EVALUATION ON THE EFFECTS OF NO-DRAINAGE BOUNDARY CONDITION TO THE GROUNDWATER PRESSURE SURROUNDING THE NATM PERMANENT CAST IN-SITU TUNNEL LINING

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by

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TABLE OF CONTENTS

CHAPTER FOUR : RESULTS AND DISCUSSION

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATION

BC	Boundary Condition
De	Equivalent Dimension
ESR	Excavation Support Ratio
FEM	Finite Element Method
GSI	Geological Strength Index
H-B	Hoek-Brown
HC	Hydrogeochemical
MC	Mohr Coulomb
MRMR	Modified Rock Mass Rating
NATM	New Austrian Tunnelling Method
PSRW	Pahang-Selangor Raw Water
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SPT	Standard Penetration Test
SRF	Stress Reduction Factor
TAM	Tube-A-Manchette
TBM	Tunnel Boring Machine
TRLC	Terzaghi’s Rock Load Classification System
UCS	Uni-axial Compressive Strength
UCT	Unconfined Compression Test

LIST OF SYMBOL

α	Increase of ε_{cp} with increase of p'
A_{el}	Size of finite element
D	Degree of disturbance
E	Young's modulus
$\varepsilon_{\infty}^{shr}$	Final shrinkage strain
ε_{cf}^{p}	Plastic failure strain in compression
ε_{cp}^{p}	Uniaxial plastic failure strain at 1h, 8h, 24h
ε^{shr}	Shrinkage strain
f_c	Compression strength
f_{c0n}	Normalized initially mobilized strength
f_{cfn}	Normalized failure strength (compression)
f_{cun}	Normalized residual strength (compression)
f_{cy}	Current compressive yield stress
F_t	Tensile force
f_t	Tensile strength
f_{tun}	Ratio of residual versus peak tensile strength
f_{ty}	Current tension yield stress
γ_{fc}	Safety factor for compressive strength
γ_{ft}	Safety factor for tensile strength
G_t	Fracture energy in tension
H_c	Normalized compression hardening / softening parameter
H_t	Normalized tension softening parameter
J_a	Joint alteration number

J_n	Joint set number
J_r	Joint roughness number
J_w	Joint water reduction factor
k_x	Permeability in x-direction
k_y	Permeability in y-direction
L	Rock bolt length from excavated tunnel dimension
L_{eq}	Equivalent length
m_i	Intact rock parameter
n_{GP}	Number of stress points per element
p^{ref}	Reference value of stress
P_{roof}	Permanent roof support pressure
Q	Value of Q-system
t_{50}^{cr}	Time for 50% of creep strains
t_{hyr}	Time for full hydration (usually 28 days)
v	Poisson's ratio
ε	Total strain
ϕ^{cr}	Ratio between creep and elastic strains
ϕ_{max}	Maximum friction angle
σ_1	Major principle stress
σ_3	Minor principle stress
σ_{ci}	Uniaxial compression strength
σ_{rot}	Intersection of the Mohr-Coulomb failure envelope and the isotropic axis
ψ	Dilatancy

PENILAIAN KESAN KEADAAN SEMPADAN TANPA ALIRAN TERHADAP TEKANAN AIR BUMI DI SEKELILING PELAPIK TEROWONG TUANGAN SETEMPAT KEKAL NATM

ABSTRAK

Pertumbuhan pesat dalam pembangunan bandar telah menyebabkan peningkatan dalam permintaan terhadap pembinaan bekalan air bagi perindustrian dan perumahan. Terowong penyaluran air merupakan salah satu cara penyelesaian bagi masalah tersebut kerana air mentah boleh disalurkan dari tempat yang mempunyai sumber air yang mencukupi ke kawasan yang kekurangan air. Pembinaan terowong merupakan satu tugas yang mencabar kerana terdapat ketidakpastian yang terlibat dalam pembinaan terowong, terutamanya pembinaan terowong di bawah tanggungan atas tinggi. Oleh itu, ramalan tirisan air bawah tanah ke dalam terowong adalah penting untuk mereka bentuk sistem perparitan terowong, lapik terowong utama dan lapik terowong sekunder, dan untuk meminimumkan kesan alam sekitar dan risiko ketidakstabilan terowong dan kerosakan penenggelaman. Dalam kajian ini, model kajian parametrik dibangunkan berdasarkan input data sifat geologi dan kejuruteraan dengan menggunakan kaedah unsur terhingga. Selain itu, satu model FEM juga dijalankan untuk membandingkan dengan kadar pertambahan kekuatan konkrit. Daripada graf hidrostatik yang dibangunkan, penilaian dilakukan berdasarkan tekanan air liang dan pembacaan instrumen sebenar dilakukan untuk menentukan hubungan sesama sekali. Kesan beban air bawah tanah yang mengawal perilaku terowong di bawah keadaan sempadan yang berbeza telah ditentukan. Berdasarkan analisis, beban air pada lapisan terowong akan berkurangan

