Engineering geological properties of Jurong formation for underground cavern excavation in Singapore

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**ENGINEERING GEOLOGICAL PROPERTIES OF
JURONG FORMATION FOR UNDERGROUND
CAVERN EXCAVATION IN SINGAPORE**

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

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ENGINEERING GEOLOGICAL PROPERTIES OF JURONG FORMATION FOR UNDERGROUND CAVERN EXCAVATION IN SINGAPORE

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School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University in partial
fulfilment of the requirement for the degree of Doctor of Philosophy

Statement of Originality

I hereby certify that the work embodied in this thesis is the result of original research, is free of plagiarised materials, and has not been submitted for a higher degree to any other University or Institution.

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21-1-2019

Date

Low Bak Kong
(Associate Professor)

Authorship Attribution Statement

This thesis contains material from 2 papers published in the following peer-reviewed journals where I was the first author.

- Chapter 3 is published as Kar Winn, Wong, N.Y., Khin Zaw, Thompson, J. (2018). "The Ayer Chawan Facies, Jurong Formation, Singapore: Age and observation of syndepositional pyroclastic sedimentation process with possible peperite formation". Bulletin of the Geology Society of Malaysia, V. 66, pp: 25-31

The contributions of the co-authors are as follows:

- I prepared the manuscript drafts based on the field observation with supporting photographs and petrological studies.
 - The manuscript was edited by A/P Louis Wong and Professor Khin Zaw.
 - Professor Khin Zaw and Jay Thompson carried out radiometric dating of the rock samples, which was sent by me, at their university laboratory.
-
- Chapter 4 is published as Kar Winn, Wong, N.Y. (2019). "Quantitative GSI Determination of Singapore's Sedimentary Rock Mass by Applying Four Different Approaches". Geotechnical and Geological Engineering. <https://doi.org/10.1007/s10706-018-0748-8>

The contributions of the co-authors are as follows:

- I prepared the manuscript drafts based on the field observation with photographs, determination of quantitative GSI results with tables and graphs.
- The manuscript was edited by A/P Louis Wong.

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ABSTRACT

The excavation of underground hydrocarbon storage rock cavern project had recently been completed in Singapore's sedimentary Jurong Formation. Geologically speaking, it belongs to Ayer Chawan Facies based on constituted rocks, location and stratigraphic position.

Based on the field relationship and petrographic studies, it is very likely that the pyroclastic lava intruded into the muddy sediments of Ayer Chawan Facies while they were deposited in a wet and unconsolidated condition. As a result, peperitic textures were generated. Subsequent thermal metamorphism made the rocks harder than the similar type of typical sedimentary rocks elsewhere in the world.

The LA ICP-MS U-Pb zircon age of pyroclastic rocks (lithic tuff) of the Ayer Chawan Facies is determined to be 240.6 ± 1.2 Ma (Middle Triassic) in this study. It is in the same age range assigned after age determination from previous finding on collected fossil species.

The engineering geological properties of intact rocks were measured in different stages before and during the excavation work. Joint analysis was carried out with 350 numbers of representative joints, bedding and fault information, which have been recorded from the geological mapping in one storage cavern. Joint sets were analyzed by DIPS program and 1 bedding set associated with 4 joint sets were identified.

The strength properties of intact rocks in laboratory such as point load strength, unconfined compressive strength, triaxial compressive strength, Brazilian tensile strength and shear strength of natural rock joints were determined.

The hydrogeological properties of rock mass was analysed with the records of water ingress rate versus pressure measured from the observatory probe holes, water bearing zones and water recharging holes in water curtain galleries. The calculated k value is in the order of 1×10^{-6} m/s and the average value is 1.73×10^{-6} m/s.

Detailed geology mapping was conducted on every excavated rock faces to evaluate the rock mass quality using Q and RMR systems and to determine qualitative

Geological Strength Index (GSI). The quantitative GSI was also calculated by four different approaches and all the results are approximately in the same range from 45 to 55. The quantitative and qualitative GSI varied within a range of ± 10 .

Afterward, the strength and deformation properties of jointed rock mass were determined based on the intact rock properties with the input of measured GSI and excavation disturbance factor.

In this study, an intelligent approach based on the Mamdani inference mechanism was employed to predict UCS and quantitative GSI determination of sedimentary rock mass. The attained results are evaluated with those from statistical model using coefficient of correlation (R^2) and root mean square error (RMSE).

The joint sets analysis results were applied to the wedge analyses in full cavern excavation case to evaluate the possibility of formation of wedges due to bench excavation. The results shows that no additional wedges can be formed with investigated joint sets due to steep dip angle of joints and faults.

2 D finite element program for discontinuum (intact rock parameters) and continuum analyses (rock mass parameters) and 3D finite element program for continuum analysis were conducted to evaluate the cavern stability in subsequent excavation stages. The calculated vertical displacements were in the comparable range with measured values. The safe strength factors after full face of excavation were recorded from all analyses. The calculated rock bolts and shotcrete strength are much lower than the design capacity of them. The high horizontal to vertical stress ratio measured in the project also provided tunnel crown stability condition.

Having competent sub-horizontal bedded layers intersected with sub-vertical joints, flat-roofed tunnel profiles were occasionally observed in the project. The stability of such tunnels were evaluated by analytical (Voussoir beam algorithm), empirical (stability graph) and numerical approaches. The abutment relaxation is unlikely to be occurred in the project since the sufficient pillar width between adjacent tunnels is designed to balance the overstressing caused by the combination of rock mass strength and in situ stresses.

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List of symbols used

Symbol	Unit	Definition
2D		2 dimensional
3D		3 dimensional
a		Hoek-Brown constant depend on rock mass character
BHTV		Bore hole televiewer survey
c, c _{peak} , c _{res}	MPa	Cohesion, peak cohesion, residual cohesion
CR		Core Run
CS		Cavern storage (for oil & gas)
D		Disturbance factor
E	MPa/GPa	Elastic modulus
E _i	MPa/GPa	Intact rock deformation modulus
E _{rm}	MPa/GPa	Rock mass deformation modulus
EW	degree	East-west direction
GPa		Giga Pascal
GSI		Geological Strength Index
J _a		Joint alteration number
J _c		Joint surface condition
J _n		Joint set number
J _r		Joint roughness number
J _s		Small scale joint smoothness
J _w		Joint waviness
J _v		Volumetric joint count
JCond ₇₆		Joint condition rating (Bieniawski, 1976)
JCond ₈₉		Joint condition rating (Bieniawski, 1989)
JRC		Jurong Rock Cavern Project
k	m/s	Coefficient of permeability
k _h	m/s	Coefficient of permeability in horizontal direction
k _v	m/s	Coefficient of permeability in vertical direction
LDP		Longitudinal displacement profile
m _i , m _b		Hoek-Brown constant of intact rock and rock mass
MN		Mega Newton
MPa		Mega Pascal (MN/m ² = MN per square meter)
NNE	Degree	North-north-east direction
NS	degree	North-south direction
Q		Tunnel Quality Index
R ²		coefficient of correlation
RMSE		root mean square error
R _f		Infilling rating
R _r		Roughness rating
R _w		Weathering rating
RMR		Rock Mass Rating System
RQD		Rock Quality Designation
s		Hoek-Brown constant depend on rock mass character
SCR		Surface condition rating

σ_h	MPa	Minimum horizontal stress
σ_H	MPa	Maximum horizontal stress
σ_v	MPa	Vertical stress
VBH		Vertical Bore Hole
V_b		Block volume
X_A		a unique membership function
γ	kN/m ³	unit weight
$\mu_A(x)$		a membership function of a fuzzy set
σ_{ci}	MPa	Uniaxial compressive strength of the intact rock
$\sigma_1 - \sigma_3$	MPa	Deviator stress
σ_1	MPa	Major compression stress
σ_3	MPa	Minor compression stress
$\phi, \phi_{peak}, \phi_{res}$	Degree (°)	Friction, peak friction, residual friction angle

Symbols used in the determination of stability and deflection of a Voussoir beam

F_m	kPa	the maximum horizontal stress in the beam
F_{avg}	kPa	the average horizontal stress in the beam
F.S _{crushing}		Factor of safety against crushing
F.S _{sliding}		Factor of safety against sliding
L	m	Voussoir beam length
ΔL	m	Difference in Voussoir beam length
$(\Delta L - \Delta L_{previous} = 0)$	m	End Loop
N		the ratio of true to effective beam thickness
s	m	Span width
T	m	thickness
Z_0	m	the moment lever arm before deflection
Z	m	the moment lever arm after deflection
Zcheck	m	Calculated lever arm to check (negative value if failure)
$(Z_0 - Z)$	m	Midspan deflection
α	degree	Dip angle

CHAPTER 1

Introduction

1.1 Background

Jurong Formation is the second most widely distributed rock formation in Singapore which covers about a quarter of the surface area of the city state. The formation is well exposed in western and south-western parts of the country. It is composed of different varieties of sedimentary to meta-sedimentary rocks with minor amount of basic lava and volcanic ash. It was deposited and formed in Late Triassic to early Jurassic age. Currently, at Jurong Island, underground hydrocarbon storage caverns namely Jurong Rock Caverns Project (JRC Project) having capacity of about 1.47 million cubic meters have been excavated in this rock formation. It was also known that planning is underway for excavation of caverns for industrial and commercial usage in this formation at two different locations.

Although Jurong Formation covers about a quarter areas of the country, almost all surface rock outcrops had been covered by residential and industrial development projects since a few decades ago. In the JRC project, about three million cubic meters rocks have been excavated at about 100 to 135 m depth below existing ground surface. The linear length of excavation is about 18 kilometres. Hence it is worthy to study and document the engineering properties of those rocks from this project site.

The main thoughts and philosophy of the thesis are;

- Documentation of the new findings on geological setting and radiometric age dating result of Ayer Chawan Facies which contribute valuable information in Singapore Geology.
- Reanalysis of the engineering properties determined by laboratory test and field measurement.
- Analysis of the engineering geological properties of rock mass in the project.
- Determination of the rock mass geomechanical properties with input of field assessed parameters such as GSI and D.
- Establishment of prediction model for UCS and GSI by soft computing

method

- 2D and 3D numerical analysis with varying rock mass parameters and alternative designs
- Establishment of longitudinal displacement profile from 3D analysis for future reference
- Provide the knowledge about the stability and workability of unforeseen flat-roofed tunnel profile in the project after analysis by analytical, numerical and empirical approach

1.2 Objectives and scope of research

From the field observation, study, laboratory testing and numerical analysis on the first large scale underground excavation work in Singapore, the extensive engineering geological properties of rock masses of Ayer Chawan Facies, Jurong Formation have been recorded, analysed and documented as a reference for future similar works.

To accomplish the above objective, the following research approach was adopted.

1. Rock type classification by in situ rock specimen identification and petrographic thin section study
2. Determination of in situ (Field) physical and mechanical properties of intact rock and rock mass
 - Study and analysis of sedimentary structures such as bedding planes, joints sets, faults with their associated properties and to set up the geological model of the area
 - In situ rock stress magnitudes measured at site and its relation to regional tectonics structure and stability of underground excavation
 - Hydrogeological properties of rock mass analysed from water standpipes/piezometers and water bearing beds encountered at site
 - Determination of general rock mass quality incorporating the above properties
3. Engineering properties of intact rocks by laboratory testing

- Strength properties determination such as point load test, unconfined compression tests, triaxial compression test (c , ϕ values), Brazilian tensile test
 - Shear strength of natural rock joint by direct shear test
 - Permeability of intact rocks
4. Feasibility of Jurong Formation Rocks as a medium for underground excavation and construction
- Numerical software analysis for the stability of underground excavation in rock masses of Ayer Chawan Facies, Jurong Formation by applying the input parameters from above in situ properties and engineering properties from lab tests

To implement the above approach, the following research methodologies were applied;

1. In situ physical and mechanical properties of intact rock and rock mass
 - The measured sedimentary rock mass structures such as beddings, joints and fault planes in one selective cavern was analysed using DIPS program (Rocscience Inc.) to determine the general orientation of the area
 - Measured in situ stress was analysed and compared with the results measured in the other rock formation of Singapore, i.e. Bukit Timah Granite Formation to obtain the local stress magnitude
 - Ground water level and permeability characteristics of the rock mass was determined from the results of installed water standpipes and piezometers and seepage rate of water bearing beds at underground
 - The general rock mass quality was assessed based on the measured properties using Q and RMR system
 - Determination of quantitative Geological Strength Index (GSI) of rock mass
2. Engineering properties of intact rocks from laboratory tests
 - Strength properties were measured by point load method, uniaxial compression, triaxial compression and Brazilian tensile methods

- Direct shear test on rock joints of core samples
 - Permeability tests on rock samples
 - Soft computing and regression approaches to predict geomechanical properties of rocks
3. Feasibility and stability study of underground construction in rock masses of Ayer Chawan Facies, Jurong Formation
- Applying both intact and rock mass parameters, the stability of underground excavation with respect to different usages such as liquid hydrocarbon storage, underground power station construction, underground water reservoir construction was analysed with the following available numerical software,
- 2 Dimensional finite element geotechnical software - RS² (Rocscience Inc.)
 - 3 Dimensional finite element geotechnical software – RS³ (Rocscience Inc.)
 - 3 Dimensional software - Unwedge (Rocscience Inc.)

1.3 Organization of report

This report consists of ten chapters:

Chapter 1 of this report illustrates the motivation for research work on the engineering properties of rock masses which were excavated in large scale underground hydrocarbon storage project in terms of their feasibility for future underground excavation for similar scale projects, as well as covering the objectives and the scope of the research. Geologically the rock mass belongs to Ayer Chawan Facies of Jurong Formation.

Chapter 2 shall focus on relevant previous research and study on the Geology of Ayer Chawan Facies, Jurong Formation, in situ physical and mechanical properties of intact rocks and rock masses, engineering properties of formation and feasibility of underground cavern excavation in the formation. Those engineering properties measurements on site and laboratory works were carried out during the preliminary

and detailed feasibility studies of the proposed Jurong Rock Cavern Project between 2004 and 2007. 2D and 3D numerical analyses by previous works, Voussoir beam theory which applied in analytical solution and modified stability charts which used for empirical stability analysis of flat roofed caverns will be presented here. Geological and engineering geological properties of Singapore's igneous formation are also presented. The objective of the current research work by the candidate based on his working experience and observation in JRC project will be described in this chapter.

Chapter 3 will describe the geological setting of the area. The age of pyroclastic rocks yielded a U-Pb zircon age of 240.6 ± 1.2 Ma (Middle Triassic) as determined by Laser Ablation Inductively Couple Plasma Mass Spectrometer (LA ICP-MS). Based on the stratigraphic and sedimentological evidence of the presence of peperitic textures/structures, it is interpreted that the pyroclastic rocks were emplaced into the sediments during deposition which were still in a wet and unconsolidated condition.

Chapter 4 shall document the methodology on how to collect and record the geology data and field testing results during construction period, study and analysis of those data such as thin section study, permeability characteristics, in situ stress determination and rock mass quality. Results of rock strength determination on different types of rocks carried out by NTU research team are presented in this chapter. It will describe the calculated quantitative Geological Strength Index (GSI) from 420 tunnel rock faces in the project by four different approaches available in the literature. These GSI will be compared with the qualitative GSI based on assessment on the rock faces using the chart by [Hoek et al. \(1995\)](#).

Chapter 5 will present the determination of rock mass properties (Hoek-Brown criterion) based on the laboratory intact rock properties and properties of the rock mass such as discontinuities. Triaxial compression results of 84 core samples will be reanalyzed to compute intact uniaxial compressive strength (σ_{ci}) and Hoek-Brown constant (m_i). The Geological Strength Index (GSI) and disturbance factor (D) of rock mass were estimated on excavated tunnel faces. Finally the rock mass strength parameters (m_b , s and a) and deformation modulus (E_{rm}) were determined by applying well established equations.

Chapter 6 will present the application of soft computing and regression approaches to predict the geomechanical parameters of rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project. In this study, the Mamdani fuzzy algorithm was selected in prediction of unconfined compression strength and the Geological Strength Index (GSI) of sedimentary rock masses since it is perhaps the most appealing fuzzy method to employ in engineering geological problems. Those predicted results were compared with the results of statistical models using determination of coefficient of correlation (R^2) and root mean square error (RMSE).

Chapter 7 will present the engineering geological assessment of one of five storage caverns, namely CS 1/1, excavated in the JRC Project. In total up to 13 zones are differentiated based on possible wedge formation. The possible geological structures controlled failure mode on tunnel roof and walls will be studied by using Unwedge software with input of measured discontinuities orientation.

Chapter 8 will describe the feasibility and stability study of cavern excavation in rock masses of Jurong Formation. The equivalent rock mass parameters determined in chapter 5 will be used in 2D and 3D numerical finite element analyses for stability of twin storage caverns.

In Chapter 9, the stability analysis of the flat-roofed profile encountered within a horizontally bedded layered and vertically cross-jointed rock mass in JRC Project will be discussed. Calculated results between analytical approach using Voussoir beam algorithm, stability graph and numerical approach using RS² programme will be compared with results of convergence monitoring carried out in galleries.

Chapter 10 will conclude this report.

CHAPTER 2

Literature Review

2.1 Geology and Petrographic study of Jurong Formation

[Scrivenor \(1924\)](#) recorded on his 1:84,480 map that sandstone and shale with interbedded limestone underlie the west and south of Singapore. This map does not show the distribution of these rocks. The rocks were deposited in shallow water during Late Triassic times.

Later [Alexander \(1950\)](#) grouped the rocks that underlie the south and west Singapore as the Older Sedimentary Rocks. It consists of Older Schist, the argillaceous series and the arenaceous series. The distribution of these three groups of rocks is not shown on her 1:63360 geological map of Singapore.

In 1976, the Public Works Department (PWD) recognised within the Jurong Formation six facies, namely: Queenstown, Jong, Ayer Chawan, Rimau, St John and Tengah. Their distribution is shown on the 1:25,000 geological maps of Singapore by [PWD \(1976\)](#).

The Queenstown facies is mainly composed of red mudstone, the Jong facies of well-cemented conglomerates, the Ayer Chawan Facies of dark grey mudstones, the Rimau Facies of well cemented, loosely packed conglomerates and sandstones, the St. John Facies of pale grey mudstones and fine sandstones and Tengah Facies of deeply weathered poorly lithified sandstones. The rocks of Jurong Formation were deposited during Late Triassic to Early Jurassic times in a basin which was formed by uplift of the Bukit Timah Granite to the northeast and the Malayan Main Range Granite to the southwest.

Based on location and observation of lithology in the study area, the rocks belong to Ayer Chawan facies. It is characterised by tuffaceous debris, black sandstone, black mudstone and minor black to red conglomerate. Spilitic lava is present in this facies. The type section is defined as the west coast of Pulau Sakra in the south western group of islands ([PWD, 1976](#)). The engineering geology of Singapore was reviewed in [Pitts \(1984\)](#).

Subsequent to the feasibility study of cavern development in the Bukit Timah Granite Formation, Nanyang Technological University (NTU) and the former PWD joined together again to study the feasibility of underground cavern development in the sedimentary rocks of Jurong Formation. The study was sponsored by the National Science and Technology Board (NSTB). The feasibility study was conducted between 1995 and 1999 by the NTU-PWD joint project team comprising staff from both organizations, assisted by Ove Arup & Partners International and Norconsult International. The study included an extensive geological and geotechnical investigation of the Jurong Formation and engineering feasibility of cavern development in the sedimentary rocks.

The study examined (a) the geology of the Jurong Formation, (b) the engineering properties of sedimentary rocks, (c) location suitable for cavern development, (d) the technical feasibility of cavern construction and (e) the cost for cavern development in the formation. The investigation involved a total of 33 km long seismic reflection survey measurement and drilling of 17 deep site investigation boreholes.

The study reviewed the geology of the Jurong Formation by accounting the geological studies of the Formation from 1920s to the present study. The new geological model suggested by [Redding and Christensen \(1999\)](#) was discussed and compared with the PWD model. The differences between the models were highlighted in terms of the mechanism of formation, the timing of basin movement and the structural settings. The geological study was emphasized on the implications to rock cavern development in the area.

Due to the presence of various types of sedimentary rocks, the formation exhibits a wide range of engineering properties that are dependent upon the rock types, the extent of weathering and the degree of fracturing. Carbonate rocks and sandstones (including conglomerates) are generally good to fair in rock mass quality when they are fresh or slightly weathered and are generally good medium for cavern construction. Mud rocks are, on the other hand, of poor quality and therefore not suitable for cavern construction.

In 2009 Defence Science and Technology Agency (DSTA) in collaboration with NTU and Building Construction Authority (BCA) published the second edition of Geology of Singapore. They updated the observation and finding after PWD's first edition in 1976. [Lim \(1974\)](#) produced five detailed sections of sedimentary rocks and discussed the petrology he recognised in south western part of Jurong area. Those sections are from Ayer Chawan Facies with inter-beds of the Tengah Facies. The sandstone usually comprises a fine to medium-grained sand made up of quartz with a significant amount of polycrystalline quartz grains, tuffaceous clasts, secondary chalcedony and chert. He described on the presence of carbonaceous matter in mudstone. He noted boulders of spilite about 400 m north of Selat Pulau Damar. Earlier [Chin \(1965\)](#) recorded the presence of a volcanic breccia on Pulau Samulun and [Alexander \(1950\)](#) recorded lavas associated with chert at Jurong Area and Pulau Sakra. Those volcanic rocks are included in Ayer Chawan Facies.

It can be concluded that Geology of Singapore in two editions were on main Singapore Island since the study area of JRC Project was still in offshore zone.

Later, the reclamation project began in 1993 to merge seven southwestern islands - Pulau Merlimau, Pulau Ayer Chawan, Pulau Ayer Merbau, Pulau Seraya, Pulau Sakra, Pulau Pesek and Pulau Pesek Kechil - into one that came to be called Jurong Island as in [Fig. 2.1](#).

Between 2001 and 2007, three phases of extensive site investigation works were carried out at Selat Banyan basin under JTC Corporation as a developer for storage rock caverns ([Tritech, 2004, 2007a, 2007b](#)) as shown in [Fig 2.2 a & b](#). It comprised of vertical and sub-horizontal bore holes drilling, in situ testing such as permeability tests, horizontal stress measurement and geophysical surveys works.

[Zhao et al., 2004](#) carried out the feasibility study based on preliminary site investigation results and proposed for the construction of underground storage caverns in sedimentary rocks of Jurong Formation at Jurong Island.

The project consultants identified and classified two main genetic rock types, namely sedimentary and plutonic rocks, in their interpretation report ([Geostock, 2007](#)) based on the site investigation works.

Fig. 2.1 Over view plan of reclamation phases to form Jurong Island ([Tritech 2007a](#))

Fig. 2.2 a – Location of JRC project, **Fig. 2.2 b** – Location of JRC project plan, [Wong, et al. \(2009\)](#)

2.1.1 Sedimentary rocks of Ayer Chawan Facies, Jurong Formation in JRC Project

By macroscopic observation, the rocks are identified as mudstone, siltstone, sandstone and micro-conglomerate ([Geostock, 2007](#)).

Mudstone is a very fine-grained rock, often dark and homogeneous, made of dark black cement. Mud is predominantly made of clay, with a variable amount of

carbonate and fine-grained volcanic debris. Mudstone appears mostly without any internal structures and layers cannot be easily distinguished. Mudstone layers have a wide range of thickness, from very thin centimetric layers to thick metric accumulation.

As compared to mudstone, siltstone is the same rock type but grain size is a bit coarser. Layers of siltstone are generally thin, ranging from a few millimetres to a few decimetres.

Sandstone is a quartz-rich rock, often containing feldspar and/or volcanic debris. Elements are often rounded and well sorted. Quartz grains have a homogeneous size with the cementing materials between grains consists of clay minerals.

Micro-conglomerate is coarse-grained sandstone including rounded or angular pebbles. The size of these elements being small (generally from a few millimetres up to half a centimetre) the term “micro” has been added to conglomerate. Elements are volcanic rocks, minerals (such as feldspar) or limestone.

In addition to these standard facies, a specific rock type often called “limestone” is observed in cores. This rock is a conglomeratic rock composed of limestone fragments “floating” into the cement made of black mudstone. Limestone fragments are generally big enough to be easily visible (generally 5 to 10 cm; up to 15 cm) and angular (very few sub-angular pieces are found). The origin of limestone is either shallow marine (very few fossils such as coral were observed) or continental (lacustrine limestone and calcrete).

Two types of elements incorporated into the sedimentary layers can be distinguished, based on their origin, distant or local:

- Fragments of distant origin observed within micro-conglomerate and limestone indicate that the corresponding rocks have origins outside the sedimentary basin: they are mainly volcanic fragments and locally marine and continental limestone.
- Fragments of local origin may also be included in sedimentary rocks: they are essentially mud pebbles, representing residual parts of eroded layers,

incorporated at the base of new overlying ones. The shape and deformation of these pebbles confirm that eroded mud was not yet lithified but simply compacted.

These observations are essential for analysing and reconstructing the depositional environment (paleo environment) of Selat Banyan.

Microscopic observation performed on the thin sections provides a great help for determining rock type. The main finding is that all selected samples show rocks composed of two different parts ([Geostock, 2007](#));

- Terrigenous sediments (eroded material derived from land): sediments with various grain size and essentially containing quartz plus some feldspar and a variable amount of clay; quartz is more frequent in coarse-grained facies (sandstone and siltstone) and clay is more represented in mudstone facies. Clay minerals are hydro-micas and sericite, with some very fine calcite.
- Tephra (fragmental volcanic products): the volcanic debris and ash contain feldspar, quartz (often devitrified) and some biotite.

The amount of terrigenous sediments versus tephra is generally equal but the first group can be very high up to 90% of the total amount observed in some boreholes. The volcanic origin of Jurong rocks at Selat Bayan is therefore clearly evidenced. Thus strictly speaking, the rock has to be named tuffite or at least tuffaceous or volcanic mudstone.

Some cracks in the samples show a filling with minerals having a metamorphic origin such as zeolite, chlorite, epidote and calcite.

One limestone sample was studied. By thin section study, it reveals the existence of echinoderm and hemibranchiate bioclasts (fossil fragments) which confirms a shallow marine origin ([Geostock, 2007](#)). The cement of the rock matrix is made of calcite with a small quantity of charcoal and detrital minerals, which reveals a detrital origin and the proximity of the continent.

2.1.2 Plutonic rocks of Ayer Chawan Facies, Jurong Formation in JRC Project

Igneous rock types are less frequent than sedimentary rocks, both in spatial distribution and petrographic types in the study area.

Igneous rocks are only found in dykes and sills. One main dyke plus several sills are clearly observed at Selat Banyan. The dyke is more or less oriented in the axis of the Banyan basin.

The dyke is made of acid material (quartz rich). When fresh, it has a whitish to pure white colour, composed mainly of quartz and feldspar with minor amount of ferromagnesian minerals and pyrite (FeS_2). The most common petrographic type observed on core samples is micro-granite with minor amount of aplite.

The main dyke observed at site is a thick and long feature as;

- Thickness ranges from 15 to 25 meters
- Length is more than 1 kilometre
- The dyke is cut by faults in several parts
- The attitude is almost vertical (steep angle, estimated at 81 degree)

Dyke represents sub-vertical features while sills are sub-horizontal ones. Thin sills are observed on cores, thickness is generally centimetric. They are more frequent when close to the dyke, confirming that sills are originating in the dykes and intrude the surrounding sedimentary rock mass along pre-existing discontinuities such as the bedding planes.

2.1.3 Metamorphism of Ayer Chawan Facies, Jurong Formation in JRC Project

Two types of metamorphism are evidenced on rocks of the site, namely regional metamorphism and contact metamorphism essentially due to dyke and sill intrusion.

2.1.3.1 Regional metamorphism

The main result of metamorphic process is a hardening of the rocks. This is clearly evidenced by the results of rock mechanical laboratory tests for sandstone and mudstone. In parallel the behaviour of the rock becomes more brittle.

Without this general metamorphism, it might not have been possible to consider excavating underground caverns with such large sections in similar rock types.

2.1.3.2 Contact metamorphism

In addition to general metamorphism, localised metamorphism is also found. This type is restricted to rocks in contact with or close to dykes and sills.

During dyke intrusion, acid and dyke material was fluid enough to impregnate and/or intrude surrounding rocks. Close to dykes, the most porous rock types show impregnation with whitish material composed of silica and pyrite. In addition, intrusions into small fractures are also visible. As similar effects by regional metamorphism, contact metamorphism also provides hardening of the rock and homogenization of different facies.

2.1.4 Petrographic study of Ayer Chawan Facies, Jurong Formation in JRC Project

Petrographic thin section analysis has been conducted on a number of JRC rocks. Selected core samples were studied at the Beijing University of Science and Technology (China). The objective is to describe and identify the rocks. Some of thin section analysis are shown in [Fig. 2.3 a to c](#).

2.2 In situ physical and mechanical properties of intact and rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

2.2.1 Study and analysis of sedimentary structures,

In their interpretation report ([Geostock, 2007](#)), the local structure of Jurong Formation at Selat Banyan can be described as sedimentary layers locally cut by dykes as in [Fig. 2.4](#).

Fig. 2.3.a Photomicrograph of specimen Borehole VBH 4-2 CR 62 Crystal and vitric fragment Tuff (Left – Plane Polarised Light, PPL; Right – Cross Polarise Light, XPL), [Tritech \(2007a\)](#)

Fig. 2.3.b Photomicrograph of specimen Borehole VBH 4-3 CR 25 Gravel contained Tuffite (L – PPL; R – XPL) [Tritech \(2007a\)](#)

Fig. 2.3.c Photomicrograph of specimen Borehole VBH 4-3 CR 68 Lithic fragment contained andesite Tuff (L & R – XPL) [Tritech \(2007a\)](#)

GEOLOGICAL MAP OF BANYAN BASIN

Fig. 2.4 Geological Map of Banyan Basin (Geostock, 2007)

The main acid dyke and the bedding planes of the sedimentary layers were used as two markers for assessing the global structure. More in detail, several attempts were made in order to clarify this arrangement in particular regarding:

- The attitude of each part of the main dyke
- The attitude of the sedimentary layers
- The identification of main sedimentary units, if any.

