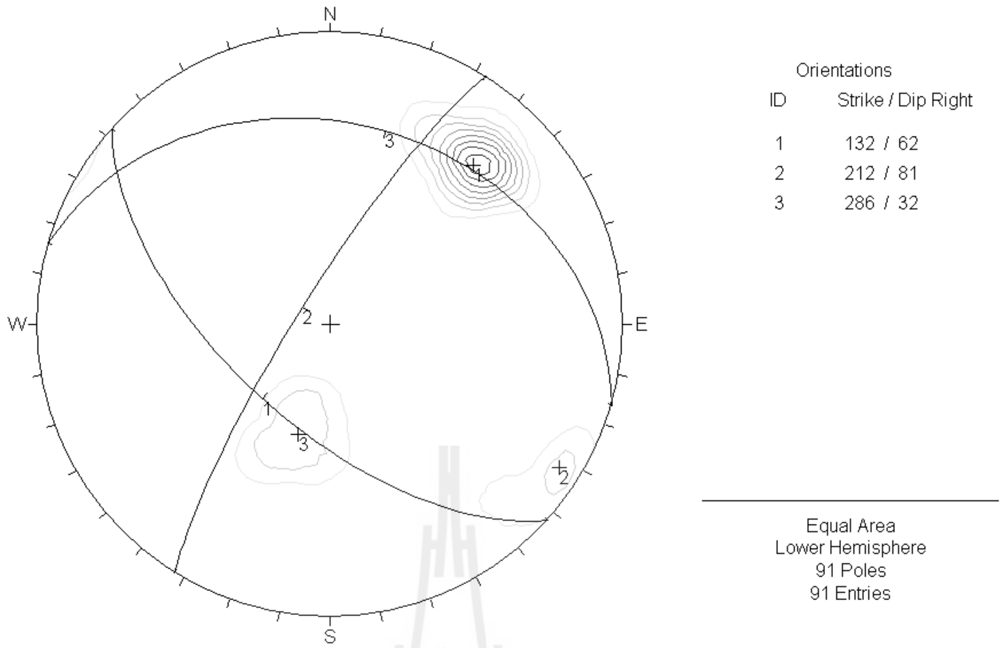


Figure 3.6 Dominant discontinuity sets of bedded limestone.

3.4 Laboratory testing

Laboratory testing is carried out to determine the physical and mechanical properties of intact rock including unit weight, Young's modulus, Poisson's ratio, uniaxial compressive strength, friction angle and cohesion. All laboratory tests are conducted in accordance with the relevant ASTM standards and the ISRM suggested methods (Brown, 1981). Test results are presented in Table 3.1

Table 3.1 Physical and mechanical properties of intact rocks.

Rock type	Density (g/cc)	Uniaxial compressive strength test (MPa)	Friction angle (Degree)	Cohesion (MPa)	Young's modulus (GPa)	Poisson's ratio
edded limestone	2.68	37.4±6.6	37	0.036	6.5±1.8	0.29
Kakirite	2.66	66.7±11.9	40	0.012	8.9±2.0	0.25
Spatic limestone	2.67	59.6±13.8	38	0.049	10.9±2.6	0.27

CHAPTER IV

ROCK MASS CHARACTERIZATIONS

4.1 Introduction

This chapter describes the characterizations of rock mass in the proposed adit area 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, 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 mass along the tunnel alignment is classified by four individual rock mass classification systems included 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 studies, became available and conforms to international standards and procedures (Bieniawski, 1979). In this research, the 1989 version of the classification table has been used by considering the uniaxial compressive strength of intact rock (UCS), rock quality designation (RQD), discontinuity spacing, discontinuity conditions, groundwater 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 water tunnel alignment are shown in Table 4.1.

Table 4.1 Rock mass rating of rock mass along adit in the study area.

Section	RMR Rating value	RMR Class	Description
1	49	III	Fair Rock
2	36	IV	Poor Rock
3	71	II	Good Rock
4	71	II	Good Rock
5	71	II	Good Rock
6	71	II	Good Rock
7	71	II	Good Rock
8	71	II	Good Rock
9	71	II	Good Rock
10	71	II	Good Rock
11	71	II	Good Rock

The results from RMR rock mass classification show the rock class range in the study area is from good to poor rock classes. In the study area, section 1 is classified as fair rock class which rating value at 49, section 2 is classified as poor rock class which rating value at 36 and section 3 to 11 are classified as good rock class with rating at 71. The UCS, RQD, and the discontinuities are the main factors governing the rock class in the study area.

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). 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 adit in study area.

Section	Q Index value	Q-Class	Description
1	1.42	D	Poor
2	0.43	E	Very poor
3	39.33	B	Good
4	39.33	B	Good
5	39.33	B	Good
6	39.33	B	Good
7	39.33	B	Good
8	39.33	B	Good
9	39.33	B	Good
10	39.33	B	Good
11	39.33	B	Good

The results from Q-system rock mass classification show rock class as good and very poor class. The poor (D) rock class has rating value of 1.42 from the sections 1. The very poor (E) rock class has rating value of 0.43 from the sections 2. While in sections 3 to 11 is classified as good (B) class with the rating value at 39.33. The result is governed by the RQD, the discontinuities, and SRF in this study area.

4.4 Rock mass index (RMi)

Palmström (1995) proposed rock mass index (RMi) for general characterization. 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 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 Palmström (1995):

$$V_b = \beta \times J_v^{-3} \tag{4.1}$$

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 water tunnel alignment are described in Table 4.3

Table 4.3 RMi index value and class of rock mass along adit in study area.

Section	RMi index value	Description
1	0.47	Poor
2	0.31	Very poor
3	11.30	Good
4	11.30	Good
5	11.30	Good
6	11.30	Good
7	11.30	Good
8	11.30	Good
9	11.30	Good
10	11.30	Good
11	11.30	Good

The results from RMi rock mass classification show the rock class in the study area fall in poor, very poor and good classes. The poor rock class has index value of 0.47 found in the section 1. The very poor rock class has index value of 0.31 found in the section 2. The good rock class has index value at 11.30 found in the section 3 to 11.

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 (e.g. 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 adit alignment are described in Table 4.4. Based on the modified quantitative GSI, the classes in the study area are very poor to good class rock. The fair rock class has index value 55 and in section 1. The very poor rock class has index value 15 and in section 2. The good rock class has index value range at 65 found in the section 3 to 11.

Table 4.4 GSI index value and class of rock mass along adit in study area.

Section	GSI index value	Description
1	55	Fair rock
2	15	Very poor rock
3	65	Good rock
4	65	Good rock
5	65	Good rock
6	65	Good rock
7	65	Good rock
8	65	Good rock
9	65	Good rock
10	65	Good rock
11	65	Good rock

4.6 Comparison of the rock mass classification results from four different rock mass classification systems.

The rock mass classes along the study area adit are classified by four rock mass classification systems. There are summarized in Table 4.5. The three different rock class zones are defined by the results of four rock mass classification systems, Zone 1 is identified as fair rock, Zone 2 is very poor rock and Zone 3 is generally identified as good rock.

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

Section	**Zone**	**RMR**	**Q**	**RMi**	**GSI**
1	Zone 1	Fair	Poor	Poor	Fair
2	Zone 2	Poor	Very poor	Very poor	Very poor
3	Zone 3	Good	Good	Good	Good
4		Good	Good	Good	Good
5		Good	Good	Good	Good
6		Good	Good	Good	Good
7		Good	Good	Good	Good
8		Good	Good	Good	Good
9		Good	Good	Good	Good
10		Good	Good	Good	Good
11		Good	Good	Good	Good

The utility parameters of the four different rock mass classification systems are varied. Therefore classify different rock mass class in accordance with their utilized parameters. In RMR and GSI systems have no input parameter for rock stress but Q and RMi system include the stress factor in the estimated value. The number of joint set is considered indirectly in RMR classification system. The Q system is a function of three parameters which are measured from block size, inter-block shear strength and active stress. The RMi system has similar input parameters with those of Q-system, jointing parameter. The GSI system classifies the rock mass based on the surface condition rating, such as roughness rating, weathering and infilling rating. All systems consider the condition of discontinuities. The RMR and Q systems consider groundwater condition which is indirectly considered in RMi and GSI systems.

The class of each rock mass classifications are different, The RMR-system 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. The 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.

CHAPTER V

DETERMINATION OF ROCK MASS PARAMETERS AND STABILITY ANALYSIS

5.1 Introduction

This chapter describes the determination of geotechnical rock mass parameters and stability analysis. The rock mass parameters are evaluated by empirical equations which are developed by many researchers based on the rock mass classification systems. The stability of the tunnels is evaluated in terms of stand-up time, 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 tunnels. Reliable input parameters to distinct element method can produce accurate 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 values 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 adit are important and a numerical analysis of these deformations requires an estimation 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 the 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 et al. (1999) has proposed:

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

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:

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

E_m is expressed by Barton (2002) as:

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

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

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

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

Using the geological strength index (GSI), provided that 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 et al., 2002).

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

For $\sigma_c > 100$ MPa, use equation 5.8.

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

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 raw material transportation adit constructions, control blasting method is used but blasting in hard rock adit results in severe local damage, extending 2 or 3 m in surrounding rock mass. Therefore, the value of D is 0.8. The results of the deformation modulus of rock mass for all sections of the adit 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 study area.

Section	From RMR			From Q		From RMi	From GSI	Avg. (GPa)
	Eq.5.1 (GPa)	Eq.5.2 (GPa)	Eq.5.3 (GPa)	Eq.5.4 (GPa)	Eq.5.5 (GPa)	Eq.5.6 (GPa)	Eq.5.7 (GPa)	
1		9.44	11.76	3.81	8.10	4.22	4.89	7.04
2		4.77	4.67		6.59	3.61	0.65	4.00
3	42.00		35.79	39.87	28.62	13.90	10.98	28.53
4	42.00		35.79	39.87	28.62	13.90	10.98	28.53
5	42.00		35.79	39.87	28.62	13.90	10.98	28.53
6	42.00		35.79	39.87	28.62	13.90	10.98	28.53
7	42.00		35.79	39.87	28.62	13.90	10.98	28.53
8	42.00		35.79	39.87	28.62	13.90	10.98	28.53
9	42.00		35.79	39.87	28.62	13.90	10.98	28.53
10	42.00		35.79	39.87	28.62	13.90	10.98	28.53
11	42.00		35.79	39.87	28.62	13.90	10.98	28.53

5.2.2 Hoek and Brown parameters

The Hoek and Brown failure criterion for rock mass 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 et al. (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.

Palmström (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 .

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 but blasting in hard rock adit results in severe local damage, value D is 0.8. The calculated Hoek and Brown constants of rock mass, 'm_j' and 's_j', for all sections of tunnel is shown in Tables 5.2 and 5.3, respectively.

Table 5.2 Calculated Hoek and Brown constant of rock mass (m_j) for all sections of the study area.

Section	m_j				
	Eq.(5.9)	Eq.(5.13)	Eq.(5.16)	Eq.(5.18)	Average
	RMR	Q	RMi	GSI	
1	0.26	3.82	3.55	0.69	2.08
2	0.06	0.53	0.03	0.04	0.17
3	1.26	4.01	4.29	1.25	2.70
4	1.26	4.01	4.29	1.25	2.70
5	1.26	4.01	4.29	1.25	2.70
6	1.26	4.01	4.29	1.25	2.70
7	1.26	4.01	4.29	1.25	2.70
8	1.26	4.01	4.29	1.25	2.70
9	1.26	4.01	4.29	1.25	2.70
10	1.26	4.01	4.29	1.25	2.70
11	1.26	4.01	4.29	1.25	2.70

Table 5.3 Calculated Hoek and Brown constant (s_j) for all sections of the study area.

Section	S_j				
	Eq.(5.10)	Eq.(5.14)	Eq.(5.17)	Eq.(5.19)	Average
	RMR	Q	RMi	GSI	
1	$2.03E^{-4}$	$4.53E^{-2}$	$8.94E^{-2}$	$1.09E^{-3}$	$3.40E^{-2}$
2	$2.33E^{-5}$	$5.67E^{-4}$	$5.70E^{-6}$	$2.55E^{-6}$	$1.50E^{-4}$
3	$7.96E^{-3}$	$5.24E^{-2}$	$1.39E^{-1}$	$4.98E^{-3}$	$5.11E^{-2}$
4	$7.96E^{-3}$	$5.24E^{-2}$	$1.39E^{-1}$	$4.98E^{-3}$	$5.11E^{-2}$
5	$7.96E^{-3}$	$5.24E^{-2}$	$1.39E^{-1}$	$4.98E^{-3}$	$5.11E^{-2}$
6	$7.96E^{-3}$	$5.24E^{-2}$	$1.39E^{-1}$	$4.98E^{-3}$	$5.11E^{-2}$
7	$7.96E^{-3}$	$5.24E^{-2}$	$1.39E^{-1}$	$4.98E^{-3}$	$5.11E^{-2}$
8	$7.96E^{-3}$	$5.24E^{-2}$	$1.39E^{-1}$	$4.98E^{-3}$	$5.11E^{-2}$
9	$7.96E^{-3}$	$5.24E^{-2}$	$1.39E^{-1}$	$4.98E^{-3}$	$5.11E^{-2}$
10	$7.96E^{-3}$	$5.24E^{-2}$	$1.39E^{-1}$	$4.98E^{-3}$	$5.11E^{-2}$
11	7.96E-3	5.24E-2	1.39E-1	4.98E-3	5.11E-2

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 the 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:

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

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 in 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. Roc Data software version 3.0 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 water tunnel 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 study tunnel area 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 study area.

Section	From RMR Eq.(5.21) (MPa)	From Q Eq.(5.22) (MPa)	From RMi Eq.(5.23) (MPa)	From GSI RocData 4.0 (MPa)	Average (MPa)
1	2.464	0.950	0.470	4.143	2.007
2	2.197	0.123	0.310	1.077	0.927
3	12.692	0.623	11.300	9.286	8.475
4	12.692	0.623	11.300	9.286	8.475
5	12.692	0.623	11.300	9.286	8.475
6	12.692	0.623	11.300	9.286	8.475
7	12.692	0.623	11.300	9.286	8.475
8	12.692	0.623	11.300	9.286	8.475
9	12.692	0.623	11.300	9.286	8.475
10	12.692	0.623	11.300	9.286	8.475
11	12.692	0.623	11.300	9.286	8.475

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 major limestone belt conveyor access adit and portal intersection, ESR value is 1.0. 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 study area 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 study area.

<table>
<tr><th rowspan="2">Section</th><th colspan="3">RMR</th><th colspan="2">Q-System</th></tr>
<tr><th>RMR Value</th><th>Maximum Unsupported span</th><th>Stand-up time</th><th>Q Value</th><th>Maximum Unsupported span</th></tr>
<tr><td>1</td><td>49</td><td>8.0 m</td><td>2 days</td><td>1.42</td><td>2.5 m</td></tr>
<tr><td>2</td><td>36</td><td>4.0 m</td><td>7 hrs.</td><td>0.43</td><td>1.80 m</td></tr>
<tr><td>3</td><td rowspan="9">71</td><td rowspan="9">18 m</td><td rowspan="9">2 months</td><td rowspan="9">39.33</td><td rowspan="9">9.5 m</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td></tr>
<tr><td>6</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td></tr>
<tr><td>9</td></tr>
<tr><td>10</td></tr>
<tr><td>11</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,

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

for the tangential stress at side wall,

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

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 studied adit, 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 are calculated by the following equation:

$$\sigma_v = \gamma H \qquad (5.26)$$

After estimating the overall stresses for all sections of tunnel, 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 tunnel is 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 sections are stable. They do not need to be supported to increase safety factor. However, stability problems in blocky jointed rock mass are generally associated with gravity falling and sliding of blocks from roof and sidewalls. Rock stress at shallow depth are generally low that does not control the failure mechanism.

Table 5.6 Calculated induced stress and factor of safety for all sections.

Section	**H (m)**	σ_v **(MPa)**	$\sigma_{\theta roof}$ **(MPa)**	$\sigma_{\theta wall}$ **(MPa)**	σ_{cm} **(MPa)**	**FS (roof)**	**FS (wall)**
1	1.00	0.03	0.06	0.04	2.01	36.34	44.90
2	5.27	0.14	0.29	0.24	0.93	3.18	3.93
3	25.86	0.68	1.42	1.15	8.48	5.97	7.38
4	56.53	1.48	3.11	2.51	8.48	2.73	3.37
5	84.47	2.21	4.64	3.76	8.48	1.83	2.25
6	115.12	3.01	6.33	5.12	8.48	1.34	1.65
7	121.09	3.17	6.66	5.39	8.48	1.27	1.57
8	112.02	2.93	6.16	4.99	8.48	1.38	1.70
9	99.63	2.61	5.48	4.44	8.48	1.55	1.91
10	98.14	2.57	5.40	4.37	8.48	1.57	1.94
11	97.76	2.56	5.38	4.35	8.48	1.58	1.95

CHAPTER VI

SUPPORT DESIGN

6.1 Introduction

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

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 studied adit.

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

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

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 follows:

$$P_{roof} = \frac{2.0}{J_r} Q^{-1/3} \tag{6.2}$$

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 the adit. The results are given in Table 6.1.

Table 6.1 Calculated support pressure for all sections of the studied adit.

Section	**From RMR**	**From Q**
	Eq. (6.1), kN/m^2	**Eq. (6.2), kN/m^2**
1	67.04	1.78
2	83.50	2.65
3	37.98	0.10
4	37.98	0.10
5	37.98	0.10
6	37.98	0.10
7	37.98	0.10
8	37.98	0.10
9	37.98	0.10
10	37.98	0.10
11	37.98	0.10

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 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 adit. The method of excavation is provided based on the rock mass rating values. The suggested support system assessed based on rock mass rating system (RMR) are presented in Table 6.2.

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

<table>
<tr><th>Section</th><th>RMR Value</th><th>Rock Bolt</th><th>Shotcrete</th><th>Steel Set</th></tr>
<tr><td>1</td><td>49</td><td>Systematic cable bolts long 4 m, spaced 1.5-2 m in crown and wall with wire mesh in crown.</td><td>50-100 mm in crown and 30 mm in sides.</td><td>None</td></tr>
<tr><td>2</td><td>36</td><td>Systematic cable bolts 4-5 m long, spaced 1-1.5 m in crown and wall with wire mesh.</td><td>100-150 mm in crown and 100 mm in sided.</td><td>Light ribs spaced 1.5 m.</td></tr>
<tr><td>3</td><td rowspan="9">71</td><td rowspan="9">Locally bolts in crown, 3 m long, spaced 2.5 m with occasional wire mesh.</td><td rowspan="9">50 mm in crown where required.</td><td rowspan="9">None</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td></tr>
<tr><td>6</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td></tr>
<tr><td>9</td></tr>
<tr><td>10</td></tr>
<tr><td>11</td></tr>
</table>

6.3.2 NGI tunneling quality Index (Q system)

The NGI tunneling quality index (Q system) is related to adit 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). 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 fiber reinforced shotcrete. The summary of the support measures for all sections based on Q system is given in Table 6.3

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

<table>
<tr><th>Section</th><th>Q Value</th><th>Rock bolt</th><th>Shotcrete</th><th>Steel Set</th></tr>
<tr><td>1</td><td>1.42</td><td>Bolt length 1.7-2.4 m
spacing 1.7-2.1 m</td><td>Un-reinforce
Shotcrete 40-100 mm</td><td rowspan="2">None</td></tr>
<tr><td>2</td><td>0.43</td><td>Bolt length 1.7-2.4 m
spacing 1.5-1.7 m</td><td>Fiber reinforce
Shotcrete 50-90 mm</td></tr>
<tr><td>3</td><td rowspan="9">39.33</td><td rowspan="9" colspan="3">Unsupported</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td></tr>
<tr><td>6</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td></tr>
<tr><td>9</td></tr>
<tr><td>10</td></tr>
<tr><td>11</td></tr>
</table>

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 (Palmström, 2001) is used to get direct assessment of support types for all sections of the adit. The support measures evaluated based on rock mass index (RMi) is summarized in Table 6.4. The suggested support types based on rock mass index (RMi) include rock bolts and fiber reinforced shotcrete.

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

<table>
<tr><th>Section</th><th>RMi value</th><th>Rock bolt</th><th>Shotcrete</th></tr>
<tr><td>1</td><td>0.47</td><td>Systematic bolt spacing 2 x 2 m for roof and systematic bolt spacing 2.5 x 2.5 m for walls.</td><td>Fibre reinforced shotcrete thickness 80-100 mm for roof and 60-80 mm for walls.</td></tr>
<tr><td>2</td><td>0.31</td><td>Systematic bolt spacing 3 x 3 m for roof and systematic bolt spacing 2.5 x 2.5 m for walls.</td><td>Reinforced shotcrete thickness 60 mm for roof.</td></tr>
<tr><td>3</td><td rowspan="9">11.30</td><td rowspan="9">Systematic bolt spacing 2 x 2 m for roof and systematic bolt spacing 2.5 x 2.5 m for walls.</td><td rowspan="9">Fibre reinforced shotcrete thickness 60 mm for roof and shortcrete 50-60 mm for walls.</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td></tr>
<tr><td>6</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td></tr>
<tr><td>9</td></tr>
<tr><td>10</td></tr>
<tr><td>11</td></tr>
</table>

6.4 Support design using numerical method

The Universal Distinct Element Code (UDEC) is a two-dimensional numerical program based on the distinct element method for discontinuum modeling. UDEC simulates the response of discontinuous media subjected to either static or dynamic loading. The discontinuous medium is represented as an assemblage of discrete blocks. The discontinuities are treated as boundary conditions between blocks; large displacements along discontinuities and rotations of blocks are allowed. Individual blocks behave as either rigid or deformable material. Deformable blocks are subdivided into a mesh of finite-difference elements, and each element responds according to a prescribed linear or non-linear stress-strain law. The relative motion of the discontinuities is also governed by linear or non-linear force-displacement relations for movement in both the normal and shear directions. UDEC has several built-in material behavior models, for both the intact blocks and the discontinuities, which permit the simulation of response representative of discontinuous geologic. UDEC is well-suited to model the large movements and deformations of a blocky system.