jika kondisi sempadan keadaan longkang lapisan terowong disediakan. Untuk pembinaan terowong di bawah beban air bawah tanah yang tinggi, reaksi air bawah tanah akan menyebabkan keretakan konkrit jika kekuatan konkrit awal diperoleh tidak mencukupi. Pemahaman geologi dan kejuruteraan yang mengawal tindak balas terowong dalam membolehkan jurutera untuk membuat persediaan atau langkah kawalan bagi mengatasi keadaan yang tidak diinginkan semasa pembinaan terowong.

EVALUATION ON THE EFFECTS OF NO-DRAINAGE BOUNDARY CONDITION TO THE GROUNDWATER PRESSURE SURROUNDING THE NATM PERMANENT CAST IN-SITU TUNNEL LINING

ABSTRACT

Urbanisation of city and urban area has resulted in urging needs for the construction of new water resources for industrial and residential area. Raw water transfer tunnel is one of the solutions for this problem, as it can transfer water from region with abundant water to water scarcity regions. Tunnelling is a challenging task as there are many uncertainties involved during the construction, especially under moderate to high overburden. Thus, tunnel groundwater flow prediction is crucial during design stage of primary and secondary tunnel lining under the drained or no-drainage boundary conditions in order to minimize the subsidence damage, risk of the tunnel instabilities and environmental impacts. In this study, the parametric study model is developed according to the input of engineering properties data and geological conditions using finite element method (FEM). Besides, a FEM model also carried out to compare with concrete gain strength curve. From the developed hydrostatic graph, the evaluation is done based on the pore water pressure. Additionally, comparison between real instrumentation reading and numerical result is carried out based on tunnel displacement. The effects of groundwater load that control the behaviour of tunnel under different boundary condition were determined. From the analyses, the water load on tunnel lining will decrease if drain condition boundary condition of tunnel lining is provided. For deep excavated tunnel under high groundwater load, the recharge of groundwater will induce cracking of concrete if provided concrete early gain strength is insufficient. The understanding of tunnel

lining boundary condition that control the behaviour of moderate to deep tunnel allow engineer to make better preparation or mitigations to overcome the unfavourable geological condition that will occur during the tunnel construction.

CHAPTER 1

INTRODUCTION

1.1 Background of the study

The prediction of water seepage into tunnel is crucial for tunnelling works during design stage of tunnel drainage system, primary tunnel lining and secondary tunnel lining, in order to minimize subsidence damage, the risk of the tunnel instabilities and impacts to environmental. Butscher (2012) had summarized tunnel seepage can be calculated by setting drainage type and no-drainage type boundary conditions into tunnel perimeter. Besides, analytical solutions and numerical models also been recognized to calculate tunnel inflow. Besides, steady state water inflow into tunnel also can be calculated by using analytical solutions (Kolymbas & Wagner, 2007; Lei, 1999; Park, Owatsiriwong & Lee, 2008).

PSRW Transfer Tunnel Project is a project to build a tunnel for conveying raw water from rich of water source area of Pahang state to Selangor state. The total length of the proposed tunnel is about 44.6 km. This study covered the evaluation of controlled tunnel boundary condition for medium to deep excavated tunnels under high water loads.

For deep mountain tunnels, the tunnels will experience high water pressure. The external water pressure becomes very important role to the stability of deep underground tunnel. The water pressure exert on the lining is referred to hydrostatic pressure. A drainage system can decrease the external water pressure, meanwhile the watertight lining can results to increment of hydrostatic pressure. Wang *et al.* (2008) summarised that the controlled drainage would help in the reduction of hydrostatic pressure on the lining. Besides, there are a few methods had been developed to compute the external water pressure. There are reduction coefficient method, analytical method, semi-analytical method, hydrogeochemical method, seepage and stress coupling methods (Lamas, Leitão, Esteves, *et al.*, 2014; Qin, *et al.*, 2015) and numerical method.

The purpose of this study is to develop a FEM seepage analysis model and to evaluate the effect of groundwater flow to tunnel lining under different tunnel boundary condition for the Pahang-Selangor Raw Water (PSRW) Transfer Tunnel Project.

1.2 Statement of Problems

In all, this dissertation covers numerous investigations on the hydraulic and mechanical behaviour of no-drainage and drained concrete-based tunnels. Parametric study of a shallow to moderate excavated tunnel of circular tunnel and NATM horse shoe shaped tunnel will be highlighted, so that plane strain two-dimensional finite element models can be applied.