To reach the above objective, the following data were used;

- Observation of cores with measurements, essentially dip angle since dip direction cannot be estimated (except oriented cores)
- Analysis of Bore Hole Televier (BHTV) results: measurement of joints, bedding planes and contacts including dip direction and dip angle and correlation with core logs
- Analysis of seismic surveys and in particular the last marine refraction survey

2.2.1.1 The main acid dyke

The main acid dyke has a global NW-SE trend. In the study area, five segments of it cut by fault have been identified. Depending on the considered segment, the attitude of the faults ranges from N110°E to N135°E for the azimuth and from 70° to 85° towards the SW for the dip angle.

Considering the dyke was initially a unique segment, this confirms the movements induced by the faults, including a combination of rotation in the horizontal direction as well as the vertical one.

2.2.1.2 Structure of the sedimentary layers

The structure appears as the alternation of sedimentary layers lying almost horizontally. Dip angles of the bedding planes are relatively low, ranging in undisturbed areas from 0° to 25°. However, in some areas, bedding planes have been measured with higher dip angles (45° to 65°)

From the BHTV measurements as well as observation of cores, the following points are concluded (Geostock, 2007);

- Sedimentary layers had initially relatively low, sub-horizontal dip angles.
- Higher dipping angles of bedding planes do not correspond to initial conditions: such angles are due to rotation and/or tilting of blocks
- Apart from this, no major structure was identified and there is no obvious parameter allowing to group together sedimentary layers into main units nor to correlate them at the scale of the site.

2.2.1.3 Fault identification and its characteristics

Fig. 2.5 Example of fault limit assessment using BHTV images, (Geostock, 2007)

Several faults, faulted zones or brecciated zones have been logged on core samples. To start the analysis of tectonic conditions and try to locate and address movement of faults, a “marker” was needed. Since no sedimentary marker was found at the scale of the site area, the only “continuous” feature was the dyke crossed by several boreholes on site. Five parts were identified, displaced by faults, essentially with the right lateral (= clockwise) strike-slip movement.

BHTV provided very accurate images and easy to treat measurements, which allowed identifying faults as well as correlating them from hole to hole. As in Fig. 2.5, a fault was identified using striations and taking into account higher density of joints. Three main fault trends were identified as shown below;

Table 2.1: Orientation of identified fault sets (Geostock, 2007)

Fault sets	Orientation		Remarks
	Dip angle	Dip direction	
1	70 - 90	190 – 200	Main set affecting dyke, clockwise movement
2	70 – 85	70 – 80	NA
3	0 - 15	NA	Associated with brecciated quartz and calcite

2.2.1.4 Discontinuities

Apart from faults, two types of discontinuities are found; bedding planes and joints. Bedding planes represent potential discontinuities since they are mostly closed but can be broken during coring for example. Joints exist in all the rock types, sedimentary and plutonic;

- In the sedimentary rocks, fractures cross the layers (and thus the bedding planes). They are generally relatively undulated and rough except when the material is affected by contact metamorphism (close to dykes). Joints can be coated and slightly filled with various minerals, mainly calcite but also chlorite, epidote. In addition, tension cracks, systematically sealed with calcite, can also be found.
- Inside the acid dyke, the existing joints are interpreted as the results of cooling down of the material after setting. In the great majority, such fractures are rough and without coating / filling.

Around 7500 discontinuities recorded from BHTV and core orientation were treated and analysed using stereographic projection.

2.2.2 In-situ rock stress magnitudes

In-situ stress measurement was conducted to determine actual stress regime, e.g. the magnitude of horizontal and vertical stress components as well as their orientations. In addition, vertical gradient is identified in the range of depth of the future caverns.

A series of 10 hydraulic injection tests was conducted in the uncased section of the marine borehole no. VBH 4-10 (Fig. 2.6) between 99 m and 184.4 m depth below barge floor (corresponding depth below sea floor: 81 m – 166.4 m). The in-situ tests were carried out by using the MeSy wireline technique to move the hydro fracturing and impression packer unit PERFRAC II within the 76mm diameter borehole section (Tritech 2007a).

Although all of the tests showed typical hydraulic fracturing test features such as pronounced breakdown – (fracture initiation) events and distinct fracture re-opening and shut-in phases, the impression packer tests showed that fractures with different spatial orientation were induced or stimulated. Therefore, the in-situ stress analysis was conducted on the basis of the Hydraulic fracturing on pre-existing fractures (HTPF) – or Shut-in Pressure (P_{si}) – method using the MeSy – code SESAM. The results of the inversion calculations can be summarised by the following stress – depth relations between 81 m and 167 m below sea floor.

$$\sigma_h \text{ [MPa]} = (2.6 \pm 0.3) + (0.026 \pm 0.003) \times (z \text{ [m]} - 81) \quad (2.1)$$

$$\sigma_H \text{ [MPa]} = (4.4 \pm 0.2) + (0.049 \pm 0.001) \times (z \text{ [m]} - 81) \quad (2.2)$$

Where σ_h and σ_H are the minimum and maximum horizontal stresses (in MPa) and z is the depth below sea floor (in m).

The vertical stress σ_v was calculated for an average sea level of 15 m, and mean densities of 2.0 g/cm³ for the marine soils to 45 m below barge floor and 2.5 g/cm³ for rock mass below:

$$\sigma_v \text{ [MPa]} = 0.7 + 0.0245 \times (z \text{ [m]} - 45) \quad (2.3)$$

The direction of the maximum horizontal stress σ_H was determined as N 10° ± 3° (NNE – SSW).

Fig. 2.6 Location of borehole V BH 4-10 at Banyan Basin, Jurong Islands ([Tritech, 2007a](#))

2.2.3 Hydrogeological properties of rock mass

In-situ hydrogeological testing from 3 phases of site investigation (Tritech, 2007a) showed that there is no correlation between depth and hydraulic conductivities (for example, there is no permeability reduction with increasing depth). 107 short duration packer tests were performed on small intervals (about 11.7 m interval). The analysis allowed correlating hydrogeological test results and geological characteristics of the tested interval and establishing the geological and hydrogeological model. Laboratory tests have determined that the hydraulic conductivity of the matrix was in the 10^{-11} m/sec to 5×10^{-9} m/sec range and long duration packer tests have confirmed that the global hydraulic behaviour of the rock mass was related to the different geological features and jointing networks.

Long duration tests indicated large scale heterogeneous hydraulic conductivity distribution, which suggests that large zones with very different hydrogeological characteristics may be encountered during cavern excavation works. Representative hydraulic conductivities were determined for each type of geological class defined in the geological and hydrogeological model and are summarised in Table 2.2.

Table 2.2 Representative values for hydraulic conductivities of the different geological classes (Geostock, 2007)

	Geological class		Representative hydraulic conductivity (m/sec)	Percentage of tests performed per facies (total 107 tests)
	Main class	Sub-class		
Fresh bedrock	Weathered	Weathered	5×10^{-5}	9%
	Low confined zone	LCZ	4×10^{-6}	6%
		Low K sedimentary	8×10^{-10}	32.5%
	Untectonized bedrock	High K sedimentary	2×10^{-6}	5.5%
	Metamorphosed bedrock		3×10^{-9}	15%
	Tectonized bedrock	Dyke	4×10^{-7}	2%
		Fractured wall of dyke	4×10^{-6}	25%
		Faulted (H)	1×10^{-6}	7%
		Faulted (V)	4×10^{-6}	21%

Horizontal hydraulic conductivities have been measured for 6 plugs, one plug having its horizontal and vertical hydraulic conductivity measured. The six plugs belong to the sedimentary class. It appears that the permeability of the matrix ranging between 4×10^{-11} m/sec and 10^{-10} m/sec, except for the plug tested for both the vertical and horizontal hydraulic conductivities (6H and 6V), which exhibits higher values. The higher hydraulic conductivities measured for plug no. 6 is due to the presence of alternation of thin layers of siltstone, mudstone and sandstone whereas the other plugs were located in homogeneous facies.

The range of hydraulic conductivity found for the matrix is comparable to the range of hydraulic conductivity values measured for the “low K sedimentary” class. This comparison suggests that the “low K sedimentary” class hydraulic conductivities measured during the in-situ tests might be representative of the matrix whereas the “high K sedimentary” class might be representative of micro-fracturing in the matrix. In any case, matrix hydraulic conductivity is low and therefore water flow is determined by the jointing network, either at the local scale or at larger scale represented by the fractures identified by the geological survey.

Local scale anisotropy was measured on one plug sampled on VBH 4-3a for which the lithology corresponds to an alternation of thin layers of siltstone, mudstone and sandstone. Horizontal hydraulic conductivity k_h was found to be higher than the vertical one k_v ($k_h = 1.7 \times k_v$). This result indicates that the local anisotropy is not very significant but cannot be considered as representative since it is based on one test only.

2.2.4 Determination of general rock mass quality

2.2.4.1 Method used

The strength measured on rock core samples only represents the rock matrix property only. In order to estimate the rock mass behaviour, the following parameters have to be taken into account:

- Joint set characteristics
- Water flow conditions
- Features such as stress regime and existence of broken zones

Various empirical methods are available for the calculation of large scale physical parameters. Often used is the one proposed by [Hoek \(2007\)](#), for which GSI (the Geological Strength Index) values are combined with measurements on samples mainly to obtain:

- The rock mass Unconfined Compressive Strength σ_{rm}
- The rock mass tensile strength σ_{trm}
- The rock mass yield criterion to describe the limit between elastic behaviour and post-failure behaviour (c_{rm} , ϕ_{rm})
- The rock mass elastic modulus of deformation, E_{rm}

The Geological Strength Index is determined using standard rock mass classification system. Rock Mass Rating proposed by [Bieniawski \(1989\)](#) and Q index proposed by [Barton et al. \(1974\)](#) has been used for this study. For both systems, low values correspond to poor rock conditions. It should be noted that the Q system is a logarithmic scale system whereas the RMR value varies linearly between 0 and 100.

Correlation relationship to link RMR and Q is proposed in the literature. However, caution is needed when using such correlation between empirical methods. A better way is to perform RMR and Q evaluation independently making a comparison of the rock mass quality given by both methods.

Two levels of description are proposed:

- Qualitative identification of the rock mass according to both systems mentioned above
- Evaluation of rock mass mechanical characteristics (i.e. UCS, Young's modulus, yield criterion) by combining values of characteristics measured on samples with the obtained indices. These rock mass characteristics can then be manipulated in models.

2.2.4.2 Results

As mentioned earlier, rock matrix of Jurong Formation rocks at study area is fairly strong, regardless of the geological facies, compared to standard sedimentary rocks

(Geostock, 2007). The rock mass quality, in terms of geotechnical engineering, will be mainly related to the degree of fracturing. Geotechnical classes will therefore be defined by combining rock matrix with fracturation degree and joint characteristics.

The following Q and RMR assessments have not been established along boreholes but were derived from the geological observations on cores, synthesized on determined rock classes (Geostock, 2007):

- The good competency of the intact rock plays a significant role in the RMR whereas joint condition greatly prevails in the Q index.
- GSI is calculated from the RMR index using the following expression:

$$GSI = RMR' - 5 \quad (2.4)$$

With $RMR' = RMR$ considering adjustment for joint orientation set to 0 and ground water rating set to 15.

- GSI is also calculated from the Q index using the following expression:

$$GSI = 9 \log_e Q' + 44 \quad (2.5)$$

$$\text{where } Q' = Q \times \frac{SRF}{JW} = \left(\frac{RQD}{J_n} \right) \times \left(\frac{J_r}{J_a} \right)$$

2.3 Engineering properties of Ayer Chawan Facies, Jurong Formation in JRC Project

2.3.1 Strength properties of intact rocks

An extensive laboratory program has been carried out to quantify the physical parameters and rock mechanical characteristics of matrix material. Five rock classes plus weathered material were classified (Geostock, 2007).

These classes are the followings:

- Coarse grained dominant: this class corresponds to coarser sedimentary rocks, essentially micro-conglomerate and sandstone,
- Complete sequences: this rock class corresponds to thicker sequences,

- Fine grained dominant: this rock class consists of finer sedimentary rock types such as fine grained sandstone, siltstone with locally mudstone,
- Mudstone and limestone: this specific rock class corresponds to thick mudstone sequences, including those having limestone fragments incorporated into the mudstone,
- Acid dyke: this rock mechanical class corresponds strictly to the geological rock type
- Weathered: this class integrates all data from weathered material,

Mechanical strength tests ([Tritech, 2007a](#)) include:

- Brazilian tests for indirect tensile strength evaluation
- Uniaxial compressive tests for evaluation of the uniaxial compressive strength and elastic parameters,
- Triaxial tests for estimation of the Mohr-Coulomb failure criterion and elastic deformation parameters

2.3.1.1 Brazilian tests for tensile strength evaluation

A total of 48 Brazilian tests for tensile strength determination were carried out from selected core samples from 3 vertical boreholes ([Tritech, 2007a](#)). The histogram of tensile strength results of different rock types is shown in [Fig. 2.7](#).

In the histogram in [Fig. 2.7](#), the numbers in rock type groups represent the density of samples which fall in that strength range. Except for weathered rock which shows a specific population, the wide ranges of tensile strength in other rocks were observed and did not really relate to types of rock.

Fig. 2.7 Histogram of tensile strength values (Brazilian test) (Geostock, 2007)

2.3.1.2 Uniaxial compression tests

A total of 47 uniaxial compression and deformation tests were carried out from selected core samples from 3 vertical boreholes (Tritech, 2007a).

The photograph of core sample (CR 12 of VBH 4-1) with failure plane before and after uniaxial compression testing was shown in Fig. 2.8. The histogram of uniaxial compression test results of different rock types is shown in Fig. 2.9.

In the histogram, the numbers in rock type groups represent the density of samples which fall in that strength range. Except for weathered rock which shows a narrow population range, the wide ranges of uniaxial compressive strength in other rock types are observed and do not really relate to types of rock.

Fig. 2.8 Borehole VBH 4-1 CR 12 sample before and after UCS testing (Tritech, 2007a)

Fig. 2.9 Histogram of uniaxial compressive strength values (Geostock, 2007)

2.3.1.3 Triaxial compression test

A total of 39 triaxial compression and deformation tests were carried out on selected core samples from 3 vertical boreholes (Tritech, 2007a).

The applied minor compression stress (σ_3) and major compression stress (σ_1) and calculated Mohr-Coulomb failure criteria of cohesion (c), friction angle (ϕ), elastic modulus (E) from one of the testing is shown in Table 2.3.

Table 2.3 Triaxial compression test and deformation test (VBH 4-1, CR 15)
(Tritech, 2007a)

BH Ref.	Core run ref.	Depth	Rock description	Sample No.	Die-meter (mm)	Height (mm)	σ_3 MPa	$\sigma_1 - \sigma_3$ MPa	σ_1 MPa	E GA
VBH 4-1	CR 15	75.9-77.5	Moderately to slightly weathered fine SNDSTONE	151-1	50.38	101.73	2	42.88	44.88	78.67
				153-1	50.47	102.13	2	60.48	62.48	80.03
				151-2	50.34	101.95	6	68.60	74.60	39.74
				154-1	50.36	101.68	10	124.61	134.61	58.92
				152-2	50.35	79.01	20	175.07	195.07	59.71
Cohesion (c) = 6.57 MPa and Friction angle $\phi = 51.30^\circ$										

The photograph of 5 pieces of core samples (CR 15) with failure plane after testing, marked with white paint, was shown in Fig. 2.10. The histogram of calculated deformation parameter (elastic or Young's modulus, E) values of different rock types is shown in Fig. 2.11. The elastic modulus values of all rock types are observed between 30-80 GPa.

In the histogram in Fig. 2.11, the numbers in rock type groups represent the density of samples which fall in that strength range. Except in acid dyke, the rest groups show wide ranges of modulus values.

Fig. 2.10 Core samples with failure planes after triaxial compression test (Tritech, 2007a)

Fig. 2.11 Histogram of Young modulus values (Geostock, 2007)

2.3.2 Shear strength of rock joints

There was no report on the shear strength measurement of rock joints in previous site investigation work.

2.4 Numerical analyses in underground rock excavation

Numerical simulation for excavation in rock have been performed by many researchers. Under the research program between JTC Corporation and Nanyang Technological University (Contract: JTC C0550), both two dimensional and three dimensional numerical analyses were carried out using the distinct element programs UDEC and 3DEC ([Chua et al., 2009](#) & [NTU, 2013](#)) to investigate the various factors that influence the stresses and displacements around underground caverns constructed in the sedimentary rocks of the Jurong Formation of Singapore in JRC Project. The Mohr-Coulomb failure criterion was assumed in all the analyses. Sequential construction of the rock cavern with initial top heading with subsequent benching procedures was considered. Both continuous and discontinuous numerical modellings were carried out. Those intact rock properties were taken from the results of feasibility study before the project commenced.

In continuous modelling, the rock mass is treated as a continuous medium and only very limited number of joints/faults were considered. The critical input parameters are the strength and deformation properties of the rock mass. The geotechnical properties of the equivalent homogenous rock mass were typically estimated using [Bieniawski's \(1984\)](#) empirical equation. The input parameters were calculated based on the tunnel quality index (Q) values mapped inside one storage cavern. The continuous modelling was able to give detailed information about the stresses and deformations, and the forces in the rock bolts. Compared to discontinuous modelling, the computational time is very fast. In discontinuous modelling, the rock mass is modelled as a system of individual rock blocks interacting along their boundaries.

As the underground storage facility involved multiple caverns being constructed in close proximity to each other, analyses were also performed to ensure minimal

interaction effects of adjacent cavern construction on the existing cavern as repair of any damage to the existing cavern is both disruptive and expensive.

[Chua et al., 2009](#) & [NTU, 2013](#) carried out parametric study on the effects of rock support parameters, excavation sequences, fault location and orientation with respect to cavern, discontinuities parameters and late installation of rock supports. They concluded that the interaction between rock masses surrounding the cavern and the rock support system affects the stresses and displacements induced around the cavern. In addition, the stresses and displacements that develop around the cavern are also significantly influenced by the support installation, properties of the rock mass and joints as well as the in-situ stresses.

[Mandal et al., 2013](#) performed two-dimensional (2D) elastoplastic analysis using the finite- element code RS² to study the rock mass behaviour caused by staged excavation of the cavern (20 m wide 33 m high) in India. The excavation of the cavern is simulated with different stages, i.e., excavation of a pilot tunnel and subsequent side slashing followed by excavation for a pilot bench and subsequent side slashing with a total of three benches. Displacement values obtained analytically are compared with the observed displacement as the excavation progresses. Because the 2D FEM code is based on the plain strain concept, it could not take into account the effect of excavation in other directions that is very obvious at the intersection with tunnels and shafts. Therefore, these intersections were analyzed using a linear three-dimensional (3D) boundary element code, Examine3D. The stability analysis of rock wedges formed from various joint patterns is also studied using the UnWedge software based on block theory for each section of the cavern.

They observed that almost 90% of the vertical displacement occurred at the crown during heading excavation, and this displacement remained almost unchanged during Bench 2 and Bench 3 excavations, although there is a significant increase in the horizontal wall movement during bench excavation. Predicted values of displacements are generally higher than the observed values for poor rock conditions.

[Nie et al., 2018](#) studied and proposed to evaluate the support design based on rock cavern performance. They applied rock mass parameters estimated

using Bieniawski's (1984) empirical equation as in analysis by Chua et al., 2009 & NTU, 2013. 2D plane strain models are adopted to simulate the excavation effects of horse shoe cross-section rock caverns using the progressive core replacement method. The performance of rock cavern is investigated using CCM. Parametric studies are carried out to analyze the effects of rock condition and sequential excavation. It shows the roof displacement is not changing significantly during excavation if $Q > 10$. It also presents the sequential excavation can reduce the range of the Excavation Damage Zone (EDZ), but there are no obvious relationships for different subdivision methods. Using the data from numerical analysis, the relationship, the relationships among the rock conditions, the sequential excavation parameters and the cavern performances are mapped using artificial neural network (ANN). An evaluation chart for the support design of a rock cavern is proposed by integrating the ANN models into EXCEL software (Nie et al., 2018). A case study is presented to verify the accuracy of the proposed method. It illustrates the feasibility of the proposed approach for practical applications with much less computing time.

As a full 3D numerical analysis is usually too time-consuming for the preliminary support design, the suitable 2D finite element (FEM) plane strain models are often used to consider the 3D excavation effects (Cai, 2008). The key difficulty in the application of the 2D model is how to determine the correspondence between the simulation stage and distance from the excavation face (Alejano et al., 2010). To solve this problem, an improved Longitudinal Displacement Profile (LDP) has been proposed by Vlachopoulos and Diederichs (2009).

2.4.1 3D numerical analysis in underground rock excavation

During tunnel and cavern excavations, the deformation of the rock mass caused by the unloading of itself will vary with time. In the past few decades, the radial shape of the displacements around the tunnel face has been estimated. In the mid-1970s, numerical methods were used to analyse the displacement around the tunnel face, thereby obtaining

valuable information on the stability of the tunnel (Ranken and Ghaboussi, 1975; Hocking, 1976). From the 1980s, monitoring measurement has been extensively used to assess the response of the rock mass based on the field measurements of the rock mass deformation using special instruments (e.g., tap extensometer and precision level) (Sakurai and Takeuchi, 1983; Gioda and Sakurai, 2010). The measured results can reflect the behaviour of the rock mass and supporting structures, as well as evaluation of the stability of the tunnel structures (Luo et al., 2016). However, the measured displacement of the rock mass is only a part of the complete deformation. Complete deformation includes those before, during, and after the tunnel excavation. Hoek (1998) used a three-dimensional finite element method (FEM) to analyze the rock mass deformation that surrounds a circular tunnel advancing through a weak rock mass, thereby showing a deformation pattern in the rock mass that surrounds an advancing tunnel. Carranza-Torres and Fairhurst (2000) established a longitudinal deformation profile (LDP) to analyze and graph the radial displacements along the axis of the tunnel in sections located ahead and behind the tunnel face (Fig. 2.12).

With respect to distance from the tunnel face, LDP includes three components: (1) Y_1 , pre-deformation, (2) Y_2 , loss displacement, which cannot be measured before the installation of the monitoring instrument, and (3) Y_3 , measured displacement. LDP for a tunnel is important to determine the appropriate timing for the installation of support or to optimize the installation of support with specific displacement capacity (Song et al., 2016).

Two methods are used to obtain LDP, namely, field measurement and numerical simulation. Many attempts were used to measure LDP in the field, however, the results were incomplete because the monitoring measurement is difficult to obtain particularly in deeply buried tunnels. Therefore numerical simulation is extensively used with the development of computer technology. Numerical simulation includes

finite element method (FEM), finite difference method and discrete element method (Jiao et al., 2005).

Fig. 2. The radial displacements along the axis of the tunnel in sections located ahead and behind the tunnel face.

in sections located ahead and behind the tunnel face (Carranza-Torres and Fairhurst, 2000)

However, the material parameters of the ground are difficult to determine in a numerical simulation. The material properties are affected by the inhomogeneous nature of the soil mass or rock mass, and the influence of complicated geological factors (Liang et al., 2003). Alternatively, the back analysis method was first introduced to obtain the actual material parameters during tunnel construction in 1980s (Cividini et al., 1981). Luo et al., 2018 studied on a weak rock mass tunnel in the Mingyazi tunnel in Shaanxi Province of China to measure the tunnel displacement at site, including the crown settlement (vertical displacement) and horizontal displacement. The LDP of this tunnel was analyzed by numerical simulation using the FLAC

(ITASCA Inc.). Back analysis method was used to obtain the rock mass parameters utilizing the measured data in a weak rock mass tunnel.

Fig. 2.13 Longitudinal displacement profile (LDP) of JRC (Lu et al., 2012)

Lu et al. (2012) studied on JRC project and proposed LDP applying FLAC 3D code (Fig. 2.13). About 75% ($u/u_{\max}$) of total displacement was observed at 2 m behind the excavated rock face. It represents Y2 (loss displacement) as mentioned in Fig. 2.12.

2.5 Flat-roofed cavern design

Underground excavations are usually designed with an arched roof of either a horseshoe or near the circular profile. These types of roof are classically used in the design of wide span caverns. They provide optimal uniform stress distributions around the excavation which often result in a more stable condition. However, an arched roof design is occasionally not optimal in some geological settings consisting of low-angle bedded sedimentary rocks, horizontally-foliated metamorphic rocks and

horizontal jointed and fractured rocks. In cases of heterogeneous materials, in situ stress distributions can become particularly complex as the stresses are not able to transfer uniformly around the excavation, and will often concentrate within the layers, units or at discontinuities surrounding the excavation.

Even after adopting an arched roof design in such layered ground conditions, auxiliary support system such as reinforced concrete materials or steel structures is required to prevent roof fall-out. In many cases this involves the design of a full spanned flat-roofed cavern, where the rock mass behaves as a natural structural beam in the roof of the cavern (Diederichs and Kaiser, 1999; Crockford and Diederichs, 2012).

Nowadays, it is becoming more common to adopt a flat-roofed design in large span excavations in sedimentary rocks. Due to the favorable geological setting in Australia, flat-roofed large span caverns have become a widely accepted design option. Perhaps the most well-known example in Australia is the underground parking structure at the Sydney Opera House as shown in Fig. 2.14 (Pells et al., 1994); though the design method has been applied in several other underground transit infrastructure projects as well (Pells, 1994 & 2002).

The impact of the arch profile geometry in comparison to the flat-roofed profile within a layered and cross-jointed rock mass is shown in Fig. 2.15.

To maintain the arched geometry roof, the rock materials from the sub-horizontal layers are excavated, thus reducing the capacity for natural arching. By removing the central portion of a layer within the roof (the midspan), two cantilever beams form in the sides of the roof which, in a cross-jointed rock mass, become unstable and potentially lead to partial roof collapse (Pells et al., 1994). By designing a flat-roofed cavern, fewer layers are disturbed by the excavation, and the inherent strength of the rock material can be exploited.

In addition to reducing excessive stress concentrations around the roof, a flat-roofed cavern minimizes the extent to which layers are weakened by the removal of materials from the center of the cavern's crown. Where a final pressure-resistant lining is not required, a combination of rock bolts and shotcrete are sufficient for a

large span excavation with a flat roof, which represents a significant reduction in support installation and cost requirements ([Bertuzzi and Pells, 2002](#)).

Fig. 2.14 Double deck tunnel of 3 km long Eastern Distributor with 24 m wide span (Sydney Opera House underground car park, [Pells, et al., 1994](#))

Fig. 2.15 Comparison of two cavern profile geometries: arch-roofed and flat-roofed. The shaded areas indicate the rock mass which becomes unstable due to cavern excavation (Pells et al., 1994, Crockford and Diederichs, 2012)

An optimal design of flat-roofed caverns involves special considerations of the material mechanics such as an analytical approach using the Voussoir beam theory. The design procedure is based on the systematic iteration procedure developed by Diederichs and Kaiser, 1999, after their extensive research. The relevant support methods are then discussed and verified using two-dimensional numerical simulations.

2.6 Voussoir beam theory

It was initially noted by Fayol that when layered systems are destabilized, the layers will delaminate and the system may be treated as a stack of merely supported beams as in Fig 2.16 a. The gravitational load of the overlying strata is distributed laterally to the abutments through the individual beams.

Fig. 2.16 (a) Vertically and horizontally jointed rock mass dissociates and deflects in the crown of an excavation; (b) a compression arch forms to oppose the deflection of two discrete blocks, a Voussoir beam analogue ([Diederichs & Kaiser, 1999](#))

Conventional beam mechanics is applicable exclusively to homogeneous elastic beams. As the rock mass strength is limited in tension, midspan cracks in an intact layer or joint separation in a jointed beam will often form as a result of a slight beam deflection, but the beam will not fail completely. This lack of complete failure is due to the formation of a compressional arch within the layer, as shown in [Fig. 2.16 b](#). The compressional arch which allows for the formation of a solid discontinuum beam was first introduced as a Voussoir beam by [Evans, 1941](#).

The Voussoir beam relies on the compressional stress to maintain a solid beam structure composing of discrete blocks, which distributes the load laterally to abutments. Due to the discontinuous nature of the Voussoir beam, it has no tensile capacity ([Sophianos, 1996 & Sophianos and Kapanis, 1998](#)).

Rock masses are inherently discontinuous due to natural jointing or induced fracturing. Layered rock masses which usually contain vertical or sub-vertical discontinuities are thus dissected into discrete blocks before any initial beam

deflection. In these cases, the Voussoir beam mechanics becomes especially important in view of the principle described below.

In order to maintain the stability of a large span excavation, the Voussoir beam has to remain intact when the following two primary criteria are met:

- sufficient horizontal stress and
- adequate beam thickness.

However, in many cases, laminations are not sufficiently thick to reach equilibrium. Additional support is required to maintain the stability as described in the Voussoir beam mechanics. Rock bolt support may be used to minimize the slip between layers, hence reducing the resultant dissociation of the layers. This enables the multiple layers to act as one large beam of sufficient thickness forming an adequate compression arch to oppose the moments acting on the beam.

However, to optimize the cavern design, preventing the initial shear displacement along the discontinuities due to the beam deflection, high horizontal stress is crucial at and near the abutments ([Diederichs and Kaiser, 1999](#)).