An important aspect of the geomechanical analysis and design is the use of structural support to stabilize a rock mass. The term support describes engineered materials used to restrict displacements in the immediate vicinity of an opening. The support systems are composed of reinforcement and surface support. Reinforcement consists of bolts installed in holes drilled in the rock mass. Reinforcement acts to conserve inherent rock mass strength so that it becomes self-supporting. Surface support consists of shotcrete that are placed on the surface of an excavation, the weights of individual blocks isolated by discontinuities or zones of loosened rock. For this study, boundary conditions are defined as restrained X and Y for both sides boundary. The finite element mesh and boundary conditions for the analysis sections show in Figure 6.1.

For the numerical simulations in study area, 11 sections were done with four types of models, unsupported and supported by RMR, Q, and RMi suggested are simulated for each section of the adit. The rock mass parameters calculated by empirical methods, described in Chapter 5, are used as input parameters in numerical simulations.

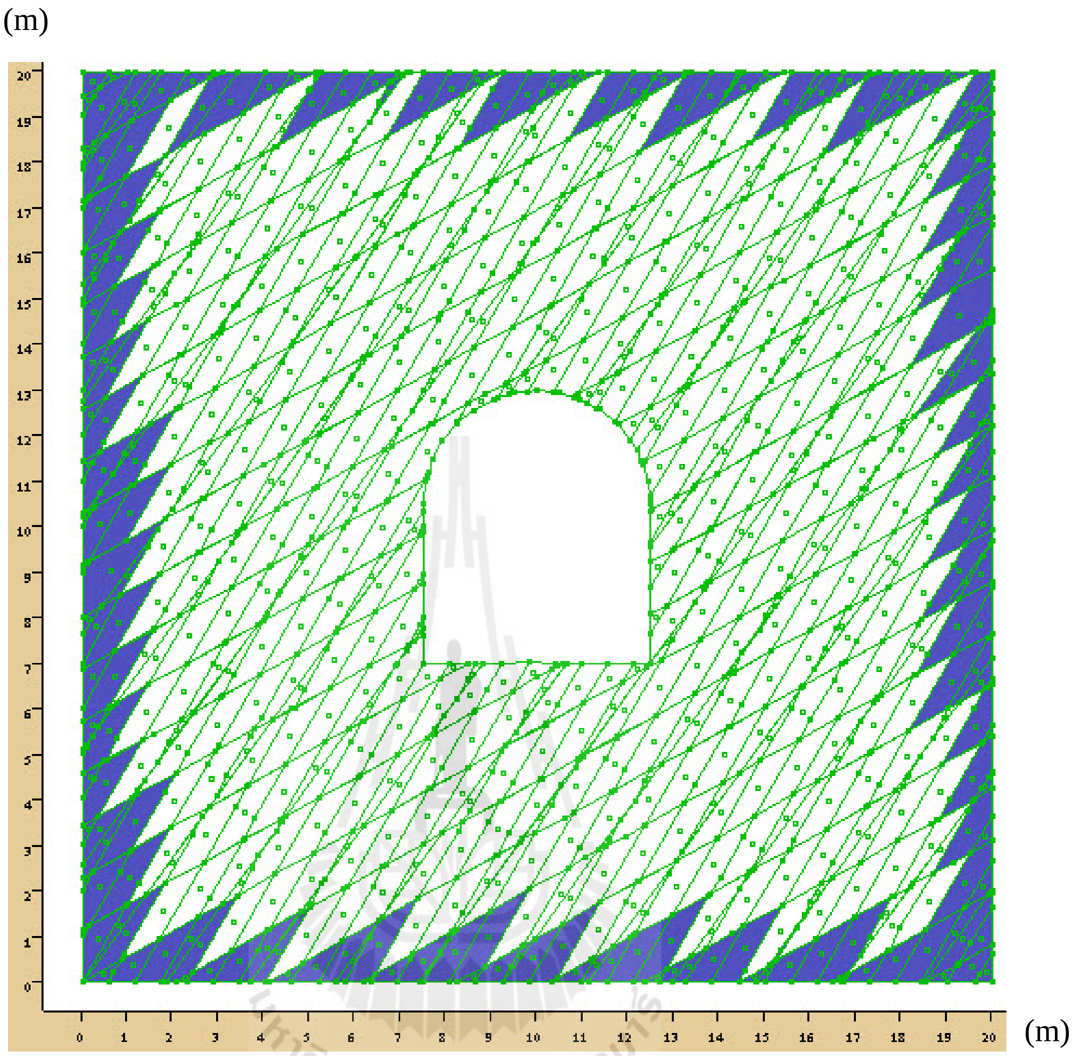


Figure 6.1 Finite element mesh and boundary conditions for the analysis.

The maximum displacement vectors of the adit before support and support by RMR, Q, and RMi suggested given in Table 6.5.

Table 6.5 UDEC simulation results for maximum displacements vector before support and support by empirical suggested.

<table>
<tr><th>Section</th><th>Unsupported (m)</th><th>RMR supported (m)</th><th>Q supported (m)</th><th>RMi supported (m)</th></tr>
<tr><td>1</td><td>0.058</td><td>0.020</td><td>0.026</td><td>0.024</td></tr>
<tr><td>2</td><td>23.45</td><td>0.606</td><td>6.184</td><td>6.126</td></tr>
<tr><td>3</td><td rowspan="8">0.008</td><td rowspan="8">0.004</td><td rowspan="8">-</td><td rowspan="8">0.005</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td></tr>
<tr><td>6</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td></tr>
<tr><td>9</td></tr>
<tr><td>10</td></tr>
<tr><td>11</td><td>17.53</td><td>0.099</td><td>-</td><td>0.120</td></tr>
</table>

Because UDEC is a two-dimensional program, the three-dimensional effect of regularly spaced elements is accommodated by scaling their material properties in the out-of-plane direction. In the case of adit support design in this study area, it considered only the magnitude of displacement. After support installation, the maximum displacement vectors as shown in Figures 6.2 through 6.29.

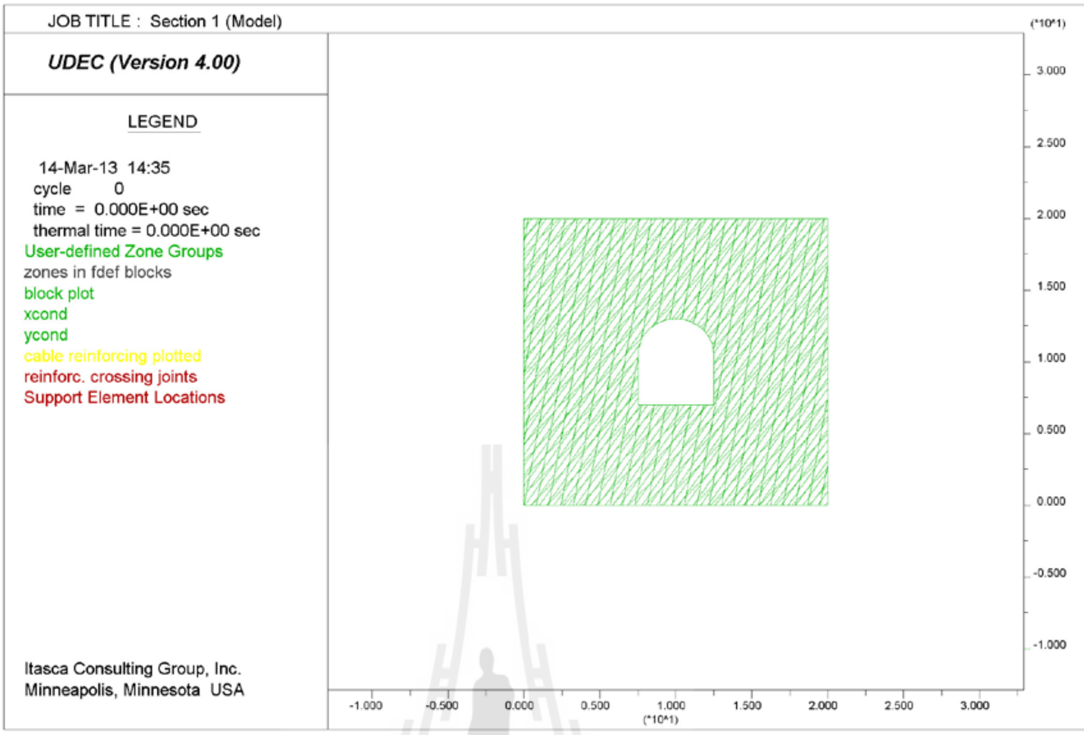


Figure 6.2 Joint sets and excavation boundary for section 1.

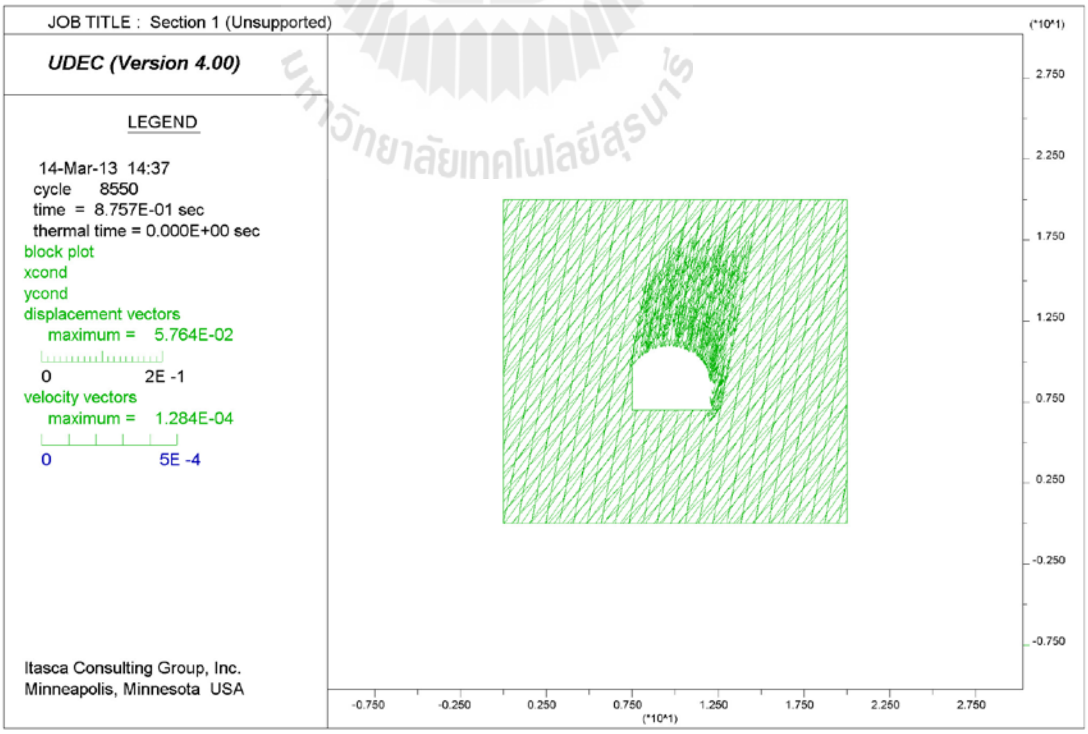


Figure 6.3 Maximum displacement vectors unsupported for section 1.

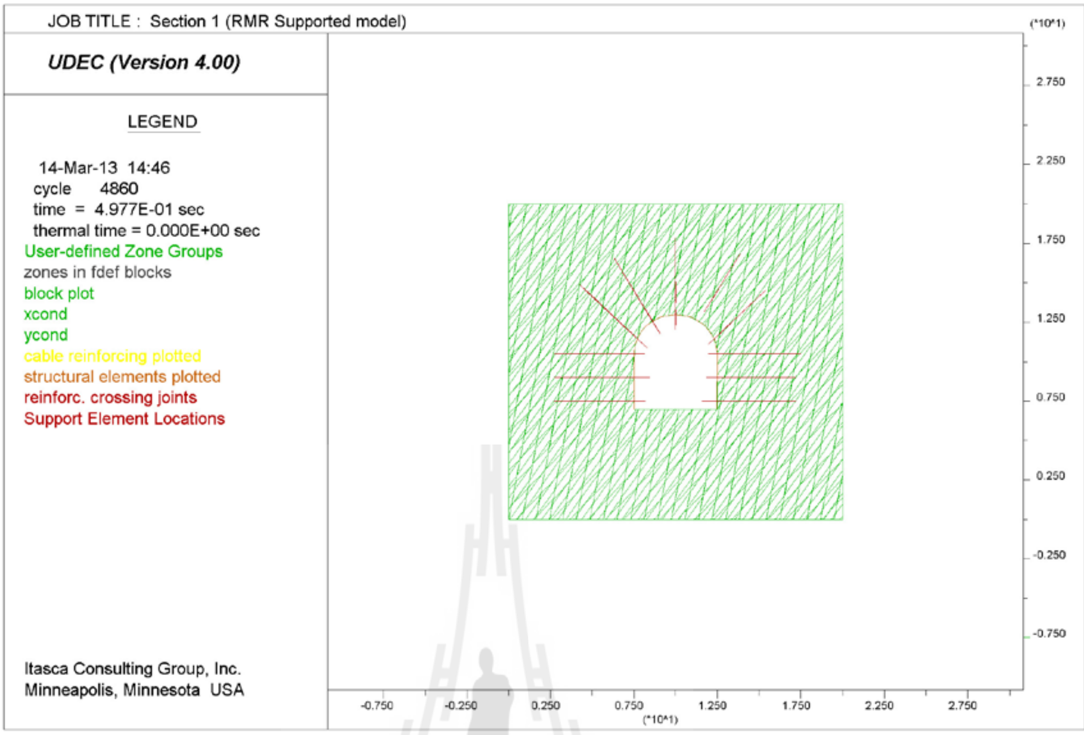


Figure 6.4 Structure adit supported as RMR suggested for section 1.

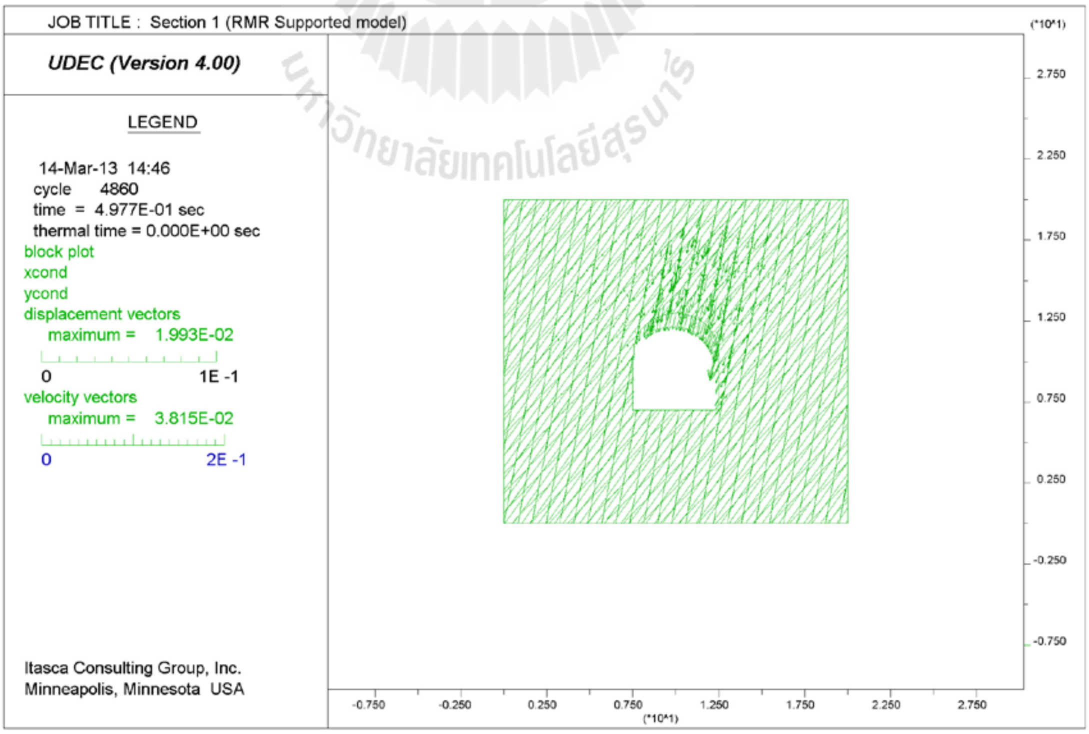


Figure 6.5 Maximum displacement vectors supported as RMR suggested for section 1.

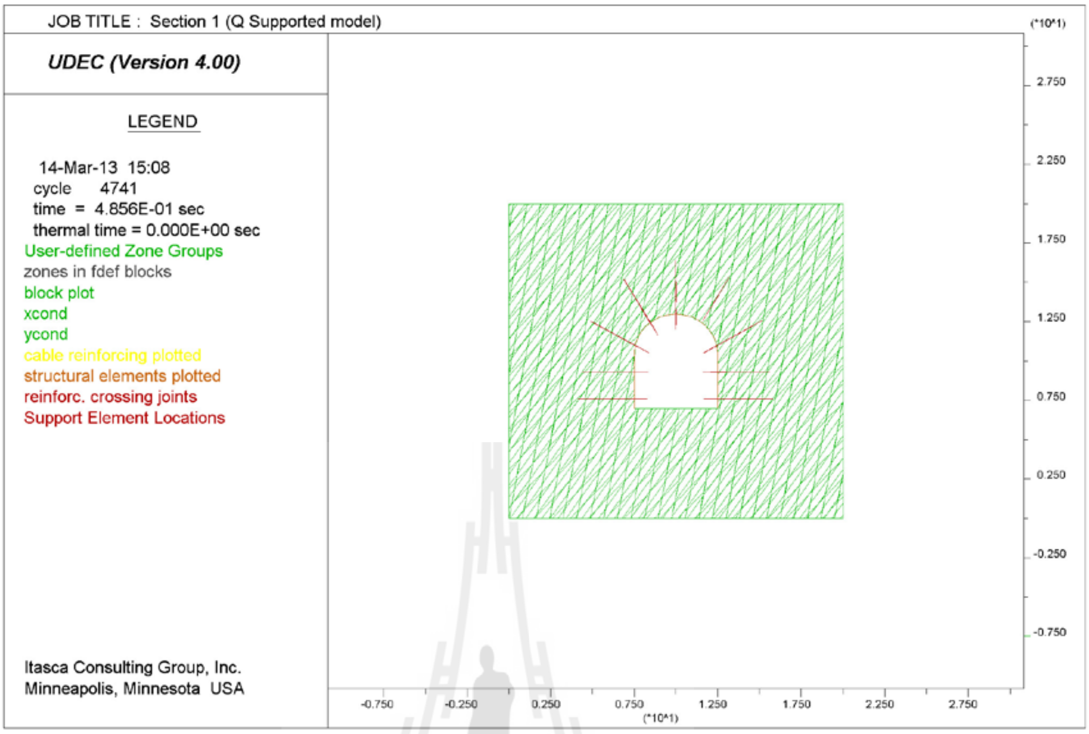


Figure 6.6 Structure adit supported as Q suggested for section 1.

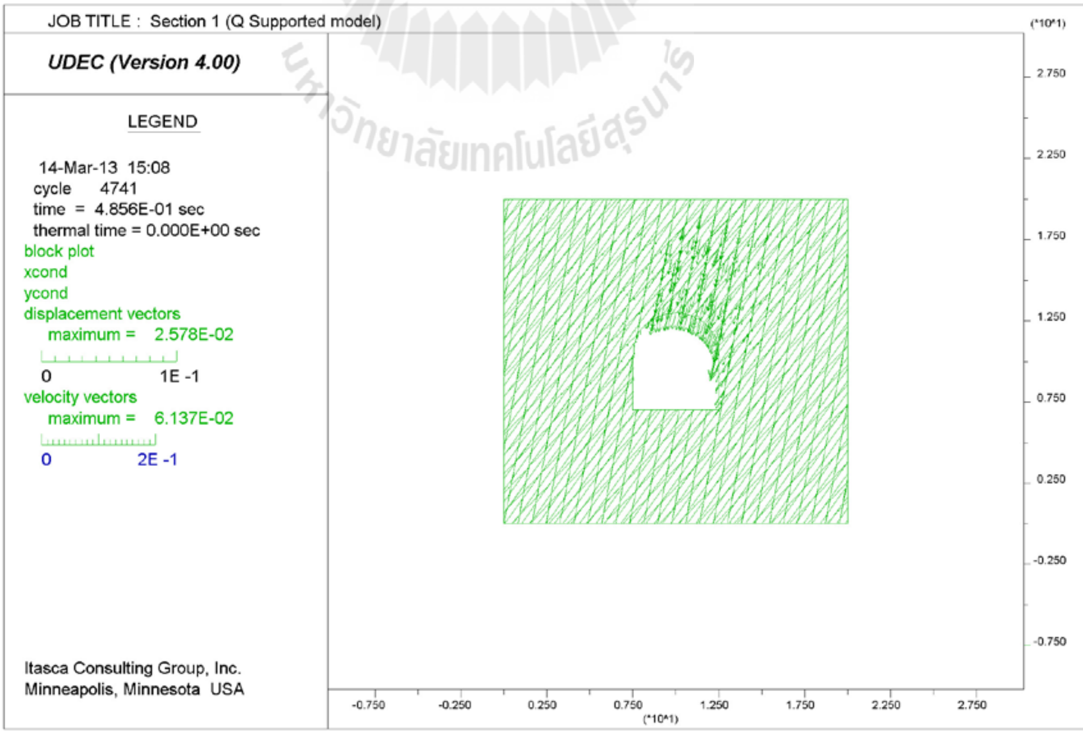


Figure 6.7 Maximum displacement vectors supported as Q suggested for section 1.