Insufficient information of the ground condition such as rock type, engineering properties, geological condition and groundwater condition of the rock mass under moderate to high overburden is critical to establish the hydrostatic and overburden loading for application of tunnel permanent lining design where uncertainty of problems will arise during tunnel excavation stage. Ground investigation was unable to proceed before the tunnel construction as tunnel was constructed in high overburden where it located under more than 100m height of mountain. Besides, groundwater and displacement monitoring were difficult to be carried out due to restriction in assess of jungle area and not recommended in term of economical effectiveness. As such, it will cause unexpected load case to tunnel lining during construction stage. Due to uncertain and unknown groundwater behaviour, different drainage type of tunnel lining system will perform different role of design approached in tunnel lining. Hence, effect of hydrostatic loading onto drained tunnel lining and no-drainage tunnel lining urge to being studied and predicted before tunnel lining design. Thus, appropriate tool to simulate and compute graph of hydrostatic stress versus time shall be adopted so that overall case of hydrostatic loading could be considered during tunnel lining design.

This research is intended for cases where groundwater level of tunnel is situated below existing ground level, to serve as worse scenario case for tunnelling works. Without the drain properties in concrete lining, the structural capacity of tunnels concrete lining depends to the progressively development of hydrostatic pressure. Distinction is made based on whether the concrete lining can withstand the full hydrostatic pressure. There are few research questions which appeared related to the main domain research topics:

i. Insufficient information of the ground condition is critical to establish the hydrostatic and overburden loading applied to the permanent lining design of the tunnel and uncertainty of problems will arise during tunnel excavation stage.
ii. Pore water pressure monitoring was unable to be carried out under deep mountain caused unexpected load case to tunnel lining during construction stage and subsequently induced cracking of tunnel lining.
iii. Necessarily in computation of graph of hydrostatic stress versus time so that overall case of hydrostatic loading could be considered during tunnel lining design.

iv. Necessarily in assess on the relationship between permeability of tunnel lining permeability to the maximum ground water pressure for tunnel lining.

v. Necessarily in assess on the time dependant behaviour of concrete lining strength with porewater pressure increment effect.

In order to provide the solution to the problem, a few tunnel boundary conditions are proposed to study the relationship between tunnel boundary condition to tunnel groundwater inflow and surrounding tunnel groundwater pressure. For this purpose, FEM seepage analysis is used to simulate the groundwater pressure in various tunnel boundary conditions. In additional, actual instrumentation monitoring results such as strain gauge monitoring result is used to compare and verify with the analysis result.

1.3 Objectives

The main objectives of the study are listed out as below:

i. To develop graph of hydrostatic stress versus time using 2D FEM seepage analysis.

ii. To determine the effect of groundwater pressure to drainage and no-drainage tunnel lining stability.

iii. To evaluate discharge and recharge of groundwater that create different pore water pressure induced tunnel lining cracking.

iv. To assess time dependent behavior of concrete tunnel lining strength with pore water pressure increment due to the effect of groundwater recharge surrounding the tunnel.

1.4 Expected Outcomes

This study comprises of evaluation on the effects of drainage condition and no-drainage condition to the groundwater pressure and permanent cast in-situ tunnel lining. From the study, FEM seepage analysis model will be generated. Several tunnel boundary conditions will be developed in the model to identify the hydraulic stress exerted to the primary lining and secondary lining during the excavation of tunnel. The study will provide a good interpretation to the tunnel lining design, where no-drainage and drainage type lining system during tunnel lining design. Besides, shotcrete model will be developed to identify the concrete gain strength overtime behaviour for modelling to actual construction condition.

1.5 Content of the Study

The content of the study is summarized as below.

In chapter 2, the mode of potential factor for cracking of tunnel lining, tunnelling methods, numerical methods, analytical method and tunnel lining properties were highlighted based on the previous studies. Based on the literature review carried out, the mode of potential factors for cracking of tunnel lining can be classified into six important aspects, such as concrete shrinkage and effect of groundwater on creep induced tunnel lining cracking. The effect of groundwater induced tunnel lining cracking is very important in this study as it was the main factors to investigate the real construction case happened in PSRW Transfer Tunnel Project. For the tunnelling methods, previous studies had summarized it into TBM Method and NATM. Previous researches have been summarized the evaluation of tunnelling method under different ground condition such as soft soil, rock and mixed ground condition. Besides, the literature review provides the overview over the numerical methods such as reduction coefficient method, Finite Element Methods (FEM) and Hydrogeochemical Methods; and analytical methods. The theories behind the numerical methods and analytical methods were also had been further discussed in this chapter.