2.6.1 Analytical approach of roof design using Voussoir beam theory

The Voussoir arch concept was first introduced by [Evans \(1941\)](#), into rock engineering to explain the stability of a jointed rock beam. Although this concept initially raised a great deal of controversy, it is now generally accepted and it was later extended by [Beer and Meek \(1982\)](#). [Brady and Brown \(1985 & 1993\)](#) made a significant contribution in presenting the Voussoir arch theory in the form of a simplified design tool. Several authors have studied the Voussoir beam through numerical, experimental and analytical approaches. However, in these solutions only a single layer is modeled and the influence of spacing and friction between the vertical joints are ignored. Limited research has been performed on the mechanical strength of a Voussoir beam. [Passaris et al. \(1993\)](#) studied the crushing strength of the beam, and the mechanism of shear sliding alongside walls has been investigated by [Ran et al. \(1991\)](#), both of which have used nonlinear finite element analysis. In both studies the analysis was extended to the case of multiple mid joints and the

spacing between joints was considered, but friction along the discontinuities was not modeled. The influence of joint spacing on the stability of a Voussoir beam having a fixed span, total thickness and individual bed thickness is studied using DDA (Discontinuous Deformation Analysis) by [Hatzor and Benary \(1998\)](#) and [Mazor et al. \(2009\)](#).

Another vital contribution to the theory was made by [Diederichs and Kaiser \(1999\)](#) who corrected and improved the iterative solution scheme presented by [Brady and Brown \(1993\)](#). They also performed a more realistic yield threshold for snap-through failure to replace the ultimate rupture limit initially proposed by [Evans \(1941\)](#).

[Diederichs and Kaiser \(1999\)](#) provided a clear and concise explanation of the Voussoir beam theory. The most significant difference between the Voussoir beam theory and the elastic beam theory is the fact that the Voussoir beam material has no tensile strength in the horizontal direction. Besides, the moment that needs to be generated for stability is created by forming a compressional arch inside the stratified rock as shown in [Fig. 2.16 b](#).

A generalized version of the solution scheme proposed by Diederichs and Kaiser is shown in [Fig 2.17](#). The solution scheme systematically increases the value of N (the ratio of true to effective beam thickness) from 0 to 1 and calculates the values of Z_0 (the moment lever arm before deflection), Z (the moment lever arm after deflection) and F_m (the maximum horizontal stress in the beam) corresponding to each value of N . If the z_{chk} (check) value becomes negative, the calculation is restarted with next N value. The lowest and highest values for N are recorded to calculate the buckling limit. The smallest calculated value of F_m and the associated values of N , Z and Z_0 are assumed to be the parameters for Voussoir beam formation at equilibrium state. These values are then used to check the stability of the beam against buckling, crushing and sliding.

The midspan deflection (vertical displacement) can also be calculated. The nomenclatures used in the calculation are shown in [Fig. 2.18](#).

[Diederichs and Kaiser \(1999\)](#) also proposed yield limit as Displacement Ratio (mid-span deflection/bed thickness) to be 10% and Buckling Limit (BL) to be 35%;

whereas Collapse Limit as Displacement Ratio to be 25% and BL to be between 80% to 100% (Fig. 2.19).

Fig. 2.17 Flow chart for the determination of stability and deflection of a Voussoir beam (Diederichs & Kaiser, 1999)

Fig.2.18 Voussoir beam (half-span above) with the nomenclature used in the calculation (a) initial geometry, (b) deformed geometry (Brady and Brown, 1985 & 1993)

Fig. 2.19 Determination of yield and collapse limit (Diederichs and Kaiser, 1999)

Alejano et al. (2008) studied a roof bed failure in an underground mine in Spain, which was excavated in a stratified rock mass, by multi-approach back-analysis applying empirical stability graph method, the analytical Voussoir beam techniques and the numerical discrete element method with UDEC (Universal Discrete Element Code, Itasca Inc.).

2.7 Assessment of tunnel condition by the modified stability graph technique

The stability graph method was first proposed by Mathews et al. (1981) and later modified and calibrated for Canadian mining conditions by Potvin (1988). Additional data has been added by Suorineni (1998) and Nickson (1992) and the method is now a popular tool in Canadian open stope design. The characterization of the rock mass by means of a stability number N' is based initially on the Q system (Barton et al. 1974). The stress reduction factor is dropped (set to 1) as is the joint water factor (since deep mines are typically dry).

Fig. 2.20 Stability parameters for the modified stability graph (Hutchison and Diederichs, 1996)

Three new factors A, B and C are defined to account for induced stress, relative joint orientation and gravity loading respectively such that:

$$N' = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times A \times B \times C = Q' \times A \times B \times C \text{ ----- (2.6)}$$

Where A, B and C are defined in [Fig. 2.20](#).

Excavations of different geometries are equated through the use of a shape factor or hydraulic radius, HR, which describes the geometry and size of the stope face in question ([Fig. 2.21](#)). For a rectangular stope face of dimensions x and y:

$$HR = \frac{\text{Area}}{\text{Perimeter}} = \frac{xy}{2x + 2y} \text{ ----- (2.7)}$$

$$HR = \frac{\text{Area}}{\text{Perimeter}} = \frac{x \times y}{2x + 2y}$$

Fig. 2.21 Calculation of "hydraulic radius," HR for use in the stability chart ([Hutchison and Diederichs, 1996](#))

Using this relationship the hydraulic radius, HR, of a tunnel roof (with length much longer than the span) is given as one half of the span. For a square excavation, the hydraulic radius equivalent to one-quarter of the span. The hydraulic radius

expression, therefore, defines that a horizontal tunnel is equally as stable as a square excavation surface of twice the linear span (measured orthogonal to the face edge).

2.7.1 Calibration of the Voussoir model to the stability graph

A calibration database of 189 case histories involving unsupported stopees has been assembled by [Potvin \(1988\)](#) and [Nickson \(1992\)](#). The database and the resulting no-support limits are shown in [Fig. 2.22](#).

Fig. 2.22 Unsupported stope data from [Potvin \(1988\)](#) and [Nickson \(1992\)](#) and No-support limits for modified stability graph ([Hutchison and Diederichs 1996](#))

2.8 Other rock formations in Singapore

During recent years, several extensive investigations including geological mapping, seismic reflection and refraction surveys, resistivity survey, deep coring, geophysical borehole logging and cross-hole tomography, as well as laboratory testing have been

conducted on the Bukit Timah Granite ([Sharma et al., 1999](#)). New geological and geotechnical data and information have been gathered, which lead to new understandings of the geology and engineering geology of the Bukit Timah Granite ([Zhao et al., 1994](#); [Zhao, 1996](#)).

2.8.1 Geological setting of the igneous rocks

The Bukit Timah Granite (BTG) and the Gombak Norite underlies about one-third of the Singapore Island and the whole of Pulau Ubin. The BTG is mainly an acid igneous rock formed during the Triassic period ([PWD, 1976](#); [DSTA, 2009](#)) and forms a batholith at the centre of Singapore Island extending some 8 km in north to south and 7 km east to west. Its dominant, granitic component is grey and medium to coarse grained (2-5 mm) and consists of cream or pale yellow feldspar (60-65%), smoky quartz (30%) and smaller proportions of reddish brown biotite and dark hornblende. The granite in the central Singapore includes entire suite of acidic igneous rocks, ranging from granodiorite, adamellite and true granite. There is considerable hybridization within the formation and there is much evidence of assimilation.

The Gombak Norite is an association of noritic and gabbroic rock which outcrops in a restricted area, around Bukit Panjang and Bukit Gombak in the centre of the island. The rocks are coarse-grained and plagioclase-rich with varying amounts of clino- and orthopyroxene minerals in intergranular texture. The rock also has very high strength.

2.8.2 General geological and engineering geological property of the igneous rocks

The geological and engineering geological properties of the granite and the norite can be summarized as follows;

- The weathering of the granite and the norite is very extensive, with the depth of weathering varying between 10 to 70 m with an average of around 30 m.
- Both rocks have very high strength (UCS average of 190 MPa)

- Faults in the granite are sub-vertical with variable width. Some of the faults are re-cemented by calcite and other minerals.
- There are generally four to five joint sets in the granite rock masses. The dominant joint set is sub-vertical with strike in the direction NNW-SSE.
- The permeability of the jointed granite rock mass is very low with average permeability of 10^{-7} to 10^{-8} m/s. Groundwater flow is only likely in the fractured zones and faults. Permeability of the completely weathered granite and the residual soil is in the order of 10^{-5} m/s.
- Recent in-situ stress measurements have shown that the minimum and maximum horizontal stresses are two to four times the vertical stress respectively (Zhao et al., 2005). The major principal stress is in the NNE-SSW direction. The direction of the major principal stress seems to be related to the tectonic activity of the Jurong Formation which has been folded with strike in the NE-SW direction.

2.9 Summary and discussion

In this chapter, the literature reviews on the previous works including general geology of the project and engineering geological properties of intact rock are presented. However, general geology of the area was based on the observation of surface rock exposures at surrounding areas and engineering geological properties are obtained by the testing of drilled core specimens during the feasibility study of the project.

During working from the beginning of the project till the completion of the underground rock excavation work which involved about 3 million cubic meters of sedimentary rocks with 18 km linear length, the candidate gained valuable knowledge, findings and new observation of unexpected geological natures of Ayer Chawan Facies. Additional laboratory testing and numerical analyses were also carried out by Nanyang Technological University engaged as research and development on the project assigned the developer, JTC Corporation.

Hence the main objective of this current work is to document the new findings of the project including observation after extensive study and analyses on the engineering properties and stability of the project. The subsequent numerical analyses are carried

out using the actual rock mass parameters determined from the project. Those knowledge will be worthy as references for future underground rock excavation projects in Singapore.

CHAPTER 3

The geological setting of the area

3.1 Introduction

The Ayer Chawan Facies is one of the volcano-sedimentological Facies belonging to the Jurong Formation. The geology of the south-western Jurong Island had never been documented before the island was reclaimed in 1998. The present geological, sedimentological and geochronological study of the Ayer Chawan Facies was undertaken as part of an underground rock caverns storage project which was carried out from 2009 to 2014. The Ayer Chawan Facies is composed mainly of grey to black mudstone interbedded with grey siltstone/sandstone. In addition, the felsic pyroclastic rocks of varying thickness are also intercalated with the sedimentary rocks. The age of pyroclastic rocks yielded an U-Pb zircon age of 240.6 ± 1.2 Ma (Middle Triassic) as determined by Laser Ablation Inductively Couple Plasma Mass Spectrometer (LA ICP-MS). Based on the stratigraphic and sedimentological evidence of the presence of peperitic textures/structures, it is interpreted that the pyroclastic rocks were emplaced into the sediments during deposition which was still in a wet and unconsolidated condition.

The aims of the study are to undertake age relationships, geological and stratigraphic mapping of the Jurong Formation focusing on the Ayer Chawan Facies, to document sedimentological and petrological features of the different lithologic units and to determine the nature, setting and origin of the syndepositional pyroclastic rocks and their sedimentation process.

3.2 Previous study

The two previous editions of “Geology of Singapore” focused on the main Singapore Island ([PWD, 1976](#) and [DSTA, 2009](#)), while the present study focuses in the area of rock caverns storage project that is located offshore and the geological and sedimentological aspects have never been described and discussed in the literature.

3.3 Present study and methods

The Laser Ablation Inductively Coupled Plasma Mass Spectrometry (LA ICP-MS) U-Pb zircon age dating method was used to determine the age of the pyroclastic unit at CODES, University of Tasmania, Hobart, Tasmania.

The LA-ICPMS method is now widely used for measuring U, Th and Pb isotopic data (e.g. [Fryer et al., 1993](#); [Compston, 1999](#); [Black et al., 2003](#); [Kosler, 2001](#); [Black et al., 2004](#); [Jackson et al., 2004](#); [Chang et al., 2006](#); [Harley & Kelly, 2007](#)).

Approximately 200 -400 g of rock was crushed in a Cr-steel ring mill to a grain size <400 microns. Non-magnetic heavy minerals were then separated using a gold pan and a Fe-B-Nd hand magnet. The zircons were hand-picked from the heavy mineral concentrate under the microscope in the cross-polarised transmitted light. The selected crystals were placed on double sided sticky tape and epoxy glue was then poured into a 2.5 cm diameter mould on top of the zircons. The mould was dried for 12 hours and polished using clean sandpaper and a clean polishing lap. The samples were then washed with distilled water in an ultrasonic bath.

The analyses in this study were performed on an Agilent 7900 quadrupole ICPMS with a 193 nm Coherent Ar-F excimer laser and the Resonetics S155 ablation cell at the University of Tasmania in Hobart. The downhole fractionation, instrument drift and mass bias correction factors for Pb/U ratios on zircons were calculated using 2 analyses on the primary (91500 standard of [Wiendenbeck et al. 1995](#)) and checked on 1 analysis on each of the secondary standard zircons (Temora standard of [Black et al. 2003](#) & JG1 of [Jackson et al. 2004](#)) analysed at the beginning of the session and every 15 unknown zircons (roughly every 1/2 hour) using the same spot size and conditions as used on the samples. Additional secondary standards (The Mud Tank Zircon, Penglai zircons and the Plesovice zircon) were also analyzed. The correction factor for the $^{207}\text{Pb}/^{206}\text{Pb}$ ratio was calculated using large spots of NIST610 analyzed every 30 unknowns and corrected using the values recommended by [Baker et al. \(2004\)](#).

Each analysis on the zircons began with a 30-second blank gas measurement followed by a further 30 seconds of analysis time when the laser was switched on.

Zircons were sampled on 32-micron spots using the laser at 5 Hz and a density of approximately 2 J/cm². A flow of He carrier gas at a rate of 0.35 liters/minute carried particles ablated by the laser out of the chamber to be mixed with Ar gas and carried to the plasma torch. Isotopes measured were ⁴⁹Ti, ⁵⁶Fe, ⁹⁰Zr, ¹⁷⁸Hf, ²⁰²Hg, ²⁰⁴Pb, ²⁰⁶Pb, ²⁰⁷Pb, ²⁰⁸Pb, ²³²Th, ²³⁵U and ²³⁸U with each element being measured every 0.17 s with longer counting time on the Pb isotopes compared to the other elements. The data reduction used was based on the method outlined in detail in and is similar to that described in [Black et al. \(2004\)](#) and [Paton et al. \(2010\)](#). Uncertainties were calculated using methods similar to that outlined [Halpin et al. 2014](#) and [Paton et al. \(2010\)](#).

Element abundances on zircons were calculated using the method outlined by [Kosler \(2001\)](#) using Zr as the internal standard element, assuming stoichiometric proportions and using the NIST610 to standard correct for mass bias and drift.

3.4 Geology of Ayer Chawan Facies, Jurong Formation in JRC Project

Based on location and observation of lithology in the study area, the rocks belong to Ayer Chawan Facies. It is composed of well-bedded marine muddy sandstone and mudstone, often black. Red roundstone conglomerate is common and all beds are tuffaceous. Spilitic lava is present in this Facies. The type section is defined as the west coast of Pulau Sakra in the southwestern group of islands ([PWD, 1976](#)).

Fig. 3.1 Dark grey to black mudstone interbedded with a minor amount of black sandstone (Ayer Chawan Facies).

Fig. 3.2 An alternate sequence of mudstone (black) and muddy sandstone (light)

A typical example of dark grey to black mudstone exposed on the excavated face in JRC Project, which is the primary rock type of Ayer Chawan Facies is shown in Fig. 3.1 and sandstone interbedded with mudstone type is shown in Fig. 3.2.

3.5 Observation of pyroclastic (Lithic Tuff) rocks and its boundary relationship with mudstone

a.

b. An enlarged view of xenolith structure

Fig. 3.3 Alternate sequence of mudstone (dark) and pyroclastic (light) rocks showing xenolith structure; mudstone xenolith fragments inside pyroclastic layers

Fig. 3.4 Morphology of base of pyroclastic (light) layer with mudstone (dark) showing – flame-like structure; quench margin of pyroclastic rocks in contact with mudstone and the mutual inclusion of both rocks at the base. The formation of fluidal peperite is by quenching and magma-sediment density contrasts.

The interbedded sequences of mudstone and pyroclastic rock layers as shown in Figs. 3.3 a & b were also encountered with the xenolith inclusion of mudstone in pyroclastic bed (Fig. 3.3b close-up view). The irregular basal contact boundary

between pyroclastic rocks and underlying mudstones was often observed. The flame-like structures were found at the boundary between two rock layers on some exposed faces (Fig 3.4 a & b). The generalized stratigraphic column prepared from 63-meter long rock cored from one vertical borehole inside tunnel gallery is presented in Fig. 3.5 which showed that pyroclastic (lithic tuff) constitutes about 50 percent of the core run. The individual bed thickness of both rocks varied widely from a few centimeters to over 2 meters. Mutual inclusions of one type of rock fragments in the other rock type were commonly observed in this area.

3.5.1 Petrography and texture

According to the I.U.G.S classification, the volcanic rocks consisting of ash and lapilli (fragments size from 2 mm to 64 mm in diameter) are called tuff. Lithic tuff refers to one in which rock fragments predominate. The lithic tuffs are white to light grey with varying thickness from about 10 cm to over 1.5 m interbedded with dark grey mudstone as shown in rock core photo (Fig. 3.6).

The thin section analysis reveals that the rock is composed of various kinds of rock fragments (grain size 1 mm to 4 mm) and a minor amount of opaque iron ore minerals (pyrites). Almost rock fragments are of volcanic origin as evident by their composition and textures (Fig. 3.7).

Since volcanic rock fragments predominate, strictly speaking, the rock has to be named lithic tuff or tuffites. Although the occurrence of lithic tuff (pyroclastic rock) was documented in two issues of Singapore Geology, the depositional relationship between lithic tuff and sedimentary mudstone was not recorded.

Fig. 3.5 Generalised stratigraphic column of 63-meter rock cores drilled at one tunnel gallery.

Fig. 3.6 Core samples of 5 m length showing lithic tuff (light grey color) interbedded with well-bedded mudstone (dark color)

Fig 3.7. Photomicrograph of lithic tuff showing volcanic rock (rhyolite, andesite) fragments embedded in fine-grained groundmass (a) PPL – (b) XPL

3.6 Observation of Peperites in the project area

3.6.1 The definition of “Peperite”

The term ‘peperino’ was defined by [Scrope \(1827\)](#) to term the clastic rocks from the Limagne d’Au-vergne region of central France that includes mixtures of lacustrine limestone with basalt which look like ground pepper. This area is now placed as the type locality for ‘peperite.’ [Scrope \(1858\)](#) interpreted the rocks as having originated by a “violent and intimate union of volcanic fragmentary matter with limestone while yet in a soft state,” whereas [Michel-Levy \(1890\)](#) more particularly interpreted them as having produced by the intrusion of magma into the wet lime mud. The term ‘peperite’ is now most commonly used to refer to clastic rocks including both igneous and sedimentary components, which were produced by intrusive processes, or along the contacts of lava flows or hot volcanoclastic deposits with unconsolidated, typically wet, sediments.

[White et al. \(2000\)](#) defined as below;

Peperite (n): a genetic term employed to a rock formed essentially in situ by the disintegration of magma intruding and intermingling with unconsolidated or poorly consolidated, characteristically wet sediments. The term also refers to similar mixtures originated by the same processes activating at the contacts of lavas and hot pyroclastic flow deposits with such sediments.

Peperites are commonly associated with intrusions in submarine sedimentary sequences (Goto and McPhie, 1996), lacustrine successions associated with lavas (Khalaf et al., 2015; Martin and Nemeth, 2007), at the base of pyroclastic flow deposits in subaerial successions (Leat, 1985) and at the base of lava flow in arid setting (Karla et al., 2007).

The magma involved in the formation of peperites can have a broad chemical variation, from basaltic to rhyolitic compositions (Skilling et al., 2002) and sediments can vary from fine to coarse-grained (Busby-Spera and White, 1987; Squire and McPhie, 2002).

3.6.1.1 Gross characteristics of peperites

The volume of peperite body range in volume from less than one cubic meter along contacts between sediments and intrusions, lavas and hot volcanoclastic deposits, to several km³ (Snyder and Fraser, 1963; Hanson and Wilson, 1993, Khalaf et al., 2015).

3.6.1.2 Characteristics of juvenile clasts

Fig 3.8a Formation of blocky peperites in coarse-grained sediments host (Khalaf et al., 2015)

Busby-Spera and White (1987) recognized two types of peperite, which were called blocky and fluidal, with reference to the dominant shape of the juvenile clasts. Various shapes of blocky (Fig. 3.8a), fluidal (Fig. 3.8b) and other juvenile clasts are mentioned (Skilling et al., 2002). In this area, fluidal peperites in elongated globular shape (Fig. 3.9, 3.10 and 3.14), tendril and irregular shapes (Figs. 3.13 a & b) are common since host sediments are fine-grained mud.

Fig. 3.8b Formation of fluidal peperites in fine-grained sediments host (Khalaf et al., 2015)

3.6.1.3 Characteristics of the sediments of Ayer Chawan Facies

In this study area, fine-grained mud and clay with low porosity and permeability are the dominant types of host sediments (Fig. 3.1).

Evidences showing that the host sediments were unconsolidated or poorly consolidated and most likely wet, at the time of interaction with the magma or lava are as follows;

- the scarcity of pyroclastic and mudstone aggregates at the boundary (Figs 3.9, 3.10, 3.13 a & b) and the presence of sediment (mudstone) in fractures of the pyroclastic beds (Fig. 3.14).
- Mutual inclusions of mudstone and pyroclastic rock fragments in alternate layers at the contact boundaries (Figs 3.4a & b, 3.11, 3.12 and 3.14)

3.6.2 Peperite-forming processes

Peperite formation consists of fragmentation or breakdown of magma/lava to appear juvenile clasts and intermingling of these clasts with a sediment host. Fragmentation and intermingling are probably often synchronized, but some juvenile clasts, particularly those formed by processes such as quenching and auto-brecciation during the intrusion, may intermingle with the adjacent sediment after initial disruption (Skilling et al., 2002). Thus the mechanisms for fragmentation and intermingling are inter-related and occurred instantaneously.

3.6.2.1 Fragmentation or breakdown of magma/lava in Ayer Chawan Facies

The main mechanisms for generation of pyroclastic fragments at the contacts after magma/lava intruding wet sediment in the study area;

- magma-sediment density contrasts (Figs. 3.4 a & b, 3.9 and 3.11) and
- quenching (Figs. 3.10 and 3.13)

3.6.2.2 Mingling of juvenile clasts and host sediment

The mingling of juvenile clasts and host sediment is promoted by fluidization of sediment, the forceful intrusion of magma, hydromagmatic explosions, magma-sediment density contrasts and sediment liquefaction and liquification (Skilling et al., 2002). The following two mechanisms are the main sources for the mingling of juvenile pyroclastic clasts and host sediments in the study area.

Sediment fluidization process of Ayer Chawan Facies

Fluidization of host sediment, in this sense, is probably an important process accompanying intrusion of magma into wet sediment, and probably gives rise to the mingling of sediment and juvenile components (Skilling, et al., 2002)

The main evidence of sediment fluidization during peperite formation of the Ayer Chawan Facies observed in the study area are the presence of flame like structures at the base of pyroclastic layers (Figs. 3.4a & b, 3.9, 3.10 and 3.11). The flame-like structure was formed by the upward movement of fluidized wet mud sediments into the hot pyroclastic layer. The flame structure at the contact was also developed due

to the density contrast between the two layers.

Magma-sediment density contrasts

Mingling may also be motivated by magma-sediment density contrasts at the lower boundary of pyroclastic layers (Figs 3.4a & b, 3.9 and 3.10). The observation that peperite is better developed or restricted to basal contacts was reported by Brooks et al. (1982); White and Busby-Spera, (1987) respectively. The occurrences show that magma-sediment density contrasts are an essential control on mingling, at least in these contact areas.

Fig. 3.9 Morphology of base of pyroclastic (light) layer with mudstone (dark) showing – flame-like structure; mutual inclusion of both rocks; mudstone inside cracks at the bottom of the pyroclastic rock. The formation of fluidal peperite is by quenching and magma-sediment density contrast.

Fig. 3.10 Morphology of base of pyroclastic (light) layer with mudstone (dark) showing – flame-like structure; elongate globule of fluidal clasts at base; mudstone inside cracks at the bottom of the pyroclastic rock. The formation of fluidal peperite is by quenching and magma-sediment density contrast.

a

b

Fig. 3.11. Wavy boundary contact, mutual inclusions, flame structure (magma sediment density contrast)

a

b

Fig. 3.12 Wavy boundary contact, mutual inclusions (magma sediment density contrast)

a

b

Fig. 3.13 Juvenile clasts morphology – elongate globule, tendril and irregular shapes

a

b

Fig. 3.14 Elongate and circular globular juvenile clasts in mudstone showing mutual inclusions

3.6.3 Thermal and mechanical effects on host sediment

Hot pyroclastic lava flows with peperite generation resulted in contact metamorphism, local dewatering, cementation and compaction on host sediments which gave rise to the formation of higher strength mudstone (UCS = 140 MPa) in JRC project than the similar type of typical sedimentary rocks elsewhere on the world (40 MPa; [Waltham, 1994](#)). Due to the high strength gained in meta-sedimentary rocks, the wide span (20 m width) of storage caverns could be excavated successfully in Ayer Chawan facies in JRC Project.

3.7 Age of Ayer Chawan Facies, Jurong Formation in JRC Project

The LA ICP-MS U-Pb zircon age of the pyroclastic rock, determined at CODES ARC Centre of Excellence in Ore Deposits University of Tasmania Hobart Tasmania, is 240.6 ± 1.2 Ma (Middle Triassic) as the results are plotted on Tera Wasserburg concordia diagrams ([Fig. 3.15](#)), and the data are given in [Table 3.1](#).

There was no fossil found during the construction of storage caverns in the study area. However, six stratigraphic columns from Ayer Chawan Facies with collected fossils species recorded by [Lim \(1974\)](#) were reproduced in [DSTA \(2009\)](#). The age of this Facies was given as Late Triassic to Early Jurassic. Based on fine-grained sediments, black coloration and laminated bedding nature, the facies was likely to have deposited in anaerobic conditions with later phases of volcanic activity which were responsible for the elimination of the biota present.

As reported in [DSTA \(2009\)](#), Dr. I.G. Speden remarked after studying the Lim's collection that the sediments were deposited on the shallow shelf to enclosed marine bay habitats of normal salinity, turbidity, oxygenation and low to moderate energy conditions. He also pointed out that one type of fossil species is only known from late Jurassic to early Cretaceous.

globular, tendril and irregular shaped juvenile clasts were observed. In this study area fluid instabilities within vapour films, magma-sediment density contrasts and fluidization of host sediment mechanisms were likely the primary mechanisms to form the fluidal juvenile clasts and mingling with host sediments.

Table 3.1 Detrital zircon U-Pb ages for the pyroclastic rocks (Lithic Tuff) from JRC Project

Analysis No.	²⁰⁶ Pb/ ²³⁸ U Age (Ma)		²⁰⁶ Pb/ ²³⁸ U		²⁰⁸ Pb/ ²³² Th		²⁰⁷ Pb/ ²⁰⁶ Pb	
	²⁰⁶ Pb _{cor}	+/-1 std. error	Ratio	% rise	Ratio	% rise	Ratio	% rise
AP21C805	232	3.23	0.0370	1.4%	0.0115	3.0%	0.0594	2.7%
AP21C814	234	2.17	0.0371	0.9%	0.0115	1.5%	0.0558	1.6%
AP21C801	237	2.71	0.0377	1.1%	0.0144	2.9%	0.0579	2.7%
AP21C808	237	1.65	0.0374	0.7%	0.0119	1.3%	0.0511	1.2%
AP21C803	237	2.33	0.0377	1.0%	0.0132	1.9%	0.0560	2.2%
AP21C804	238	1.95	0.0377	0.8%	0.0120	1.5%	0.0526	1.6%
AP21C809	238	3.02	0.0381	1.3%	0.0135	2.1%	0.0607	2.5%
AP21C807	241	2.59	0.0381	1.1%	0.0123	2.6%	0.0507	2.4%
AP21C813	241	2.34	0.0382	1.0%	0.0124	2.1%	0.0510	1.9%
AP21C817	242	1.92	0.0382	0.8%	0.0119	1.6%	0.0517	1.4%
AP21C806	242	2.17	0.0383	0.9%	0.0122	2.1%	0.0508	1.9%
AP21C8 b	243	2.33	0.0385	1.0%	0.0125	2.6%	0.0523	2.3%
AP21C816	243	2.20	0.0386	0.9%	0.0125	1.8%	0.0534	1.9%
AP21C815	244	3.60	0.0394	1.5%	0.0153	3.0%	0.0690	3.1%
AP21C812	244	2.56	0.0397	1.0%	0.0153	1.9%	0.0729	1.9%
AP21C811	244	2.59	0.0387	1.1%	0.0124	3.0%	0.0522	2.7%
AP21C802	251	2.33	0.0398	0.9%	0.0128	2.2%	0.0535	1.9%
AP21C818	278	2.17	0.0441	0.8%	0.0138	1.8%	0.0513	1.4%

The LA ICP-MS U-Pb zircon age of pyroclastic rocks of the Ayer Chawan Facies is determined to be 240.6 ± 1.2 Ma (Middle Triassic) in this study. It is slightly older but close to the age range assigned after fossil species (Late Triassic to Early Jurassic) collected by [Lim \(1974\)](#).

The host sediments later experienced induration, contact metamorphism, local dewatering, cementation and compaction by simultaneous hot pyroclastic lava flows, leading to the formation of strong mudstone eventually. Having higher UCS results of mudstone in JRC (140 MPa) than the similar type of typical sedimentary rocks elsewhere on the world (40 MPa; Waltham, 1994) demonstrated the gain in strength

after thermal effect on sedimentary rocks by lava emplacement. Hence, 20 m wide span of underground storage caverns could be successfully excavated safely in such rock mass.