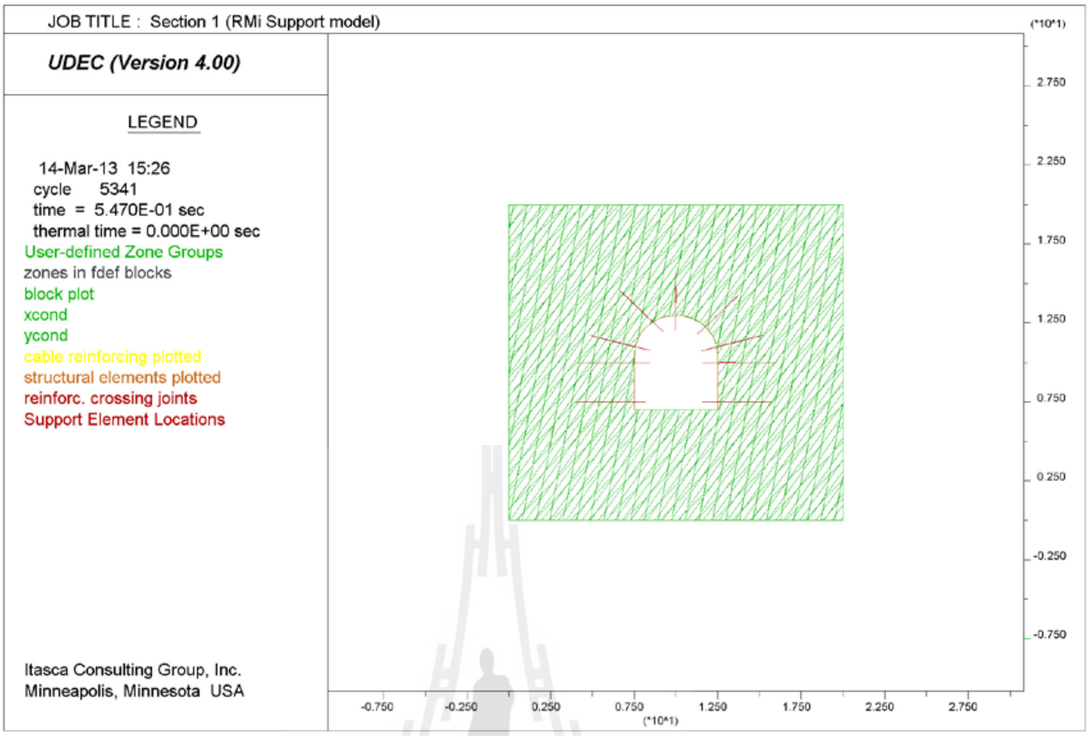


Figure 6.8 Structure adit supported as RMi suggested for section 1.

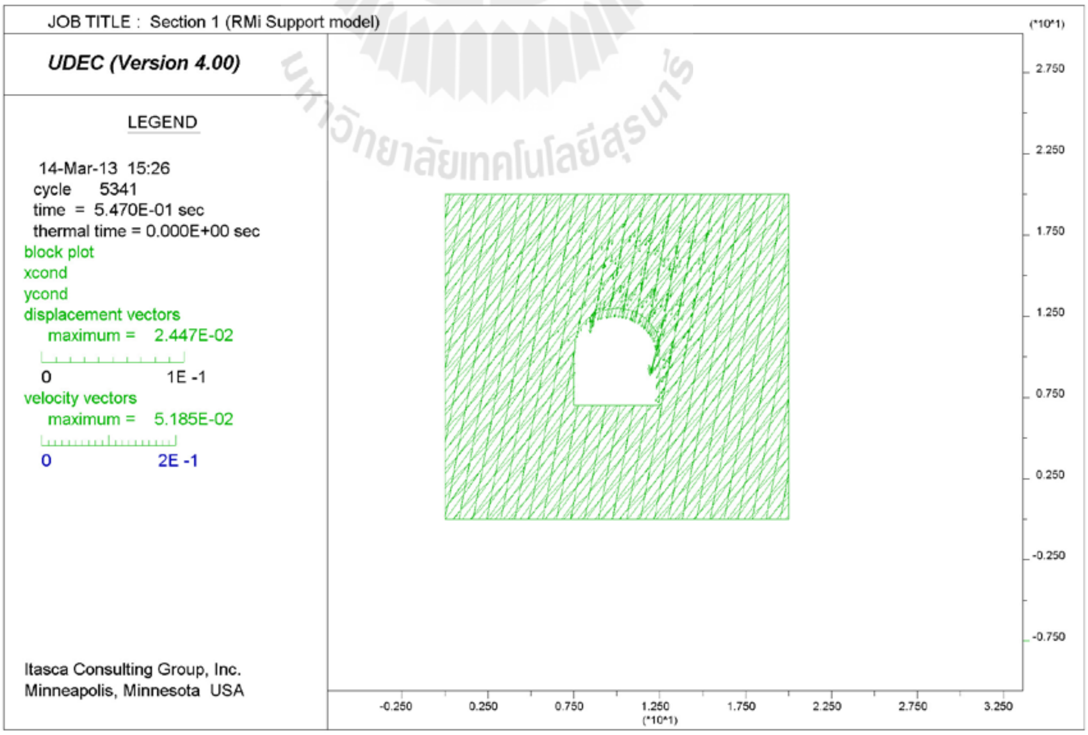


Figure 6.9 Maximum displacement vectors supported as RMi suggested for section 1.

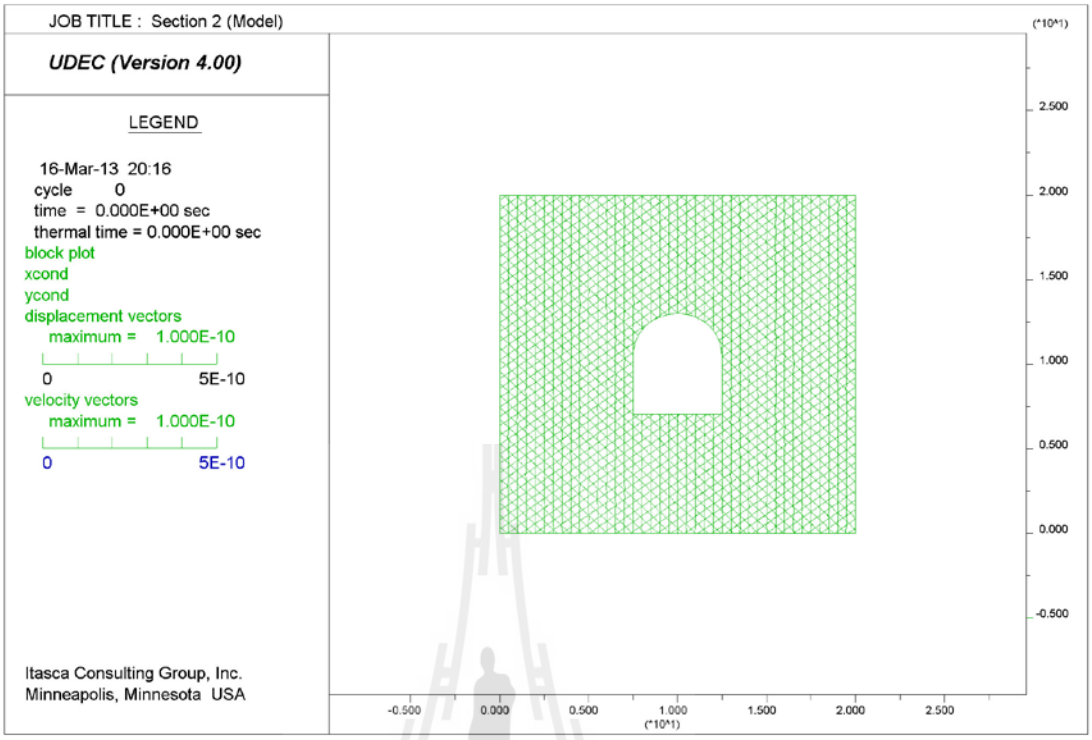


Figure 6.10 Joint sets and excavation boundary for section 2.

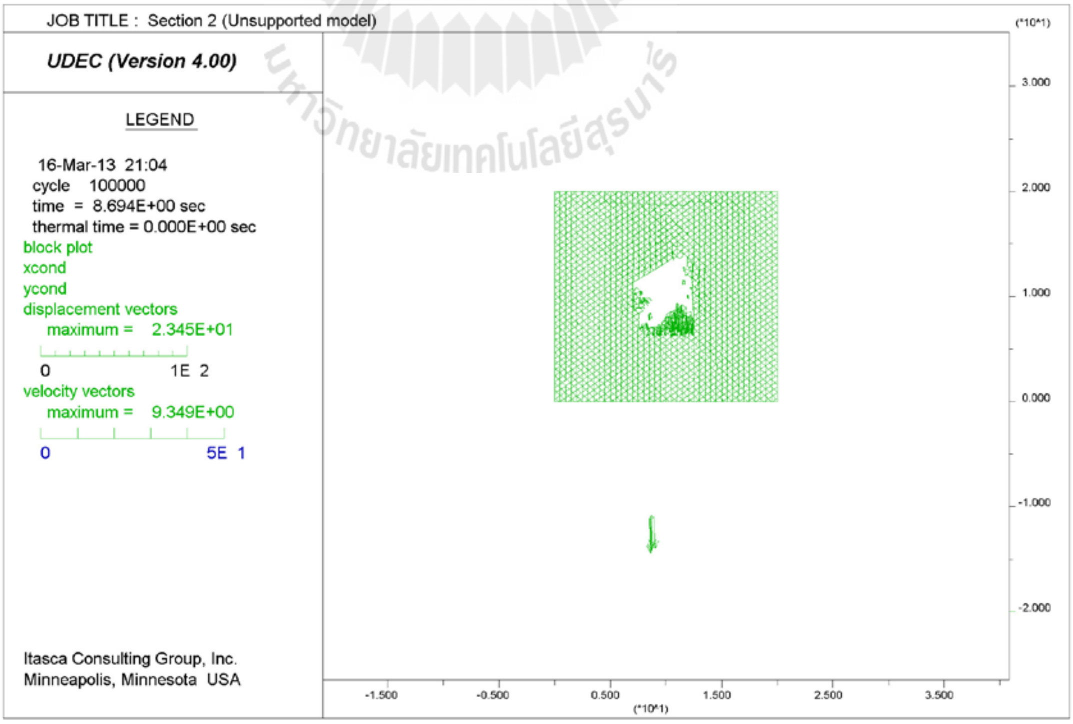


Figure 6.11 Maximum displacement vectors unsupported for section 2.

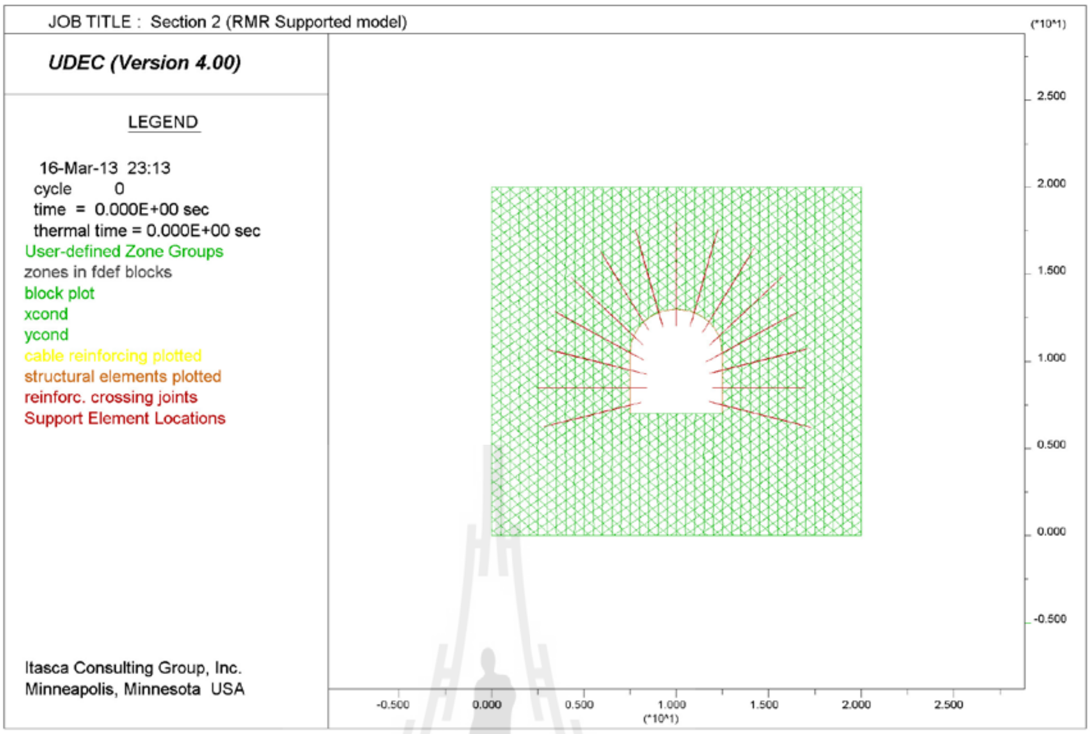


Figure 6.12 Structure adit supported as RMR suggested for section 2.

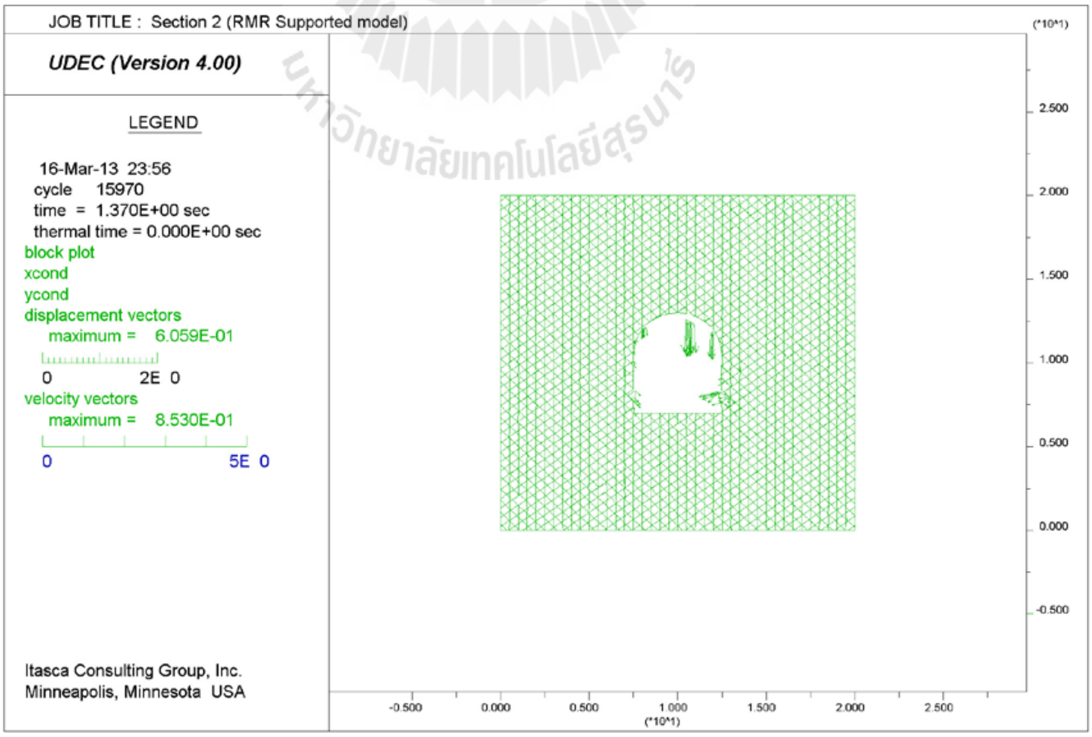


Figure 6.13 Maximum displacement vectors supported as RMR suggested

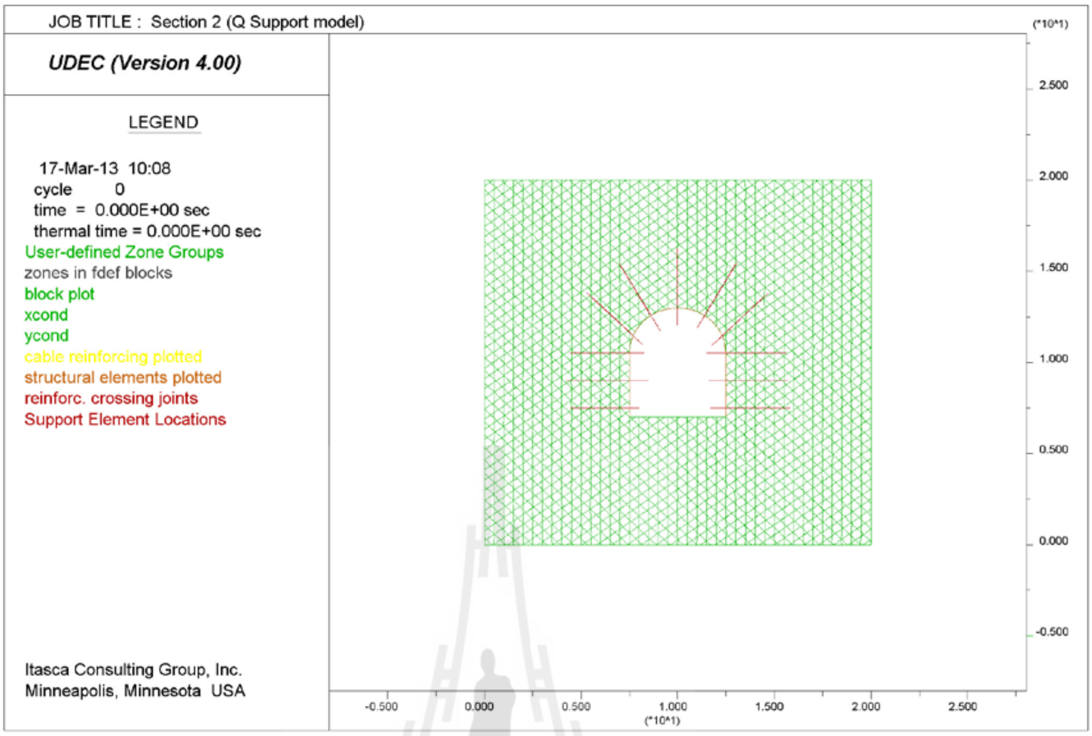


Figure 6.14 Structure adit supported as Q suggested for section 2.

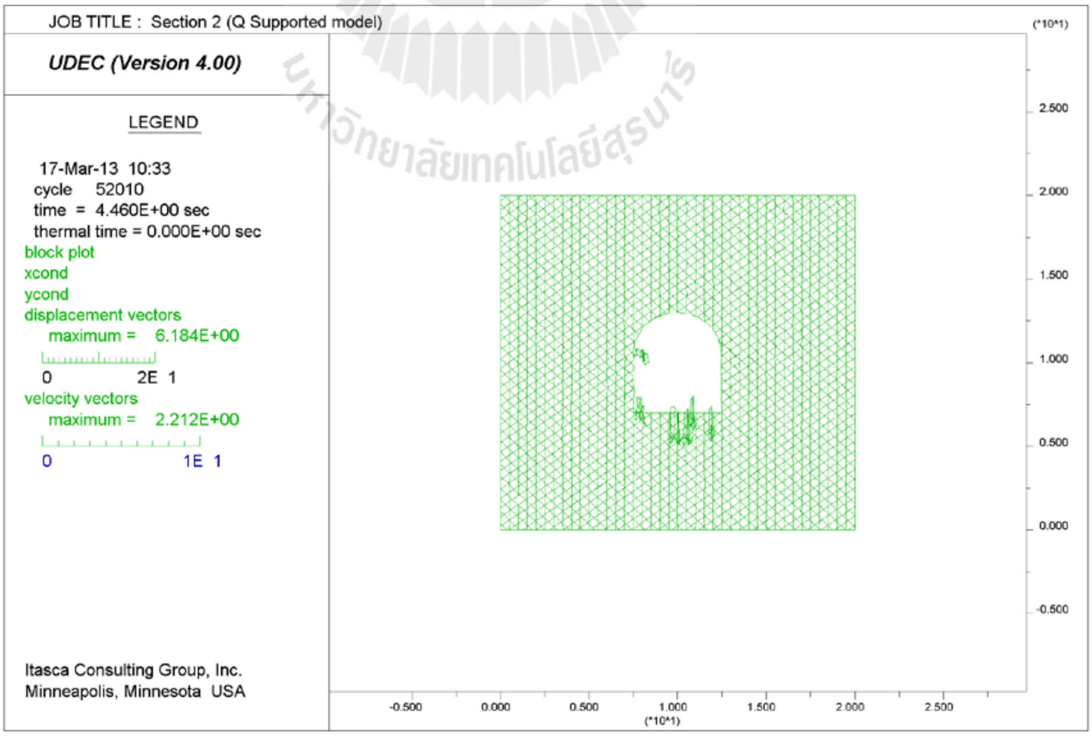


Figure 6.15 Maximum displacement vectors supported as Q suggested for section 2.