In chapter 3, the flow of the analysis is discussed in detail for this study. Several conceptual models were developed with the main parameter such as groundwater level, tunnel lining drainage properties, Lugeon value and RQD. The conceptual model was computed by using FEM with Plaxis 2D program. In the program, input parameters were justified based on collected data. Based on the conceptual model developed, the evaluation was done to understand the difference of groundwater effect to tunnel lining during construction of NATM tunnel. At last, secondary data which collected from the PSRW Transfer Project was established and compare porewater pressure of surrounding tunnel lining to groundwater pressure on site from instrumentation reading. From the analysis carried out, the porewater pressure with several tunnel boundary conditions before and during tunnel construction can be predicted. Besides, mitigation measures for tunnel excavation and tunnel lining design were able to be prepared.

In chapter 4, the generating of several graph of porewater pressure versus time based on the 1st kind boundary condition (Dirichlet type), closed boundary condition and mixed boundary condition were presented. The porewater pressure graph was generated in order to understand the difference of groundwater effect to tunnel lining during construction of NATM tunnel. It was essential to present porewater pressure so that types of tunnel boundary conditions used during the tunnel excavation can be recommended before and during the tunnel construction. Besides, comparison between the porewater pressure graph based on FEM and actual site instrumentation readings was also done in this chapter to verify the reliability of the generated porewater pressure graph.

In chapter 5, conclusion was done based on the results obtained from the analysis. Besides, the limitation during this study was also listed out. Based on the limitations that faced during the analysis, several recommendations were listed out.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Urbanisation of city and urban area has resulted to urge of new water supply source for industrial centres, commercial and residential area of the Selangor state, Kuala Lumpur and Negeri Sembilan state. Tunnels become a requirement to relief this situation and become major parts for this kind of project. To construct a tunnel, many factors and condition needed to be determined especially for the deep excavated tunnel under mountain. The boundary condition between primary lining and tunnel permanent lining for New Austrian Tunnelling Method (NATM) currently become a big challenge to the construction of moderate to deep tunnel. According to Butscher (2012), the procedure which based on analytical solutions can be used to calculate tunnel inflow. The estimation of ground water seepage into a tunnel is cricual during design stage of tunnel lining, tunnel drainage system and to eliminate the risk of subsidence damage, tunnel instabilities and impact to environmental. Besides, (Yoo, *et al.*, 2012) also reveal a case history on tunnelling induced groundwater drawdown cause excessive surface settlements.

This study includes the coupling analysis of different tunnel boundary condition to pore pressure surrounding primary lining and secondary lining by using FEM. Besides, this study also includes in the analysis of shotcrete model which to simulate real behaviour of primary lining and secondary lining concrete behaviour over time. Concrete properties and tunnel boundary condition that controls the behaviour of the NATM Tunnel under high overburden condition will be discussed.

2.2 Mode of potential factors for Cracking of Tunnel Lining

For conventional construction method of NATM tunnel, the primary lining normally casted by using shotcrete. This method is widely used in the tunnelling industry. Besides, permanent lining of NATM tunnels, where it is also considered as final lining, usually formed by cast in-situ method. The cracking of the tunnel lining always drawn much attention of designers and researchers in the engineering field and some measurements have been applied to the practical use. Many currents methods for evaluating the conditions of the existing tunnel linings are still on the stage of qualitative analysis. More extensive studies and comprehensive understanding for mode of potential factors to cracking of tunnel lining is essential to be considered into proper standard of designing the new tunnel linings and of maintaining the existing one under various service conditions.

2.2.1 Concrete Shrinkage

New Austrian Tunnelling method (NATM) is mainly used for constructing the tunnel in mountainous area. In this method, cast-in-place plain concrete has been generally used as a secondary lining. In many cases, the small cracks induced by the thermal, autogenous and drying shrinkage were occurred on inner surface (contact with air) at an early age. It can be considered that this kind of cracks would not reduce an overall tunnel stability, but it could cause concrete spalling in the long term if cracks run cross one another and could affect to the safety of tunnel user. The water effect onto damaging of tunnel lining during design life of tunnel was summarised by Howard (1991). He mentioned that a forecast of shrinkage must be made during tunnel lining design.

2.2.2 Effect of Groundwater on Creep induced lining cracking

Broch (1979) showed examples of groundwater will reduce the uniaxial compressive strength (UCS) of some isotropic and anisotropic rock. For anisotropic rocks, the failure strength will be reduced for water saturated rock based on triaxial test result. The rock behaviour will be affected if the cracking between rock filled with water, especially in anisotropic rock. The groundwater in rock cracking area will deteriorate the component between rock by stress corrosion in the tips of crack and due to changing in the rock humidity through capillary and adsorption. and result to dispersion of rock crack especially rock under stress. This process will slowly reduce the long-term strength of rock.