CHAPTER 4

Measurement of engineering geological properties of rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

4.1 Petrographic study of rocks under thin section

Representative samples from different rock types were collected both from drilled core samples and excavated block samples. Selected rock samples from those collections were prepared for thin slides of thickness 0.03 mm. Those were studied using petrographic microscope under plane polarized light and crossed polarized light. The basic principle of study the rock under thin section is as follows;

- Study of morphological properties of minerals such as grain shape and symmetry, cleavage and fractures, twinning, inclusion, intergrowth, alteration products.
- Identification of minerals based on optical properties such as colour and pleochroism, light refraction (relief), double refraction (extinction behaviour, interference colour),
- Naming of rocks according to above two characters of minerals those composed of the rocks

The scheme for mineral identification upon examining the minerals under plane polarised light mode and crossed polarised mode proposed by [Raith et al., 2012](#) is shown in the appendix [Fig. A.1](#). Detailed study of microscopic petrography and petrogenesis (origin of rocks) are discussed in [Chapter 3](#).

4.2 Collection and analysis of general trend of discontinuities using “Dips” program

The magnitudes of discontinuities such as sedimentary bedding planes, joints and fault planes were measured and recorded during the course of face geological mapping after excavation. Those recorded magnitudes, dip amount and direction, of such discontinuities were analysed by using “Dips” programme ([Rocscience Inc., 2013a](#)). “Dips” is a software program designed for the interpretative analysis of

orientation based geological data. It allows the user to analyse and visualise structural data following the same techniques used in manual stereo nets.

Fig. 4.1 a – d show the pole plots with overlay contours of different discontinuities such as bedding planes, joint planes and fault planes measured and recorded during geological face mapping from one of the storage rock caverns namely CS 1/1. From the contour plot analysis, 1 set of bedding with varying dip angles, 4 sets of joints and 2 sets of fault are grouped as prominent sets in the cavern and are presented in Table 4.1.

The analysis of the geometry and stability of underground wedges formed by intersecting structural discontinuities in the rock mass surrounding an underground excavation are essential for safety of the people working in the project. The tunnel roof stability analysis in terms of possible wedge roof formation is carried out in the Chapter 7.

Fig. 4.1a Contour plot of bedding planes (equal Area, lower hemisphere)

Fig. 4.1b Contour plot of joint planes (equal Area, lower hemisphere)

Fig. 4.1c Contour plot of fault planes (equal Area, lower hemisphere)

Table 4.1 Identified Beddings, Joints and Faults sets in CS 1/1

Sets	Bedding	Joints	Faults
		Dip amount / dip direction	
Set 1	25/055 (B1)	85/267 (J1)	80/175
Set 2		78/176 (J2)	80/270
Set 3		85/130 (J3)	
Set 4		83/228 (J4)	

4.3 Analysis and correlation of measured horizontal stress with regional structures

In many large scaled civil and mining engineering projects, especially those involving underground excavation, the initial state of stress prior to any excavation or construction are essential to understand for excavation stability and hydraulic suitability. The first in situ stress measurement in Singapore was performed in Bukit Timah Granite rock in around 2000. The second measurement was started carried out in Sedimentary Jurong Formation in 2006 followed by 2011 during construction period of the JRC project, inside one tunnel gallery at level 1 which is about 98 m below ground level as shown in Fig. 4.2 (Hyundai Engineering & Construction, 2011a). The ground surface is located at +5 m above the sea level (ACD, above chart datum = 0 m). The vertical depth, Z and ACD have a difference of 5 m in depth expression. Thus the depth level expressed in Z can be expressed in ACD as $[Z=ACD+5]$.

HF (Hydraulic Fracturing) and HTPF (Hydraulic Test on Pre-existing Fractures) were carried out in 3 NX boreholes with core samples taken (one vertical and 2 sub-vertical bore holes) which were drilled inside the excavated gallery with the depth of 60-70 m. The gallery was 98 m (-93 m ACD) below existing ground level.

Table 4.2 – Information of Boreholes in which HF and HTPF measurements were carried out (Hyundai Engineering & Construction, 2011a)

Borehole No.	Bearing (degree)	Inclination (degree)	Borehole depth (m)	Corresponding ACD (m)	HF/HTPF tests
BH -1	133	70	74	-167	7
BH -2	348	80	64	-157	8
BH -3	0	90	60	-153	4

Fig 4.2.a – General geology showing stratigraphic layers with properties (Hyundai Engineering & Construction, 2011a)

Fig 4.2.b - Orientation of the test boreholes (Hyundai Engineering & Construction, 2011a)

The borehole information in which stress measurement was carried out was presented in Table 4.2. The general geological description and orientation of test boreholes are shown in Fig. 4.2a & b.

4.3.1 Test procedure of hydraulic fracturing tests for in situ stress determination

First of all, the acoustic borehole televiewer was run in from the bottom of the hole to the top to record the fractures pattern on the borehole wall. After checked and compared the televiewer images of the fracture patterns and drilled core samples, the depths for hydraulic fracturing test were predetermined (Fig. 4.3). The test zones may be either at original fractures existing or fracture free interval.

After the hydraulic fracturing tool was assembled and tested on the surface, it was lowered into the testing hole to predetermined depth via the depth-counted electrical winch. The inflatable packers were wet by hydraulic pressurization. Attention was given to keep at least 2 MPa higher pressure in the packers to prevent leakage around packers. Also before commencement of hydraulic fracturing tests, pre-test pressurization to about 2 MPa was done. The pump was shut in as soon as the present pressure was reached and the behaviour of interval pressure with time was

examined. The purpose of the pre-test was to confirm that the interval was intact and was not taking fluid.

Direction of borehole : bearing/inclination = 133/70
Fracture image is represented as appeared on borehole circumference.

Fig. 4.3 Fracture image by borehole televiewer

Interval pressurization for hydraulic fracturing was conducted at an approximately constant flow rate until a critical (breakdown) pressure was achieved. The pump was shut off, usually after some additional pumping used to extend the induced fracture away from the hole, and the pressure was allowed to decay. The shut-in pressure was later recorded from this portion of the pressure record. Several minutes into the shut-off phase the pressure can optionally be released and allowed to return to atmospheric conditions. The pressurization cycle was then repeated several times, recording the fracture reopening pressure and additional readings of shut-in pressure (Fig. 4.4). At the completion of all the pressurization cycles, the packers were deflated and the straddle packer was removed to the next test zone. The procedure outlined above was repeated for each test. These methods of data reduction and pressure parameter determination are well documented in previous works (Cornet, 1993; Haimson, 1993; Haimson and Cornet, 2003)

< Pressure vs. Time Record by Hydraulic Fracturing Test >

< Pressure vs. Flowrate Record by Stepwise Flowrate Test >

Upper line in pressure record – pressure in packer for isolating the test section
Lower line – pressure in the test section for fracturing & reopening fracture

Stepwise flow rate test is carried out continuously after several cycles of fracturing & reopening

Fig. 4.4 BH 1 – Test section depth $38.5 \pm 0.5\text{m}$ (HTPF – hydraulic test on pre-existing fracture)

Each fracturing test was carried out starting from the lowest test zone to the highest test zone. After all the fracturing tests were completed, fracture delineation test should be carried out by the borehole televiewer. The knowledge of hydraulic fracture orientation is essential to establishment in the orientation of the principal stresses. In the study area, a borehole televiewer was used to obtain an oriented trace of the induced hydraulic fractures on the borehole wall.

Fracture Reopening Pressure, P_r

Shut-in Pressure, P_s

Normal Pressure on fracture plane, P_n

Fig. 4.5 Pressure parameter analysis BH 1 – Test section depth $38.5 \pm 0.5\text{m}$ (HTPF – hydraulic test on pre-existing fracture)

4.3.2 The results of in situ stress measurement

The above pressure data (Fracture Reopening Pressure, P_r ; Shut-in Pressure, P_s ; Normal Pressure on fracture plane, P_n ; Fig. 4.5) including the direction of the test borehole and the orientations of the fracture traces are used for the generalized 3D

stress analysis by the code 3DHF (KIGAM) which has the nonlinear optimum minimization routine using a modified Levenberg-Marquardt algorithm and a finite-difference jacobian. 24 sets of input data are finally used in stress analysis.

The trends of 3D stress states with depth are expressed with the correlation equations as below. In the following equations of three principal stresses and two max/min horizontal stresses, the depth boundary of validity is adopted as 100m below ground surface. The reason is that field tests are carried out in the depth range of 100 m below ground surface and so the correlation equations estimated from these test data is to be valid in the region below 100 m depth from the ground surface.

Max Principal stress:

$$PS_1 \text{ (MPa)} = (5.40 \pm 0.37) + (0.0393 \pm 0.0043) (Z[\text{m}] - 100) \quad (4.1)$$

Med Principal stress:

$$PS_2 \text{ (MPa)} = (3.25 \pm 0.22) + (0.0475 \pm 0.0052) (Z[\text{m}] - 100) \quad (4.2)$$

Min Principal stress:

$$PS_3 \text{ (MPa)} = (2.33 \pm 0.16) + (0.0263 \pm 0.0029) (Z[\text{m}] - 100) \quad (4.3)$$

Max Horizontal stress:

$$\sigma_H \text{ (MPa)} = (5.38 \pm 0.36) + (0.0388 \pm 0.0041) (Z[\text{m}] - 100) \quad (4.4)$$

Min Horizontal stress:

$$\sigma_h \text{ (MPa)} = (3.14 \pm 0.21) + (0.0463 \pm 0.0049) (Z[\text{m}] - 100) \quad (4.5)$$

Vertical stress:

$$\sigma_v \text{ (MPa)} = 2.04 + 0.028 (Z[\text{m}] - 85) \quad (4.6)$$

The range of maximum, minimum horizontal stresses and vertical stress are 6.2-7.8 MPa, 4.2-6.1 MPa and 2.9-3.9 MPa respectively in test depth of 120-160 m from ground. The bearing and inclination of those stresses are N26±2°E/-5±2°, S62±2°E/-17±2° and S81±3°E/-71±2° respectively. It is clearly shown that the maximum stress is in the trend of NNE-SSW direction.

The stress ratio of $PS_1 : PS_2 : PS_3 = 2.3 : 1.4 : 1$ (at 100 m depth) to $2 : 1.5 : 1$ (at 160 m depth) and $\sigma_H : \sigma_h : \sigma_v = 2.2 : 1.8 : 1$ (at 100 m depth) to $1.9 : 1.7 : 1$ at 160 m depth) were determined. Both of the stress ratios showed trend of gradual decrease with increasing depth.

The measured stress values and ratio are crucial in the stability analysis applying numerical analysis. From the above test results, the horizontal stress in sedimentary Ayer Chawan Facies is high which favour the safe rock mass condition for excavation of wide-spanned storage caverns.

4.3.3 Comparison of in-situ stress measurement in Bukit Timah Granite and South Asia

Eight hydro fracturing in situ stress measurements were performed in the Bukit Timah Granite Formation in two vertical boreholes at depth ranging between 60 m and 120 m below the ground in late 1990s. Those were reported as the first in situ stress measurement that have been conducted in the rocks of Singapore ([Zhao, et al., 2005](#)).

Results of the stress measurements indicated that ratio of $\sigma_H : \sigma_h : \sigma_v$ is approximately 3 : 2: 1 with maximum horizontal in situ stress in the direction of NNE-SSW.

Although the stress ratios in Bukit Timah Granite is in the higher range than those of Jurong Formation from Jurong Island, the general trend of maximum stress in both formations are similarly oriented in NNE-NE direction ([Kar Winn and Ng, 2013](#)). It is also in the same general direction of maximum horizontal stress in Sumatra and southern part of Peninsular Malaya, i.e. NE-NNE ([Heifbach, et al., 2005](#)). It is due to tectonic stresses developed at the convergent boundary (Sumatra-Java trench) between the Euro-Asian and the Indian Ocean Plate as shown in [Fig. A.2](#).

The higher horizontal stresses in both Formations provide general favorable stable conditions for the construction of storage caverns with large span width.

4.4 Permeability characteristics of rock mass determined from results of water bearing zones and boreholes in water curtain gallery

4.4.1 Determination of coefficient of permeability from water bearing zones

Permeability characteristics of rock mass was also determined from water seepage through some water bearing zones encountered in caverns and galleries.

To measure the water ingress rate and flowing pressure, a number of probe holes were drilled into those water bearing zones. The 79 numbers of water flow rate versus pressure were measured from 5 sections in storage cavern CS 1/2a and the result from one is shown in [Fig. 4.6](#).

The coefficient of permeability (k) can be determined using the following formula proposed by [Lei, 1999](#);

$$k = (2.3 q / 2 \pi L H) \times \log (L/r) \quad (4.7)$$

where k = coefficient of permeability (m/sec)

q = measured water flow rate at the location (m³/sec)

L = drilled probe hole length (m)

r = drilled probe hole radius (m)

H = the hydraulic head (m)

From the above formula, if the k value of the area is uniform, the water flow rate is approximately proportional to flow pressure. It is supported by the drawn straight fitting line on the each graph in [Fig. 4.6](#). [Fig. 4.7](#) shows the cumulative distribution of the 79 coefficient of permeability data which indicates that k value is in the order of 1×10^{-6} m/s and the average value is 1.73×10^{-6} m/s.

Fig. 4.6 Water flow rate versus pressure at one location in CS 1/2a (NTU, 2013)

Fig. 4.7 Cumulative distribution of k values in water bearing zone (NTU, 2013)

4.4.2 Determination of coefficient of permeability from boreholes at water curtain gallery WG 1/1

The water ingress rate (l/min) and coefficient of permeability (k , m/sec) versus flowing pressure measured from recharging wells in water curtain gallery WG 1/1 was drawn and shown in Fig. 4.8.

Recharging boreholes were drilled at 10 m interval along the water curtain gallery, WG 1/1 as shown in Fig. A.3. They were drilled to maximum 70 m depth and water flow rate against flow pressure were measured at each borehole. The rock mass coefficient of permeability, k (m/s) was calculated using the same equation Eq. (4.7) as in section 4.4.1. The results show that there is no high permeable water zone in this gallery.

Fig. 4.8 Water flow rate (l/min) and coefficient of permeability (m/sec) versus pressure (bar) of water recharging boreholes

4.5 Determination of coefficient of permeability on intact rock

Three different types of permeability tests were conducted on twenty-three intact and singly-jointed rock specimens, which were cored from rock blocks collected from the JRC. They are interbedded metamorphosed mudstone with sandstone and pyroclastic rocks (lithic tuff) having high unconfined compressive strength but having very low porosity.

The transient pulse water flow method was applied to determine the permeability of intact meta-sandstone and lithic tuff under a confining pressure up to 30 MPa. It shows that the magnitude order of meta-sandstone's intrinsic permeability is about 10^{-18} m^2 . The steady-state gas flow method was used to measure the permeability of both intact meta-mudstone, meta-sandstone and lithic tuff in a triaxial cell under different confining pressures ranging from 2.5 MPa to 10 MPa. The measured permeability of rock types ranges from 10^{-21} to 10^{-20} m^2 . The influence of a single natural joint on the permeability of rock types was studied by using the steady-state water flow method under different confining pressures spanning from 1.25 MPa to 5.0 MPa, including loading and unloading phases. The measured permeability of both jointed rocks ranges from 10^{-13} to 10^{-11} m^2 , where the permeability of jointed meta-mudstone is usually slightly lower than that of jointed meta-sandstone and lithic tuff.

The in-situ permeability tests which were carried out in six boreholes by the injection test gave the in-situ permeability of rock mass ranged between 10^{-18} m^2 and 10^{-11} m^2 . The lower result of the in situ permeability is higher than that of the present laboratory tested intact rock specimens, while the upper results of the in situ permeability is lower than that of the present laboratory tested jointed rock specimens. The in situ permeability test results are thus well-matched with NTU laboratory permeability results of both intact and jointed rock specimens.

4.6 Strength properties of intact rocks by point load tests, uniaxial compression test, triaxial compression tests and Brazilian tests

4.6.1 Point load tests

Point load test which serves as an index test was conducted to establish the relationship between the uniaxial compressive strength and the point load index. Point load test is a low-cost and convenient testing method to measure the strength of rocks. Point load tests have been carried out on JRC rocks (including meta-siltstone and meta-sandstone from 12 rock blocks) in the laboratory at NTU. The Point Load Strength Indexes ($I_{s(50)}$) of these rock specimens have been determined. The strength classification of the JRC rocks can be decided based on the $I_{s(50)}$ value.

The testing equipment with mode of testing is shown in [Fig. A.4](#) whereas a typical failure mode is shown in [Fig. A.5](#) respectively.

4.6.2 Uniaxial compression test

Uniaxial compression strength tests were conducted on the rock cores from Borehole No. DI-OT-0/1d-07 at AS3, Level 1. Most of them are broadly classified into sandstone, conglomeratic sandstone/conglomerate and limestone. The failure modes are classified as shear failure, longitudinal splitting and combined cataclasis and cleavage. The test was conducted in general accordance with the suggested method by the ISRM ([Bieniawski and Bernede, 1978](#))

Table 4.3 Conversion factors between Point Load Strength Indexes and UCS values in JRC

Specimen condition	Rock type	I_s (50) (MPa)	UCS (MPa)	Calculated conversion factors for UCS
Air dried	Meta siltstone/mudstone	9.38	188.0	20.0
	Pyroclastic rocks	12.84	269.9	21.0

4.6.3 Triaxial compression tests

4.6.3.1 The Triaxial testing System

The testing system was manufactured by MTS Inc. (USA), which included a loading frame, a Triaxial cell, a hydraulic power supply, an electronic signal conditioning and control system and a data acquisition. Two major types of rock, namely siltstone and sandstone in both dry and wet conditions were tested and results of cohesion (c , MPa) and friction angle (ϕ , in degree) at peak and residual condition are shown in Table 4.4. The triaxial compression tests were done in accordance with the ISRM suggested method (ISRM, 1973 & 1983) and its subsequent revised version (1983)

Table 4.4 Cohesion and friction angle associated with the peak and residual shear strength of two tested rocks under dry and wet condition (NTU, 2013)

Water condition and rock type	Peak cohesion, c_{peak} (MPa)	Peak friction angle ϕ_{peak} (°)	Residual cohesion c_{res} (MPa)	Residual friction angle ϕ_{res} (°)
Dry Siltstone	36.1	43.4	5.3	41.6
Wet Siltstone	68.6	10.3	8.4	32.4
Dry Sandstone	43.2	51.5	5.9	47.6
Wet Sandstone	58.0	42.2	5.8	45.4

4.6.4 Brazilian tensile strength test

Brazilian tensile tests were successfully conducted on 88 specimens, in which 56 of them were sandstone and 32 of them were mudstone. For both sandstone and mudstone, specimens with transverse bedding planes at three water conditions and specimens with longitudinal beddings at three water conditions were all tested. The water contents were measured for both sandstone and mudstone. The water contents of both rock types were very low, which were no more than 0.5% for sandstone and no more than 0.21% for mudstone.

For wet condition I, the specimens were weighed first to determine their natural mass under room conditions. Then these specimens were immersed into seawater under vacuum less than 1apm (760mmHg) for at least 2 months. Before test, the wet specimens were weighed again to calculate the water content. For wet condition II,

the specimens were first weighed by a balance. They were then oven-dried at 105°C for 48 hours and allowed to cool down to room temperature. After cooling down, each specimen was weighed again to obtain its dry mass. Finally, these specimens were immersed into seawater under vacuum less than 1apm (760mmHg) for at least 2 months. To calculate the water content, the specimens were weighed again to obtain their wet masses before test.

Table 4.5 Brazilian tensile strength results (average and standard deviation)

Bedding orientation	Schematic sketch	Water condition	Brazilian strength			
			Average strength (MPa)		Standard deviation (MPa)	
			Sandstone	Mudstone	Sandstone	Mudstone
Longitudinal		Dry	7.0	9.56	1.71	0.98
		Wet I	4.72	7.97	1.12	1.43
		Wet II	4.47	7.94	0.68	1.59
Transverse		Dry	8.88	8.41	2.69	2.43
		Wet I	7.76	7.23	2.81	1.98
		Wet II	6.43	6.62	1.41	1.81

The tests were conducted according to the following procedures. Before test, each face of the specimen was affixed with a strain gauge at the centre (Fig. 4.9). For mudstone and sandstone, the lengths of the strain gauge are 10mm and 5mm respectively. After that, two steel rods were attached to Brazilian test specimen by means of adhesive tapes. Finally, they were located into the loading jaws.

The loading jaws, together with the specimen, were placed onto the loading frame. The compressive load increased at a constant rate of 0.5 kN/s except the mudstone with longitudinal bedding planes. The compressive loading rate of the latter was 0.2 kN/s till 20 kN, when was then changed to 0.001mm/s until the failure of the specimen. Actually, no obvious different failure mode was observed due to the different loading patterns.

Fig. 4.9 Position of strain gauge in Brazilian tensile test

During the test, the lateral strains were measured by strain gauges, which were recorded by a data logger at a rate of 1Hz. Based on these data, the Brazilian tensile strength was calculated according to [Eq. 4.8](#) provided by ISRM (1978).

$$\sigma_x = -\frac{2P}{\pi DT} \quad (4.8)$$

Where σ_x : Brazilian tensile stress; P: Compressive load at failure; D: Diameter of the specimen and T: Thickness of the specimen at the centre.

According to the ISRM suggested equation ([ISRM, 1981 & 2014](#)), Brazilian tensile (BT) strengths were obtained as shown in [Table 4.5](#). For sandstone with longitudinal bedding planes at dry condition, Wet I condition, and Wet II condition, the Brazilian tensile strengths were 7.00 MPa, 4.72 MPa, and 4.47 MPa respectively. For sandstone with transverse bedding planes at dry condition, Wet I condition, and Wet II condition, the Brazilian tensile strengths were 8.88 MPa, 7.76 MPa, and 6.43 MPa respectively. For mudstone with longitudinal bedding planes at dry condition, Wet I condition, and Wet II condition, the Brazilian tensile strengths were 9.56 MPa, 7.97 MPa, and 7.94 MPa respectively. For mudstone with transverse bedding planes at dry condition, Wet I condition, and Wet II condition, the Brazilian tensile strengths

were 8.41 MPa, 7.23 MPa, and 6.62 MPa respectively. The Brazilian tensile strengths and the stiffness of JRC rock decreased when the water condition changed from dry to wet.

The results of Brazilian tensile strength which are tested before and during project are in the majority range between 6 MPa to 10 MPa. The Brazilian tensile strength results in JRC are higher than the similar type of typical sedimentary rocks elsewhere on the world (5 MPa for sandstone and 2 MPa for mudstone; [Waltham, 1994](#)). The ratio of UCS/BTS falls in the range of 15 to 20 which is in agreement the ratio of 20 by [Waltham \(1994\)](#). Tensile strength of rock is one of the important criterions to determine the tunnel roof stability.

4.7 Direct shear tests on rock joints of core samples

A total of 16 specimens with a single natural joint associated with different mineral veins or traces were tested at NTU, ([NTU, 2013](#)). The direct shear test method contains the requirements and the laboratory procedures for performing direct shear strength tests on rock specimens that are homogeneous, or have planes of weakness, including natural discontinuities, under constant normal load. The discontinuities may be open, partially or completely healed or filled (that is, clay fillings and gouge). Only one discontinuity per specimen can be tested under this method. The test is performed on an open discontinuity in dry condition under a constant normal load. The rock specimen with an open discontinuity is set in the shear machine with the shear surface to be tested aligned with the horizontal and sheared along the desired direction, and within the shear gap.

While maintaining a constant load normal to the nominal shear surface of the specimen, an increasing external shear load is applied along the designated shear surface to cause a shear displacement. The applied normal and shear loads and the corresponding normal and shear displacements are measured and recorded. These data are the basis for deriving the following required parameters and curves:

Peak shear strength – the maximum shear stress in the complete shear stress–displacement curve.

Residual shear strength – the shear stress at which no further rise or fall in shear strength is observed with increasing shear displacement. A true residual shear strength may only be reached after considerably greater shear displacement than can be achieved in testing. The test value should be regarded as approximate and should be assessed in relation to the complete shear stress–displacement curve.

The curves depicting the relationships of shear stress versus shear displacement, peak shear strength versus normal stress, and residual shear strength versus normal stress, can be plotted for all test specimens.

Strictly speaking, due to the indefinable area of contact and the small relative displacement between top and bottom halves of the specimen during testing, the term “normal load” is used here instead of normal stress. The actual contact areas during testing change, but the actual total contact surface is unmeasurable. Therefore, nominal area is used for loading purposes and calculations, which can be obtained by measuring or calculating the cross-sectional area of the shear surface after its relevant cross-sectional dimensions are determined. The nominal area is also corrected to account for shear displacement during the direct shear test.

Table 4.6 Peak and residual strength parameters (NTU, 2013)

Test No.	Peak friction angle (°)	Peak cohesion (kPa)	Residual friction angle (°)	Residual cohesion (kPa)
1	29.9	299	29.5	79
2	35.5	238	32.3	228
3	34.7	383	36.9	0
4	37.3	223	37.6	0
5	38.6	17	37.2	0
6	21	1436	29.2	372
7	38.2	362	39.6	73
8	27.5	993	33.5	0
9	29.5	977	32.5	385
10	28.4	1074	33.8	0
11	33.6	957	38.5	334
12	34.3	1031	39.2	0
13	36.9	1206	35.9	510
14	17.2	3036	22.1	1222
15	43.3	10	37.1	0
16	30.5	2150	37.5	0

In the test, the specimen surfaces were elliptical shape and in dry conditions. A shear

strength determination should preferably comprise at least five repeated shear tests at a different but constant normal stress on the same shear surface with each specimen tested. Taking account of the in situ stress condition of JRC, normal stresses of 1, 2, 3, 4 and 5 MPa were applied to each specimen for multiple runs. The normal stresses of some of the most recently conducted tests were extended to 6, 7, 8, 9, 10 MPa as advised by the client. However, due to the shear damage on the specimen joint surfaces during multiple runs, some specimens were tested fewer than the predetermined five stages of normal stresses. The shearing direction was along the long axis of its elliptical joint surface. The shear load was applied continuously at the selected shearing rate of 0.5 or 1 mm/min. The peak and residual strength parameters are summarized in [Table 4.6](#).

From the results of 16 direct shear tests, the mean values of cohesion (0.5 MPa) and friction angles (33°) were calculated. After consideration of scale factor, 0.3 MPa & 30° for bedding and 0.2 MPa & 30° for joints are reasonable to use in the analyses. It was discussed and agreed with the consultants of JRC Project to use for further analyses in the project.

4.8 Determination of general rock mass quality using Q and RMR system

After evaluation of a large numbers of underground excavation case history, [Barton et al. \(1974\)](#) proposed a tunnelling Quality Index, here called Q system, for determination of rock mass characteristics and tunnel support requirements. The numerical value of Q varies on a logarithmic scale ranging from 0.001 to a maximum 1,000 and is defined by

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (4.8)$$

where RQD is the Rock Quality Designation, J_n is the joint set number, J_r is the joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor and SRF is the stress reduction factor.

Rock Mass Rating (RMR) was developed by [Bieniawski \(1989\)](#), it is an index of rock mass competency based on the rating of six parameters namely, intact rock strength (UCS of intact rock), rock quality designation (RQD), joint spacing, joint surface condition, ground water condition and rating adjustment for discontinuity

orientation. Each of these parameters is given a value. All the values are algebraically summed for the first five given parameters and then adjusted by the sixth parameter depending on the joint and excavation orientation. RMR value ranges from 0 to 100.

4.8.1 Q-RMR correlation

Throughout the last decades, many authors have published empirical correlations. The most widely used examples are given in [Table 4.7](#).

Table 4.7 Equations correlating Q-RMR geomechanical classifications from previous works

Correlation	Reference
$RMR = 9 \ln (Q) + 44$	Bieniawski (1976)
$RMR = 5.9 \ln (Q) + 43$	Rutledge and Perston (1978)
$RMR = 5 \ln (Q) + 60.8$	Cameron Clarke and Budavari (1981)
$RMR = 10.5 \ln (Q) + 41.8$	Abad et al. (1984)

[Fig. A.6](#) showed correlation of two systems by [Bieniawski \(1976\)](#). The equation for mean regression line of data collected from the above figure is as follows;

$$RMR = 9 \ln (Q) + 44 \quad (4.9)$$

4.8.2 Correlation from Jurong Rock Cavern

Two rock classification systems (Q and RMR) were determined independently during geological mapping on excavation faces at some designated locations. As per contract obligation, two systems determination were carried out at total 394 faces (up to 6% of tunnel progress). After each new excavation was completed, the geology of the face was mapped and recorded required parameters for determination of Q values on a record sheet as shown in [Fig. 4.10](#). The photograph of rock face was shown on the map. Based on the Q values, the rock support system was assigned as per recorded rock class system.

The two values were plotted as shown in Fig. 4.11 with 95% of RMR recorded between 60 and 80 whereas Q values are scattered between 4 and 70. The general correlation equation is;

$$\text{RMR} = 6.95 \ln (Q) + 49.5 \quad (4.10)$$

This correlation is comparable with that proposed by Rutledge and Perston (1978). It is recommended to be used as reference in other project.

4.9 The Geological Strength Index (GSI) determination for rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project by qualitative and quantitative approaches

Hoek-Brown empirical failure criterion along with Geological Strength Index (GSI) classification system has been used to determine the deformation modulus of jointed rock mass.

The strength and deformation modulus of moderately to highly jointed rock mass are not possible to determine directly since the dimension of the representative specimen are beyond the workable size in rock mechanics laboratory. This difficulty gave limitation to set reliable design parameters in the analysis of jointed rock mass. Hoek and Brown (1980a & b) proposed an empirical failure criterion to determine strength and deformation parameters of highly jointed rock mass based on uniaxial compressive strength and a constant (m_i) of intact rock. The rock mass rating (RMR) classification system (Bieniawski, 1976), which define the quality of rock mass, was applied in the Hoek-Brown criterion (Hoek and Brown, 1988) as an input parameter.