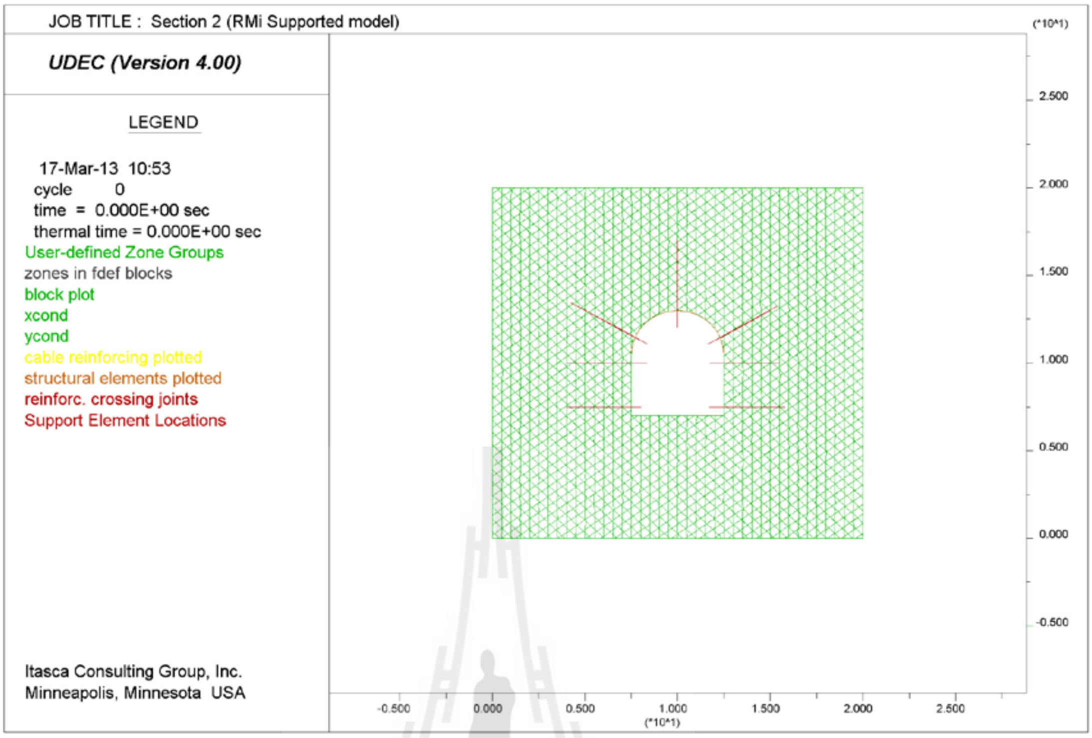


Figure 6.16 Structure adit supported as RMi suggested for section 2.

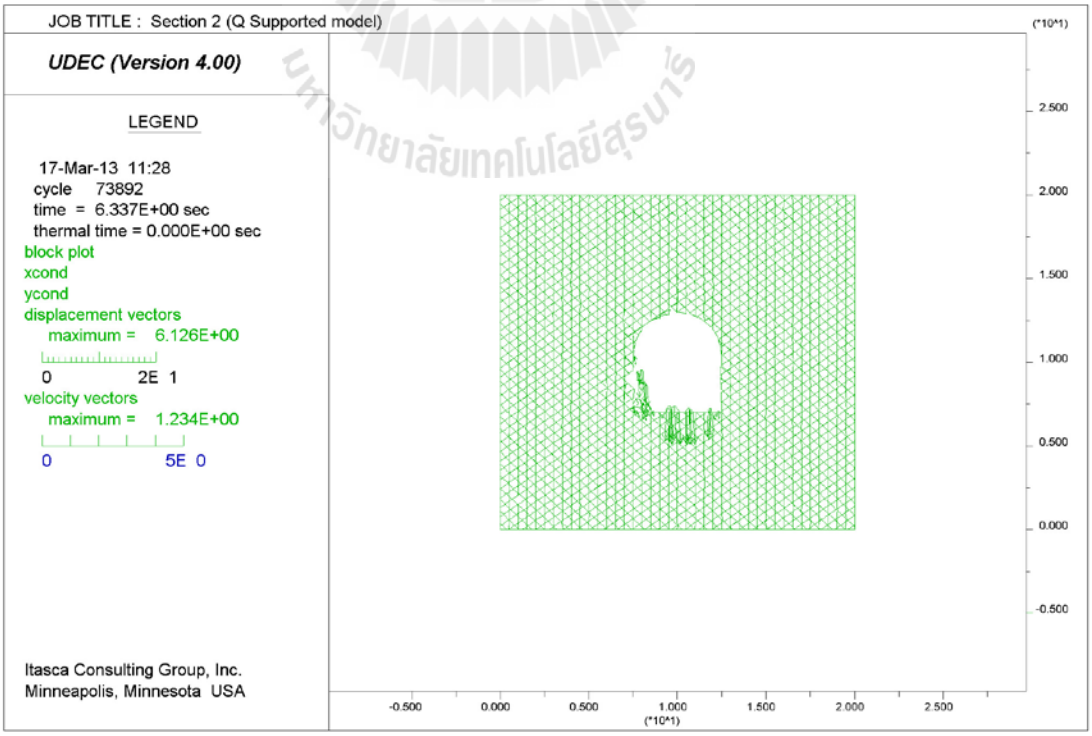


Figure 6.17 Maximum displacement vectors supported as RMi suggested for section 2.

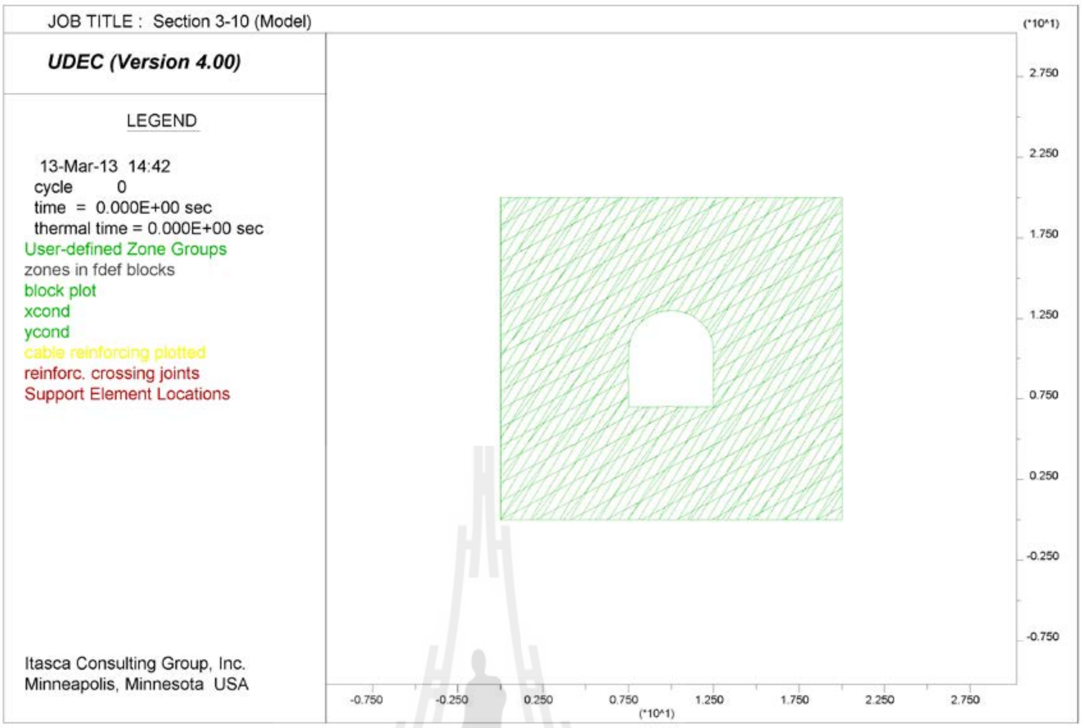


Figure 6.18 Joint sets and excavation boundary for section 3-10.

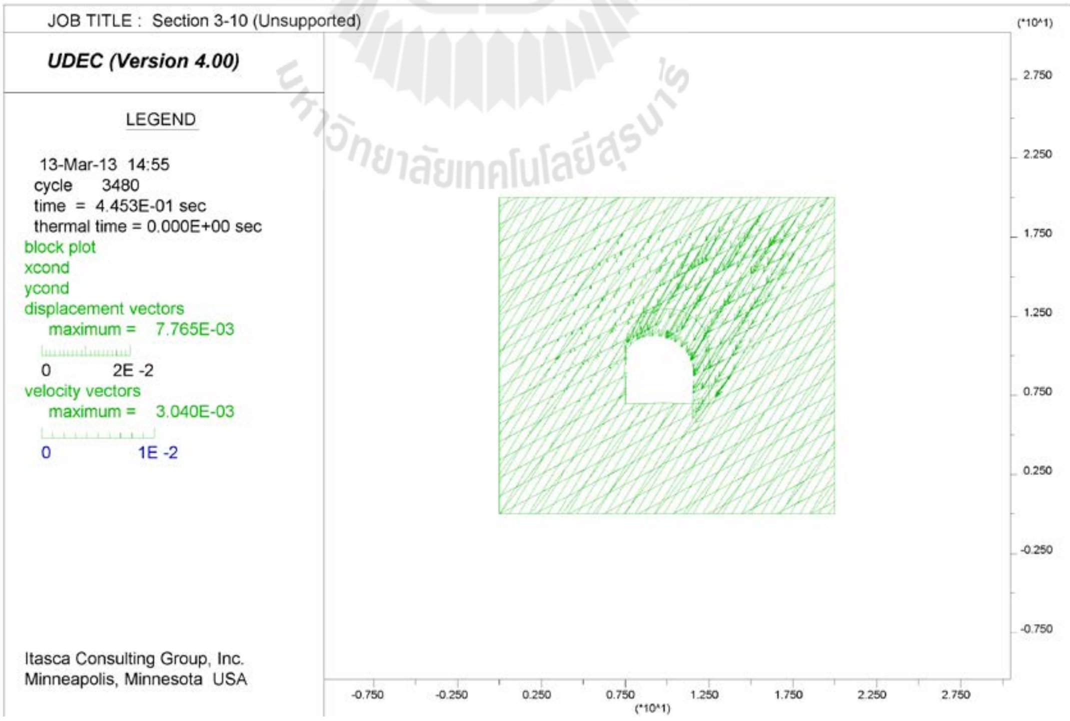


Figure 6.19 Maximum displacements vector unsupported for section 3-10.

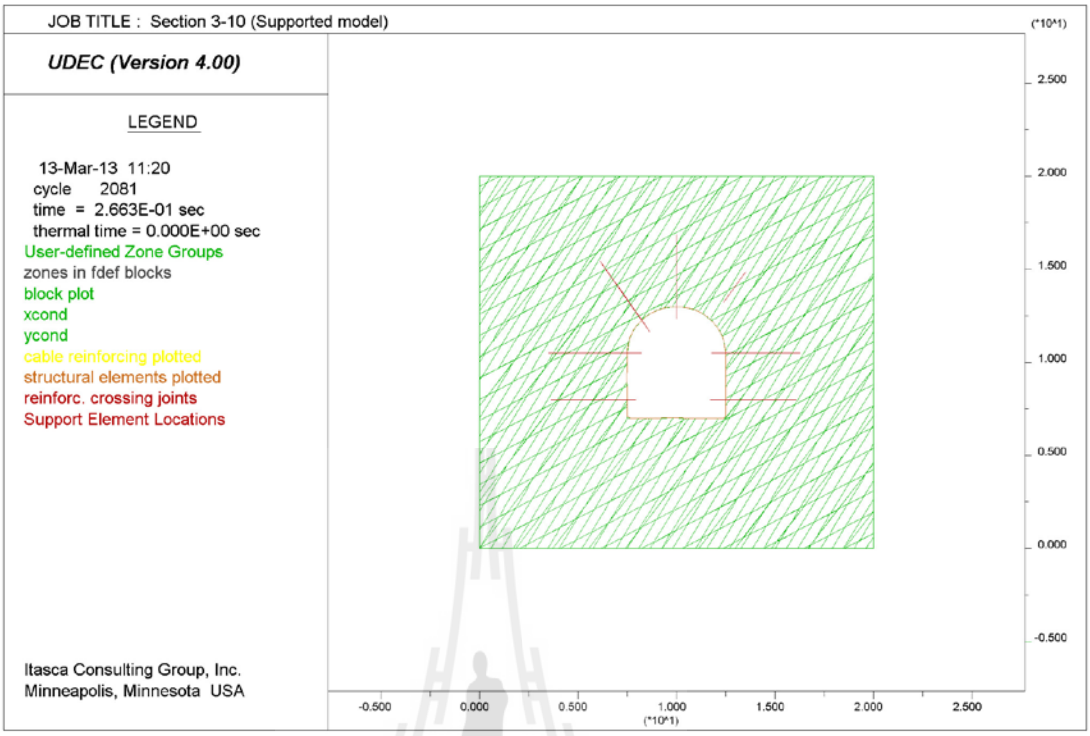


Figure 6.20 Structure adit supported as RMR suggested for section 3-10.

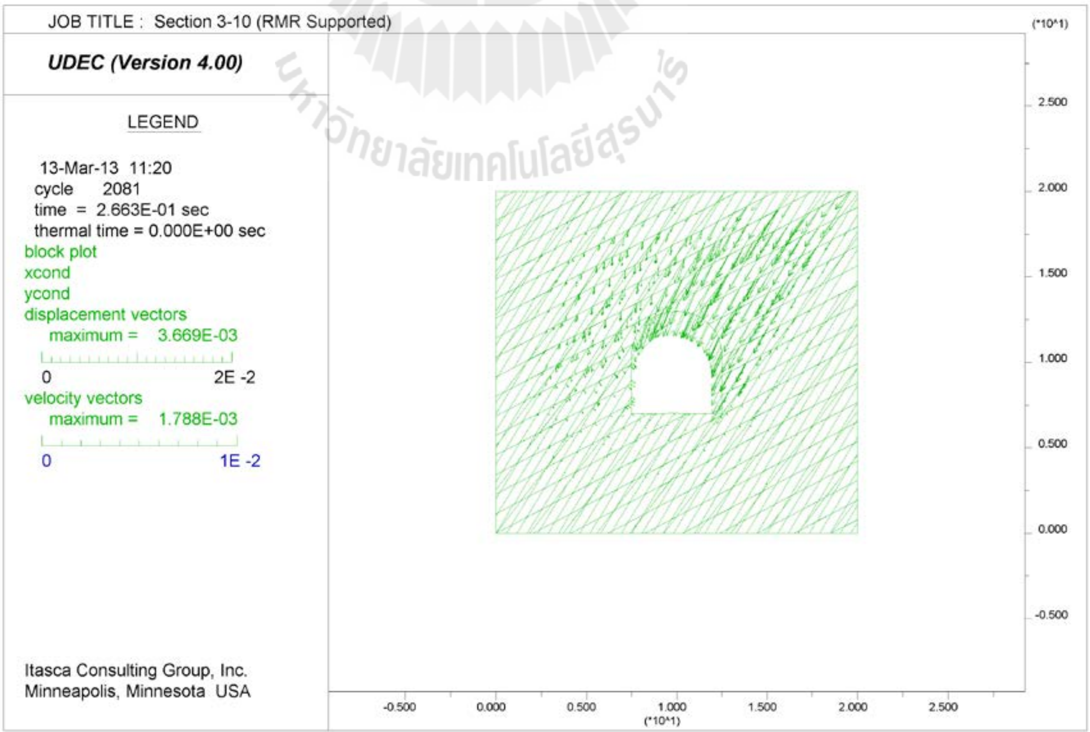


Figure 6.21 Maximum displacement vectors supported as RMR suggested for section 3-10.

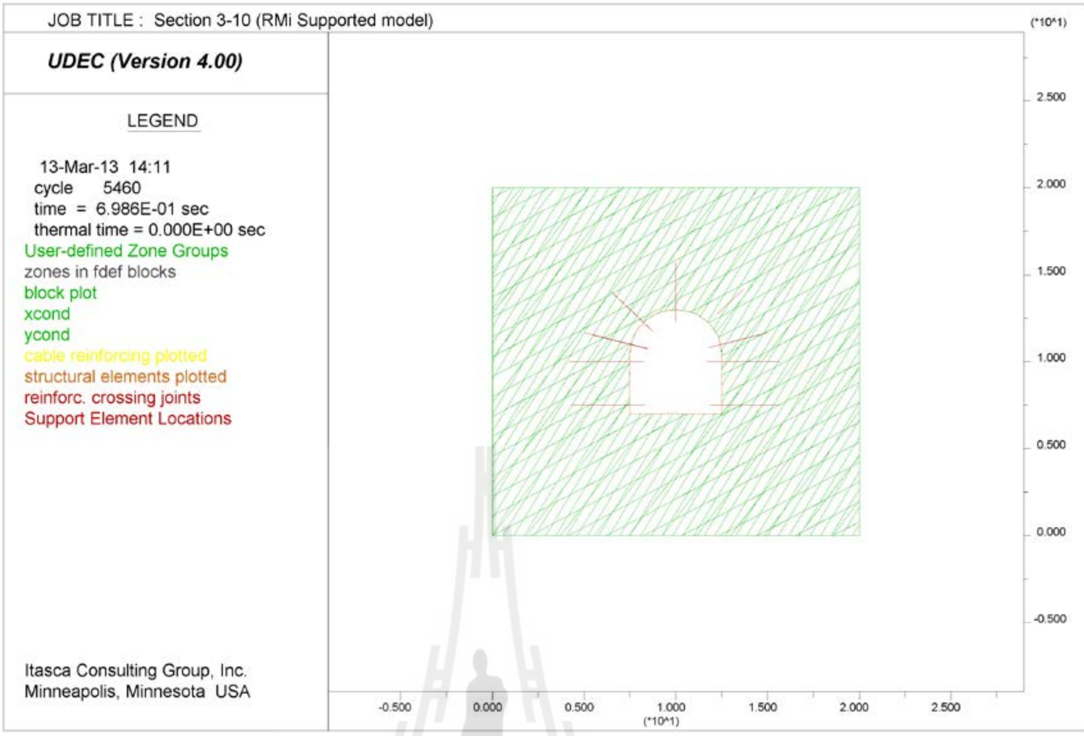


Figure 6.22 Structure adit supported as RMi suggested for section 3-10.

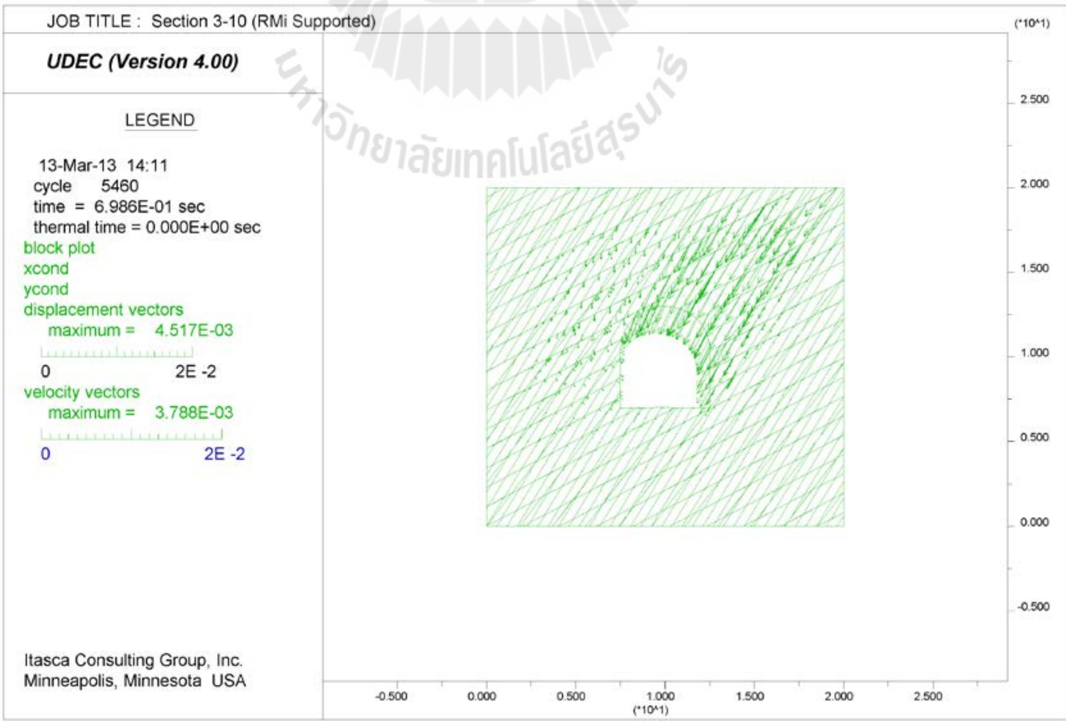


Figure 6.23 Maximum displacement vectors supported as RMi suggested for section 3-10.