Besides, if there is water seepage in between the rock crack, the drainage effect will fasten the wearing of rock crack, hence increase the rate of rock strain. Brace & Martin (1968) had carried out drained compression test in low porosity of anisotropic rocks. Their finding is the effect of water seepage to the strength of isotropic rock is negligible during the rock experienced low in strain rates. If the rock porosity is increase, then subsequent effect is the strength (UCS) of rock will be reduced due to seepage of water. There was some reduction in UCS strength by 33% to isotropic rock, 42% to anisotropic rock, 53% for perpendicular to foliation rock and 38% for parallel to foliation rock. In isotropic rocks, the failure strength is same for both cases of saturated rock and dry rock, but it was not applying to gneiss specimens case (*Figure 2-1*). For the rock which under high porewater pressure, the effect to the rock will be significant. There will be increment in porewater pressure if clay is existence in the discontinuity plane within the tunnel vicinity.

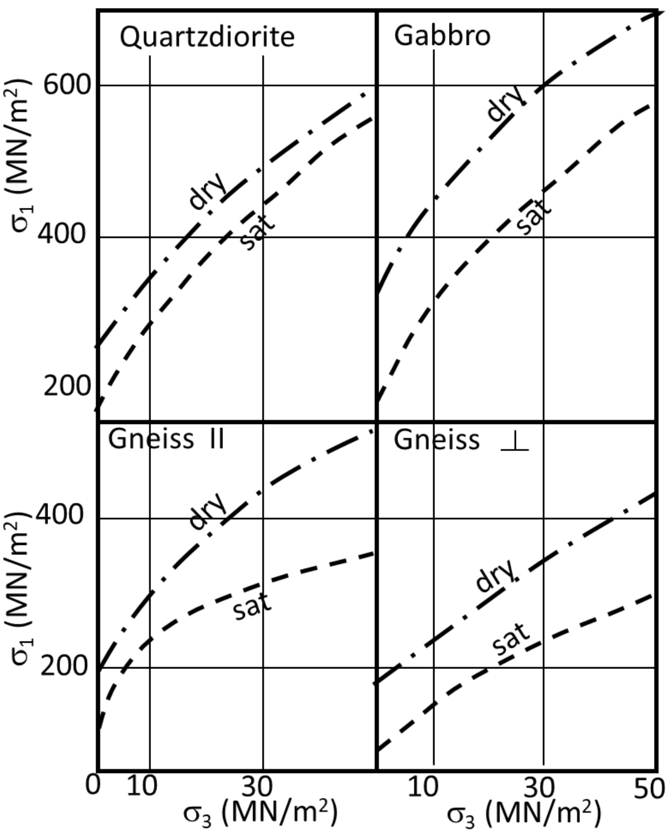

Figure 2-1: Failure curves for dry and water saturated rocks (Broch, 1979)

2.3 Derivation of groundwater properties by in-situ measurement

In this section, groundwater properties are defined as the heterogeneous distribution of hydraulic properties for flow and mass transferring. Meanwhile, diffusivity and dispersity are beyond this discussion.

In order to grasp the groundwater properties in the field, geologists and engineers measure and estimate the distribution of hydraulic conductivities in rock mass. However, unlike sedimentary rocks, hydraulic conductivity of igneous rock varies several orders of magnitude in short distance between fractures and matrix, thus, increased the susceptibility of uncertainty in estimating its distribution.

Hydraulic test, such as pumping test, slug test and constant injection provide simple and rapid methods to assess the hydraulic properties of the anisotropic or isotropic rock. Pumping test consists of a pumping well and at least a monitoring well where installed constant flow rate pump withdraws water in the pumping well and continues groundwater level measurement in monitoring one. Theis (1935) initially gave solution so that hydraulic properties (transmissivity and ability of water storage) of confined aquifers can be certified by the analysis to match the Theis solution for groundwater level fluctuation data collected from pumping test.

Slug test is conducted by removing or adding a slug of liquid such as water in the well, then monitoring the groundwater level or pressure fluctuation. Slug test assesses the hydraulic properties surrounding test borehole without monitoring borehole.

Constant head test hydraulic test provides a stable control condition in experiment for interpretation and is essential to geotechnical and hydrogeological site characterization. The test was originally developed by Lugeon (1932) also named Lugeon test. Constant injection has been adopted on fractured rock (Shapiro & Hsieh, 1998) and relatively low permeability clayey soil (Tavenas, Diene & Leroueil, 1990). Studies interpreted the patterns of results to derive the hydraulic conductivity and permeability (Lancaster-Jones, 1975). Parkers also have been applied in hydraulic test to isolate the target-rock for deriving the local hydraulic properties (Pickens, *et al.*, 1987; Rehbinder, 1996). In the heterogeneous medium, the parker hydraulic test result is quoted in terms of a support scale which equals to the length of the packed-off interval. For the low permeability and high stiffness rock, high pressure (>10Mpa) injection triggered a transient pulse in packed-off interval to examine the hydraulic properties and provides a rapid hydraulic permeability assessment (Neuman & Di Federico, 2003).