The modification to improve this empirical criterion has been made for years due to the constraints to apply RMR in the criterion for very poor-quality rock masses (Hoek and Brown, 1988, 1997; Hoek et al., 1992; Hoek, 1994). Later Hoek (1994), Hoek et al. (1995) introduced a new rock mass classification system called “Geological Strength Index, GSI” to improve those shortcomings. Actually this new classification had been initiated in Toronto after David Wood’s input on engineering geological features of rock block structure and joint surface condition (Hoek et al., 1992) but no proper name had been given (Fig. A.7).

The GSI system seems to be more functional than the other classification systems such as Q and RMR to be incorporated in the Hoek-Brown failure criterion. Therefore, the GSI system has become more commonly used input parameter in the Hoek-Brown criterion to estimate the strength and deformation properties of the jointed rock masses.

However due to the complex nature of the rock masses, there were still uncertainties in determination of GSI values. Hence the engineering judgement is usually to be added by experienced engineers or geologists. The value of GSI in range, rather than one single figure, seems to be estimated when using the original GSI chart by [Hoek \(1994\)](#), [Hoek et al. \(1995\)](#). It was advised not to be too precise with single value but obtaining a range of GSI from 46 to 52 is more reasonable than stating $GSI = 48$.

4.9.1 Historical development of GSI in the need to modify Hoek-Brown failure criterion

The initial Hoek-Brown failure criterion (Hoek and Brown, 1980a & b) which was developed for jointed rock masses of tightly interbedded without fines under the confined condition at underground excavation as follows;

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \sqrt{m \frac{\sigma'_3}{\sigma_{ci}} + s} \quad \text{----- (4.11)}$$

σ'_1 and σ'_3 are major and minor effective principal stresses at failure respectively and

σ_{ci} is the uniaxial compressive strength of the intact rock

m and s are material constants (s=1 for intact rock)

It is necessary to adopt the parametric relationship between non-linear type, m and a of Hoek-Brown criterion and linear type, c and ϕ of Mohr-Coulomb criterion. An exact theoretical solution, developed by Dr John W. Bray at the Imperial College of Science and Technology, was first introduced in the 1983 Rankine lecture (Hoek, 1983).

Earlier Hoek and Brown realized that the development of the failure criterion without consideration of geological features of rock mass was not practical. Although they have tried to develop a new classification system, they later decided to use RMR (Bieniawski, 1976) as geological input since it had been widely known in the rock mechanics community.

Thus Hoek and Brown (1988) modified their initial criterion with RMR to define m and s parameters as follows;

$$\frac{m_b}{m_i} = e^{\frac{(RMR-100)}{14}} \quad \text{----- (4.12)}$$

$$s = e^{\frac{(RMR-100)}{6}} \quad \text{----- (4.13)}$$

Later on, it was very obvious that Bieniawski's RMR system was not reliable in the application for very poor quality rock masses and the Eq. (4.12) and (4.13) are questionable as the relationship between RMR and m and s is no longer linear in these very low ranges.

Hence, Hoek, 1994; Hoek, et al., 1995 introduced the concept of Geological Strength Index (GSI) (Fig. A.8) in the replacement of RMR system as following;

For $GSI > 25$

$$\frac{m_b}{m_i} = e^{\frac{(RMR-100)}{28}} \text{-----} (4.14)$$

$$s = e^{\frac{(RMR-100)}{9}} \text{-----} (4.15)$$

$$a = 0.5 \text{-----} (4.16)$$

For $GSI < 25$

$$s = 0 \text{-----} (4.17)$$

$$a = 0.65 - \frac{GSI}{200} \text{-----} (4.18)$$

Hoek, et al. (1998) expanded the range of GSI down to 5 for extremely poor quality schistose rock masses observed in the Athens Metro excavation site and the graphitic phyllites found in some of the tunnels in Venezuela.

Marinos and Hoek (2000) proposed a new GSI system to accommodate weak and tectonically disturbed materials such as flysch which were observed in the major projects in northern Greece.

Hoek, et al. (2005) introduced a new GSI chart for molassic rock masses which composed of a group of tectonically undisturbed sedimentary rocks of sandstones, conglomerates, siltstones and marls exposed on mountain ranges after erosion. They highlighted the difference between molassic rock masses and the flysch type which have been heavily disturbed by orogenic processes.

Marinos, et al. (2006) presented a new GSI chart for ophiolitic rock masses with discussion about geological model, various petrographic description and tectonic deformation by over-thrusting effect. They considered various rock masses including massive strong type to sheared weak type and discontinuities' surface condition.

4.9.2 Characteristics of the GSI System

In the original chart of the GSI System (Hoek, et al., 1995), the rock mass is divided into 20 different categories with a letter code and the GSI values ranging between 10 and 85. It is purely qualitative engineering geological evaluation by visual observation of the blockiness of the rock mass and the surface conditions of discontinuities.

Hoek et al. (1998), Hoek (1999) and Marinos and Hoek (2000) added two more rock masses namely foliated/ laminated rock mass structure and massive or intact rock into initial GSI system (Fig. A.8).

Due to the anisotropic geological characters of the foliated/ laminated rock mass, Marinos and Hoek (2000, 2001) also proposed a special GSI chart only for the heterogeneous rock masses such as flysch (Fig. A.9).

Hoek et al. (1998) suggested providing appropriate range of GSI values rather than a single value as some ambiguity existed in geological interpretation on rock mass. Thus original GSI chart in Fig. A.8 & A.9 may be more appropriate especially for experienced engineers and engineering geologists. Moreover based on own personal experience, he/she will estimate differently from small to wide ranges in GSI values on the same rock mass.

4.10 Qualitative GSI determination of rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

The GSI determination of rocks from JRC were carried out based on a mostly qualitative engineering geological assessment of the blockiness of the rock mass and the surface conditions of discontinuities with reference to the chart in Fig.A.9 since it is applicable to the sedimentary and associated rocks in project. In addition, the results were counter-checked with the original GSI chart (Fig. A.8).

The majority results of GSI are recorded between 40 (Fig. 4.12 b & c; class C in Fig. A.9) and 45 (Fig. 4.12 a; class B in Fig. A.9) which are equivalent to the blocky/disturbed structure with good surface condition as in Fig A.8. As low as GSI = 25 was seldom recorded at faulted zone (Fig 4.12 d) where rock layers are sheared with smooth slickensided surface condition.

(a) GSI = 45

(b) GSI = 40

(c) GSI = 40

(d) GSI = 25

Fig. 4.12 Determination of GSI for common rock masses in JRC Project

4.11 Quantitative GSI determination of rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project by applying four different approaches

The Geological Strength Index (GSI) chart, which was developed in 1995 (Hoek et al., 1995) is based on a mostly qualitative engineering geological assessment of the blockiness of the rock mass and the surface conditions of discontinuities. Though the

system generally works well for geologists exercising professional observation and judgement in the field, some engineers are uncomfortable with it because it does not contain parameters that can be readily measured quantitatively. Hence GSI system has been modified since 1999 to provide quantitative assessment of the above two fundamental parameters (blockiness of rock mass and surface conditions of discontinuities).

In this study, rock mass properties of 420 tunnel faces excavated by drill & blast methods in sedimentary rocks at about 135 m below ground level were analyzed to obtain the quantitative GSI. Relevant input parameters were used in four different approaches of assessing the two basic parameters proposed by authors namely [Hoek et al. \(2013\)](#), [Cai et al. \(2004\)](#), [Sonmez and Ulusay \(1999\)](#) and [Russo \(2009\)](#). This study intends to demonstrate the applicability of the four approaches in characterizing the Singapore's sedimentary rock masses.

4.12 First approach: Determination of quantitative GSI in terms of RQD and joint condition proposed by [Hoek et al. \(2013\)](#)

The chart proposed by [Hoek et al. \(2013\)](#) for calculating quantitative GSI is presented in [Fig. A.10](#). The x-axis represents discontinuity surface condition, while the y-axis represents the blockiness of the rock mass. The Joint Condition (JCond₈₉) rating defined by [Bieniawski \(1989\)](#) and the Rock Quality Designation (RQD) defined by [Deere \(1963\)](#) stand for both x-axis and y-axis scales. These ratings are given in [Table A.1](#).

The JCond₈₉ rating relates well with the surface conditions defined in the text boxes of the x-axis of the GSI chart in [Fig. A.10](#). These rating parameters have been in use for many years and users have found it simple and reliable to apply in the field.

The RQD rating has been used for 50 years and it has been found reliable. Hence these two ratings are applied as the x-axis and y-axis for the determination of quantitative of GSI.

[Fig. A.10](#) shows a chart in which the x-axis scale is defined by 1.5 JCond₈₉ while the y-axis scale is defined as RQD/2. The value of GSI is given by the sum of these scales which results in the relationship;

$$GSI = 1.5 JCond_{89} + RQD/2 \text{ ----- (4.19)}$$

4.12.1 Alternative joint condition scale

In the event that the values of $JCond_{89}$ are not available in the data collected from field geology mapping, two other determination of surface quality was recommended by [Hoek et al. \(2013\)](#) as an alternative approach.

The first proposal is to be replaced with Joint Condition rating ($JCond_{76}$) proposed by [Bieniawski \(1976\)](#) as shown in [Table A.2](#). Regression analysis of a plot of individual values assigned to $JCond_{76}$ and $JCond_{89}$ gives $JCond_{89} = 1.3 JCond_{76}$. Upon substitution into Eq. (4.19) gives

$$GSI = 2 JCond_{76} + RQD/2 \text{ ----- (4.20)}$$

The second alternative is the quotient J_r/J_a which is included in the Tunneling Quality Index (Q) of [Barton et al. \(1974\)](#). The ratings for J_r and J_a are given in [Table A.3 \(Barton et al., 1974\)](#). This quotient represents the roughness and frictional characteristics of the joint walls or fillings.

Comparing the ratings for $JCond_{89}$ with those allocated to J_r and J_a by [Barton et al. \(1974\)](#) gives the relationship ([Hoek et al., 2013](#))

$$JCond_{89} = 35 \frac{\frac{J_r}{J_a}}{(1 + \frac{J_r}{J_a})} \text{ ----- (4.21)}$$

Substitution of this relationship into Eq. (4.19) yields;

$$GSI = \frac{52 J_r/J_a}{(1 + \frac{J_r}{J_a})} + RQD/2 \text{ ----- (4.22)}$$

4.12.2 Input parameters for rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

Four hundred and twenty excavated tunnel faces at various locations in JRC project were evaluated to calculate GSI. Ratings for discontinuities length, separation (aperture) gap, roughness, infilling inside aperture, weathering on plane based on guidelines in [Table A.1](#) (after [Bieniawski, 1989](#)) were determined using Geological map ([Fig. 4.10](#)). The sum of 5 ratings, which is the value of joint surface condition $JCond_{89}$, is calculated to be between 13 and 21. Selective examples of joint surface condition rating and RQD in JRC rock mass is presented in [Table 4.8](#).

Discontinuity length (persistence)

Bedding planes were continuously traceable along the tunnel faces and side walls. Two major joint sets with traceable length of 10 m (rating = 2) were common but short traceable length (rating = 3) was also recorded in some faces.

Separation (aperture) width of discontinuities

Majority of beddings and joints are moderately to closely sealed with aperture width about 1 mm (rating = 4).

Roughness

Generally the planes of discontinuities (beddings and joints) are rough with planar to undulating surfaces (rating = 5). In some places slickensides on minor fault planes (rating = 1) were encountered.

Infilling

The gap of bedding or joint planes were mostly filled with quartz or calcite minerals in width less than 5 mm (rating = 4).

Weathering nature of discontinuities

Faces of joints and bedding were observed in fresh to slightly weathered condition (rating = 5 to 6).

Rock quality designation (RQD)

RQD of rock masses in JRC was in the high range with values between 70 and 95 but the lowest value of 50 was also recorded on faulted and sheared faces. RQD values of individual faces were determined by the following correlation ([Palmstrom, 1974 & 1982](#)) using the volumetric joint count, J_v , a measure of the number of joints crossing a cubic meter of rock;

$$RQD = 112 - 3.3 J_v \text{ ----- (4.23)}$$

4.12.3 Determination of quantitative GSI for rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

The intersection points of 1.5 times JCond89 and factored RQD (RQD/2) values, plotted on chart (Fig. A.10) by Hoek et al., 2013, gave quantitative GSI. After following it, the quantitative GSI results were much higher than those determined on excavated rock faces during construction time.

Table 4.8 Selective examples of quantitative GSI determination of 420 excavated rock faces (Modified after Hoek et al., 2013)

Serial Numbers	Discontinuity Length	Separation (aperture)	Rating			JCond ₈₉	1.5JCond ₈₉	RQD/3	(modified after Hoek et al., 2013 approach)
			Roughness	Infilling	Weathering				
1	2	4	1	4	5	16	24	28	52
2	3	4	5	4	5	21	31.5	32	63
3	3	3	5	4	5	20	30	32	62
4	3	4	1	4	5	17	25.5	30	55
5	3	4	5	4	5	21	31.5	30	55
6	2	2	5	4	3	16	24	20	44
7	2	1	1	4	5	13	19.5	27	46
8	2	4	1	4	5	16	24	28	53
9	2	3	1	4	5	15	22.5	26	48
10	2	3	5	4	5	19	28.5	28	57
11	2	4	5	4	3	18	27	27	54
12	2	4	5	4	3	18	27	28	55
13	2	3	5	4	3	17	25.5	23	49
14	2	4	5	4	3	18	27	25	52
15	2	4	5	4	6	21	31.5	30	62
16	2	3	5	4	5	19	28.5	22	50
17	2	3	5	4	5	19	28.5	20	49
18	2	3	1	4	3	13	19.5	23	43
19	2	4	1	4	5	16	24	21	45
20	2	2	5	4	3	16	24	23	47

Therefore, the following relationship was proposed (Kar Winn and Wong, 2017a) by replacing RQD/2 Eq. (4.19) by RQD/3 to obtain better correlation with the field measurements using the Hoek et al. (1995) GSI chart;

$$GSI = 1.5 JCond_{89} + RQD/3 \text{ ----- (4.24)}$$

The calculated GSI results by Eq. (4.24) are between 45 and 55 in majority as plotted in Fig. 4.13. Selective examples of quantitative GSI are presented in Table 4.8.

Fig. 4.13 Quantitative GSI of rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project by Hoek et al., 2013, approach with modification of vertical scale as $RQD/3$. (Kar Winn and Wong, 2017a)

4.12.4 Comparison of quantitative GSI with qualitative GSI (field assessment using chart by Hoek et al. (1995))

Table 4.9: Selective examples of comparison of quantitative GSI (by modified after Hoek et al., 2013; Cai et al., 2004; Sonmez & Ulusay, 1999; Russo, 2009 approaches) with qualitative GSI (field assessment using Hoek et al., 1995) of 420 excavated rock faces

Serial Num-bers	(modified after Hoek et al., 2013 approach)	(Cai et al., 2004 approach)	(Sonmez & Ulusay, 1999 approach)	(Russo, 2009 approach)	(Hoek et al., 1995 approach)
<i>Column a</i>	<i>Column b</i>	<i>Column c</i>	<i>Column d</i>	<i>Column e</i>	<i>Column f</i>
1	52	43	52	40	45
2	63	65	61	73	65
3	63	60	61	64	65
4	55	57	60	60	65
5	62	60	61	64	65
6	55	53	53	55	55
7	55	50	53	51	55
8	55	55	58	57.5	60
9	44	47	46	45	45
10	46	45	47	44	45
11	53	43	50	40	50
12	48	43	47	40	45
13	55	47	53	46	50
14	57	47	56	46	50
15	55	44	45.5	40	50
16	54	49	52	51	50
17	57	52	56	56	55
18	55	55	54	60	55
19	49	46	47	43	45
20	52	47	48	46	50
21	62	47	51	46	55
22	50	47	50	46	55
23	49	47	49	46	50
24	43	41	43.5	34	45
25	45	43	45	40	45
26	47	44	47.5	40	50

Determination of GSI was carried out in geological mapping of excavated rock faces in JRC project based on the chart proposed by Hoek et al. (1995). Those qualitative GSI and calculated GSI by Eq. (4.24) from the same rock faces were compared. Both results are presented in Table 4.9 columns b & f and graphically plotted in Fig. 4.14

which shows that results of quantitative GSI are slightly higher than qualitative GSI with a maximum of 7. However, both results can be considered to be within comparable range.

Fig. 4.14 Comparison of quantitative GSI (by modified after [Hoek et al., 2013](#); [Cai et al., 2005](#); [Sonmez & Ulusay, 1999](#); [Russo, 2009](#) approaches) with qualitative GSI (field assessment using Hoek et al., 1995) : selective examples from 420 excavated rock faces

4.13 Second approach: Determination of quantitative GSI in terms of block volume and joint condition factor proposed by Cai et al. (2004)

Cai et al. (2004) proposed a different concept of block size and joint conditions as input parameters to calculate quantitative GSI. After measurable quantitative input parameters are replaced in determination of quantitative output, the procedures do not rely largely on the experience and skill of the engineers or engineering geologists who are practically working for it.

The input parameter of the structure of rock block in original GSI chart (Hoek et al., 1995) is replaced with the quantitative block volume V_b , and the surface condition of the discontinuities is replaced with the quantitative joint condition factor J_c . Cai et al. (2004) made the correlation of V_b and J_c with GSI by using published data. The calculated results were later verified by back analysis carried out on two rock cavern excavation projects. In addition to the four structure categories in the original chart (Hoek et al., 1995), they added two more categories namely as massive and foliated/laminated/shear in their proposed quantitative GSI chart (Fig. A.11). Those additional two rock block categories match well with the update of the GSI chart by Marinos and Hoek (2000) for heterogeneous rock masses.

4.13.1 Block volume (V_b)

Block volume, which is determined from the joint spacing, joint orientation, number of joint sets and joint persistence, is the highly important indicator of rock mass quality. Block volume is a volumetric expression of joint density. In the cases that three or more joint sets are present and joints are persistent, the volume can be calculated (Palmstrom, 1995) as

$$V_b = \frac{S_1 S_2 S_3}{\sin \gamma_1 \sin \gamma_2 \sin \gamma_3} \text{-----} (4.25)$$

where S_1 , S_2 , S_3 and γ_1 , γ_2 , γ_3 are the joint spacing and the angle between joint sets, respectively (Fig. A.12). However, compared to the variation in joint spacing, the effect of the intersection angle between joint sets is relatively small. Hence, for practical purpose, the block volume can be approximated (Palmstrom, 1995) as

$$V_b = S_1 S_2 S_3 \text{ ----- (4.26)}$$

When irregular jointing is present, it is difficult to delineate three or more joint sets. In these cases, the block volume can be directly measured in the field after selecting some representative blocks. Other methods using RQD, the volumetric joint count J_v and weighted joint density can also be used.

4.13.2 Joint condition factor (J_c)

In the GSI system, the joint surface condition is defined by the roughness, weathering and infilling condition. The combination of these factors expresses the strength of a joint or block surface. In this study, a joint condition factor J_c which is similar to the factor proposed by [Palmstrom \(1995\)](#) is used to quantify the joint surface condition. This joint condition factor J_c is defined as

$$J_c = \frac{J_w J_s}{J_a} \text{ ----- (4.27)}$$

where J_w and J_s are the large-scale waviness (in meters from 1 to 10 m) and small-scale smoothness (in centimeters from 1 to 20 cm) respectively. J_a is the joint alteration factor. The ratings from Q and RMI system are adopted here and are listed in [Tables A.4, A.5 and A.6](#) for J_w , J_s and J_a respectively.

4.13.3 Input parameters for rock masses (Ayer Chawan Facies) in JRC project

First, block volume (V_b) and joint condition factor (J_c) of the 420 rock faces were calculated based on the information contained in the geological map ([Fig. 4.10](#)). V_b was calculated using Eq. (4.26) with spacing S_1 , S_2 and S_3 values whereas J_c was calculated using Eq. (4.27) with J_w , J_s and J_a . Selective examples of calculated V_b and J_c are presented in [Table 4.10](#).

4.13.4 Determination of quantitative GSI of rock masses (Ayer Chawan Facies) in JRC project

The quantitative GSI of rock masses in JRC is the intersection point after the calculated joint surface condition J_c and block volume V_b were plotted on the chart of [Cai et al. \(2004\)](#). Selective examples of quantitative GSI are presented in [Table](#)

4.10. The results of GSI generally ranged from 40 to 55 (Fig. 4.15). The massive limestone which occurred in limited area gave higher values between 60 and 65.

Fig. 4.15 Quantitative GSI of rock masses (Ayer Chawan Facies), JRC project, by Cai et al., 2004 approach (Kar Winn and Wong, 2017a)

Table 4.10: Selective examples of quantitative GSI determination of 420 excavated rock faces (Cai et al., 2004 approach) (Kar Winn and Wong, 2017a)

Serial Num- bers	Determination of Block volume				Determination of Joint condition factor				Quantitative GSI (Cai et al., 2004 approach)
	S ₁	S ₂	S ₃	V _b (cm ³)	J _w	J _s	J _a	J _c	
1	20	20	20	8000	1	2	2	1	43
2	40	40	100	160,000	1.5	2	1	3	65
3	40	20	20	16000	1.5	2	1	3	57
4	40	20	40	32000	1.5	2	1	3	60
5	20	20	40	16000	1	2	1	2	53
6	20	20	20	8000	1	2	1	2	50
7	20	20	20	8000	1.5	2	1	3	55
8	20	6	40	4800	1.5	2	2	1.5	47
9	20	20	40	16000	1	2	2	1	45
10	20	20	20	8000	1	2	2	1	43
11	20	20	20	8000	1.5	2	2	1.5	47
12	20	20	6	2400	1.5	2	2	1.5	44
13	20	20	40	16000	1.5	2	2	1.5	49
14	100	20	20	40000	1.5	2	2	1.5	52
15	20	40	100	80000	1.5	2	2	1.5	55
16	6	6	100	3600	1.5	2	2	1.5	46
17	20	20	20	8000	1.5	2	2	1.5	47
18	20	6	20	2400	1	2	2	1	41
19	20	20	20	8000	1	2	2	1	43
20	20	6	20	2400	1.5	2	2	1.5	44

4.13.5 Comparison of quantitative GSI (Cai et al., 2004) with qualitative GSI

(field assessment using Hoek et al., 1995)

Those quantitative GSI and field assessed qualitative GSI (chart by Hoek et al., 1995) from the same rock faces were compared (Table 4.9 columns c & f and Fig. 4.14) in a way similar to those carried out in section 4.12.4. The results of qualitative GSI were slightly higher than the quantitative GSI with a maximum of 5.

4.14 Third approach: Determination of quantitative GSI in terms of structure rating and surface condition factor proposed by Sonmez and Ulusay (1999)

Sonmez and Ulusay (1999) emphasized that surface conditions of the discontinuities and rock mass structures are the controlling factors to determine GSI accurately. Hence they suggested two terms, namely “structure rating, SR” estimated based on the volumetric joint count (J_v) and “surface condition rating, SCR”, estimated from the roughness, weathering and infilling characters of the discontinuity surfaces.

As described in Fig. A.13 (at the right upper margin) and Table A.7 (at the bottom row), the SCR values are calculated as follows;

$$SCR = R_r + R_w + R_f \quad \text{-----} \quad (4.28)$$

The rating for roughness R_r , weathering R_w and infilling R_f based on the assessment of discontinuities surface conditions are presented in Table A.7. Those are actually the same parameters involved in RMR system. As the total sum of the maximum ratings of three parameters is 18, the SCR axis in Fig A.13 is divided into 18 equal divisions (Sonmez and Ulusay, 1999).

The structure rating (SR) is determined from volumetric joint count (J_v) which is the total number of discontinuities existed in one cubic meter rock mass. The relationship of J_v to SR is shown on semi-log scale graph in Fig A.14 (Sonmez and Ulusay 1999). The quantitative GSI is then estimated from the intersection point of SCR and SR ratings after being plotted on Fig. A.13.

4.14.1 Input parameters for rock masses (Ayer Chawan Facies) in JRC project

First, the surface condition rating, SCR of the 420 rock faces were calculated using Eq. (4.28). The rating results of R_r , R_w , and R_f were determined based on the information contained in Geological map (Fig. 4.10). Secondly, the structure rating (SR) is determined from the semi-log scale graph in Fig A.14 based on volumetric joint count (J_v) recorded on (Fig. 4.10). Selective examples of calculated SCR and SR are presented in Table 4.11.

4.14.2 Determination of quantitative GSI of rock masses (Ayer Chawan Facies) in JRC project

Fig.4.16 Quantitative GSI of rock masses (Ayer Chawan Facies) in JRC project (Sonmez and Ulusay, 1999 approach)

The quantitative GSI of rock masses in JRC is the intersection point after the calculated SCR and SR were plotted on the Fig 4.16 (Sonmez and Ulusay 1999). Selective examples of quantitative GSI are presented in Table 4.11. The results of GSI generally ranged from 45 to 55 (Fig. 4.16). The massive limestone which occurred in limited area gave higher values of 60.

4.14.3 Comparison of quantitative GSI (Sonmez and Ulusay approach) with qualitative GSI (field assessment using Hoek's chart, 1995)

Table 4.11: Selective examples of quantitative GSI determination of 420 excavated rock faces (Sonmez & Ulusay, 1999 approach)

Serial Num- bers	Surface condition rating SCR				Structure rating SR		Quantitative GSI (Sonmez & Ulusay, 1999 approach)
	R _r roughness	R _w weathering	R _f infilling	SCR = R _r + R _w + R _f	Volumetr ic joint count J _v	Structur e rating SR	
1	3	6	4	13	9	54	52
2	5	6	4	15	6	64	61
3	5	6	4	15	7	61	60
4	5	6	4	15	6	64	61
5	3	6	4	13	8	58	53
6	5	6	4	15	8	58	58
7	5	5	4	14	18	34	46
8	3	6	4	13	11	41	47
9	5	5	4	14	9	54	53
10	5	5	4	14	20	32	45.5
11	5	6	4	15	9	54	56
12	5	5	4	14	9	54	54
13	5	5	4	14	14	38	47
14	5	5	4	14	12	40	48
15	5	6	4	15	12	40	51
16	5	6	4	15	15	37	50
17	5	6	4	15	16	36	49
18	3	5	4	12	14	38	43.5
19	3	6	4	13	16	36	45
20	5	5	4	14	13	39	47.5

Those quantitative GSI and field assessed qualitative GSI (chart by Hoek et al., 1995) from the same rock faces were compared (Table 4.9 columns d & f and Fig. 4.14) in a way similar to those carried out in previous approaches. The results of quantitative GSI are slightly higher than qualitative GSIs with a maximum of 6.

4.15. Fourth approach: Determination of quantitative GSI in terms of RMI parameters J_c and V_b proposed by Russo (2009)

Russo (2007 & 2009) proposed quantitative GSI based on the jointing parameters such as joint condition factor (J_c) and block volume (V_b in cm^3) as shown in Fig. A.15.

The joint condition factor, J_c , originally proposed by Palmstrom (1995) is as follows;

$$J_c = J_r \cdot J_L / J_a \text{ ----- (4.29)}$$

The ratings for J_r and J_a are shown in Table A.3 (Barton et al., 1974).

Alternatively J_r is defined as

$$J_r = J_w \cdot J_s \text{ ----- (4.30)}$$

Hence,

$$J_c = J_w \cdot J_s \cdot J_L / J_a \text{ ----- (4.31)}$$

The value of $J_L = 1$ (Table A.8) if the joint lengths are within 1 to 10 meters. By taking this Cai et al. (2004) proposed as $J_c = J_w \cdot J_s / J_a$, Eq. (4.27). Since the majority of joint lengths are within this range both equations Eq. (4.27) and Eq. (4.31) are valid for rock masses at JRC project. The block volume is taken from the product of spacing among the major discontinuities (bedding and joints) similarly in Eq. (4.26).

4.15.1 Input parameters for rock masses (Ayer Chawan Facies) in JRC project

First, block volume (V_b) and joint condition factor (J_c) of the 420 rock faces were calculated based on the information contained in Geological map (Fig. 4.10). V_b was

calculated using Eq. (4.26) with spacing S_1 , S_2 and S_3 values whereas J_c was calculated using Eq. (4.27) with J_w , J_s and J_a . Selective examples of calculated V_b and J_c are presented in Table 4.12.

4.15.2 Determination of quantitative GSI of rock masses (Ayer Chawan Facies) in JRC project

Table 4.12: Selective examples of quantitative GSI determination of 420 excavated rock faces (Russo, 2009 approach)

Serial Num- bers	Determination of Block volume				Determination of Joint condition factor				Quantitative GSI (Russo, 2009 approach)
	S_1	S_2	S_3	V_b (cm ³)	J_w	J_s	J_a	J_c	
1	20	20	20	8000	1	2	2	1	40
2	40	40	100	160,000	1.5	2	1	3	73
3	40	20	40	32000	1.5	2	1	3	64
4	40	20	20	16000	1.5	2	1	3	60
5	40	20	40	32000	1.5	2	1	3	64
6	20	20	40	16000	1	2	1	2	55
7	20	20	20	8000	1	2	1	2	51
8	20	20	20	8000	1.5	2	1	3	57.5
9	20	6	40	4800	1.5	2	2	1.5	45
10	20	20	40	16000	1	2	2	1	44
11	20	20	20	8000	1	2	2	1	40
12	20	20	20	8000	1.5	2	2	1.5	46
13	20	20	40	16000	1.5	2	2	1.5	51
14	100	20	20	40000	1.5	2	2	1.5	56
15	20	40	100	80000	1.5	2	2	1.5	60
16	6	6	100	3600	1.5	2	2	1.5	43
17	20	20	20	8000	1.5	2	2	1.5	46
18	20	20	20	8000	1.5	2	2	1.5	46
19	20	6	20	2400	1	2	2	1	34
20	20	6	20	2400	1.5	2	2	1.5	40

The quantitative GSI of rock masses in JRC is the intersection point after the calculated joint surface condition J_c and block volume V_b were plotted on the chart

of Russo (2009) Fig. A.15. Selective examples of quantitative GSI are presented in Table 4.12. The results of GSI generally ranged from 40 to 55 (Fig. 4.17). The massive limestone which occurred in limited area gave higher values of 60.