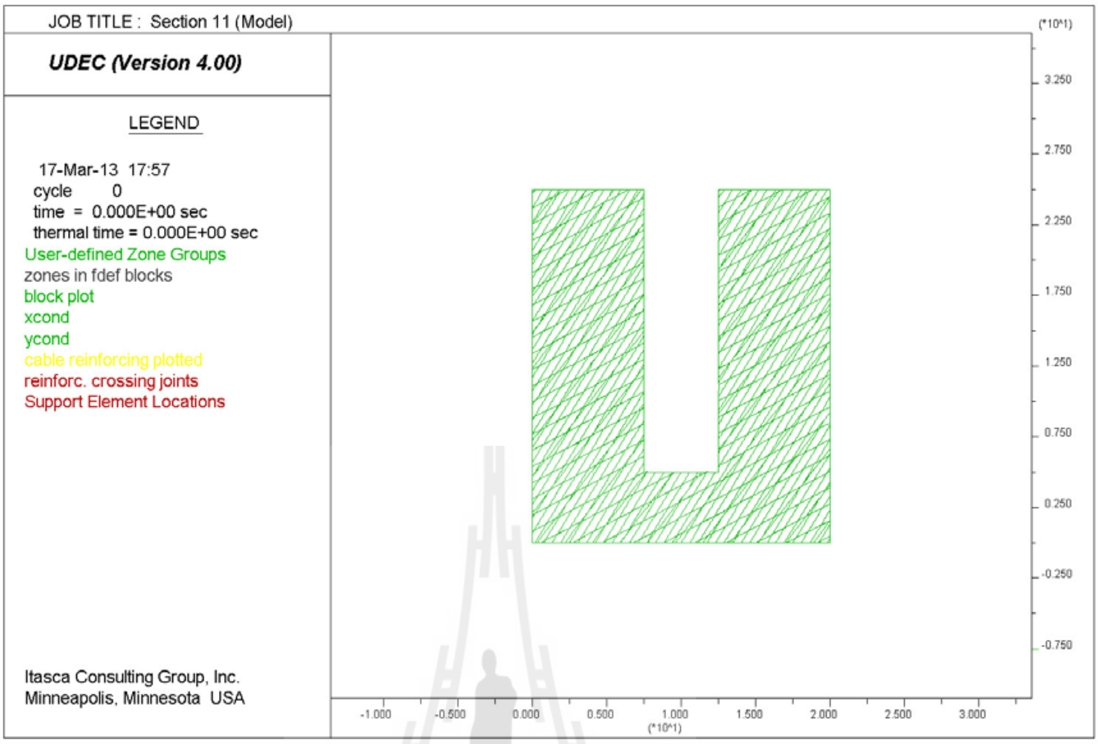


Figure 6.24 Joint sets and excavation boundary for section 11.

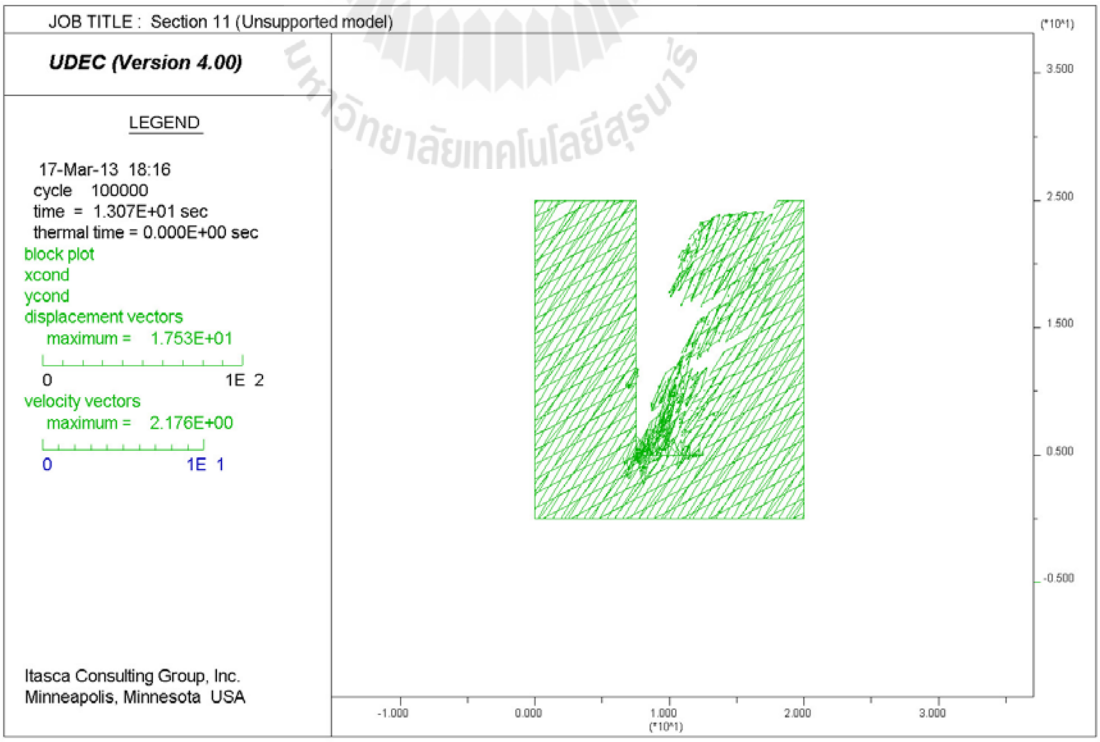


Figure 6.25 Maximum displacement vectors unsupported for section 11.

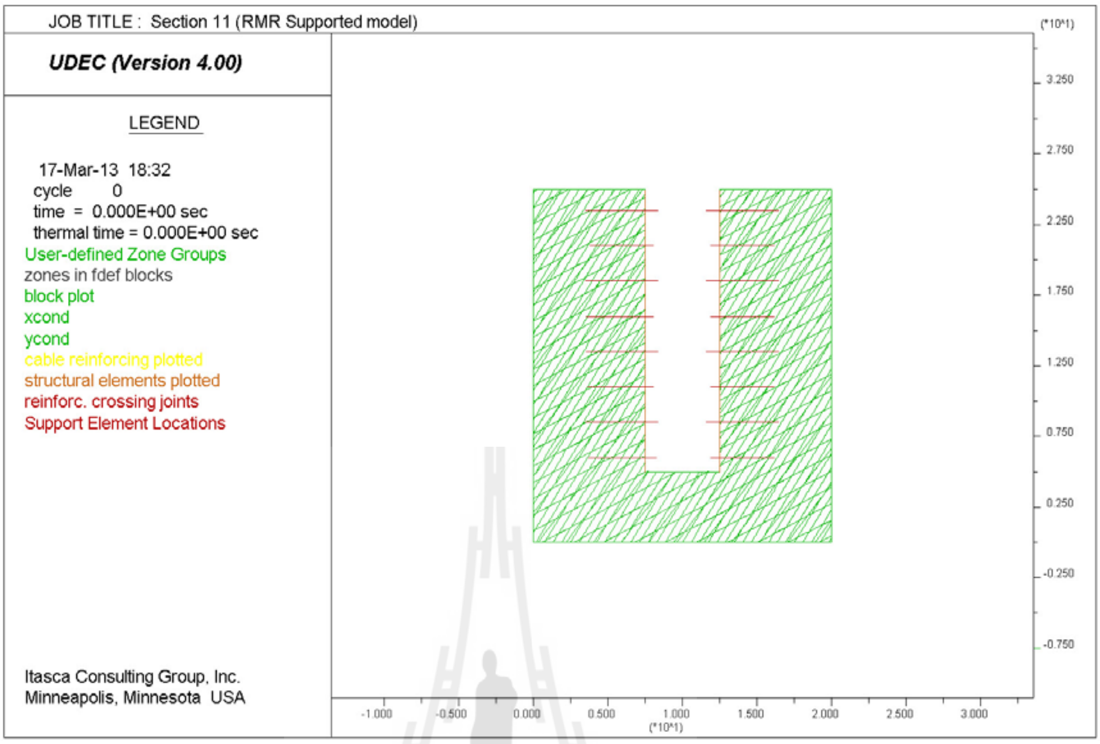


Figure 6.26 Structure shaft supported as RMR suggested for section 11.

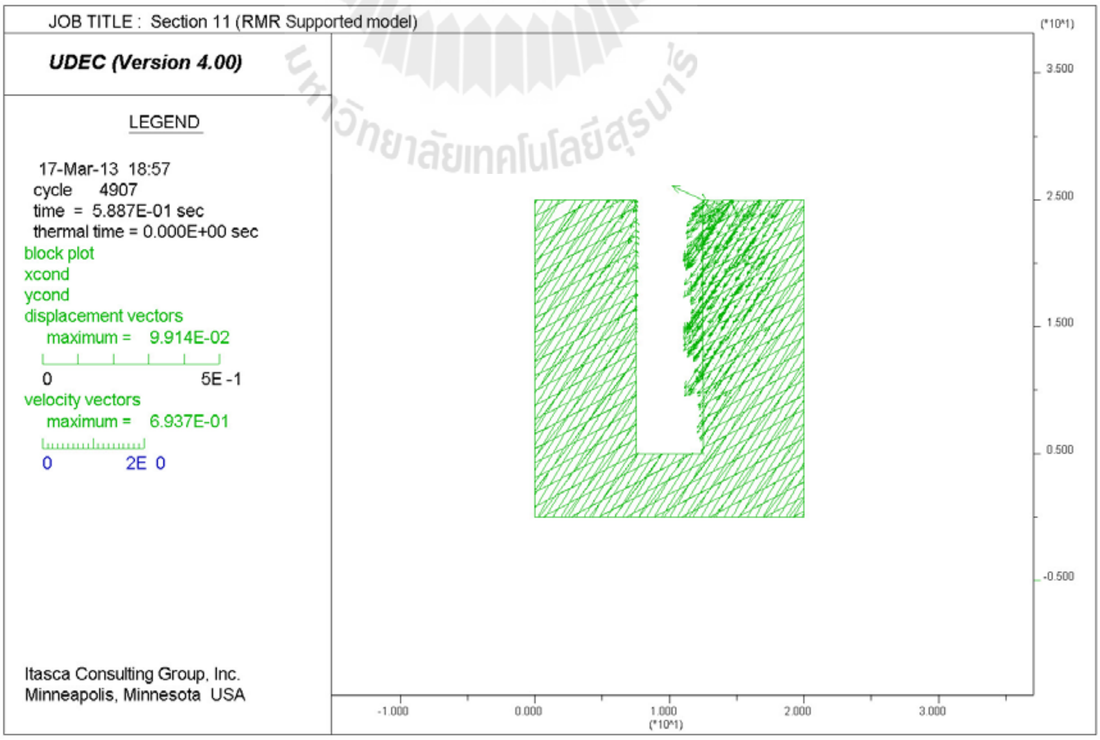


Figure 6.27 Maximum displacement vectors supported as RMR suggested for section 11.

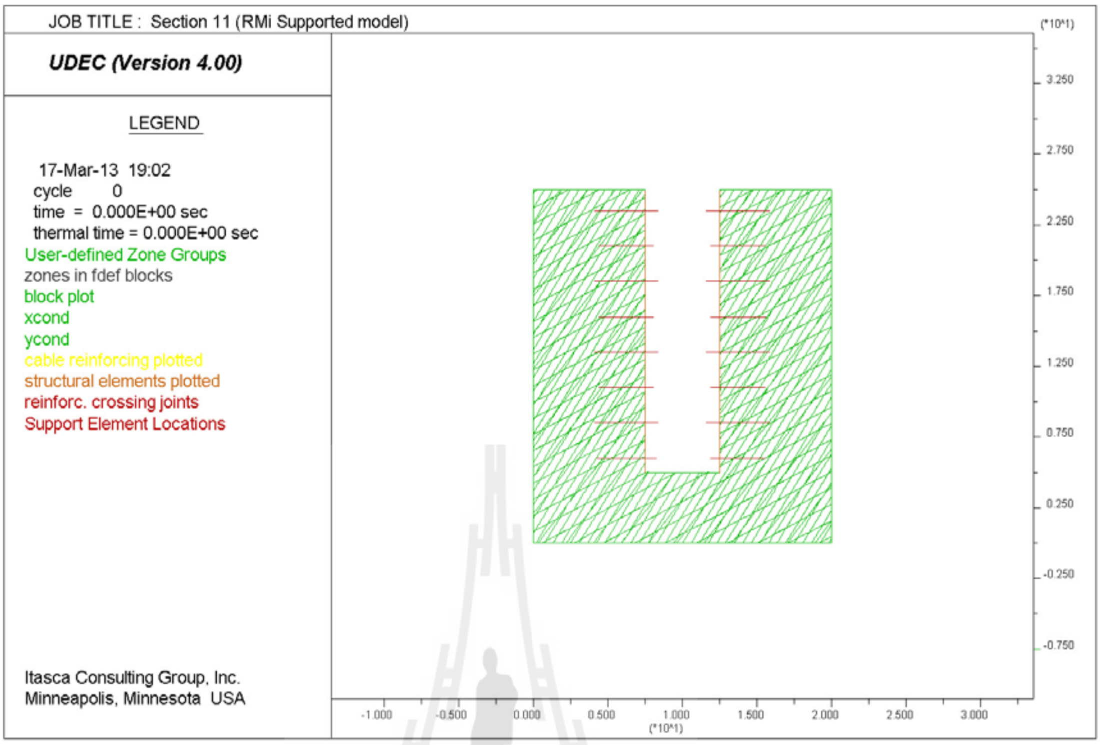


Figure 6.28 Structure shaft supported as RMi suggested for section 11.

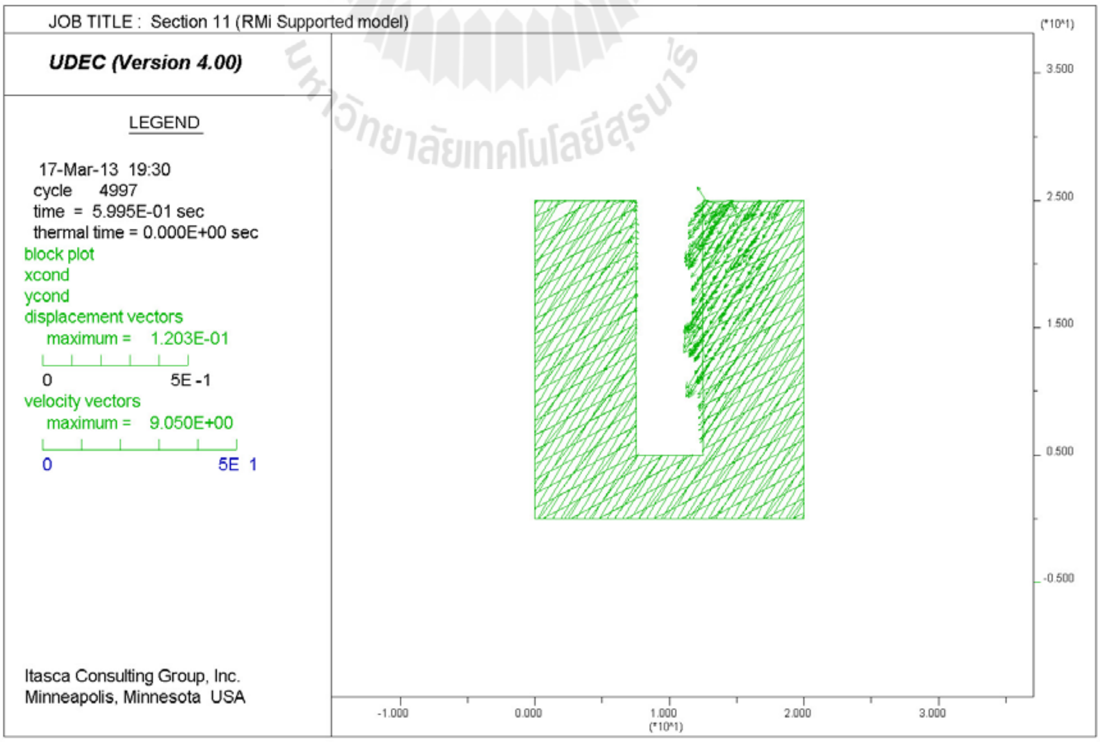


Figure 6.29 Maximum displacement vectors supported as RMi suggested for section 11.

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 this study, the empirical methods and numerical method are used to assess the reliable support system and the comparison is made each other. The comparison of the empirical is shown in Table 6.5

The rock mass rating system suggests longer rock bolts than other empirical methods. The support systems suggested by NGI tunneling quality index (Q system) has thinner shotcrete thickness than other empirical methods. The rock mass index (RMi) suggests overestimated support systems than other 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) agree reasonably well with the numerical method.

The UDEC program is used to calculate the maximum displacement vectors and the results of calculations were compared. The comparison of the support suggestions shows that the RMi gives the maximum results, RMR and Q-systems show similar results. The results are summarized in Table 6.6.

Table 6.6 Comparison of the support suggestions from empirical methods in study area.

<table>
<tr><th>Section</th><th>RMR</th><th>Q-System</th><th>RMi</th></tr>
<tr><td>1</td><td>Systematic cable bolts long 4 m, spaced 1.5-2 m in crown and wall with wire mesh in crown. Shortcrete 50-100 mm in crown and 30 mm in sides.</td><td>Bolt length 1.7-2.4 m and spacing 1.7-2.1 m. Un-reinforce shotcrete 40-100 mm.</td><td>Systematic bolt spacing 2 x 2 m for roof and systematic bolt spacing 2.5 x 2.5 m for walls. Fibre reinforced shotcrete thickness 80-100 mm for roof and 60-80 mm for walls.</td></tr>
<tr><td>2</td><td>Systematic cable bolts 4-5 m long, spaced 1-1.5 m in crown and wall with wire mesh. Shortcrete 100-150 mm in crown and 100 mm in sided. Light ribs spaced 1.5 m.</td><td>Bolt length 1.7-2.4 m and spacing 1.5-1.7 m. Fiber reinforce shotcrete 50-90 mm.</td><td>Systematic bolt spacing 3 x 3 m for roof and systematic bolt spacing 2.5 x 2.5 m for walls. Reinforced shotcrete thickness 60 mm for roof.</td></tr>
<tr><td>3</td><td rowspan="9">Locally bolts in crown, 3 m long, spaced 2.5 m with occasional wire mesh. Shortcrete 50 mm in crown where required.</td><td rowspan="9">Unsupported</td><td rowspan="9">Systematic bolt spacing 2 x 2 m for roof and systematic bolt spacing 2.5 x 2.5 m for walls. Fibre reinforced shotcrete thickness 60 mm for roof and shortcrete 50-60 mm for walls.</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td></tr>
<tr><td>6</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td></tr>
<tr><td>9</td></tr>
<tr><td>10</td></tr>
<tr><td>11</td></tr>
</table>

CHAPTER VII

DISCUSSIONS, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE STUDIES

7.1 Discussions

In this study, empirical methods are applied along with the numerical method to assess the performance of support systems for the Siam City Cement Public Company Limited (SCCC) portal and shaft with depth of about 100 m and adit is 450 m long. The RMR system considers the orientation of discontinuities and material strength, which are not directly included in the Q system. The Q system considers stresses and the joint set numbers of the rock mass, which are only indirectly considered in the RMR system. Both systems include conditions of discontinuities and groundwater. 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 with those of the Q system. The RMi system is most suitable to massive, jointed and crushed rock masses where the joints in various sets have similar properties. The GSI system is based on the visual inspection 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 adit alignment studied here are estimated by four different empirical methods and their average values are used as input parameters for the finite element analysis.

For the rock support system in the study area, it is recommended to use the RMR support system. The RMR support system suggests the longest rock bolts, with the shotcrete thickness of about 30-150 mm, and installed wire mesh and light rib. The support system results from the UDEC. It is a two-dimensional program, the three-dimensional effect of regularly spaced elements is accommodated by scaling their material properties in the out-of-plane direction. In the case of adit support design in this study area, it considers only the magnitude of displacement. The recommended supports are shown in Table 7.1

Table 7.1 Final recommended support systems for the portal adit and shaft support in study area.

<table>
<tr><th>Section</th><th>Rock Bolt</th><th>Shotcrete</th><th>Steel Set</th></tr>
<tr><td>1</td><td>Systematic cable bolts long 4 m, spaced 1.5-2 m in crown and wall with wire mesh in crown.</td><td>50-100 mm in crown and 30 mm in sides.</td><td>None</td></tr>
<tr><td>2</td><td>Systematic cable bolts 4-5 m long, spaced 1-1.5 m in crown and wall with wire mesh.</td><td>100-150 mm in crown and 100 mm in sided.</td><td>Light ribs spaced 1.5 m.</td></tr>
<tr><td>3</td><td rowspan="9">Locally bolts in crown, 3 m long, spaced 2.5 m with occasional wire mesh.</td><td rowspan="9">50 mm in crown where required.</td><td rowspan="9">None</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td></tr>
<tr><td>6</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td></tr>
<tr><td>9</td></tr>
<tr><td>10</td></tr>
<tr><td>11</td></tr>
</table>

7.2 Conclusions

Rock masses along the study adit alignment and shaft are characterized by means of rock mass classification systems based on the field mapping, vertical borehole data, engineering geological observations and laboratory test results. According to the results acquired from the rock mass characterizations and stability analysis. The empirical methods, rock mass classification systems, are also employed to estimate support requirements and required support capacities for the underground raw material access adit. The numerical models are developed for using with the Universal Distinct Element Code (UDEC) to determine the displacements around the opening to evaluate the performance of the support system recommended by the empirical methods.

The strength parameters required for the UDEC 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). The support components used here are cable bolt, rock bolts, shotcrete, steel rib, and the special support as proposed by the empirical methods. The properties of support elements including length, pattern of bolts, thickness of shotcrete and the steel rib spacing are proposed by RMR system and Q system. The distinct element analysis is 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 displacements are reduced significantly in the numerical analysis. These results indicate that the recommended applied support systems are adequate to obtain underground openings stability. They also prove that the empirical methods reasonably agree with the numerical method.

In many underground openings support designs, empirical methods are widely used due to their simplicity, however, they fail to predict interaction between the surrounding rock mass and the supporting system. Based on the findings here, it can be postulated that empirical methods should be applied together with numerical method for the safe underground openings support design. A great deal of judgment may be needed in the application of all types of rock mass classification systems in the 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 verified by comparing predictions of the rock mass quality with the actual measurements carried out during construction.