2.3.1 Groundwater control in fractured rock mass

The main purpose of the groundwater control in fractured rock mass for the underground facilities utilization is to maintain its long term performance. This concept has been used to serve the purposes of caverns for pressurized air, nuclear waste repository, oil and gas storage etc. In the underground storage cavern concept, water tight tunnel is important so that the stored products will not leaked. Normally, the intact rock mass has the hydraulic conductivity of 10^{-11} to 10^{-12} m/s. Meanwhile, fractured rock mass may have hydraulic conductivity in the range of 10^{-5} to 10^{-6} m/s. High hydraulic conductivity of fractured rock must be treated to prevent any water leakage. To prevent excessive groundwater inflow to the underground opening and thus caused disturbance to the surrounding groundwater environment, there are necessary to provide suitable solution when come to zone of high hydraulic conductivity rock zone area.

Table 2.1: Classification of Rock Mass on the Basis of Lugeon Values (Houlsby, 1977)

Lugeon Value	Strong, massive rock with continuous jointing	Weak, heavily jointed rock
0	Completely tight	Completely tight
1	Sometimes open joints up to about 1mm	Sometimes open to hair crack size of 0.3mm
3.5	Occasionally open to 2.5mm	Occasionally open to 1.2mm
20	Often open to 1.2mm	Often open to 1.2mm
50	Often open to 2.5mm	Often open to 2.5mm
100	Often open to 6.2mm	Often open to 6.2mm

Note: Joint measurements are in mm; 1 Lugeon = 1.3 x 10^{-5} cm/sec.

2.3.1.1 Groundwater controlling method

Analyses for local groundwater balance studies (Griffiths & Barker, 1993) was carried out in the recent years, to identify the sensitivity of the surroundings groundwater condition. Meanwhile, numerical modeling based on groundwater fluctuations had been carried out by Johansen (2001). Empirical formula can be used together with the numerical modelling method. Empirical formulas for analysis of ground settlement and potential damage on surface structures are also available According to Karlsrud (2001), ground settlement analysis and damage assessment analysis can be carried out according to empirical formulas. The numerical and empirical formula analysis is crucial in the groundwater control so that limit of water seepage into underground facilities can be determined and the fulfill with the limit of seepage flow.

2.3.1.2 Grouting system

Hydraulic conductivity of rock can be controlled by using grouting system such as Tube-A-Manchette (TAM) grout, fissure grout etc. The injected grout will be function as water barrier around the underground facilities by filling the hydraulic conductive fractures in the rock with cement grout, hence hydraulic conductivity of rock reduced. Grouting in rock will help to reduce the water pressure, subsequently reduce the water pressure surrounding of cavern, tunnel or shaft to closed to zero pore water pressure. Besides, the grouting also contributed to some strength improvement of rock mass, hence improving in stability of cavern, tunnel or shaft construction.

2.3.2 Grouting in fractured rock mass

Grouting in fractured rock is common to conduct surrounding the underground facilities to achieve certain degree of water tightness. The purpose of the grouting in fractured rock mass is to seal off the apertures or crack in rock mass. If the apertures of rock are connected to each other, the grouting works will be effectiveness and the complete filling of existing void will made possible.

2.3.2.1 Grouting requirement in the underground facilities

In early of 19th century, subsurface grouting was successfully carried out in France (Glossop, 1961). The purpose for the grouting applied was to improve the foundation stability. There was more frequent of application of grouting works after the first successful case. From 19th century until now, grouting methods had been improved and developed using different methods and different materials so that it suit to different ground conditions. However, until now, the grouting works methods and experience still in the trial and error stage due to uncertainty of ground condition around the world. This is because the grouting knowledge required combination of vast understanding to rheology, chemistry, rock mechanic, hydrogeology, geology and grouting technology (Fransson, *et al.*, 2007). As science of grouting involved of various field of knowledge, there are still a lot of rooms for improvement.

Many of the current and previous underground facilities required a strict groundwater inflow requirement, together with tide of cost management. With the high quality jointed rock conditions and abundance of groundwater, most of underground facilities would provide primary support such as concrete lining. One of the favored choices of sealing method is using pre-grouting with cement (Houlsby, 1990; Kutzner, 1996; Warner, 2004). For the uncertain condition of ground, fracture rock usually has typical hydraulic apertures of 10 to more than 100 μm while the water conducting fractures usually varies. In some cases of conventional grouting method and cement grouts, this standard method had fail to seal the fractured rock mass to an acceptance level due to such rock mass heterogeneities and anisotropies.