Fig. 4.17 Quantitative GSI of rock masses (Ayer Chawan Facies) in JRC project (Russo, 2009 approach)

4.15.3 Comparison of quantitative GSI (Russo approach) with qualitative GSI (field assessment using [Hoek et al., 1995](#))

Those quantitative GSI and field assessed qualitative GSI (chart by [Hoek et al., 1995](#)) from the same rock faces were compared ([Table 4.9 columns e & f](#) and [Fig. 4.14](#)) in a way similar to those carried out in previous approaches. The results of qualitative GSI were found to be slighter higher than the quantitative GSI with maximum of 10.

4.16 Summary and discussion

Microscopic petrographic identification for selected rocks was carried out to gain proper nomenclature of rocks. Lithic tuff containing different rock fragments are observed.

Measured discontinuities in one storage cavern (CS '1/1) was analysed by using DIPS programme to identify the prominent sets in JRC project. From the contour plot analysis, 1 set of bedding with varying dip angles (from 5° to 25°), 4 sets of joint and 2 sets of fault are grouped as prominent sets in the cavern. Those observed discontinuities data can be applied to check whether any unstable roof wedge may be formed on the roof and walls of the cavern which are harmful during the works.

The results of measured horizontal stress of sedimentary rocks in JRC project were compared with available stress results measured in granitic rocks in other Singapore projects. Although the range of stress ratios in Bukit Timah Granite is higher than those of Jurong Formation from Jurong Island, the general trend of maximum stress in both formation are in the same range of alignment. It is also in the same general direction of maximum horizontal stress in Sumatra and southern part of Peninsular Malaya, i.e. NE-NNE. The reason of being similarity is due to tectonic stresses developed at the convergent boundary (Sumatra-Java trench) between the Euro-Asian and the Indian Ocean Plate. The measured trend and magnitude of the horizontal stresses are useful in the determination of tunnel alignment and the crown stability. The higher horizontal stresses in both Formations provide general favorable stable conditions for the construction of large span storage caverns.

The cumulative distribution of the 79 coefficient of permeability data at water bearing zone indicated that k value is in the order of 1×10^{-6} m/s and the average value is 1.73×10^{-6} m/s. The calculated coefficient of permeability of recharging wells in one water curtain gallery was in the comparable range from 1×10^{-6} m/s to 1×10^{-7} m/s. Both results show that there is no high permeable water zone in the project.

Strength properties of intact rocks by point load tests, uniaxial compression test, triaxial compression tests and Brazilian tests were measured at laboratories in NTU and Beijing University. The conversion factor to predict UCS (MPa) from point load strength index (MPa) is calculated from 20 to 21 on Ayer Chawan rock mass. The results of Brazilian tensile strength which are tested before and during project are in the majority range between 6 MPa to 10 MPa. The Brazilian tensile strength results in JRC are higher than the similar type of typical sedimentary rocks elsewhere on the world (5 MPa for sandstone and 2 MPa for mudstone; [Waltham, 1994](#)). The ratio of UCS/BTS falls in the range of 15 to 20 which is in agreement the ratio of 20 by [Waltham \(1994\)](#). Tensile strength of rock is one of the important criterions to determine the tunnel roof stability.

Direct shear tests on rock joints of core samples were also carried out at NTU laboratory. Among the 16 direct shear tests, some were carried out at maximum 5 MPa normal stress condition whereas some are under maximum 16 MPa. The mean values of cohesion (0.5 MPa) and friction angles (33°) were calculated out of 16 results. After consideration of scale factor, 0.3 MPa & 30° for bedding and 0.2 MPa & 30° for joints are reasonable to use in the analyses. It was discussed and agreed with the consultants of JRC Project to use for further analyses in the project.

Those measured intact rock properties are fundamental data to determine the rock mass properties of the project area.

Two rock classification systems were determined independently during geological mapping on excavation faces at some designated locations. RMR was recorded between 60 and 80 while Q values scattered between 4 and 70. The general correlation equation between Q and RMR in JRC is comparable with that proposed

by [Rutledge and Perston \(1978\)](#). The correlation factor can be used as reference for future projects.

The quantitative Geological Strength Index (GSI) of selected 420 excavated rock faces in JRC was determined by four different approaches available in the literature. The required rock mass parameters for calculation were based on the records of geology face mapping works carried out during the construction period.

Two basic parameters (blockiness of rock mass and surface conditions of discontinuities) necessary for the GSI determination were defined and calculated by four different approaches.

- [Hoek et al. \(2013\)](#) followed the joint condition rating of Rock Mass Rating (RMR) proposed by [Bieniawski \(1989\)](#) for surface condition and RQD for blockiness calculation.
- In [Cai et al. \(2004\)](#) the rock mass characterization system (RMi) proposed by [Palmstrom \(1995\)](#) was used for the rating of discontinuity surface and the individual rock block volume for blockiness determination.
- [Sonmez and Ulusay \(1999\)](#) followed the rating of Roughness R_r , Weathering R_w and Infilling R_f in [Bieniawski \(1989\)](#) RMR for surface condition rating and the volumetric joint counts for blockiness rating.
- [Russo \(2007 & 2009\)](#) applied the surface condition and blockiness rating similar to those by [Cai et al. \(2004\)](#) but he proposed a different GSI graph.

Although quantitative GSI was calculated by four different means, all the results are approximately in the same range from 45 to 55. Comparison was made with the qualitative GSI which were obtained based on assessment on the rock faces using the chart by [Hoek et al. \(1995\)](#). The quantitative GSI and qualitative GSI varied within a range of ± 10 .

As a modification to the [Hoek et al. \(2013\)](#) approach, a new GSI relationship was proposed by replacing RQD/2 with RQD/3 as the former was found to provide much higher GSI which do not match the qualitative field values.

The observed qualitative and quantitative GSI in JRC project is used as a guide for other sedimentary rocks in Singapore but some justification is suggested to suit for the others. The field determination of GSI is suggested to be carried out in every

rock excavation project as it is the basic parameters for rock mass properties calculation.

The four approaches presented in this chapter for the quantitative GSI determination are all user friendly, and applicable to rock masses (Ayer Chawan Facies) in JRC Project. Thus it is recommended to choose any one of them, depending on the availability of rock mass parameters in the assigned project. In conclusion the four approaches are almost equally reliable to be used.

CHAPTER 5

Rock mass properties of Ayer Chawan Facies, Jurong Formation in JRC Project

5.1 Generalized Hoek-Brown Criteria

Determination of the reliable strength and deformation parameters of rock mass is crucial for the design analysis of slopes, foundations and underground excavations. [Hoek and Brown \(1980a, 1980b\)](#) proposed an approach to estimate the strength of jointed rock masses based on the structural nature of the interlocking rock blocks and the discontinuity surface condition between these blocks. This approach was updated for years to fulfill the necessities of the end users who were employing it to problems without consideration of the rock block structures and discontinuities surface condition ([Hoek, 1983; Hoek and Brown, 1988](#)). After further changes were required to apply this approach on very poor quality rock masses ([Hoek et al., 1992](#)), a new classification system namely the Geological Strength Index ([Hoek, 1994; Hoek et al., 1995; Hoek and Brown, 1997; Hoek et al., 1998; Marinos and Hoek, 2001.](#)) was developed eventually.

[Hoek et al. \(2002\)](#) made revision on Hoek-Brown failure criterion, relationship between major and minor principal stresses for Hoek-Brown and equivalent Mohr-Columb failure criteria. An associated Windows program called “RocData” has been developed to provide a convenient means of solving and plotting the equations to estimate intact and rock mass parameters. The determination of the rock masses deformation modulus was consequently modified by [Hoek and Diederichs \(2006\)](#).

The generalized Hoek-Brown failure criteria for jointed rock masses is defined by

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left[m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right]^a \text{-----(5.1)}$$

σ'_1 and σ'_3 are the maximum and minimum effective principal stresses at failure,

m_b is the value of the Hoek-Brown constant m for the rock mass,

s and a are constants which depend on the rock mass characteristics, and

σ_{ci} is the uniaxial compressive strength of the intact rock pieces.

To apply the Hoek-Brown failure criterion to estimate the strength and deformation characteristics of jointed rock masses, the following three properties of the rock mass have to be determined.

- Uniaxial compressive strength σ_{ci} of the intact rock pieces
- Value of the Hoek-Brown constant m_i for these intact rock pieces
- Value of the Geological Strength Index (GSI) for the rock mass

5.2 The intact rock properties (σ_{ci} and m_i)

For the intact rock pieces that make up the rock mass, Eq. (5.1) simplified to

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left[m_b \frac{\sigma'_3}{\sigma_{ci}} + 1 \right]^{0.5} \text{-----} (5.2)$$

The correlation between the principal stresses at failure for a given rock is defined by two constants, the uniaxial compressive strength σ_{ci} and a constant m_i . These constants' values can be determined after statistical analysis of the results of a set of triaxial tests on good quality core specimens.

After the five or more triaxial test results have been obtained, the uniaxial compressive strength σ_{ci} and Hoek-Brown constant m_i as defined by [Hoek and Brown \(1980a\)](#) can be determined. In this analysis, Eq. (5.2) is re-written in the form:

$$y = m \sigma_{ci} x + s \sigma_{ci} \text{-----} (5.3)$$

Where $x = \sigma'_3$ and $y = [\sigma'_1 - \sigma'_3]^2$

For n specimens the uniaxial compressive strength σ_{ci} , the constant m_i and the coefficient of determination r^2 are calculated from:

$$(\sigma_{ci})^2 = \frac{\sum y}{n} - \left[\frac{\sum xy - \left(\sum x \sum \frac{y}{n} \right)}{\sum x^2 - \left(\frac{(\sum x)^2}{n} \right)} \right] \frac{\sum x}{n} \text{-----} (5.4)$$

$$m_i = \frac{1}{\sigma_{ci}} - \left[\frac{\sum xy - \left(\sum x \sum \frac{y}{n} \right)}{\sum x^2 - \left(\frac{(\sum x)^2}{n} \right)} \right] \frac{\sum x}{n} \text{-----} (5.5)$$

$$r^2 = \frac{\left[\sum xy \frac{\sum x \sum y}{n} \right]^2}{\left[\sum x^2 \frac{(\sum x)^2}{n} \right] \left[\sum y^2 \frac{(\sum y)^2}{n} \right]} \quad (5.6)$$

The high quality triaxial test data will usually give a coefficient of determination r^2 of greater than 0.9. When the laboratory tests are not available, [Table A.9](#) and [A.10](#) can be used to obtain estimated σ_{ci} and m_i .

5.3 Determination of intact rock properties (σ_{ci} and m_i) of Ayer Chawan Facies, Jurong Formation in JRC Project

A large number of studies have revealed that a small increase in the water content may significantly lower the strength and stiffness of rocks particularly the sedimentary rocks ([Broch, 1979](#); [Chen et al., 2004](#); [Erguler and Ulusay, 2009](#)). However, few experimental studies are found in the literatures discussing the influence of water content on the triaxial compressive strength and deformability of rocks under triaxial compression.

Eighty four (84) rock core specimens from JRC Project were tested at Institute of Rock and Soil Mechanics of Chinese Academy of Science (Wuhan), during October 2010 and end January 2011 ([NTU, 2011](#)). Two different rock types of mudstone and pyroclastic rocks were tested under dry and wet condition to explore the water content effect on the strength and deformability. Those rock names were given after careful examination of the core sample photographs and sedimentary features.

The test samples were cored from fresh intact rock blocks from JRC project. Cutting and polishing work of the cylindrical cores were carried out by a diamond cutter with an acute blade and grinder. In accordance with the ISRM suggested method ([ISRM 1978, 1983](#)), the ratio of rock specimens was approximately 2 with a specimen height of 100 mm and diameter of 50 mm. After the dimensions of the prepared specimens were measured, the specimens were subjected to ultrasound cleaning in distilled water. After that, they were palced in a 105° C oven for at least 24 hours. The dry mass of the specimens were then measured after they had cooled down to room temperature. For those specimens to be tested in a wet condition, they were immersed into sea water within a container maintained at a pressure slightly

less than 1 atm (760 mm Hg) for 14 days, which were then transferred to a big plastic container filled with sufficient sea water to fully cover them.

Fig. 5.1 Photographs of rock core samples after triaxial test completed (First three samples were tested under loading direction normal to the bedding planes whereas the rest parallel direction.) (NTU, 2011)

Finally they were weighted again to obtain the wet mass of the specimens. Much effort has been made to allow those wet specimens to absorb as much as water as possible, but it is still difficult to achieve a fully saturated state by immersing them

in water or water vacuum. The water absorption of both rock types is very low, which are about 0.17% for mudstone and about 0.10% for pyroclastic rocks. Therefore the term wet was used instead of saturated for the rocks prepared and tested in the present study.

The specimens in each group were tested under four different confining pressure (σ_3). Minimum 5 samples from each group were tested under same confining pressure in which the first three with loading direction normal to the bedding planes and the remaining with loading parallel to the bedding planes as shown in Fig. 5.1. Peak and residual strength, axial and lateral failure strain of each specimen were also recorded.

5.3.1 Intact rock properties of dry mudstone

Table 5.1 Triaxial test data of dry mudstone

Triaxial compression test σ_1 (MPa)			
$\sigma_3 = 1.25$ MPa	$\sigma_3 = 5$ MPa	$\sigma_3 = 10$ MPa	$\sigma_3 = 15$ MPa
135.9	158.9	189.1	294.5
171.9	169.1	205.1	332.4
189.5	196.7	206.9	
195.1	244.7	241	
	271.3	258.9

Fig. 5.2 Mohr-Coulomb criterion and Mohr circles for the strength of dry mudstone

As shown in Table 5.1, there are maximum five σ_1 values for each four different σ_3 (1.25, 5, 10 & 15 MPa). From the data, σ_1 value is selected at each σ_3 to obtain sets of data pairs. In this case a total of sixty sets of data are obtained (e.g. $\sigma_1 : \sigma_3 = 1.25:135.9, 5:158.9, 10:189.1, 15:294.5 = 1.25:135.9, 5:159.1, 10:189.1, 15:294.5$ etc). Subsequently, a total of sixty sets of the intact uniaxial compressive strength (σ_{ci}), Hoek-Brown constant (m_i), cohesion (c , MPa) and friction angles (ϕ degree) were attained after sixty data pairs are analyzed by applying RocData 3.0 program by Rocscience Inc. (Rocscience, Inc., 2004). The sample plot of Mohr circles to determine Mohr-Coulomb criterion was presented in Fig. 5.2. Alternatively, intact uniaxial compressive strength (σ_{ci}) and Hoek-Brown constant (m_i) can also be determined by using Eq (5.4 and 5.5) respectively. The mean and standard deviation of those results are presented in Table 5.5 together with other rock types.

5.3.2 Intact rock properties of wet mudstone

Similarly in section 5.3.1, a total of thirty (30) sets of data are obtained from 4 sets of σ_3 with related σ_1 (Table 5.2) (e.g. $\sigma_1 : \sigma_3 = 1.25:143.3, 5:158.4, 10:158.8, 15:165.1 = 1.25:143.3, 5:193.1, 10:158.8, 15:165.1$ etc). Subsequently, a total of thirty sets of the intact uniaxial compressive strength (σ_{ci}), Hoek-Brown constant (m_i), cohesion (c , MPa) and friction angles (ϕ degree) were attained after thirty data pairs are analyzed by applying RocData 3.0 program by Rocscience Inc. (Rocscience, Inc., 2004). The sample plot of Mohr circles to determine Mohr-Coulomb criterion was presented in Fig. 5.3. Alternatively, intact uniaxial compressive strength (σ_{ci}) and Hoek-Brown constant (m_i) can also be determined by using Eq (5.4 and 5.5) respectively. The mean and standard deviation of those results are presented in Table 5.5 together with other rock types.

Table 5.2 Triaxial test data of wet mudstone

Triaxial compression test σ_1 (MPa)			
$\sigma_3 = 1.25$ MPa	$\sigma_3 = 5$ MPa	$\sigma_3 = 10$ MPa	$\sigma_3 = 15$ MPa
143.3	158.4	158.8	165.1
152.5	193.1	181.2	177.6
158.2	201.9	202.1	199.5
179.1			208.2
199.4			

Analysis of Rock/Soil Strength using RocData

Mohr-Coulomb Criterion

cohesion = 36.119 MPa
friction angle = 38.163 deg
tensile strength = 0 MPa
uniaxial compressive strength = 148.6 MPa
alpha = 76.71 deg

Analysis of TRIAXIAL Lab Data

No. of lab data points = 4
Sum square of errors (Residuals) = 287.656
Current strength model is NOT a 'best-fit'

Fig 5.3 Mohr-Coulomb criterion and Mohr circles for the strength of wet mudstone

5.3.3 Intact rock properties of dry pyroclastic rock

Similarly in [section 5.3.1](#), a total of one hundred and thirty (130) sets of data are obtained from 4 sets of σ_3 with related σ_1 ([Table 5.3](#)) (e.g. $\sigma_1 : \sigma_3 = 1.25:190.4, 5:210.3, 10:282.5, 15:282.8 = 1.25:190.4, 5:247.2, 10:282.5, 15:282.8$ etc). Subsequently, a total of 130 sets of the intact uniaxial compressive strength (σ_{ci}), Hoek-Brown constant (m_i), cohesion (c , MPa) and friction angles (ϕ degree) were attained after 130 data pairs are analyzed by applying RocData 3.0 program by Rocscience Inc. ([Rocscience, Inc., 2004](#)). The sample plot of Mohr circles to determine Mohr-Coulomb criterion was presented in [Fig. 5.4](#). Alternatively, intact uniaxial compressive strength (σ_{ci}) and Hoek-Brown constant (m_i) can also be determined by using [Eq \(5.4 and 5.5\)](#) respectively. The mean and standard deviation of those results are presented in [Table 5.5](#) together with other rock types.

Analysis of Rock/Soil Strength using RocData

Fig. 5.4 Mohr-Coulomb criterion and Mohr circles for the strength of dry pyroclastic rock

Table 5.3 Triaxial test data of dry pyroclastic rock

Triaxial compression test σ_1 (MPa)			
$\sigma_3 = 1.25$ MPa	$\sigma_3 = 5$ MPa	$\sigma_3 = 10$ MPa	$\sigma_3 = 15$ MPa
190.4	210.3	282.5	282.8
192.5	247.2	291.1	282.9
198.9	288.2	329.6	308
247.5	336	333.6	329
350.1		344.3	338.7
		351.4	344

5.3.4 Intact rock properties of wet pyroclastic rock

Similarly in [section 5.3.1](#), a total of sixty (60) sets of data are obtained from 4 sets of σ_3 with related σ_1 ([Table 5.3](#)) (e.g. $\sigma_1 : \sigma_3 = 1.25:217.2, 5: 308.4, 10: 316.6, 15: 348.2 = 1.25:217.2, 5: 319.4, 10: 316.6, 15: 348.2$ etc). Subsequently, a total of 60 sets of the intact uniaxial compressive strength (σ_{ci}), Hoek-Brown constant (m_i), cohesion (c , MPa) and friction angles (ϕ degree) were attained after sixty data pairs are analyzed by applying RocData 3.0 program by Rocscience Inc. ([Rocscience, Inc., 2004](#)). The sample plot of Mohr circles to determine Mohr-Coulomb criterion was presented in [Fig. 5.5](#). Alternatively, intact uniaxial compressive strength (σ_{ci}) and Hoek-Brown constant (m_i) can also be determined by using Eq (5.4 and 5.5) respectively. The mean and standard deviation of those results are presented in [Table 5.5](#) together with other rock types.

Analysis of Rock/Soil Strength using RocData

Mohr-Coulomb Criterion

cohesion = 54.284 MPa
friction angle = 46.036 deg
tensile strength = 0 MPa
uniaxial compressive strength = 269 MPa
alpha = 80.75 deg

Analysis of TRIAXIAL Lab Data

No. of lab data points = 4
Sum square of errors (Residuals) = 293.711
Current strength model is NOT a 'best-fit'

Fig. 5.5 Mohr-Coulomb criterion and Mohr circles for the strength of wet pyroclastic rock

Table 5.4 Triaxial test data of wet pyroclastic rock

Triaxial compression test σ_1 (MPa)			
$\sigma_3 = 1.25$ MPa	$\sigma_3 = 5$ MPa	$\sigma_3 = 10$ MPa	$\sigma_3 = 15$ MPa
217.2	308.4	316.6	348.2
275.1	319.4	317	367.1
288.2	326.9	331.9	

Table 5.5 Summary of cohesion and friction angles of different rock types under dry and wet condition with other intact rock properties (σ_{ci} and m_i) (Ayer Chawan Facies, Jurong Formation in JRC Project)

Water condition and rock type	Cohesion (c, MPa)		Friction angle (ϕ degree)		<i>uniaxial compressive strength</i> σ_{ci} (MPa)	Hoek- Brown constant (m_i)
	Mean value	Standard deviation	Mean value	Standard deviation		
Dry mudstone	26	6	54	3	156	32
Wet mudstone	36	3	37	3	143	7
Dry pyroclastic rock	29	7	59	4	207	34
Wet pyroclastic rock	55	12	45	6	266	13

From the above triaxial compressive tests, it was observed that the cohesion (c) increased and friction angle (ϕ degree) decreased from dry to wet conditions for both tested rock types (Li et al., 2012). Based on the Hoek-Brown failure criterion, the H-B constant of m_i was found to decrease obviously from dry to wet conditions for both tested rock types. The uniaxial compressive strength (σ_{ci}) was decreased in mudstone from dry to wet conditions but observed increase in pyroclastic rocks. It is interpreted that the bonding strength between individual rock fragments in pyroclastic rocks became weaker when they are oven dried under 105° C.

5.4 Input parameters in determination of rock mass properties from intact rock properties

The failure characteristics of jointed rock mass mainly depends on the geometrical shape of the intact rock pieces and the surface condition of discontinuities which

separated those pieces. The angular rock pieces having clean, rough discontinuity surface will make a much stronger rock mass than one with rounded particles whose surfaces are coated with weathered and altered materials.

5.4.1 The Geological Strength Index (GSI) of JRC Project

The GSI system, which was briefly introduced by [Hoek et al. \(1992\)](#) and developed by [Hoek \(1994\)](#) and [Hoek et al. \(1995\)](#), gives a number that, after combination with the intact rock properties, can be applied as a reduction factor to estimate the rock mass strength under different geological situation. The GSI estimation of rock masses in JRC Project was presented in [Chapter 4](#).

5.4.2 Disturbance factor (D) estimation of excavated faces in JRC Project

The blast damage factor D should be determined on the excavated rock faces in the field. If possible, un-damaged face is to be assessed for evaluation of damaged factor to be used. The determination of disturbance factor (D) was made following guide line in [Fig. A.16](#).

The following D value of zero and 0.5 were adopted for JRC project.

D = 0

D = 0.5

Fig. 5.6 Estimated disturbance factor (D) of common excavated rock faces at JRC

5.5 Determination of rock mass properties of Ayer Chawan Facies, Jurong Formation in JRC Project

5.5.1 Hoek-Brown criterion m_b , s and a for rock mass

The influence of blast damage factor in the determination of rock mass properties has been considered in the 2002 version of the Hoek-Brown criterion ([Hoek et al., 2002](#)) as follows;

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right) \quad (5.7)$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right) \quad (5.8)$$

$$a = \frac{1}{2} + \frac{1}{6} \left(e^{-\frac{GSI}{15}} - e^{-20/3} \right) \quad (5.9)$$

D is a damage factor which is controlled by the degree of blasting and stress relaxation as shown in [Section 5.4.2](#).

5.5.2 Rock mass deformation modulus (E_{rm})

[Serafim and Pereira \(1983\)](#) proposed a relationship between the in situ modulus of deformation and Bieniawski's RMR classification. This relationship is based upon back analysis of dam foundation deformation and it has been observed working well for better quality rocks. However, for many of the poor quality rocks it appears to predict deformation modulus values in much higher.

Based upon practical observations and back analysis of excavation behaviour in poor quality rock masses, [Hoek and Brown \(1997\)](#) modified Serafim and Pereira's equation and proposed the following relationship for $\sigma_{ci} < 100$;

$$E_{rm} \text{ (GPa)} = \sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{\left(\frac{GSI-10}{40}\right)} \quad (5.10)$$

It can be noted that GSI has been substituted for RMR in this equation and that the modulus E_{rm} is reduced progressively as the value of σ_{ci} falls below 100. This reduction is based upon the reasoning that the deformation of better quality rock masses is controlled by the discontinuities while, for poorer quality rock masses, the

deformation of the intact rock pieces contributes to the overall deformation process. Based upon measured deformations, Eq. (5.10) appears to work reasonably well in those cases where it has been applied. However, as more field evidence is gathered it may be necessary to modify this relationship.

Hoek, Carranza and Corkum (2002) again modified the original equation proposed by Hoek and Brown (1997) after adding the factor D, to consider the effects of blast damage and stress relaxation.

$$E_{rm} \text{ (GPa)} = \left(1 - \frac{D}{2}\right) \sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{\left(\frac{GSI-10}{40}\right)} \quad (5.11)$$

Later, Hoek and Diederichs (2006) re-evaluated the current empirical estimation of rock mass deformation modulus and concluded that none of these approaches provided reliable estimates over the whole range of rock mass conditions. Especially, large errors were found in the two extreme cases, e.g. for very poor rock masses and massive strong rock masses. Fortunately, the newly measured data from China and Taiwan was reliable to be used in the analysis. It was observed that the equation which gave the best fit to this data is a sigmoid function having the form:

$$y = c + \frac{a}{1 + e^{-\left(\frac{x-x_0}{b}\right)}} \quad (5.12)$$

Using commercial curve fitting program, Eq. (5.12) was fixed with the measured data from Chinese and Taiwanese and the constants a and b in the fixed equation were then substituted with GSI and the disturbance factor D. Those were adjusted to give the equivalent average curve after the upper and lower bounds were fixed over 90% of the data points. Note that the constant $a = 100,000$ in Eq. (5.13) is a scaled factor and it is not directly interrelated to the rock mass physical properties.

The following best-fit equation was derived;

$$E_{rm} \text{ (MPa)} = 100,000 \left(\frac{1 - \left(\frac{D}{2}\right)}{1 + e^{\left(\frac{75 + 25D - GSI}{11}\right)}} \right) \quad (5.13)$$

Hoek and Diederichs (2006) carried out further detailed analysis and the above Eq. (5.13) will be termed as the simplified Hoek and Diederichs equation.

The results of rock mass deformation modulus determined from China and Taiwan was integrated with geological data and the uniaxial compressive strength (σ_{ci}) of the intact rock. This information allowed a more detailed analysis including the ratio of rock mass to intact rock modulus (E_{rm} / E_i). Using the modulus ratio MR introduced by [Deere \(1968\)](#) (modified by [Hoek and Diederichs, 2006](#)) based on part on this data set and also on other correlations from [Palmstrom and Singh \(2001\)](#), it is feasible to determine the intact modulus from the following equation;

$$E_i = MR \cdot \sigma_{ci} \text{ ----- (5.14)}$$

This relationship is useful when the intact modulus (E_i) are not available or it is not possible to determine the modulus (E_i) from highly jointed rock mass. After E_i was obtained using Eq. (5.14), the deformation modulus of rock mass can be calculated from the following equation;

$$E_{rm} = E_i \left(0.02 + \frac{1-D/2}{1 + e^{\left(\frac{60+15D-GSI}{11} \right)}} \right) \text{ ----- (5.15)}$$

This equation incorporates a finite value for the parameter c (Eq. 5.12) to account for the modulus of broken rock having $GSI = 0$ such as transported rock, aggregate or soil. This equation is plotted against the average normalized field data from China and Taiwan in [Fig. A.17](#).

The modulus ratio (MR) [Table A.11](#) proposed by [Deere \(1968\)](#) can be used for estimating of the intact modulus E_i since the reliable result can hardly be obtained due to the rock specimen disturbance. The quality of specimen used for laboratory test gave a more significant impact on the results of modulus than on strength.

5.5.3 Rock mass properties of four different types of rock masses (GSI = 40 and 55)

Subsequent Hoek-Brown criterion of rock mass (m_b , s and a) can be calculated by applying Eq. (5.7, 5.8 & 5.9) based on the intact rock parameters (m_i) in [section 5.3](#) with input of GSI values ($GSI = 40$ and 55) and factor D in [Fig. 5.6](#). The rock mass modulus (E_{rm}) was calculated by using equation, Eq (5.11) by [Hoek, Carranza and Corkum \(2002\)](#); simplified equation, Eq. (5.13) by [Hoek and Diederichs \(2006\)](#) and

equation, Eq. (5.15) by Hoek and Diederichs (2006) with modulus ratio of 200 for mudstone and 300 for pyroclastic rock respectively. The calculated results of intact and rock mass properties were summarized in Table 5.6 a, b, c & d for four different rock types.