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APPENDIX A

LABORATORY TESTING

APPENDIX B1: UNIAXIAL COMPRESSIVE STRENGTH TEST (ASTM D7012-04)

Figure B1-1: UCS Machine

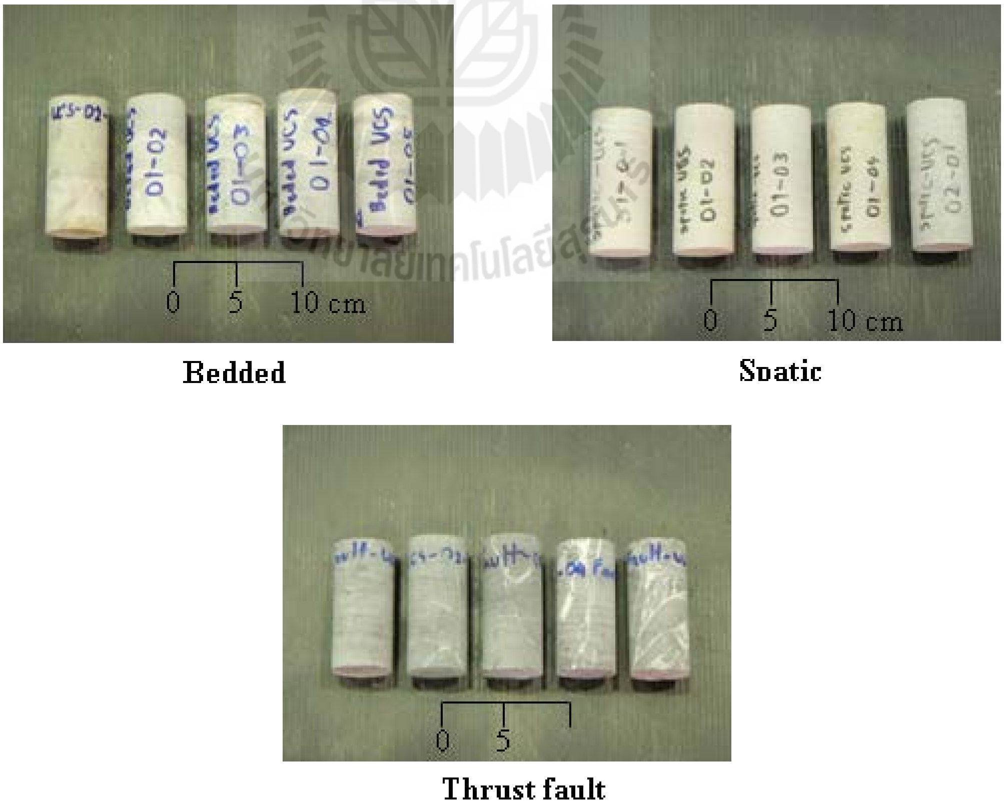


Figure B1-2: Rock Samples

Table B1-1: Testing Results of Spatic limestone

Sample No.	Diameter (mm)	Length (mm)	Weight (g)	Density (g/cc)	σ_c (MPa)	E (GPa)
Spatic(UCS-01-01)	53.90	126.87	766.96	2.65	46.0	7.2
Spatic(UCS-01-02)	53.91	129.95	786.52	2.65	48.2	10.1
Spatic(UCS-01-03)	53.91	127.55	775.56	2.67	67.9	9.9
Spatic(UCS-01-04)	53.85	127.01	772.94	2.67	57.1	12.4
Spatic(UCS-02-01)	53.89	131.27	807.73	2.70	78.9	14.1
			Average	2.67	59.6	10.7
			S.D.	0.02	13.8	2.6

Table B1-2: Testing Results of Bedded limestone

Sample No.	Diameter (mm)	Length (mm)	Weight (g)	Density (g/cc)	σ_c (MPa)	E (GPa)
Bedded(UCS-01-01)	53.93	116.64	713.39	2.68	43.8	6.7
Bedded(UCS-01-02)	53.88	120.25	732.74	2.67	30.7	4.1
Bedded(UCS-01-03)	53.81	118.99	726.25	2.69	30.7	5.3
Bedded(UCS-01-05)	53.98	120.26	736.90	2.68	37.6	8.1
Bedded(UCS-02-07)	53.61	121.23	733.98	2.68	44.3	8.2
			Average	2.68	37.4	6.5
			S.D.	0.00	6.6	1.8

Table B1-3: Testing Results of Thrust fault

Sample No.	Diameter (mm)	Length (mm)	Weight (g)	Density (g/cc)	σ_c (MPa)	E (GPa)
Fault(UCS-02-04)	53.99	119.37	729.29	2.67	63.3	7.7
Fault(UCS-02-05)	53.91	117.40	720.49	2.69	50.4	6.1
Fault(UCS-01-08)	53.77	121.40	727.40	2.64	68.2	10.8
Fault(UCS-01-07)	53.77	122.69	749.05	2.69	83.5	10.7
Fault(UCS-01-08)	53.77	124.80	733.61	2.59	68.2	9.4
			Average	2.66	66.7	8.9
			S.D.	0.04	11.9	2.0

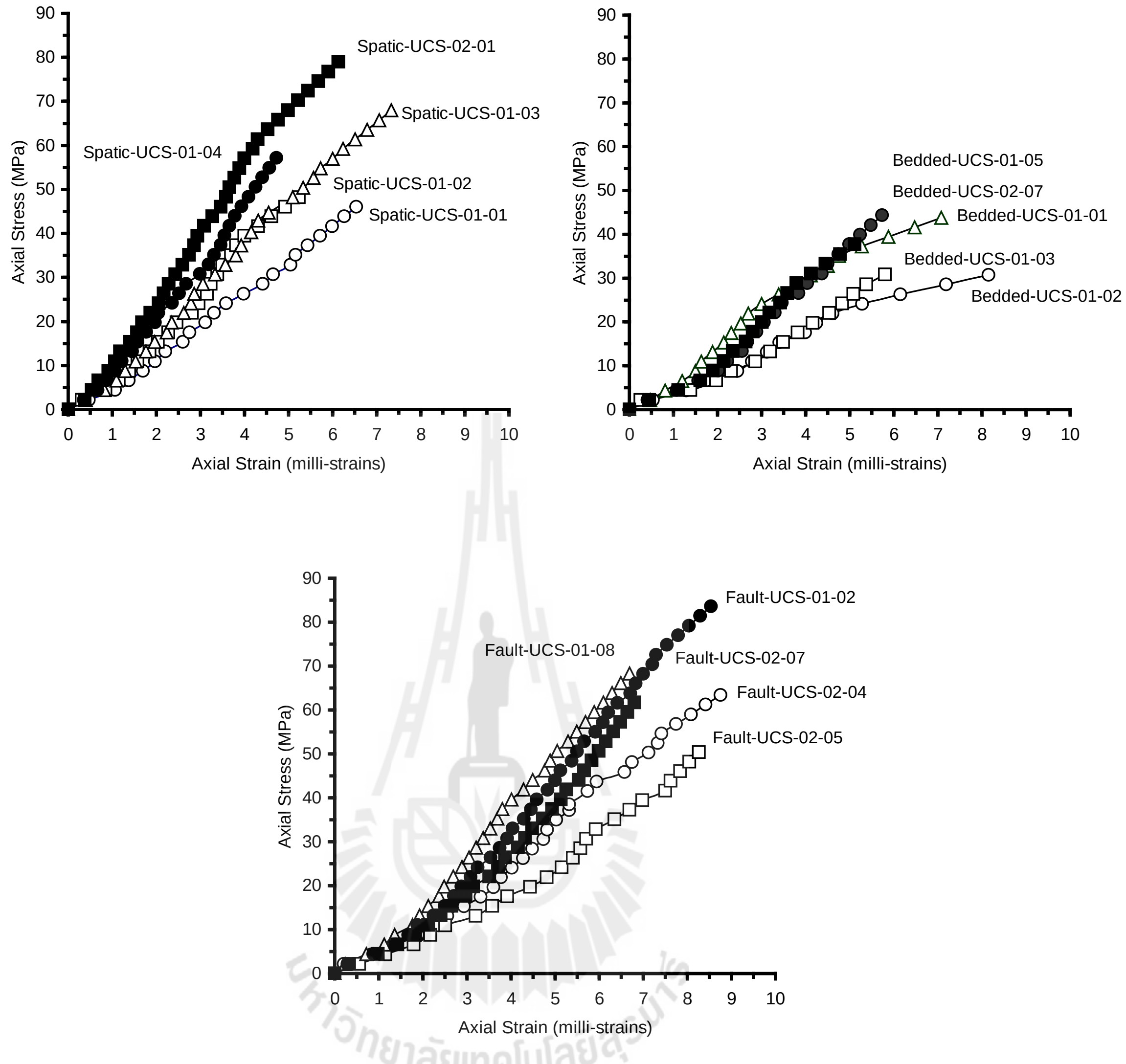


Figure B1-3: Stress-Strain plot of testing results

Conclusions:

- Uniaxial compressive strengths of Spatic limestone is 59.6±13.8 MPa, Bedded limestone is 37.4±6.6 MPa and Thrust fault is 66.7±11.9 MPa.
- Young's modulus of Spatic limestone is 10.9±2.6 GPa, Bedded limestone is 6.5±1.8 GPa and Thrust fault is 8.9±2.0 GPa .
- Poisson ratio of Spatic limestone is 0.27, Bedded limestone is 0.29 and Thrust fault is 0.25.

APPENDIX B2: DIRECT SHEAR TEST (ASTM D5607-08)

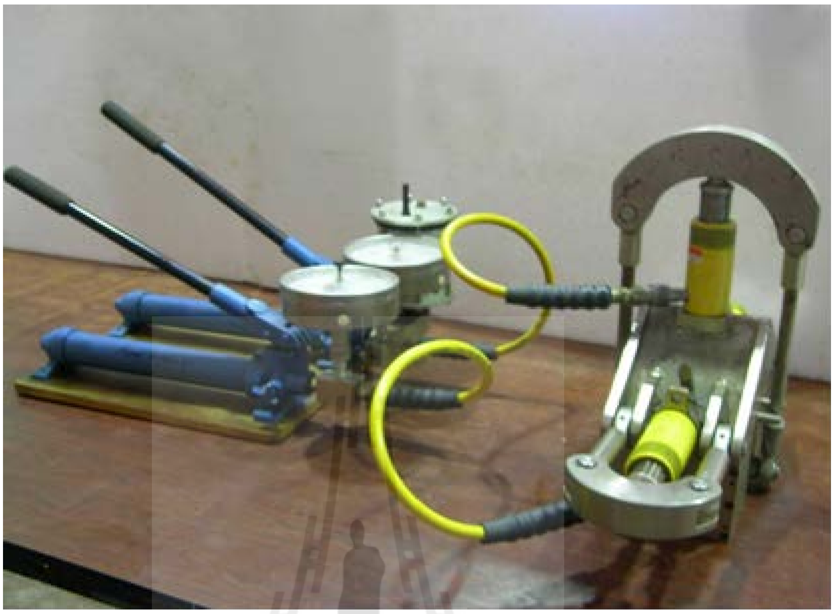


Figure B2-1: Direct Shear Machine

Spatic

Bedded limestone

Thrust

Figure B2-2: Rock Samples

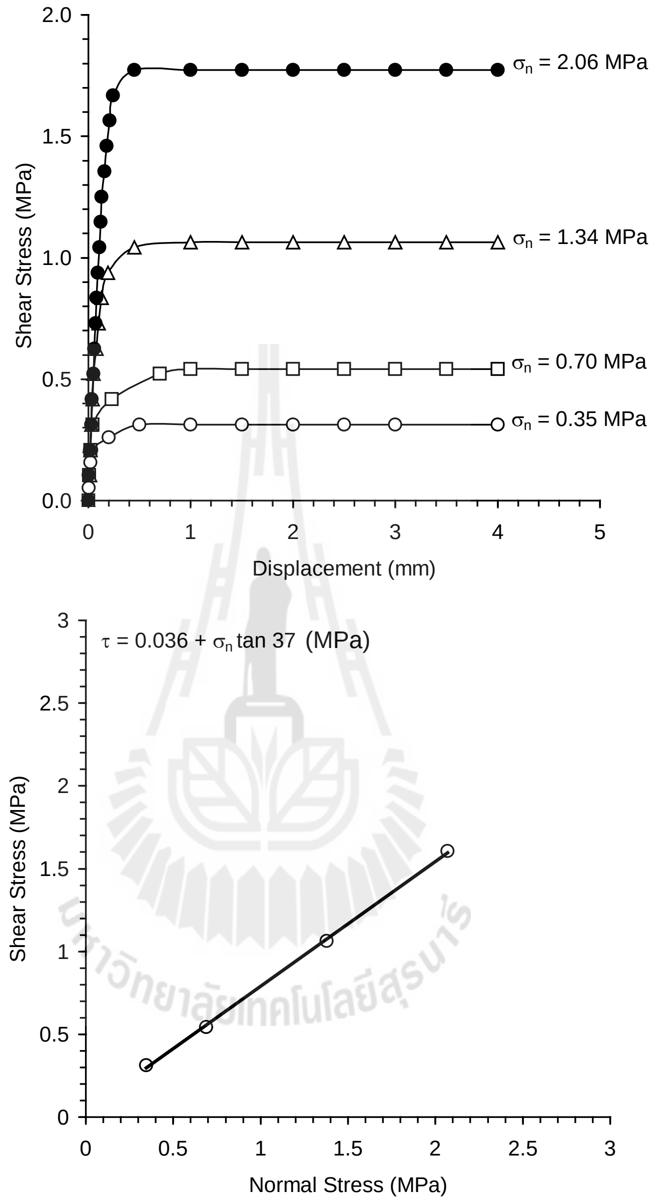
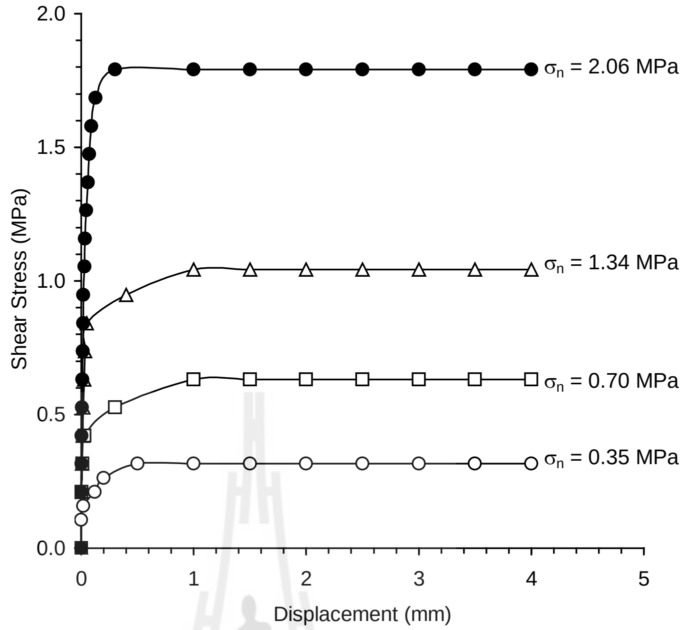


Figure B2-3: Testing Results of Bedded limestone



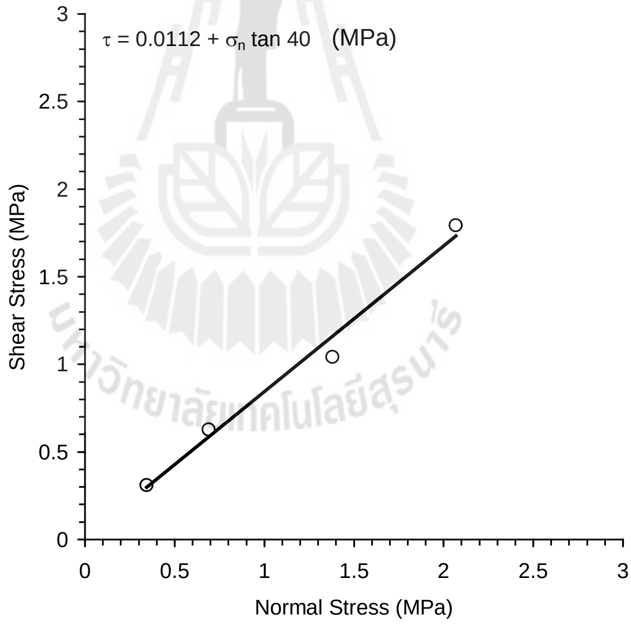


Figure B2-4: Testing Results of Thrust fault

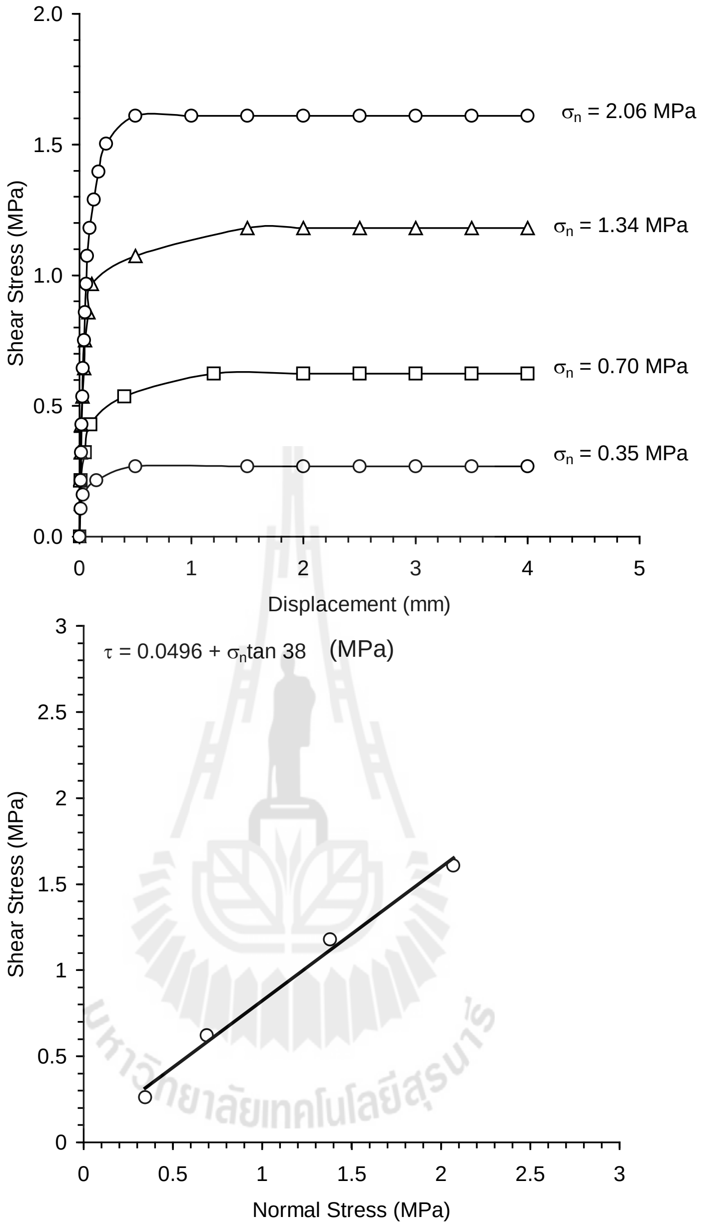


Figure B2-5: Testing Results of Spatic limestone

Conclusions:

- Friction angle of Spatic limestone is 38 degrees, Bedded limestone is 37 degrees and Thrust fault is 40°.
- Cohesion of Spatic limestone is 0.049 MPa, Bedded limestone is 0.036 MPa and Thrust fault is 0.012 MPa.

APPENDIX B

ROCK MASS CLASSIFICATION

Table A.1 Geomechanics classification parameters and their ratings

(After Bieniawski, 1998).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)						
Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS						
Rating		100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number		I	II	III	IV	V
Description		Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
D. MEANING OF ROCK CLASSES						
Class number		I	II	III	IV	V
Average stand-up time		20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)		> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)		> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions					
Discontinuity length (persistence)	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m
Rating	6	4	2	1	0
Separation (aperture)	None	< 0.1 mm	0.1 - 1.0 mm	1 - 5 mm	> 5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Ratings	6	5	3	1	0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**			
Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

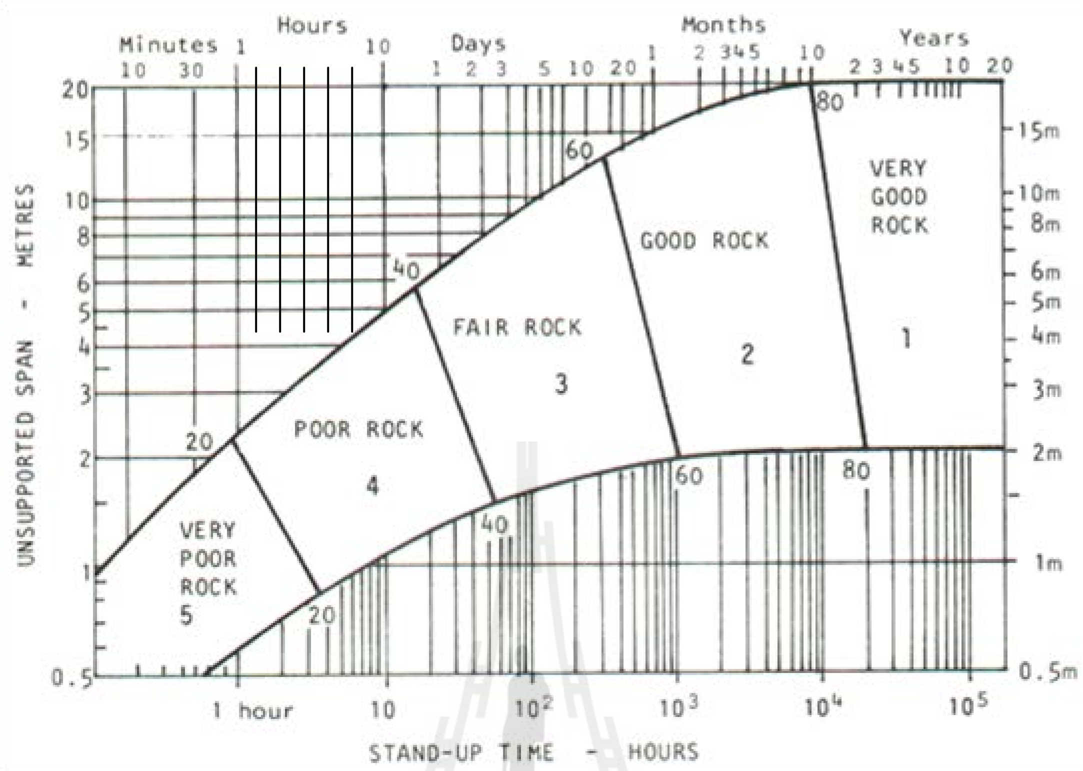


Figure A.1 Relationship between the stand-up time of an unsupported underground excavation span and the CSIR Geomechanics Classification Proposed by Bieniawski (1976).