2.3.2.2 Prediction of pre-grouting performance based in the hydrogeological classification of fractured rock mass

To improve the groundwater containment system of the underground tunnel, there will be the need to seal off fracture rock mass around the excavated tunnel by grouting works (Houlsby, 1990; Kutzner, 1996) so that excessive groundwater inflow into the tunnel excavation can be mitigated. Pre-grouting method will be one of the suitable method to be implemented during the tunnel excavation (Warner, 2004). The mechanism of pre-grouting involved injection of grout to fill through two adjacent grout holes for all the rock fracture. The grouting will be considered efficient if the particle size of grout is smaller than the aperture of the rock mass fracture (Eriksson, Stille & Andersson, 2000). To achieve the effective pre-grouting works, the fracture transmissivity of rock shall be studied. In-situ field investigation study shall be carried out to obtain the boreholes data; hence arrangement of grout holes design can be done. Advancement of tunneling works shall be carried out after pre-grouting works finish and minimum setting time achieved. After that, ground permeability test such as Lugeon test shall be carried out to check for the effectiveness of pre-grouting works. If necessary, additional grout holes to be provided to further reduce the rock hydraulic conductivity based on minimum requirement of the specific project.

However, the result from ground permeability test may not provide true view of grouting result from rock hydraulic conductivity, as grout particle are unable to be filled in very tiny aperture of rock fracture. Hence, the judgement on the suitable grouting system shall be made according to borehole individual fracture information (Gustafson & Stille, 2005). In order to obtain effective grouting procedure and grouting system, classifying the groundwater condition of the rock mass shall be carried out. The classification that will be developed should be according to rock mass generalization with parallel plate model or fracture network model, where grout and water can be flow (Hässler, Håkansson & Stille, 1992). When the generalization has been done, the grouting process and mechanism can be simulated in the numerical model.

With the availability of geological data like fracture aperture, classification of rock mass will be done by comparison of measured grouting flow and also injection result. Further classification shall be done based on past experience of previous grouting work like injection parameter, grouting procedure, grouting pressure, injection tips withdrawal speed etc. It must also be cautions that the classification is merely based on the theoretical and simplification and therefore should be verified with the results obtained in the in-situ condition.

2.4 Tunnelling Methods

Tunnelling is an underground passage that connected from one place to another place, which excavated underground through surrounding rock or soil. The tunnelling works normally carried out by using tunnel boring machine or NATM methods. Both methods will be determined based on the geological conditions, as well as tunnel length and tunnel geometry.

2.4.1 Tunnel Boring Machine (TBM)

TBM tunnelling method is an advance tunnelling methods which widely used around the world. TBM can be used to any kind of geological condition and normally it is circular in shape. Tunnel Boring Machine can be classified as Rock Tunnelling Machines and Soft Ground Tunnelling Machine. TBM are unable to carry on under uncertain ground condition such as mixed ground condition. Hence, selection of

TBM is a critical task to be done when come to uncertain ground condition. The factors to be considered during the selection of TBM are ground condition whether soil or rock, hardness of the rock (UCS), depth of tunnel, mix ground condition, and hydraulic pressure (Kovári, et al., 2004). If wrong TBM has been done on site, it will consequently effect to the tunnelling works advancement, or even cause the tunnelling works stuck underground.

2.4.2 New Austrian Tunnelling Method (NATM)

The NATM is a mining tunnelling works method for construction and excavation of tunnel. The main concept applied in the NATM was stress relief of rock surrounding the excavated tunnel and self-supporting of tunnel by rock mass itself (Herrenknecht, Baeppler & Ozdemir, 2006). The primary support of NATM tunnel included shotcrete primary lining together with or without lattice girder, rock bolt in rock tunnelling and wire mesh for general NATM tunnel. In the cases of poor in strength of rock mass or mixed ground condition, the use of forepoling or pipe roofing is also installed for crown support. The forepoling or pipe roofing will provide temporary support to prevent falling of sand; hence increase the stability of tunnel during excavation and construction of NATM tunnel. Understanding of rock mass classification and also performance of NATM tunnel during excavation will be useful for the NATM design so that the NATM design will be efficient in tern of cost when suitable tunnel support system and efficient construction sequence were proposed (Hellmich, Mang & Ulm, 2001).