Table 5.6a Rock mass properties of dry mudstone, Jurong Rock Cavern Project

Classification Disturbance factor GSI MR	Parameters	Dry mudstone			
		D = 0		D = 0.5	
		40	55	40	55
		200			
Intact rock properties	σ_{ci} (MPa)	156	156	156	156
	m_i	32	32	32	32
Rock mass properties	m_b	3.75	6.41	1.84	3.75
	s	0.0013	0.0067	0.0003	0.0025
	a	0.51	0.50	0.51	0.50
	E_{rm} (MPa) Hoek, Corranza and Corkum (2002) (Eq. 5.11)	5623	13300	4218	10000
	E_{rm} (MPa) Simplified Hoek and Diederichs (Eq. 5.13)	3986	13965	986	3714
	E_{rm} (MPa) Hoek and Diederichs (2006) (Eq. 5.15)	4981	12750	2399	6300

Table 5.6b Rock mass properties of wet mudstone, Jurong Rock Cavern Project

Classification Disturbance factor GSI MR	Parameters	wet mudstone			
		D = 0		D = 0.5	
		40	55	40	55
		200			
Intact rock properties	σ_{ci} (MPa)	143	143	143	143
	m_i	7	7	7	7
Rock mass properties	m_b	0.82	1.4	0.40	0.82
	s	0.0013	0.0067	0.0003	0.0025
	a	0.51	0.5	0.51	0.5
	E_{rm} (MPa) Hoek, Corranza and Corkum (2002) (Eq. 5.11)	5623	13300	4218	10000
	E_{rm} (MPa) Simplified Hoek and Diederichs (Eq. 5.13)	3986	13965	986	3714
	E_{rm} (MPa) Hoek and Diederichs (2006) (Eq. 5.15)	4566	11700	2199	5800

Table 5.6c Rock mass properties of dry pyroclastic rocks, Jurong Rock Cavern Project

Classification Disturbance factor	Parameters	Dry pyroclastic rocks			
		D = 0		D = 0.5	
		40	55	40	55
		300			
Intact rock properties	σ_{ci} (MPa)	207	207	207	207
	m_i	34	34	34	34
Rock mass properties	m_b	3.99		1.95	
	s	0.0013		0.0003	
	a	0.51		0.511	
	E_{rm} (MPa) Hoek, Corranza and Corkum (2002) (Eq. 5.11)	5623	13300	4218	10000
	E_{rm} (MPa) Simplified Hoek and Diederichs (Eq. 5.13)	3986	13965	986	3714
	E_{rm} (MPa) Hoek and Diederichs (2006) (Eq. 5.15)	9914	25350	4775	12550

Table 5.6d Rock mass properties of wet pyroclastic rocks, Jurong Rock Cavern Project

Classification Disturbance factor	Parameters	Wet pyroclastic rocks			
		D = 0		D = 0.5	
		40	55	40	55
		300			
Intact rock properties	σ_{ci} (MPa)	266	266	266	266
	m_i	13	13	13	13
Rock mass properties	m_b	1.53		0.75	
	s	0.0013		0.0003	
	a	0.51		0.51	
	E_{rm} (MPa) Hoek, Corranza and Corkum (2002) (Eq. 5.11)	5623	13300	4218	10000
	E_{rm} (MPa) Simplified Hoek and Diederichs (Eq. 5.13)	3986	13965	986	3714
	E_{rm} (MPa) Hoek and Diederichs (2006) (Eq. 5.15)	12740	32580	6136	16150

5.6 Rock mass properties of Bukit Timah Granite in Singapore

A three layer model, consisting of a thick layer of residual soil, the weathered granite and fresh granite bedrock can describe the geology of the Bukit Timah Granite at the Mandai site (Zhou, 2001).

Dense jointed strips often occur in the rock mass. They are generally sub-vertical and randomly distributed with a spacing of more than 5 meters between strips. The rock mass permeability at the site is also relatively low with most tunnels and chamber showing very little seepage water.

5.6.1 Geomechanical properties of intact rock

Based on more than 600 laboratory tests, the basic geomechanical properties of intact granite rock are summarized in Table 5.7.

Table 5.7 Summary of geomechanical properties of intact rock (Zhou, 2001)

Properties	Range	Average
Bulk density, g/cm ³	2.62-2.67	2.65
Uniaxial compressive strength (MPa)	108.1-224.9	163.7
Young's modulus (GPa)	49.3-111.3	65.6
Poisson's ratio	0.14-0.35	0.24
Internal friction angle (degree)	-	59.0
Point load index (MPa)	5.2-16.1	8.7
Brazilian Tensile Strength (MPa)	5.2-16.1	11.6

5.6.2 Rock mass classification

Based on results of the site investigation, the rock mass is good to very good category as per the Q classification system. During construction, results from tunnel mapping yielded slightly different results, with a higher concentration of the Q values in the fair to Good category. This could be partly due to the uniform damages on the tunnel surface that were created due to blasting.

5.7 Summary and discussion

Set of data pairs ($\sigma_{1 \text{ i-n}}$ & $\sigma_{3 \text{ i-n}}$) are selected following the suggested procedure by [Zhang et al. \(2018\)](#) from the triaxial testing of 84 rock core specimens to determine the intact rock properties such as cohesion (c , MPa), friction angle (ϕ degree), uniaxial compression strength (σ_{ci}) and Hoek-Brown parameter (m_i) using RocData program. The triaxial tests were conducted for mudstone and pyroclastic rocks under dry and wet condition to explore the effect of water content on strength properties.

From the present study, the triaxial compressive strength generally observed increase with increasing confining pressures. Meanwhile, the water content affected the triaxial compressive strength and deformability of rocks. Based on the Mohr-Coulomb failure criterion, it was found that the cohesion increased and friction angle decreased from dry to wet conditions for both tested rock types ([Li et al., 2012](#)). Based on the Hoek-Brown failure criterion, the H-B constant of m_i was found to decrease obviously from dry to wet conditions for both tested rock types.

The uniaxial compressive strength (σ_{ci}) was observed to decrease in mudstone from dry to wet conditions but increase in pyroclastic rocks. It is interpreted that the bonding strength between individual rock fragments in pyroclastic rocks becomes weaker when they are oven dried under 105° C.

Subsequently, the rock mass properties such as Hoek-Brown criterion m_b , s , a and deformation modulus (E_{rm}) were determined from intact properites with actual field input results such as GSI and D . The existing established equations are used in the determination. The rock mass deformation modulus (E_{rm}) decreased from dry to wet conditions for both rock types.

Among the three calculated results, E_{rm} by [Hoek and Diederichs \(2006\)](#) (Eq. 5.15) approach is observed reliable since the actual measured GSI, D and σ_{ci} are used as input parameters in it. Thus the results obtained by this approach are used in further numerical analyses in this study.

The present work will not only benefit the future underground space development in the Jurong Formation of Singapore, but also enrich the literature with regard to the mechanical properties of low porosity sedimentary rocks.

CHAPTER 6

The soft computing and regression approaches in geomechanical parameters prediction for rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

6.1 Prediction of unconfined compressive strength by applying the Mamdani fuzzy model in comparison with regression analysis

The determination of geomechanical properties of the rocks is the fundamental footstep to be carried out before the commencement of any civil or mining project. These properties can be measured either by laboratory tests or by applying analytical and numerical approaches. Sometimes unobtainability of standard samples necessitates researchers to ascertain the empirical correlation in critical and crucial properties with easily determinable properties. In the last a few decades, the usage of soft computing techniques such as fuzzy models, neural networks, etc. to resolve various problems related in rock mechanics and engineering geology has been increased (i.e., [Nguyen, 1985](#); [Den Hartog et al., 1996](#); [Alvarez Grima and Babuska, 1999](#); [Ouenes, 2000](#)), since the fuzzy models proposed by [Zadeh \(1965\)](#) is one of the approaches to handle with the difficulty of complicated and ill-defined systems in a flexible and reliable way ([Alvarez Grima, 2000](#)). In this study, the Mamdani fuzzy algorithm was chosen to predict unconfined compression strength and the Geological strength index (GSI) of rock masses in JRC Project since it is possibly the most fascinating fuzzy method to apply in engineering geological problems ([Alvarez Grima, 2000](#)). Those predicted results were evaluated with the results of statistical models such as coefficient of correlation (R^2) and root mean square error (RMSE).

The stability analysis of any excavated structure in rock mining project required many geomechanical properties of intact and rock mass such as unconfined compressive strength (UCS) and tensile strength. [Bieniawski \(1974\)](#) concluded that UCS is necessitated more often than any other rock properties since it is fundamental for the stability of underground excavations, tunnels, roadways, etc.

There are standard methods for determination of UCS of rock specimens ([ISRM, 2014](#) and [ASTM D7012, 2014](#)) such as the fundamental laboratory tests and correlated index tests. In the fundamental laboratory test, the deformation of a rock specimen under increment of unconfined compressive force is measured. The stress at which the specimen failure happens is taken as the peak strength. The second approach uses index tests to calculate the UCS in the place of direct measurement. The main benefits of the index tests are low costs and the measuring equipment is portable ([Meulenkamp and Alvarez, 1999](#)). The good quality specimens such as cylindrical core or block specimens are necessary to be prepared with high accuracy to carry out the standard tests. It is sometimes impossible to obtain such good quality specimens in the weak or highly fractured rock mass ([Albert and Kahraman, 2009](#)).

On top of the procedures as mentioned above, [Verwaal and Mulder \(1993\)](#) and [Hisashi and Yukinori \(2008\)](#) explored the option of UCS prediction from rock hardness determined with Equotip hardness tester. However, their research works provided statistical models (e.g., regression analysis) to predict mean values of UCS only.

To overcome the above difficulties in high-quality specimen preparation for standard laboratory tests and also the limitation in statistical models, some predictive models based on the artificial intelligence and soft computing systems would be alternative choices. Some procedures in artificial neural networks, fuzzy systems, genetic programming and evolutionary computational intelligence techniques have been positively conjoined ([Cevik et al., 2011](#); [Asadi et al., 2011](#)). These techniques become attractive in various fields since they can handle a different types of uncertainties. It is known that complex real-world problems may involve intelligent systems that possess human-like capability within a specific domain, familiarize themselves to exchanging environments and be able to clarify how they make conclusions or take actions ([Rezaei et al., 2014](#)). Among the above techniques, fuzzy modeling is one of the most proficient artificial intelligence subsystems which can handle with complex and indefinite systems in a flexible and coherent way. This method facilitates the significant capability of the human mind to aim and learn in an environment of uncertainty and imprecision. In the last two decades, fuzzy model approaches have been widely used in the field of rock mechanics, geotechnics,

mining and engineering geology (Nguyen, 1985; Finol et al., 2001; Azimi et al., 2010; Jalalifar et al., 2011; Monjezi et al., 2009; Rezaei et al., 2014). The above-mentioned researchers demonstrated the ability of the fuzzy model in modeling of complicated problems in rock mechanics, geotechnics and geo-engineering fields.

An attracting and possibly the most appealing feature of fuzzy models compared with other soft computing methods such as neural networks and genetic algorithm is that they are capable to define complicated and nonlinear multivariable problems transparently. Moreover, fuzzy models can handle with non-probabilistic ambiguities which are usual in rock engineering. Additionally, fuzzy rules can be formulated on the basis of expert knowledge of the system. In this study, a new fuzzy model was developed to predict the UCS of rock masses of Ayer Chawan Facies in JRC Project.

6.2 Fuzzy systems

After the fuzzy set theory, which was established by Zadeh (1965), emerges as an essential instrument in several engineering modeling and takes over the conventional methods of designing and modeling of a system. The rule-based models which conjoin physical perception, expert knowledge and numeric data in a transparent way can be developed based on the fuzzy theory. The developed model closely resembles the actual world. Typically, the fuzzy procedure is related to the human decision-making process which collects the plenty of imprecise data, evaluate it and give the proper decision (Zadeh, 1965).

This suggestion indicated that the outcome of the geomechanical parameters can be taken into account as fuzzy rather than crisp. For these type systems, the fuzzy sets proposed by Zadeh (1965) may deliver an efficient resolution. As demonstrated in Fig. 6.1, dissimilar to crisp sets, a fuzzy set is comprised of same objects and their related degrees of membership in the set. Crisp sets of real objects are equal to, and isomorphically defined by, a unique membership function, such as X_A in Fig. 6.1a.

However there is no set-theoretic equal to real objects related to X_A . Fuzzy sets are always functions, which map a universe of objects, say X , onto the unit interval $[0, 1]$; that is, the fuzzy set H is the function μ_H that carries X into $[0, 1]$ (Ross, 1995) in Fig 6.1b. So, the transition between a full member of the set and a non-member of the set is gradual rather than abrupt. Therefore, the fuzzy sets can be taken into

account as a more successful instrument for the modelling of human reasoning to lessen ambiguities.

In a conventional set, an element may or may not belong to a set. The membership or non-membership of an element x in the crisp set A is signified by the function μ_A of A . This representative function would be stated in the form of Eq. (6.1). The fuzzy logic, a simplification form of classical set theory is suitable to treat the indefinite data after applying a proper membership function (Jang and Gulley, 1995). The main difference from a conventional set is that degree of membership for each element is given in the unit interval between 0 and 1 in a fuzzy set. Mathematically, the fuzzy set A would be stated in the form of Eq. (6.2).

Fig. 6.1 Schematic illustration of a crisp set (a) and fuzzy set (b) with their membership functions (μ : degree of membership) (Ross, 1995)

$$\mu_A(x) = \begin{cases} 1 & \text{if } x \in A \\ 0 & \text{if } x \notin A \end{cases} \quad \text{----- (6.1)}$$

$$A = \{x, \mu(x) | x \in U\} \quad \text{----- (6.2)}$$

where $\mu_A(x)$ is a membership function of a fuzzy set.

A fuzzy system is usually comprised of four basic components including fuzzification, fuzzy "if-then" rules, fuzzy inference system (FIS) and defuzzification. This specific type of fuzzy intelligence systems is most appropriate for solving indecisiveness and complex problems by employing the fuzzy algorithm (Monjezi et al., 2009).

6.2.1 Fuzzification

Fig. 6. 2 Process of a fuzzy system construction (Monjezi et al., 2009).

The first stage in the fuzzy modeling process is the fuzzification in which the numeric values are converted into the fuzzy values after applying appropriate membership functions (Fig. 6.2). Dependent on the type of the problem to be settled and the experience of the users, either linear membership function (trapezoidal or triangular) or nonlinear membership function profile can be adopted.

The most frequently applied is the linear type (i.e. trapezoidal or triangular) (Den Hartog et al., 1996; Alvarez Grima, 2000). The membership function of a triangular and trapezoidal fuzzy set is computed by Eq. (6.3) and (6.4) correspondingly.

$$\mu_A(x) = \max[\min[\frac{x-a}{b-a}, \frac{c-x}{c-b}], 0] \quad \text{-----} \quad (6.3)$$

$$\mu_A(x) = \max[\min[\frac{x-a}{b-a}, 1, \frac{d-x}{d-b}], 0] \quad \text{-----} \quad (6.4)$$

where $\mu_A(x)$ is a membership function of a fuzzy set; a, b, c and d are the constant.

6.2.2 Fuzzy “if-then” rules

In the second stage of the fuzzy modeling procedure, the fuzzy conditional rules are to be formulated. Those restrictive rules describe the input-output relationship.

A fuzzy conditional rule is composed of a statement and a consequent (IF statement, THEN consequent). For example, “if x is good then y is valuable” in which the terms good and valuable can be characterized by fuzzy sets or more precisely by membership functions. In a fuzzy system, each rule can be expressed by the following equation ([Den Hartog et al., 1996](#)):

$$\mu_{Ri}(x, y) = I(\mu_{Ai}(x), \mu_{Bi}(y)), i = 1, 2, \dots, n \quad \text{-----} \quad (6.5)$$

where $\mu_{Ri}(x, y)$ is the relation membership degree of rule “I” according to “x” and “y” inputs, $\mu_{Ai}(x)$ and $\mu_{Bi}(y)$ are the membership degrees of “x” and “y” inputs, respectively. “I” denotes the “and” or “or” operator and “n” is the number of rules.

6.2.3 Fuzzy inference system

The third step in the fuzzy modeling procedure is to adopt a representative fuzzy inference system (FIS) to aggregate "if-then" rules ([Zadeh 1972](#)). The FIS process is to extract the end results from the applied rules used with input parameters and their membership functions. There are numerous FIS in the literature but Mamdani and Takagi-Sugeno algorithm are commonly applied in numerous purposes. The main difference of FIS is that the results of their fuzzy rules are not similar, thus the aggregation procedures would also be different ([Alvarez and Verhoef 1999](#); [Kayabasi et al., 2003](#)).

In the complex engineering geological problems, the Mamdani algorithm is one of the most commonly used FIS as it can handle most geological processes. Hence, the Mamdani algorithm is chosen in this study.

Mamdani and Assilian (1975) demonstrated that the fuzzy sets could convert a completely unstructured set of exact empirical terms into a structured one.

This structured set can be described in the form of the below equation (Iphar and Goktan, 2006):

$$\text{If } X_I \text{ is } A_{il} \dots \text{ and } X_r \text{ is } A_{ir} \text{ then } Y \text{ is } B_i \text{ for } I = 1, 2, \dots, K \quad \text{-----} \quad (6.6)$$

Where X_I and X_r are input variables, A_{il} , A_{ir} and B_i are linguistic terms (fuzzy sets), Y is output variable and K is the number of rules.

There are many fuzzy composition methods in the Mamdani algorithm such as min-max, max-min, max-max, min-min and max-mean. In this study, the most regularly applied practice, max-min composition, was utilized (Ross, 1995). In the max-min composition, producing an output from a set of disjunctive rules is based on the following equation:

$$\mu_{Ck}(Z) = \max [\min[\mu_{Ak}(\text{input}(x)), \mu_{Bk}(\text{input}(y))]] \quad \text{-----} \quad (6.7)$$

$$k = 1, 2, \dots, r$$

where μ_{Ck} , μ_{Ak} and μ_{Bk} are the membership functions of output “z” for rule “k” input “x” and input “y” respectively.

The appropriate defuzzification process is required to be applied in the next step to obtain a numeric (crisp) value as output (Ross, 1995).

6.2.4 Defuzzification

In the final step, the required output of numerical (crisp) value is to be attained from a fuzzy set. The process is termed as "Defuzzification." Among several defuzzification methods, centroid of area (COA) is applied in this study as it is the most frequently used method (Alvarez Grima, 2000). In this method, the crisp value can be gained by the following equation

$$Z_{COA}^* = \frac{\int_Z \mu_A(z)zdz}{\int_Z \mu_A(z)dz} \quad \text{-----} \quad (6.8)$$

Where Z_{COA}^* is the crisp value for the “z” output and $\mu_A(z)$ is the aggregated output membership function (Iphar and Goktan, 2006).

6.3 Application of fuzzy analysis in UCS prediction for rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

In the present study, a new fuzzy model created on the Mamdani algorithm is established for UCS prediction of rock masses at Singapore underground excavation project. To predict UCS as output, bulk density (BD), porosity (Po) and point load index test results (Plt) are used as input parameters (Columns b, c & d of [Table 6.1](#)). [Fig. 6.3](#) demonstrated the structure of proposed fuzzy model with assigned inputs and consequent output in the MATLAB environment ([MathWorks Inc., USA, 2016](#)).

Table 6.1: Input parameters and output UCS results by laboratory testing and two different analyses

Serial numbers (a)	Input parameters			UCS (MPa)		
	Bulk density (b)	Porosity (c)	Point load index (d)	Laboratory test (e)	Fuzzy analysis (f)	Statistics analysis (g)
1	2.53	7.229	1.67	53	43	45
2	2.664	2.557	1.71	55	64	86
3	2.577	3.672	7.7	76	69	80
4	2.654	0.487	13.79	127	123	117
5	2.699	0.642	11.34	140	130	118
6	2.674	1.613	8.77	93	106	105
7	2.674	1.287	21	113	104	129
8	2.687	2.727	3.15	116	104	91
9	2.657	3.097	8.61	102	102	95
10	2.748	0.964	6.83	91	101	114
11	2.754	0.719	9.08	125	102	121
12	2.748	0.548	9.83	129	125	122
13	2.758	0.614	9.51	103	102	122
14	2.73	0.934	15.71	121	125	128
15	2.751	0.167	11.34	145	130	127
16	2.748	0.505	11.89	148	133	126
17	2.728	0.968	10.62	123	123	119
18	2.726	0.591	10.89	117	126	121
19	2.741	0.745	13.8	129	136	127
20	2.744	1.56	2.2	108	109	102
21	2.726	1.179	7.88	101	102	112
22	2.752	1.778	7.31	113	110	112
23	2.73	1.541	9.32	111	104	114
24	2.751	1.426	10.98	118	126	120

Usually, fuzzification of input / output variables is implemented by a membership function. As mentioned in [Sect. 6.2.1](#), triangular and trapezoidal types are the most

regularly applied membership functions. In this analysis, the fuzzy model with triangular membership functions was applied as it has the minimum error. The membership functions of input and output parameters are presented in Fig 6.4a, b, c and d. As shown in these figures, the fuzzy sets classified from very low, low, medium, high and very high respectively.

A total of 48 rules were applied and a decision was made out of the combined input and output membership functions on the basis of obtainable data sets. Fig. 6.5 shows a fuzzy “if-then” rule editor of the model in MATLAB environment.

In this study, fuzzy inference mechanism based on the Mamdani algorithm is employed for aggregating the if-then rules. Also, centroid of area (COA) defuzzification method, the most commonly adopted method (Alvarez Grima, 2000; Hellendoorn and Thomas, 1993) is applied for attaining the numerical output value. Fuzzy logic tool box program was used in the modeling and simulation to determine UCS.

Fig 6.3. Structure of fuzzy model for UCS prediction

Fig. 6.4a The membership functions of bulk density (input parameter)

Fig. 6.4b The membership functions of porosity (input parameter)

Fig. 6.4c The membership functions of point load index test results (input parameter)

Fig. 6.4d The membership functions of UCS (output parameter)

The above Mamdani fuzzy model can predict a precise UCS values of rock masses of Ayer Chawan Facies in JRC Project. As shown in Fig. 6.6, the UCS value can be predicted after applying three input values in MATLAB environment. In this sample, the predicted output of UCS would be 102 MPa (Serial No. 13 in Table 6.1) from three input parameters such as bulk density = 2.758 kN/m^3 , porosity = 0.614 % and point load strength index = 9.51 MPa. All the predicted UCS results were tabulated in column “f” of Table 6.1.

1. If (BD is L) and (Po is VH) and (PLT is L) then (UCS is L) (1)
2. If (BD is L) and (Po is VH) and (PLT is M) then (UCS is L) (1)
3. If (BD is L) and (Po is VH) and (PLT is H) then (UCS is L) (1)
4. If (BD is L) and (Po is H) and (PLT is L) then (UCS is L) (1)
5. If (BD is L) and (Po is H) and (PLT is M) then (UCS is L) (1)
6. If (BD is L) and (Po is H) and (PLT is H) then (UCS is L) (1)
7. If (BD is L) and (Po is M) and (PLT is L) then (UCS is L) (1)
8. If (BD is L) and (Po is M) and (PLT is M) then (UCS is L) (1)
9. If (BD is L) and (Po is M) and (PLT is H) then (UCS is M) (1)
10. If (BD is L) and (Po is L) and (PLT is L) then (UCS is M) (1)
11. If (BD is L) and (Po is L) and (PLT is M) then (UCS is M) (1)
12. If (BD is L) and (Po is L) and (PLT is H) then (UCS is M) (1)
13. If (BD is VL) and (Po is VH) and (PLT is L) then (UCS is VL) (1)
14. If (BD is VL) and (Po is VH) and (PLT is M) then (UCS is VL) (1)
15. If (BD is VL) and (Po is VH) and (PLT is H) then (UCS is L) (1)
16. If (BD is VL) and (Po is H) and (PLT is L) then (UCS is VL) (1)
17. If (BD is VL) and (Po is H) and (PLT is M) then (UCS is L) (1)

Fig 6.5. The fuzzy “if-then” rule editor including samples of 17 rules in model

Fig 6.6. An example of model application in MATLAB environment to predict UCS from three input parameters

6.4 Statistical model for UCS determination

Multivariable regression, which is an extension of the regression analysis, involves extra independent variables in the predictive equation. After applying this approach, it is able to establish the correlation between the criterion (independent) variables and the predictive (dependent) variables (Jennrich, 1995). This procedure is widely employed in the fields of rock mechanics and mining engineering (Finol et al., 2001; Kumar et al., 2011).

To establish multivariate relation for similar database as applied in fuzzy modeling, the Microsoft Excel Data Analysis program was used to obtain the following correlation between input and output parameters;

$$\text{UCS} = -248.5 + 129.3 (\text{BD}) - 5.1 (\text{Po}) + 1.8 (\text{Plt}) \quad \text{-----} \quad (6.9)$$

The calculated results using Eq. (6.9) were shown in Table 6.1, column (g).

6.4.1 Performance evaluation between fuzzy and statistical models

To evaluate the proficiency of the two models, two key performance indices (KPIs) such as coefficient of correlation (R^2) and root mean square error (RMSE) were determined. The coefficient of correlation (R^2) provides the correlation between the datasets whereas RMSE index demonstrates the error between the results predicted from the models and actual measured values. Generally higher R^2 proved the relevance of a model and the lower the RMSE is the better the model functioning.

Table 6.2 Performance indices of the models

Index	Mamdani fuzzy model	Statistical model
R^2	0.8529	0.6334
RSME	14.1	14.4

The calculated performance indices of two models were summarized in Table 6.2. Also relationship between predicted and measured UCS for fuzzy and statistical models are graphically exhibited in Figs. 6.7 and 6.8 respectively. After taken into account of R^2 and RMSE, performance of fuzzy models works better than statistical model and the results are in close range with tested values.

Fig 6.7 Comparison of UCS predicted by fuzzy model (column ‘f’) versus UCS by laboratory test (column ‘e’ in [Table 6.1](#))

Fig. 6.8. Comparison of UCS predicted by statistical model (column ‘g’) versus UCS by laboratory test (column ‘e’ in [Table 6.1](#))

6.5 Summary and discussion in prediction of UCS

In this study, a new intelligent methodology applying the Mamdani inference mechanism was established to predict UCS of rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project. To prediction the output results of UCS, bulk density, porosity and point load index test results were taken into account as input parameters. To validate the predicted results from Mamdani fuzzy model, the comparison was made with results of statistical model applying coefficient of correlation (R^2) and root mean square error (RMSE). For the fuzzy model, R^2 and RMSE were calculated 0.8529 and 14.1 whereas for the statistical model, these indices were determined as 0.6334 and 14.4 respectively. These results proved that the performance of fuzzy model is significantly better than statistical model. Also fuzzy model results provided in very close range with the laboratory measured UCS.

Taking into consideration of the above, the intelligent system based on the Mamdani fuzzy model proved to have a suitable ability in predicting UCS and more economical than other pre-existing methods. Therefore, this method can be effectively applied in preliminary UCS determination of rock mass for stability analysis purposes in the fields of mining engineering and engineering geology. As it is the first attempt with limited test results, it is advisable to update the current model when additional UCS results are available in local civil and mining engineering projects.

6.6 Application of fuzzy model in quantitative Geological Strength Index (GSI) determination for rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

In this section, the fuzzy set theory is employed in GSI determination based on the Mamdani fuzzy algorithm. The fundamental parameters (block volume and joint condition factor) which were described in Cai's quantitative GSI chart ([Cai. et.al, 2004](#)) are used as two input membership functions in prediction of GSI.

6.7 Application of fuzzy sets in GSI determination

In this study, the application of fuzzy set theory to predict GSI was conducted to avoid the typical uncertainties due to either the complex characteristic features of the

rock masses or to the imperfect experience in understanding these. Two fuzzy inference algorithms were applied for GSI determination.

- First fuzzy inference algorithm - joint condition factor (J_c) was determined using total 210 “if-then” rules from three input parameters namely roughness, weathering and infilling condition
- Second fuzzy inference algorithm - GSI was determined by applying 22 “if-then” rules from two input parameters namely block volume (V_b) and joint condition factor (J_c determined from the first algorithm)

6.7.1 First fuzzy inference algorithm to predict joint condition factor (J_c)

In the GSI determination, the joint condition factor is outlined by the roughness, weathering and infilling condition. The sum of these factors specifies the strength of a joint or block surface. In this study, a joint condition factor, J_c which is the same parameters utilized by [Palmstrom 1995](#) is employed to define it. This joint condition factor (J_c) is expressed as,

$$J_c = \frac{J_w \cdot J_s}{J_a} \text{-----} (6.10)$$

where J_w and J_s are the large-scale waviness (in meters from 1 to 10 m) and small-scale smooth (in centimeters from 1 to 20 cm) and J_a is the joint alteration factor. The ratings for J_w , J_s and J_a are followed shown in the [Tables A.4, A.5 and A.6](#) respectively. Fuzzy logic tool box program ([MathWorks Inc., USA, 2016](#)) was used in the modeling and simulation to determine GSI.

The fuzzy inference system to determine joint condition factor J_c which required as one of two input parameters for the GSI system was constructed beforehand. The system calculates joint condition factor (J_c) depending on joint waviness (J_w), small scale smoothness (J_s) and joint alteration factor (J_a) as shown in [Fig. 6.9](#). The centre of gravity method for the defuzzification was applied.

The membership functions of J_w , J_s , J_a and J_c are shown in [Fig. 6.10a, b, c and d](#) respectively. The input values of J_w , J_s and J_a are treated as normalized value which is calculated by dividing the individual value by the highest one in each parameter but output value J_c is used as the same value (from 0.1 to 12) as

proposed in Cai's graph (Cai et al., 2004). As shown in Fig. 6.10, the triangular membership function was applied since each set in the input has only three crisp values (e.g. $\mu=0$ and 1). To predict J_c , a total of 210 "if-then" rules were obtained (Fig. 6.11) from the rating system proposed by Palmstrom, 1995 Tables A.4, A.5 and A.6. An example of solution rules to determine J_c was presented in Fig. 6.12.