Table A.2 Guidelines for excavation and support of 10 m span rock tunnel in accordance with the RMR system (After Bieniawski, 1989).

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock *RMR*: 81-100	Full face, 3 m advance.	Generally no support required except spot bolting.		
II - Good rock *RMR*: 61-80	Full face , 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock *RMR*: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	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.
IV - Poor rock *RMR*: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	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.
V – Very poor rock *RMR*: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Table A.3 Classification of individual parameters used in the NGI tunneling quality index (After Barton et al., 1974)

DESCRIPTION	VALUE		NOTES
1. ROCK QUALITY DESIGNATION	*RQD*		
A. Very poor	0 - 25		1. Where *RQD* is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate *Q*.
B. Poor	25 - 50		
C. Fair	50 - 75		
D. Good	75 - 90		2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.
E. Excellent	90 - 100		
2. JOINT SET NUMBER	J_n		
A. Massive, no or few joints	0.5 - 1.0		
B. One joint set	2		
C. One joint set plus random	3		
D. Two joint sets	4		
E. Two joint sets plus random	6		
F. Three joint sets	9		1. For intersections use ($3.0 \times J_n$)
G. Three joint sets plus random	12		
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15		2. For portals use ($2.0 \times J_n$)
J. Crushed rock, earthlike	20		
3. JOINT ROUGHNESS NUMBER	J_r		
a. Rock wall contact			
b. Rock wall contact before 10 cm shear			
A. Discontinuous joints	4		
B. Rough and irregular, undulating	3		
C. Smooth undulating	2		
D. Slickensided undulating	1.5		1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
E. Rough or irregular, planar	1.5		
F. Smooth, planar	1.0		
G. Slickensided, planar	0.5		2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength.
c. No rock wall contact when sheared			
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)		
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)		
4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
a. Rock wall contact			
A. Tightly healed, hard, non-softening, impermeable filling	0.75		1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25 - 35	
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30	
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25	
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)	4.0	8 - 16	

Table A.3 (continuity)

4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24	
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12	
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay	8.0		
M. conditions)	8.0 - 12.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0 - 13.0		
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
6. STRESS REDUCTION FACTOR		*SRF*	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)		10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)		5.0	
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)		2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)		5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)		2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)		5.0	

Table A.3 (continuity)

DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			*SRF*	
b. Competent rock, rock stress problems				
	σ_c/σ_1	σ_t/σ_1		2. For strongly anisotropic virgin stress field
H. Low stress, near surface	> 200	> 13	2.5	(if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c
J. Medium stress	200 - 10	13 - 0.66	1.0	to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When σ_1/σ_3 > 10,
K. High stress, very tight structure	10 - 5	0.66 - 0.33	0.5 - 2	reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where
(usually favourable to stability, may				σ_c = unconfined compressive strength, and
be unfavourable to wall stability)				σ_t = tensile strength (point load) and σ_1 and
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	σ_3 are the major and minor principal stresses.
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of
c. Squeezing rock, plastic flow of incompetent rock				crown below surface is less than span width.
under influence of high rock pressure				Suggest *SRF* increase from 2.5 to 5 for such
N. Mild squeezing rock pressure			5 - 10	cases (see H).
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

Table A.4 The relationship between ESR value and excavation category (Barton et al., 1974).

Excavation category		*ESR*
A	Temporary mine openings.	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

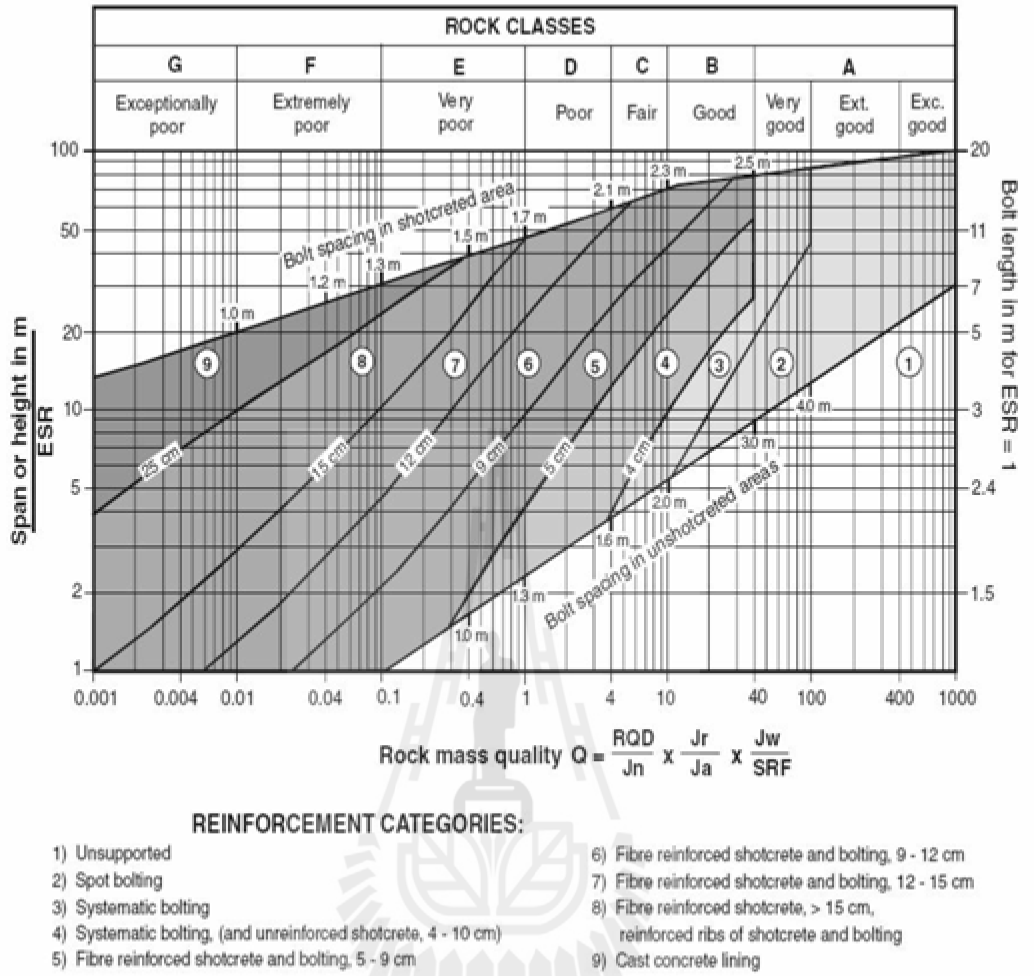


Figure A.2 Estimated support categories based on the tunnelling quality index Q (After Grimstad and Barton, 1993, reproduced from Palmstrom and Broch, 2006).

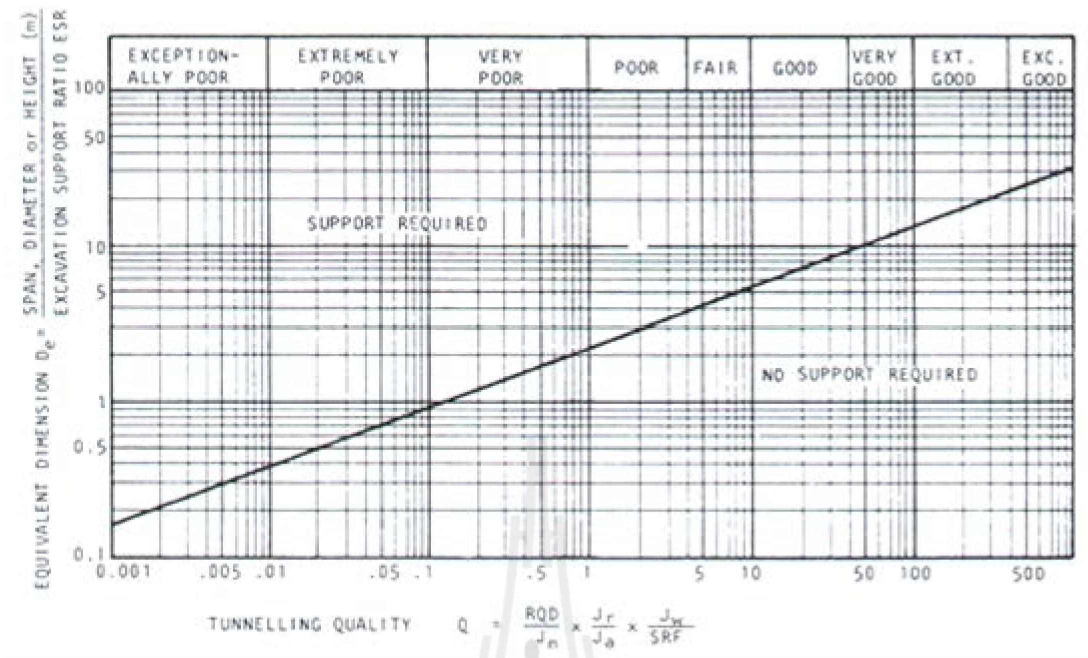


Figure A.3 Relationship between the maximum equivalent dimension D_e of an unsupported underground excavation and the NGI tunneling quality index Q (After Barton, Lien and Lunde, 1974)

Table A.5 The input parameters table of RMi-cal, versoin 2 to calculate RMi index (Palmstrom, 2002) (www.rockmass.net).

Calculating the Rock Mass index, RMi

and factors in the Hoek-Brown failure criterion for rock masses

INPUT DATA

Project: .... Location: ....

Tunnel: .... Type of rock: ....

Estimate made by:

Uniaxial compressive strength of intact rock (MPa)		σ_c =	
Hoek-Brown rock material constant	(needed for calculating the H-B factors)	m_i =	
Joint roughness	smooth & planar = 1; smooth & undulating = 2; rough & planar = 2	factor jR =	
Joint alteration	fresh = 1; coating of calcite = 3; coating of clay = 3; filling of clay = 6 - 10	factor jA =	
Joint size	partings = 6; short joints = 2 - 4; medium joints = 1 - 2	factor jL =	
Joint termination	continuous = 1; discontinuous = 2		

Block size, joint density, or degree of jointing	(use only input of ONE of these parameters)	Block volume (m^3) (note the input in m3)	Vb =	
		Rock quality designation	RQD =	
		Volumetric joint count	Jv =	
		Average joint spacing *) (m)	Sa =	
Block shape		cubical = 27; slightly long or flat = 40; long or flat = 75; very long or flat = 250	β =	
For information only	(must not be filled)	Number of joint sets in the actual location	nj =	
		Occurrence of seam(s) (= clay-filled joint with thickness < 1m)		
		Weakness zone or fault (= zone with thickness > 1m)		

*) Average spacing is Sa = (S1 + S2 + S3 + ...)/n (S1, S2, S3 .. are spacings of each joint set; n = number of joint sets)

CALCULATIONS

Rock mass index	**RMi =**	**input?**
	classification of RMi =	
Factors in the Hoek-Brown failure criterion for rock masses	*s* =	
	m_b undisturbed =	
	m_b disturbed =	

Estimate made: March 30, 2010

RMi-calc, version 2

Reference: Arild Palmstrom, www.rockmass.net

Description of the location:

Table A.6 The input parameters table and output table for support design of RMi support, versoin 3.1 (Palmstrom, 2008) (www.rockmass.net).

RMi Rock Support Method for estimates in underground openings

for experienced users

INPUT PARAMETER DATA — **Project:**

Tunnel span Dt =	12	m	**Tunnel:**	
Wall height Wt =	12	m (for circular tunnels: use Dt and Wt = 0.6 of tunnel diam.)	**Location:**	
Roof inclination		° (horizontal roof is used, when not filled)	**Rock(s):**	
Tunnel shape:	1	(1 = horse-shoe; 2 = square; 3 = circular; 4 = high horse-shoe)		

Compressive strength of intact rock (MPa)		σ_c =	26.22
Rock behaviour (competent or incompetent)	(1 = brittle; 2 = ductile or deformable)		
Joint roughness	(1 = smooth & planar; 2 = smooth & undulating; 1.5 = rough & planar; 3 = rough & undul.)	jR =	2
Joint alteration	(1 = fresh; 2 = sand coating; 4 = clay coating; 8 = clay filling)	jA =	3
Joint size **)	(2 = short (0.1-1m); 1 = medium (1-10m); 0.75 = long (10-30m); 0.5 = very long)	jL =	2
Number of joint sets	(1 = one set; 1.5 = one set + random; 2 = two sets; 2.5 = two sets + random; etc.)	nj =	
Block volume (m^3)		Vb =	0.0047
Block shape factor	(27 = cubical; 36 = slightly long or flat; 75 = long or flat; 250 = very long or flat)	β =	
Orientation of main joint set (or of zone) in roof *)		Co or Co_z =	
Orientation of main joint set (or of zone) in wall *)		Co or Co_z =	
Thickness or width of **weakness zone** (m)	(to be used for 1 - 20m wide zones)	Tz =	
Occurrence of a **seam** (filled joint) < 1 m thick (m)	(to be used for < 1m thick seams)	Ts =	
Orientation of the seam in roof *)		Co_S =	
Orientation of the seam in wall *)		Co_S =	
or Measured or estimated vertical stress (MPa)		σ_c =	8.708
Relevant overburden (m)		H =	
Ratio k = horizontal stress/vertical stress		k =	1
Ground water	(1 = little or no influence on stability; 2 = moderate influence; 5 = major influence)	GW =	
*) 1 = favourable; 1.5 = fair; 2 = unfavourable; 3 = very unfavourable	**) for discontinuous joints: double the ratings shown		

RMi = 0.93

ESTIMATED ROCK SUPPORT — **In blocky ground**

Total amount of *rock support in* ***blocky ground***	**Roof support**	*Type of ground in roof =*	*blocky*
	Only rock bolts		-
	Rock bolts combined with shotcrete	*pattern =*	1 x 1
	Shotcrete; average thickness		150 - 250mm
	Fiber reinforcement recommended?		yes
	Special designed support		
	Wall support (for vertical walls)	*Type of ground in walls =*	*blocky*
	Only rock bolts		-
	Rock bolts combined with shotcrete	*pattern =*	1.25 x 1.25
	Shotcrete; average thickness		100 - 150mm
	Fiber reinforcement recommended?		yes
	Special designed support		

In continuous ground

PERMANENT ROCK SUPPORT *in* ***continuous ground***	**Roof support**	*Type of ground in roof =*	
	Only rock bolts		
	Rock bolts combined; bolt spacing		
	Shotcrete; average thickness		
	Special designed support		
	Wall support (for vertical walls)	*Type of ground in walls =*	
	Only rock bolts		
	Rock bolts combined; bolt spacing		
	Shotcrete; average thickness		
	Special designed support		

Reference: A. Palmström, www.rockmass.net, RMi support version 3.1 2008.07.18

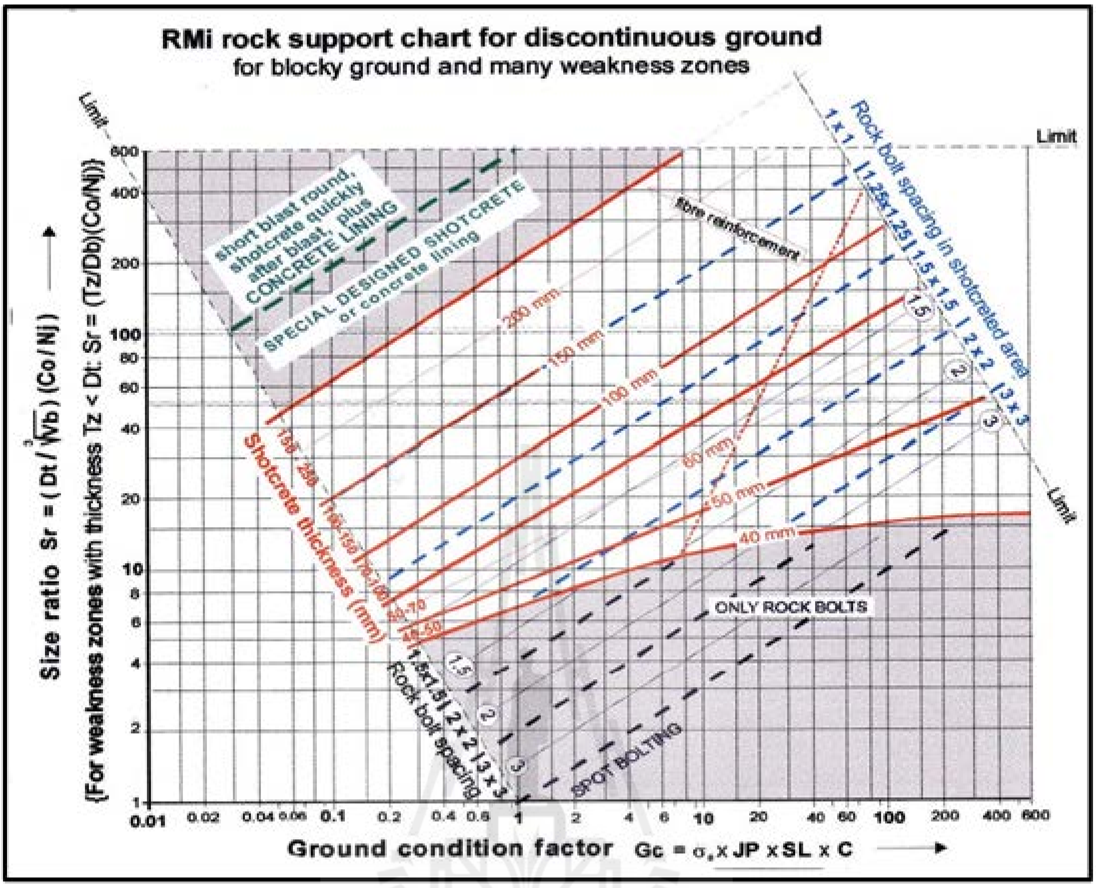


Figure A.4 RMi support chart for discontinuous ground (Palmström, 1995).

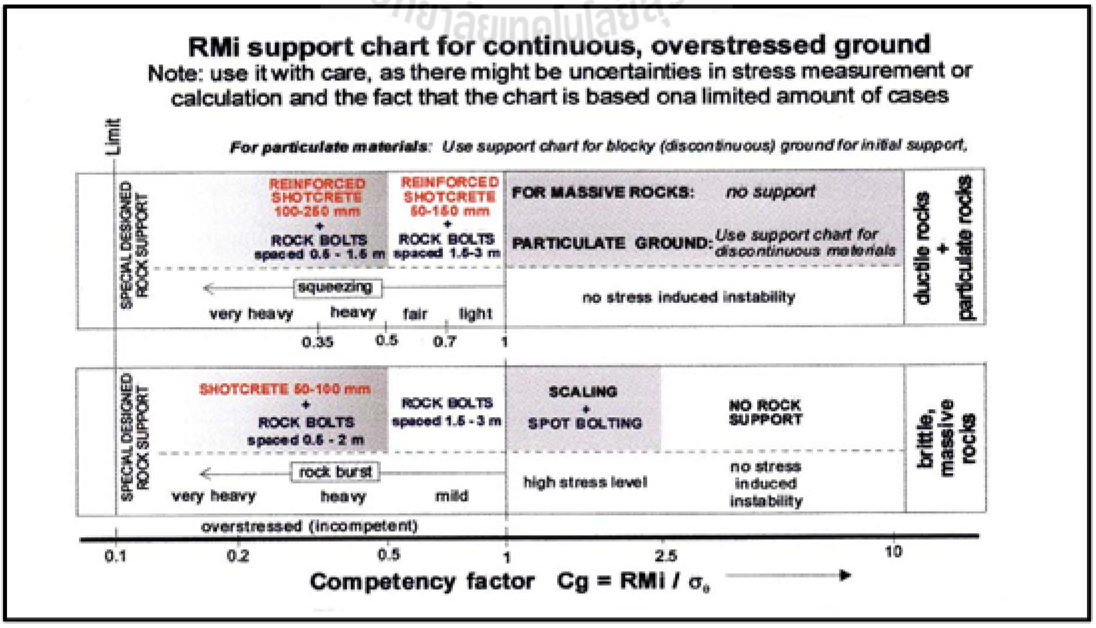


Figure A.4 RMi support chart for continuous ground (Palmström, 1995).