2.5 Rock Mass Classification

In the early of 19th century, rock mass classification had been introduced in Europe and widely used in the rock engineering. Rock mass classification was originated from empirical approach of tunnel design, which served the purpose of determining the type of tunnel support. Rock mass classification had been classified into 9 categories (Terzaghi, 1946). There are a few rock mass classification had been proposed by various researcher, such as Q-system (Barton, Lien & Lunde, 1974), Terzaghi Rock Mass Classification, RMR (Bieniawski, 1976), MRMR and etc, where input of rock rating is required by experience and research in order to obtain the most suitable tunnel support design based on rock condition. Based on case study from past experience and projects, Barton, Lien & Lunde (1974), Wickham, Tiedemann & Skinner (1972) and Bieniawski (1989) compile and classified the appropriate type of rock mass properties into their empirical approach.

2.5.1 Terzaghi's Rock Load Classification System

Terzaghi had introduced rock mass classification in year 1946 which became the fundamental for rock mass classification. TRLC involved the consideration of rock mass quality and characteristic to the proposal of tunnel support. TRLC system had been modified and improving its basic function in the RMR system and MRMR system, which precisely concern on the dominant driving force from the joint rock mass.

Terzaghi's descriptions (quoted directly from his paper) are:

- Intact rock contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as a spalling condition. Hard, intact rock may also be encountered in the popping condition involving the spontaneous and violent detachment of rock slabs from the sides or roof;

- Stratified rock consists of individual strata with little or no resistance against separation along the boundaries between the strata. The strata may or may not be weakened by transverse joints. In such rock the spalling condition is quite common;
- Moderately jointed rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type, both spalling and popping conditions may be encountered;
- Blocky and seamy rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support;
- Crushed but chemically intact rock has the character of crusher run. If most or all of the fragments are as small as fine sand grains and no recommendation has taken place, crushed rock below the water table exhibits the properties of water-bearing sand;
- Squeezing rock slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or clay minerals with a low swelling capacity; and
- Swelling rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks that contain clay minerals such as montmorillonite, with a high swelling capacity.

2.5.2 Rock Mass Rating (RMR)

RMR had been proposed by Bieniawski in year 1976 (Bieniawski, 1974). There are improvement and modification of RMR which according to various successful case in the project from time to time until year 1989 (Bieniawski, 1989). The RMR system can be used to rock tunnelling, stability of slope (Bieniawski, 1988; Romana, 1993; Robertson, 1988), foundations of dam (Bieniawski & Orr, 1976; Serafim & Pereira, 1983) and also cavern construction (Laubscher, 1990; Newman & Bieniawski, 1985). RMR had divided rock mass into 6 categories of parameters.

1. Uniaxial compressive strength of rock material.
2. Rock Quality Designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities
5. Groundwater conditions.
6. Orientation of discontinuities.

Tunnel support in rock can be computed based on rock mass classification shown in Table 2.2 (Bieniawski, 1989). RMR system is a combination based on previous research of (Wickham, Tiedemann & Skinner, 1972), together with the RQD parameter from Deere, *et al.* (1967).

2.5.3 Modification to Rock Mass Rating (MRMR) for Mining

As describe in chapter 2.5.1, MRMR is the improvement and modification of RMR according to various successful case in the past project until year 1989. The improvement of RMR had been carried out in order to satisfy the mining condition and also to have more economically and efficient NATM design. A comprehensive summary of these modifications was compiled by (Bieniawski, 1989) had summarised on the improvement of RMR. MRMR also had been explained in details by Laubscher (1990) in the application of mining works. As MRMR system is an improvement from RMR, the parameters used in RMR were remain, with additional consideration for rock mass in-situ stresses, changes in stress during tunnel excavation and the construction method of tunnel. Based on the computed MRMR results, type of tunnel support will be recommended.

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) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 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	

*Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

** Modified after (Wickham, Tiedemann & Skinner, 1972).

Figure 2-2: Rock Mass Rating System (after Bieniawski, 1989)

Table 2.2: 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 20m from face	Locally, bolts in crown 3m long, spaced 2.5m 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 10m from face.	Systematic bolts 4 m long, spaced 1.5-2m 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 0.5-1.5m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6m long, spaced 1-1.5m 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 0.75m with steel lagging and fore poling if required. Close invert.

Table 2.3: Classification table for the RMR

RMR	Rock Quality
0-20	Very poor
21-40	Poor
41-60	Fair
61-80	Good
81-100	Very Good

2.5.4 Q-System

(Barton, Lien & Lunde, 1974) introduced Q-system based on previous projects and experience of mining works. The purpose of Q-system is to classify the rock mass, subsequently suggested suitable type of tunnel support. Q-value can be computed based on equation of:

$$Q = \frac{RQD}{J_n} x \frac{J_r}{J_a} x \frac{J_w}{SRF} \qquad \text{Eq. 2.1}$$

where

RQD is the Rock Quality Designation

J_n is the joint set number

Jr is the joint roughness number

J_a is the joint alteration number

J_w is the joint water reduction factor

SRF is the stress reduction factor.