Table A.7 The modified quantitative GSI system (Sonmez, 2001)**.**

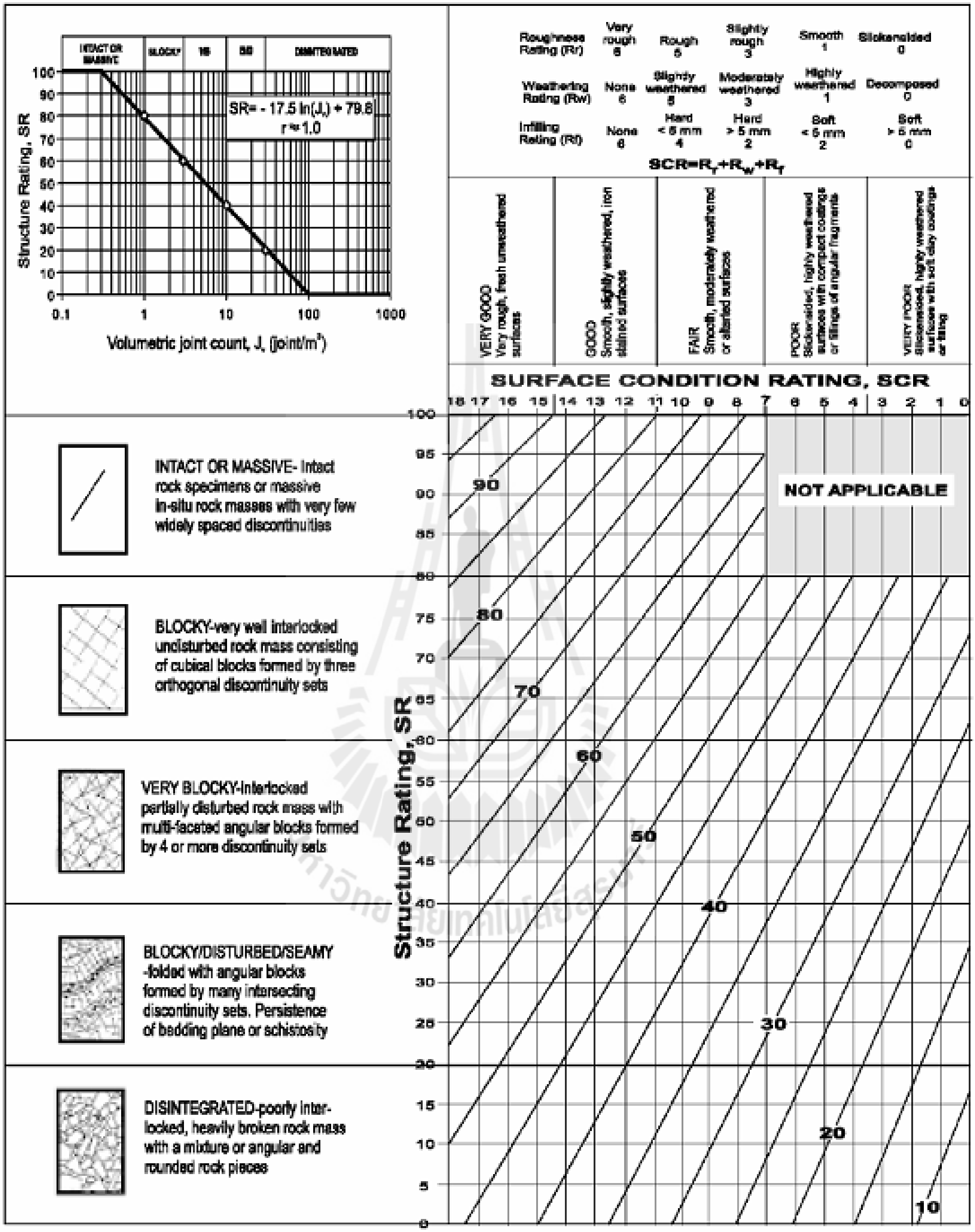


Table A.8 Field estimates of uniaxial compressive strength of intact rock (Marinos and Hoek, 2000)

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

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

Table A.9. Values of the constant m_i for intact rock (Marinos and Hoek, 2000)

Rock type	Class	Group	Texture: Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates * Breccias *	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6	Gneiss 28 ± 5	
	Foliated**			Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

* Conglomerates and breccias may present a wide range of m_i values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone, to values used for fine grained sediments (even under 10).

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

APPENDIX C

PUBLICATION

Publication

Boonbatr, A. and Fuenkajorn, K., 2012, **Design and analysis of adit for limestone quarry of Siam City Cement Public Company Limited**. Thailand Rock Mechanics Symposium. Sprindfield@Sea Resort and Spa, Cha-am, Thailand, 10-11 March 2011, pp. 229 – 238.

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Design and analysis of adit for limestone quarry of Siam City Cement Public Company Limited

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Keywords: Adit, rock support, limestone, rock mass

ABSTRACT: This paper presents the stability analysis and support design for an adit for limestone transportation of Siam City Cement Public Company Limited (SCCC), Saraburi, Thailand by using empirical approaches and numerical method. The geological evaluation of the SCCC adit 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 objective of this study is to perform stability analysis and support design for adit to access limestone quarry and limestone transportation of Siam City Cement Public Company Limited (SCCC), Saraburi province, Thailand. This access opening is intended to reduce the cost of haulages and transportation. The horseshoe sharp adit is a span of 5 m, 6 m high and 450 m long with slope of 3%. It is driven underneath the west side of limestone quarry. SCCC transportation adit project is located near limestone pre-blending pile of SCCC cement plant 3, 129 km north of Bangkok and along the highway number 2 (Figure 1). The SCCC limestone quarry has an annual production of 16.5 million tons, the largest quarry pit in Thailand. The proposed study involves geological data collection, laboratory rock mechanics testing, development of the design criteria, and mechanical stability analysis using empirical approaches and numerical methods.

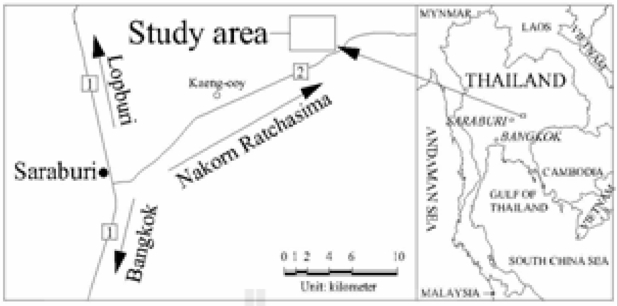

Figure 1. Location map of the project area (Scale 1: 5000).

The geologic data collection accomplished by core and borehole logging and geotechnical surface mapping. The laboratory characterization testing includes point load index strength testing, uniaxial strength testing and direct shear testing. The results are used in the analysis of the performance of the support system (e.g. rock bolts, shotcrete, and lining). The functional requirements, design parameters and constraints for the support systems are derived based on the design methodology of rock engineering. The conventional empirical approaches including rock mass rating system (RMR), NGI tunneling quality index (Q system), rock mass index (RMi), and geological strength index (GSI) are applied for adit design. The finite element method (using Phase^2) has been employed to assess the stability conditions of the limestone mass around the adit. Comparisons are made to optimize the design solutions (support system) obtained from the two approaches.

2 GEOLOGY OF THE PROJECT AREA

The rocks of the project area are mainly carbonates and siliciclastics. The different rock types reflect a marine environment for sedimentation. The fossils found show a Permo-Carboniferous age. The general trend of the site geologic structures lies in the northwest-southeast direction. Adjacent to the thrust zone on the hanging wall is a thin bed, 30 to 40 m thick of dark silicified shale which can be easily detected on site. Bedded limestone lies on top of the shale. On the footwall, limestone adjacent to the thrust zone sometimes shows heavily fracturing. Away from the thrust zone most spatic limestone mass shows well-defined discontinuities (bedding plane and joints).

3 ENGINEERING GEOLOGY

Geological data collection is carried out to classify the rock mass as accurately as possible. In order to constitute the geological model and to determine the engineering geological properties of the SCCC transportation adit grounds was performed within surface mapping and four boreholes closed to adit axis. Engineering geological descriptions of the rock masses are based on the procedures suggested by ISRM. The major trend of the joint is 130/60 (strike/dip) and the tunnel direction is N 8°E. The adit axis is divided into three different zones of rock mass, bedded limestone, thrust fault zone and spatic limestone. Each zone has different engineering geological properties and lithologic types. Total 11 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 adit axis are described in Figure 2.

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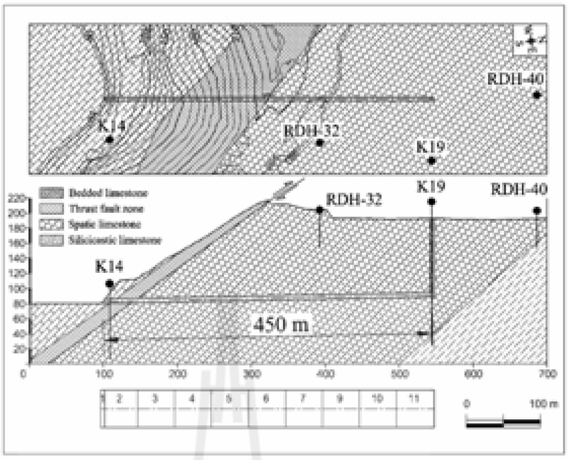

Figure 2. Geological map and cross-section of the SCCC transportation adit.

Zone 1: Bedded limestone will be cut for adit entrance. Sequence outcrops is compact, dark grey micritic limestone (with some crinoidal biosparits), chert layers, calcareous siltstone and siltstone. Some layers are very hard and seem to be silicified. Clear dipping towards southwest with an angle of about 40 to 70 degrees. Apertures are 2.5–10 mm wide and non-material infilling. Average joint spacing ranges between 30 and 50 cm. Discontinuity surfaces are tight. The uniaxial compressive strength (UCS) is measured as 93.6 ± 38.4MPa. The rock quality designation (RQD) is 51%.

Zone 2: Thrust fault zone thickness about 10 to 30 m is observed. This unit consists mainly of heavily brecciated black shale and reddish and bright grey sandstone. Also some tectonised limestone may occur. The tectonic breccia crosses the limestone quarry from SE to NW and dips clearly towards southwest with an angle of about 30 to 60 degrees. Apertures are 0.5–2.5 mm wide. Average joint spacing ranges between 5 and 20 cm. Discontinuity surfaces are slickenside with occasional calcite and clay infilling. The UCS is measured as 57.6 ± 36.0 MPa. The RQD is 31%.

Zone 3: Spatic limestone is light grey, partly pinkish-violet. The limestone is biosparites or biomiclasts. Also some siliciclastic intercalations where cross bedding (sediment structures) have been observed. Sporadically well-rounded micritic extra-clasts are found. Thick massive bedding and a very homogeneous appearance are typical. At the contact to the siliciclastic unit, a clear dip of the layers towards southwest with an angle of about 40 to 60 degrees is observed. Apertures are 2.5–10 mm wide. Average joint spacing ranges between 50 and 150 cm. Discontinuity surfaces are tight. The UCS is measured as 50.3±9.3 MPa. The RQD is 59%.

A total of 216 discontinuities were measured in the field. The discontinuity orientations have been processed by computer software DIPS 5.1, based on equal-area stereographic projection and dominant discontinuity set. The determined dominant discontinuity sets are illustrated in Figure 3. The strike and dip angle of main discontinuities are determined as Table 1.

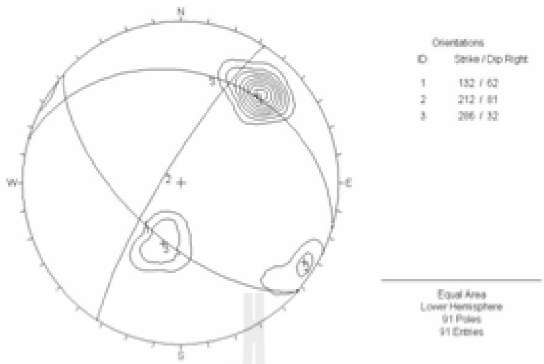

Figure 3. Dominant discontinuity sets of bedded limestone.

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

Rock type	RMR		Q		RMi		GSI	
	Value	Class	Value	Class	Value	Class	Value	Class
Bedded limestone	66	Good	21.25	Good	28.00	Good	55	Good
Thrust fault	38	Poor	0.39	Very poor	0.04	Extremely poor	15	Poor
Spatic limestone	74	Good	39.33	Good	18.80	Good	65	Good

4 LABORATORY TESTING

Rock mechanics testing is carried out to determine the physical and mechanical properties of intact rock. The uniaxial compressive strength of intact rock for zones 1, 2 and 3 are 93.6 ± 38.4, 57.6 ± 36.0 and 50.3 ± 9.3 MPa, respectively. Unit weight of zone 1, 2 and 3 are 0.027 MN/m^3. Young's modulus of zone 1 is 24.72 GPa, zone 2 is 3.81 GPa and zone 3 is 30.39 GPa. Poisson ratio of zones1, 2 and 3 are 0.25, 0.25 and 0.24, respectively. Hoek and Brown constant (m_b) of intact rock of zones 1, 2 and 3 are 1.

5 ROCK MASS CHARACTERIZATIONS

The rock mass characterization has been performed to assess the rock mass quality in accordance with the existing engineering rock mass classification systems. Four types of rock mass classification systems including RMR, Q, RMi and GSI, are applied. The classification table of Bieniawski and the modified quantitative GSI table 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 are applied. The rock mass classes for every zone based on these four rock mass classification systems are summarized in Table 1.

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 adit. Field tests to determine some parameters directly are time consuming and expensive. Some empirical relations used here to calculate rock mass parameters given in Tables 2 through 4. The calculated values are presented in Tables 5 through 8.

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

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

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

Eq. no.	Equations	Unit	Notes
(9)	$\sigma_{cm} = \sigma_c \exp\left(\frac{RMR - 100}{18.75}\right)$	(MPa)	
(10)	$\sigma_{cm} = \frac{5.5\gamma (Q_N)^{1/3}}{\sigma_{ci} B^{0.1}}$	(MPa)	B = Width of the tunnel
(11)	$\sigma_{cm} = RMi = \sigma_c\, JP$	(MPa)	JP = Jointing parameter

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

Eq. no.	Equations	Notes
(12)	$m_j = m_i \exp\left(\frac{RMR - 100}{14}\right)$	For disturbed rock mass
(13)	$s_j = \exp\left(\frac{RMR - 100}{6}\right)$	For disturbed rock mass
(14)	$m_j = m_i \exp\left(\frac{RMR - 100}{28}\right)$	For undisturbed rock mass
(15)	$s_j = \exp\left(\frac{RMR - 100}{9}\right)$	For undisturbed rock mass
(16)	$\frac{m_j}{m_i} = 0.135\, Q_N^{1/3}$	$\left(Q_N = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}\right)$
(17)	$s_j = 0.002\, Q_N$	
(18)	$m_j = m_i\, JP^{0.64}$	For undisturbed rock mass
(19)	$m_j = m_i\, JP^{0.857}$	For disturbed rock mass

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

Eq. no.	Equations	Notes
(20)	$s_j = JP^{2.0}$	
(21)	$m_j = m_i \exp\left(\frac{GSI-100}{28-14D}\right)$	D = Disturbance factor
(22)	$s_j = \exp\left(\frac{GSI-100}{9-3D}\right)$	
(23)	$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$	

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

Rock type	RMR			Q		RMi	GSI	Avg.	SD
	Eq. 1	Eq. 2	Eq. 3	Eq. 4	Eq. 5	Eq. 6	Eq. 7		
Bedded limestone	32.00	-	28.75	33.18	27.09	19.54	7.74	24.72	9.61
Thrust fault	-	5.01	5.49	-	4.37	-	0.37	3.81	2.34
Spatic limestone	48.00	-	40.52	39.87	27.05	16.83	10.09	30.39	14.90

Table 6. Rock mass strength (σ_{cm}).

Rock type	Eq. 9	Eq. 10	Eq. 11	(RocData 3.0)	Avg.	SD
Bedded limestone	15.2671	0.3823	28.0000	11.3260	13.7438	11.4008
Thrust fault	0.7914	0.3728	0.0400	0.3490	0.3883	0.3085
Spatic limestone	12.5703	0.7467	18.8000	8.5120	10.1573	7.5671

Table 7. Hoek and Brown parameter (m_j).

Rock type	RMR	Q	RMi	GSI	Avg.	SD
	Eq. 14	Eq. 16	Eq. 18	Eq. 21		
Bedded limestone	1.0580	4.5847	5.5409	0.8239	3.0019	2.4134
Thrust fault	0.0716	0.5159	0.1259	0.0381	0.1879	0.2216
Spatic limestone	1.8734	4.8129	6.3821	1.4942	3.6406	2.3537

Table 8. Hoek and Brown parameter (s_j).

Rock type	RMR	Q	RMi	GSI	Avg.	SD
	Eq. 15	Eq. 17	Eq. 20	Eq. 22		
Bedded limestone	0.022873	0.045333	0.089379	0.001094	0.039670	0.037742
Thrust fault	0.001019	0.000517	0.000006	0.000003	0.000386	0.000486
Spatic limestone	0.055638	0.052444	0.139019	0.004976	0.063019	0.055711

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.

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Table 9. Suggested support systems based on empirical approaches.

Rock type	From RMR	From Q	From RMi
Bedded limestone	BL: 3 m (Locally), BS: 2.5 m with occasional wire mesh CS: 50 mm where required	Unsupported	CBS: 2-2.5 m WBS: 2.5-3 m
Thrust fault	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: 1.5-2.4 m BS: 1.5-1.7 m CS(W): 5-90 mm	CBS: 2-2.5 m WBS: 1.2-1.5 m CS(W) : 80-100 mm WS(W): 150-175 mm
Spatic limestone	BL: 3 m (Locally), BS: 2.5 m with occasional wire mesh CS: 50 mm where required	Unsupported	CBS: 2-2.5 m WBS: 1.2-1.5 m CS(W) : 50-60 mm WS(W): 50-60 mm

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

8 SUPPORT DESIGN USING NUMERICAL METHOD

Phase^2 version 6.0, a finite element program developed by Rocscience, has been used to perform a series of numerical simulations. Support elements used are consisted of rock bolts and shotcrete as proposed by the empirical methods. Hoek and Brown failure criterion is used to estimate yielding zone around the adit 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. In the case of SCCC transportation adit 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 4. Maximum total displacement is also reduced for the supported cases. This suggests that the applied support systems are adequate to obtain adit stability.

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 have been estimated by four different empirical methods and their average values are used as input parameters for finite element analysis.

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

Rock type	Yielded finite elements (no)		Radius of plastic zone (m)		Maximum total displacement (mm)	
	unsupported	supported	unsupported	supported	unsupported	supported
Bedded limestone	297	187	3.67	3.20	5.15	4.87
Thrust fault	575	383	5.79	4.29	4.45	2.86
Spatic limestone	306	87	4.37	3.02	4.24	3.76

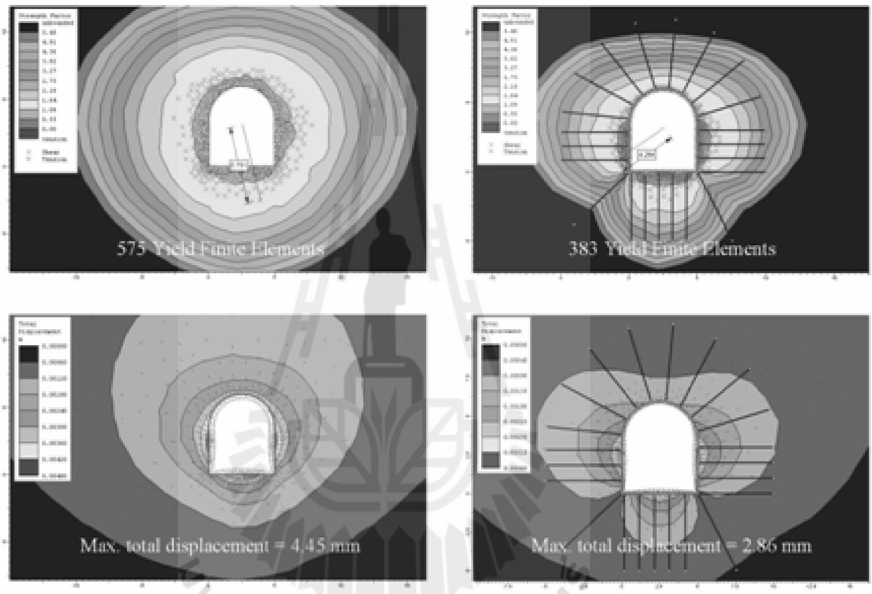

Figure 4. Yielded radius of plastic zone, strength contour, maximum total displacement and displacement vectors for unsupported and supported case of thrust fault zone.

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 unit weight of 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 and bench cut excavation method is recommended of Bedded limestone and Spatic limestone are 1-1.5 m advance full face and install support concurrently with excavation, 20 m from face. Thrust fault zone is recommend 0.5-1.5 m advance in top heading, install support concurrently with excavation; shotcrete as soon as possible after blasting.

Table 11. Recommended support systems for SCCC transportation adit.

Rock type	Rock bolt	Shotcrete
Bedded limestone	Length = 2 m, Spacing = 1.0 x 1.0 m	No
Thrust fault	Length = 5 m, Spacing = 1.0 x 1.0 m	Thickness = 200 mm with wire mesh
Spatic limestone	Length = 2 m, Spacing = 1.0 x 1.0 m	No

10 CONCLUSIONS

Rock masses along the SCCC limestone transportation adit axis is 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 is a stability problem in the Thrust fault zone. The empirical methods, rock mass classification systems, are employed to determine the required support systems for the SCCC transportation adit. 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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