Fig. 6.9 Main component of Joint condition factor (J_c) fuzzy inference system

Fig. 6.10a Joint waviness (J_w) input of joint condition factor (J_c) fuzzy inference system

Fig. 6.10b Small scale smoothness (J_s) input of joint condition factor (J_c) fuzzy inference system

Fig. 6.10c Joint alteration (J_a) of joint condition factor (J_c) fuzzy inference system

Fig. 6.10d Output of joint condition factor (J_c) fuzzy inference system

1. If (Joint_waviness_Jw is IL) and (Smoothness_Js is VR) and (Joint_alteration_JA is UW) then (Joint_condition_factor_Jc is VG) (1)
2. If (Joint_waviness_Jw is IL) and (Smoothness_Js is VR) and (Joint_alteration_JA is MW) then (Joint_condition_factor_Jc is G) (1)
3. If (Joint_waviness_Jw is IL) and (Smoothness_Js is VR) and (Joint_alteration_JA is SC) then (Joint_condition_factor_Jc is G) (1)
4. If (Joint_waviness_Jw is IL) and (Smoothness_Js is VR) and (Joint_alteration_JA is HW) then (Joint_condition_factor_Jc is G) (1)
5. If (Joint_waviness_Jw is IL) and (Smoothness_Js is VR) and (Joint_alteration_JA is CmCI) then (Joint_condition_factor_Jc is F) (1)
6. If (Joint_waviness_Jw is IL) and (Smoothness_Js is VR) and (Joint_alteration_JA is SfCI) then (Joint_condition_factor_Jc is F) (1)
7. If (Joint_waviness_Jw is IL) and (Smoothness_Js is VR) and (Joint_alteration_JA is SwCI) then (Joint_condition_factor_Jc is F) (1)
8. If (Joint_waviness_Jw is IL) and (Smoothness_Js is R) and (Joint_alteration_JA is UW) then (Joint_condition_factor_Jc is VG) (1)
9. If (Joint_waviness_Jw is IL) and (Smoothness_Js is R) and (Joint_alteration_JA is MW) then (Joint_condition_factor_Jc is G) (1)
10. If (Joint_waviness_Jw is IL) and (Smoothness_Js is R) and (Joint_alteration_JA is SC) then (Joint_condition_factor_Jc is G) (1)
11. If (Joint_waviness_Jw is IL) and (Smoothness_Js is R) and (Joint_alteration_JA is HW) then (Joint_condition_factor_Jc is F) (1)
12. If (Joint_waviness_Jw is IL) and (Smoothness_Js is R) and (Joint_alteration_JA is CmCI) then (Joint_condition_factor_Jc is F) (1)
13. If (Joint_waviness_Jw is IL) and (Smoothness_Js is R) and (Joint_alteration_JA is SfCI) then (Joint_condition_factor_Jc is F) (1)
14. If (Joint_waviness_Jw is IL) and (Smoothness_Js is R) and (Joint_alteration_JA is SwCI) then (Joint_condition_factor_Jc is P) (1)
15. If (Joint_waviness_Jw is IL) and (Smoothness_Js is SR) and (Joint_alteration_JA is UW) then (Joint_condition_factor_Jc is G) (1)
16. If (Joint_waviness_Jw is IL) and (Smoothness_Js is SR) and (Joint_alteration_JA is MW) then (Joint_condition_factor_Jc is G) (1)
17. If (Joint_waviness_Jw is IL) and (Smoothness_Js is SR) and (Joint_alteration_JA is SC) then (Joint_condition_factor_Jc is F) (1)
18. If (Joint_waviness_Jw is IL) and (Smoothness_Js is SR) and (Joint_alteration_JA is HW) then (Joint_condition_factor_Jc is F) (1)
19. If (Joint_waviness_Jw is IL) and (Smoothness_Js is SR) and (Joint_alteration_JA is CmCI) then (Joint_condition_factor_Jc is F) (1)
20. If (Joint_waviness_Jw is IL) and (Smoothness_Js is SR) and (Joint_alteration_JA is SfCI) then (Joint_condition_factor_Jc is P) (1)
21. If (Joint_waviness_Jw is IL) and (Smoothness_Js is SR) and (Joint_alteration_JA is SwCI) then (Joint_condition_factor_Jc is P) (1)
22. If (Joint_waviness_Jw is IL) and (Smoothness_Js is S) and (Joint_alteration_JA is UW) then (Joint_condition_factor_Jc is G) (1)
23. If (Joint_waviness_Jw is IL) and (Smoothness_Js is S) and (Joint_alteration_JA is MW) then (Joint_condition_factor_Jc is F) (1)
24. If (Joint_waviness_Jw is IL) and (Smoothness_Js is S) and (Joint_alteration_JA is SC) then (Joint_condition_factor_Jc is F) (1)

Fig. 6.11 “If-then” rules for the J_c fuzzy inference system

Fig. 6.12 Solution of the J_c fuzzy inference system

6.7.2 Second fuzzy inference algorithm for GSI

The second fuzzy inference algorithm calculates GSI by applying the input parameters of block volume V_b and joint condition factor J_c as shown in Fig 6.13. GSI determination is based on the quantitative GSI chart established by Cai et al., 2004 as shown in Fig. A.11. In this study, block volume V_b input was applied with the spacing (cm) between adjacent discontinuities (bedding and joints) of the rock mass as shown on left hand side vertical scaled column of chart (Cai et al., 2004). Joint condition factor J_c is on horizontal scale of the chart. The membership functions of block volume V_b , joint condition factor J_c and GSI are given in Fig. 6.14 a, b and c respectively. The scaled values of block volume (1 cm to 150 cm) and J_c (0.1 to 12) are treated as original value but GSI is used as normalised value which is calculated by dividing the individual value by highest one (= GSI 100) as proposed in Cai's graph. As shown in Fig 6.14, the triangular membership function, which was used as each set in the inputs, has only three crisp values (e.g. $\mu = 0$ and 1) whereas the membership functions of the block volume V_b are taken as trapezoid since all rock masses display their own structures in a range. The initial GSI chart (Hoek et al, 1992) has 25 subclasses after foliated/laminated rock mass structure category is not considered due to the anisotropic and heterogeneous nature of this rock mass category. Three of the 25 sub-classes namely in intact or massive structure are not relevant because of the unexpected combinations between the surface conditions of the discontinuities and the rock mass structure. Hence, the fuzzy inference system to predict the GSI has a total of 22 if-then rules as given in Fig. 6.15. The control surface of the system is displayed in Fig. 6.16. A sample of rule to determine GSI was presented in Fig. 6.17.

Fig. 6.13 Main component of Geological Strength Index (GSI) fuzzy inference system

Fig. 6.14a Block volume V_b input of the GSI fuzzy inference system

Fig. 6.14b Joint condition factor J_c input of the GSI fuzzy inference system

Fig 6.14c Output of GSI in the GSI fuzzy inference system

6. If (Block_size_(cm) is BD) and (Joint_condition_factor is VP) then (GSI is BD/VP) (1)
7. If (Block_size_(cm) is BD) and (Joint_condition_factor is P) then (GSI is BD/P) (1)
8. If (Block_size_(cm) is BD) and (Joint_condition_factor is F) then (GSI is BD/F) (1)
9. If (Block_size_(cm) is BD) and (Joint_condition_factor is G) then (GSI is BD/G) (1)
10. If (Block_size_(cm) is BD) and (Joint_condition_factor is VG) then (GSI is BD/VG) (1)
11. If (Block_size_(cm) is VB) and (Joint_condition_factor is VP) then (GSI is VB/VP) (1)
12. If (Block_size_(cm) is VB) and (Joint_condition_factor is P) then (GSI is VB/P) (1)
13. If (Block_size_(cm) is VB) and (Joint_condition_factor is F) then (GSI is VB/F) (1)
14. If (Block_size_(cm) is VB) and (Joint_condition_factor is G) then (GSI is VB/G) (1)
15. If (Block_size_(cm) is VB) and (Joint_condition_factor is VG) then (GSI is VB/VG) (1)
16. If (Block_size_(cm) is B) and (Joint_condition_factor is VP) then (GSI is B/VP) (1)
17. If (Block_size_(cm) is B) and (Joint_condition_factor is P) then (GSI is B/P) (1)
18. If (Block_size_(cm) is B) and (Joint_condition_factor is F) then (GSI is B/F) (1)
19. If (Block_size_(cm) is B) and (Joint_condition_factor is G) then (GSI is B/G) (1)
20. If (Block_size_(cm) is B) and (Joint_condition_factor is VG) then (GSI is B/VG) (1)
21. If (Block_size_(cm) is M) and (Joint_condition_factor is G) then (GSI is M/G) (1)
22. If (Block_size_(cm) is M) and (Joint_condition_factor is VG) then (GSI is M/VG) (1)

Fig. 6.15 “If-then” rules for the GSI fuzzy inference system

Fig. 6.16 Control surface of the GSI fuzzy inference system

Fig. 6.17 Solution of the GSI fuzzy inference system

6.8 Application of fuzzy analysis in GSI determination for rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

Four hundred and twenty excavated tunnel faces at various locations in the project were analyzed to calculate GSI. The required spacings of individual blocks were taken from the respective geological mapping records (Fig. 4.10) which were presented in columns (b, c and d) in Table 6.3 where only 14 selective faces were tabulated. The discontinuities' properties such as J_w , J_s and J_a , taken from the same face mapping records, were shown in column (e, f and g) with calculated J_c in column (h) in same table. The calculated normalised results, i.e divided by highest of individual rate, of each properties were shown in coulmmns (i, j and k) of same table. By following the solution rules shown in Fig 6.12, joint condition factor (J_c) was determined and tabulated in column (l).

Table 6.3: GSI determined by fuzzy inference system analysis; selective examples from 420 excavated rock faces

Sr. No.	Determination of Block volume			Determination of joint condition factor (by calculation)				Normalized joint condition factor (by fuzzy analysis)				GSI (Fuzzy analysis)
<i>a</i>	<i>b</i>	<i>c</i>	<i>d</i>	<i>e</i>	<i>f</i>	<i>g</i>	<i>h</i>	<i>i</i>	<i>j</i>	<i>k</i>	<i>l</i>	<i>m</i>
	S_1	S_2	S_3	J_w	J_s	J_a	J_c	J_w	J_s	J_a	J_c	
1	40	40	100	1.5	2	1	3	0.5	0.67	0.08	3.27	64
2	40	20	40	1.5	2	1	3	0.5	0.67	0.08	3.27	57
3	20	20	20	1.5	2	1	3	0.5	0.67	0.08	3.27	56
4	20	6	20	1.5	2	1	3	0.5	0.67	0.08	3.27	56
5	20	20	40	1	2	1	2	0.33	0.67	0.08	2.92	54
6	20	20	20	1	2	1	2	0.33	0.67	0.08	2.92	54
7	20	40	100	1.5	2	2	1.5	0.5	0.67	0.17	1.35	57
8	100	20	20	1.5	2	2	1.5	0.5	0.67	0.17	1.35	57
9	20	20	20	1.5	2	2	1.5	0.5	0.67	0.17	1.35	47
10	20	20	40	1.5	2	2	1.5	0.5	0.67	0.17	1.35	47
11	20	20	6	1.5	2	2	1.5	0.5	0.67	0.17	1.35	47
12	20	20	6	1	2	2	1	0.33	0.67	0.17	1.2	47
13	20	20	20	1	2	2	1	0.33	0.67	0.17	1.2	47
14	20	20	40	1	2	2	1	0.33	0.67	0.17	1.2	47

Next step is to determine the GSI of representative faces by following second fuzzy inference system. The average spacings (cm) to determine block volume (columns b, c and d in [Table 6.3](#)) and earlier calculated J_c values (column l) were applied as two input parameters ([Fig 6.13](#)). The fuzzy rules shown in [Fig. 6.17](#) was applied to determine required GSI of representative faces and results are described in column (m) of [Table 6.3](#) and also in column (e) of [Table 6.4](#).

Table 6.4: Comparison among quantitative GSI ([Cai et al., 2004 approach](#)), GSI (fuzzy analysis) and qualitative GSI (field assessment using [Hoek et al, 1995](#) chart); selective examples from 420 excavated rock faces

Sr. No. (a)	Block size (cm ³) V _b (b)	Joint condition factor J _c (c)	Quantitative GSI (Kar Winn and Wong, 2017a) (d)	GSI (Fuzzy analysis) (e)	Qualitative GSI (Hoek et al (1995) Chart) (f)
1	160,000	3	65	64	65
2	32000	3	60	57	65
3	8000	3	55	56	60
4	2400	3	52	56	55
5	16000	2	53	54	55
6	8000	2	50	54	55
7	80000	1.5	55	57	55
8	40000	1.5	52	57	55
9	8000	1.5	47	47	50
10	16000	1.5	49	47	50
11	2400	1.5	44	47	50
12	2400	1	41	47	45
13	8000	1	43	47	45
14	16000	1	45	47	45

6.9 Comparison of GSI (fuzzy analysis) with quantitative GSI (Cai et al., 2004 approach) and qualitative GSI (field assessment using chart by Hoek et al., 1995)

Quantitative GSI of same 14 representative faces in above were determined by Cai et al., 2004 approach (Kar Winn and Wong, 2017a), shown in column (d) of Table 6.4. Qualitative GSI values which were determined on rock faces using chart by Hoek et al., 1995 during construction work were presented in column (f) of same table.

The comparison graph between quantitative GSI and those predicted by fuzzy analysis was plotted as shown in Fig 6.18 with coefficient of correlation, R^2 value of 0.8477. Similarly, the comparison graph between qualitative GSI with those predicted by fuzzy analysis was drawn and shown in Fig 6.19 with coefficient of correlation, R^2 value of 0.7945.

Both sets of comparison graphs demonstrated that GSI values determined by three different methods are in the well comparable ranges respectively.

6.10 Summary and discussion in prediction of GSI

In this study, the fuzzy set theory is employed in quantitative GSI determination following the Mamdani fuzzy algorithm. At first, joint condition factor (J_c) was determined by first fuzzy algorithm using a total of 210 “if-then” rules from three input functions namely joint waviness, small-scale smoothness and joint alteration factors as proposed by Palmstrom, 1995.

In second fuzzy algorithm, the fundamental parameters (block volume and joint condition factor) which were used in Cai’s quantitative GSI chart (2004) are used as two input membership functions in the determination of GSI after setting 22 “if-then” rules.

It was observed that GSI determined by fuzzy analysis for Singapore’s sedimentary rock mass in JRC project was observed in the comparable range with quantitative GSI (Cai et al., 2004 approach) and on-site qualitatively assessed results (field assessment using Hoek et al, 1995).

The coefficient of correlation, R^2 between quantitative GSI and fuzzy-based predicted GSI is calculated as 0.8477 whereas qualitative GSI and fuzzy-based predicted GSI is calculated as 0.7945. It demonstrated that GSI values determined by three different methods are in the well comparable ranges respectively.

Fig 6.18 Comparison of predicted GSI (fuzzy analysis) with quantitative GSI (Cai et al., 2004 approach); selective examples from 420 excavated rock faces

Fig 6.19 Comparison of predicted GSI (fuzzy analysis) with qualitative GSI (field assessment using [Hoek et al, 1995](#)); selective examples from 420 excavated rock faces

CHAPTER 7

Engineering geological assessment of one storage cavern

7.1 Engineering geological assessment of storage cavern

Engineering geological assessment was carried out for storage cavern CS 1/1. After thorough study of face geological map and compilation map, 13 geological homogeneous zones were classified in storage cavern CS 1/1. Each zone is again analyzed to evaluate the relative potential in terms of wedge formation by combination of geological information, such as joint roughness, orientation, persistence, aperture and number of joint set. From the engineering geological assessment, 4 zones were classed based on the potential to form wedges.

Joint analysis was carried out with 350 numbers of representing joints, bedding and fault information, which have been recorded from the geological mapping. Joint sets were analyzed by Dips program ([Rocscience Inc., 2013a](#)) and 1 bedding set and 4 joint sets were identified ([Table 4.1](#)). In addition, random joints were also observed locally.

The joint sets results were applied to the wedge analyses for full cavern to evaluate the possibility of formation of wedges due to bench excavation. The results show that no additional wedges can be formed with investigated joint sets due to steep dip angle of joints and faults.

Fold is major and global geological structure in CS 1/1 and other areas of tunnels and galleries. It is observed that dip angle of bedding is homogeneous except in the end part of CS 1/1 TH where steeper angled bedding is found.

[Table 7.1](#) shows comprehensive engineering geological assessment with respect to the potential wedge formation after assessing the joint sets and their condition in each zone. The class column shows the engineering geological classes in relation to potential wedge such as N (none), L (low), M (moderate) and H (high) respectively.

Table 7.1 Engineering geological assessment for CS 1/1 TH

Class	From (m)	To (m)	Engineering geology assessment
N	1	8	Rock mass of this zone is intrusive igneous rock. Thus bedding is not present. Two joint sets observed in this zone are unlikely to form wedge. Hence this zone is classed as N (none = not possible to form wedge) (class = N)
M	8	31	Joint surface is rough ($J_r = 1.5$) and RQD is lower than other zone ($RQD = 85$). The sedimentary bedding plane is not significant in this zone. The spacing of joint is wide. Fault and joint planes are filled with quartz. The possibility of wedge formation is moderate and thus classed as M. (class = M)
L	31	66.5	There are some persistent faults in this zone but they are filled with calcite and tight. Joint surface is rough ($J_r = 3$) and tight. RQD is from 90 to 95. The possibility of wedge formation is lower and thus classed as L (class = L).
M	66.5	92	The bedding can be considered as the random joint set and RQD becomes lower than previous zone. Two joint sets are well developed. The possibility of wedge formation is medium (class = M).
L	92	112.5	The condition of bedding is same as in previous zone. The spacing of joints is wider than previous and RQD is 90. Joint surface condition, J_r , is 3. The aperture of joint plane is close to very close and filled with quartz. The wedge formation possibility is low (class = L).
M	112.5	133.5	Two joint sets and bedding are well developed. Minor joint set and fault are also observed. Thus wedge formation possibility in this zone is moderate (class = M).
L	133.5	159.5	Two joint sets are developed with minor random joint set. Joint surface condition, J_r , is rough ($J_r=3$) and filled with quartz. Joint surface aperture is close and tight. Thus wedge

formation chance is low (class = L).

H	159.5	183.5	Two joint sets with bedding plane are well developed. Bedding plane surface is soft and planar type. Joint surface is irregular and planar ($J_r = 1.5$). The water bearing layer is observed in this zone ($J_w = 0.66$). Thus chance of failure along water bearing beds is high and classed as H for this zone (class = H).
M	183.5	193	Two joint sets with bedding plane are well developed but spacing of planes is wider than previous zone. Joint surface is rough ($J_r = 3$). This zone is classified as in moderate wedge formation possibility (class = M).
H	193	227	Two joint sets with bedding plane are well developed. Surface of bedding and joints are soft and planar ($J_r = 1.5$). RQD is lower than other zone and spacing of bedding is closer than others. Thus it is classed as high wedge formation possibility (class = H).
M	227	274	The joint surface condition, J_r , is from 1.5 to 3. Two joint sets are well developed but bedding plane is randomly observed. Hence it is classed as medium (class = M).
H	274	290	Two joint sets and bedding are well developed with surface condition is soft and planar ($J_r = 1.5$). Thus the wedge formation possibility is classed as high (H) (class = H).
L	290	340	The bedding is randomly observed and tightly closed. The surface condition of joints is rough ($J_r = 3$) and RQD is high. Thus the wedge formation possibility in this zone is low (L) (class = L).

The above 13 classified zones are plotted in [Fig. 7.1](#).

The objective of geological mapping is to observe the geological structure and to determine rock mass quality through rock mass classification and support system that is able to ensure the stability of the excavation works. Basically, geological map will be

prepared on the rock type, rock structure, discontinuities and groundwater condition. On the other hand, rock quality map and support map will be produced by Q values and geological map.

Geological mapping was carried out after blasting of each advance to assess the engineering geology condition to assign for tunnel support application. The tunnel quality index “Q” value was determined after required parameters such as RQD, J_n , J_r , J_a , SRF and J_w are measured. The determination of Q value is presented in [section 4.8](#) with Eq. 4.8. The sketch of the excavated face with photograph is to be placed on geological map. On the sketch, measured information on bedding, joints and water condition are described.

Once Q value is rated from geological mapping works, the rock class will be determined as per below table;

Table 7.2 Classification of Rock Grade

Q value	$Q < 1$	$1 \leq Q < 4$	$4 \leq Q < 10$	$10 \leq Q < 40$	$Q \geq 40$
Rock Grade	Very poor	Poor	Fair	Good	Very Good
Rock Class	5	4	3	2	1

Other required information about types, roughness, filling, aperture, spacing, water condition, weathering and persistence of discontinuities are measured at site after following the guide lines on the geological map.

The assessment classes already described in [Table 7.1](#) were differentiated based on the discontinuity properties from the geological mapping works. The following geological maps ([Fig 7.2a](#) to [7.2d](#)) present how N (none), L (low), M (moderate) and H (high) classes are classified based on the information of face geology mapping, tunnel quality index “Q” value and other discontinuity parameters.

Geological Assessment of CS 1/1

Fig. 7.1 Engineering geology assessment classes of CS1/1

7.2 Wedge analysis

The joint sets results ([Table 4.1](#)) were applied to the wedge analyses for full cavern to evaluate the possibility of formation of wedges due to bench excavation. The results shows that no additional wedges can be formed with investigated joint sets due to steep dip angle of joints and faults.

Two wedge analyses have been carried out for CS 1/1 cavern using UnWedge program ([Rocscience Inc., 2013b](#)). UnWedge is a 3D stability analysis and visualization program for underground excavations in rock containing intersecting structural discontinuities. Safety factors are calculated for potentially unstable wedges and support requirements can be modeled using various types of pattern and spot bolting and shotcrete. UnWedge is used to quickly create a model, perform a safety factor analysis, place reinforcement and interpret the results.

First case of wedge analysis is done with bedding joint (B1) combined with two prominent joint sets, J1 and J2. This case represented for moderate wedge formation possibility (class = M). The next case is analyzed with bedding joint (B1) combined with all four joint sets (J1 to J4). It represented for high possibility of wedge formation (class = H).

The input parameters for properties of bedding and joints, tunnel support rock bolt and shotcrete and in situ stress parameters were used those from the previous chapters for this analysis.

7.2.1 Analysis of wedges formed by B1 with J1 & J2

The Unwedge analysis for wedges formed by intersecting of B1 with J1 and J2 are presented in [Fig. 7.3](#) with the results of safety factors for each wedge is shown in [Table 7.3](#).

Total six wedges are formed on the roof, both side walls and floor of the cavern. The Unwedge analysis showed that the wedges formed by intersecting B1 with J1 and J2 are very stable after tunnel support applied. The weight of individual formed wedge and safety factor of them are presented in [Table 7.3](#).

Fig. 7.3 Analysis of wedges formed by B1 intersection with J1 and J2.

Table 7.3 Safety factor and weight of wedges (B1 with J1 & J2)

Wedge Number	Safety Factor	Weight of wedge formed (MN)
1 (floor wedge)	Stable	1.647
2 (lower right wedge)	66.24	1.436
3 (lower left wedge)	129.448	0.105
6 (upper right wedge)	61.126	0.026
7 (upper left wedge)	16.418	0.446
8 (roof wedge)	9.566	0.551

7.2.2 Analysis of wedges formed by B1 with all four joint sets (J1, J2, J3 & J4)

Fig. 7.4 Analysis of Wedges formed by B1 intersection with all joint sets

The Unwedge analysis for wedges formed by intersecting of B1 with all joint sets are presented in Fig. 7.4 with the results of safety factors for each wedge are shown in Table 7.4.

Total six wedges are formed on the roof, both side walls and floor of the cavern. The Unwedge analysis showed that the wedges formed by intersecting B1 with four joint sets are very stable after tunnel support applied. The weight of individual formed wedge

and safety factor of them are presented in [Table 7.4](#). The safety factor of individual wedge is lower than the first wedge type.

Table 7.4 Safety factor and weight of wedges (B1 with all joint sets)

Wedge Number	Safety Factor	Weight of wedge formed (MN)
1 (floor wedge)	Stable	1.647
2 (lower right wedge)	43.83	1.436
3 (lower left wedge)	58.539	0.105
6 (upper right wedge)	10.342	0.026
7 (upper left wedge)	8.427	0.446
8 (roof wedge)	2.603	0.551

7.3 Summary and discussion

In this chapter, 13 geological homogeneous zones were classified in storage cavern CS 1/1, as a sample, after assessments on recorded field geological data are carried out. Each zone is again analyzed to evaluate the relative potential wedge formation by combining geological information, such as joint roughness, orientation, persistence, aperture and number of joint set. From the engineering geological assessment, 4 zones (none, low, medium and high potential) were classed based on the possibility to form wedges. Based on the engineering geological zones, the tunnel systems in the tunnel can be evaluated whether it is adequately applied.

After 350 numbers of representing joints, bedding and fault information are analyzed by Dips program ([Rocscience Inc., 2013a](#)), 1 bedding set and 4 joint sets were identified ([Table 4.1](#)). In addition, random joints were also observed locally.

The joint sets results are again applied to the wedge analyses using UnWedge program for full cavern to evaluate the possibility of formation of wedges due to bench excavation. UnWedge is applicable to quickly create a model, perform a safety factor analysis, apply reinforcement and interpret the output results.

Two different cases were studied; one case using bedding and two major joint sets, the other using bedding with all 4 joints. Both of the results shows that no additional wedges can be formed beyond using those discontinuities encountered in the cavern.

It is advisable to perform engineering zone classification in the tunnel to further evaluate the geological and hydrogeological condition. Based on the results, the quality control checking for applied tunnel support system in the tunnel should be analysed. Unwedge analysis is also to be carried out for wedge stability using assessed discontinuities sets.

CHAPTER 8

Feasibility and stability analysis of caverns excavation in JRC

Project by 2D & 3D finite element programs

8.1 2D numerical analysis using RS² (Rocscience Inc.) in JRC Project

In this study, both intact rock properties and equivalent rock mass properties determined in [Chapters 4 and 5](#) are used as input parameters in the analyses. The reason for choosing RS² and RS³ (Rocscience Inc.) is

- To compare the results by 2 D and 3D analyses
- The working file size in 3DEC (Itasca Inc.) is beyond control and difficult to model the 340 m long cavern.
- Both discontinuum and continuum analyses can be performed successfully with RS² code.
- The limitation is that only continuum analysis can be performed with RS³.

RS² (formerly known as Phase², [Rocscience Inc., 2012](#)) is a powerful 2D finite element program for soil and rock application. Complex, multi-stage models can be easily created and quickly analysed such as tunnels in weak or jointed rock.

The overall bird's eye view of Jurong Rock Cavern (JRC) is presented in [Fig. 8.1a](#). JRC project comprised of two levels of excavation, namely level one (upper level) and level zero (lower level). The dimension of each storage cavern is 340 m in length, 20 m in width and 27 m in height as presented in [Fig. 8.1b](#). Excavation is carried out into 3 stages namely top heading (TH), bench 1 (B1) and bench 2 (B2) with 9 m height in each stage as presented in [Fig 8.1c](#).

Excavation and tunnel support application works of one twin storage cavern was modelled for study. The size of the domain of analysis is rectangular shape with 400 x 300 meter in dimension. The floor of both caverns is set at zero elevation level with about 150 m each of rock mass above and below. The two dimensional (2D) finite element geotechnical software RS² ([RS² V 8.0, Rocscience Inc., 2012](#)) was used in the study as it is a user friendly powerful program to design and analyse of underground excavation in rock and soil.

The following different approaches are applied for analysis

1. Discontinuum analysis
2. Continuum analysis ($GSI = 40$)
3. Continuum analysis ($GSI = 55$)
4. Continuum analysis (layered rock mass)

Fig 8.1a Birds eye view of Jurong Rock Cavern Project

Fig. 8.1b Schematic diagram of twin storage cavern (plan view)

Fig. 8.1c Sectional view of one cavern showing 3 stages of excavation

8.2 Discontinuum finite element analysis (intact rock with bedding & joint properties)

Among four rock mass properties presented in [Chapter 5](#), only wet mudstone properties shown in [Table 5.5](#) “Summary of cohesion and friction angles of different rock types under dry and wet condition with other intact rock properties” and representative bedding and joint cohesion and friction angle from [Table 4.6](#) were used in this current analysis as mudstone is the common rock mass in the project and testing under wet condition reflects the actual project site situation since the jointed rock mass is saturated with sea water. The applied intact rock and discontinuities properties are presented again in [Table 8.1](#).

When a bolt crosses a joint, force will develop in the bolt to resist joint movement. The section of the bolt that crosses the joint also resists joint movement through a dowel force. In RS², the formulation of [Dight \(1982\)](#) is used to compute this force. Thus Bolt-joint interaction in RS² program is applied for the bonded bolt type those used in JRC project

Table 8.1 – Properties of wet mudstone intact rock and discontinuities

	Cohesion (c, MPa)	Friction angle (ϕ , deg.)
Wet mudstone Peak strength	36	37
Bedding plane	0.3	30
Joint planes	0.2	30
Young Modulus (MPa)	20,000	
Poisson's ratio	0.3	
Normal stiffness of bedding & joint	100,000 (MPa/m)	
Shear stiffness of bedding & joint	10,000 (MPa/m)	

The properties of tunnel support materials are shown in [Table 8.2](#) and [8.3](#) and will also be applied for other analyses in [Sections 8.3, 8.4, 8.5](#) and [8.6](#).

Table 8.2 - Shotcrete Properties

Liner Type	Formulation	Thickness	Young's Modulus	Poisson's ratio
Standard Beam	Timoshenko	0.08m	20000 MPa	0.15

Table 8.3 - GFRP Rock Bolt Properties (Glass Fibre Reinforced Polymer Bolts)

Bolt Type		Swelllex / Split Sets	
Tensile Capacity	0.35 MN	In plane spacing	2.2 m
Tributary area	380 mm ²	Out of plane spacing	2.2 m
Bolt modulus	50000 MPa	Bond shear stiffness	100 (MN/m/m)
Bolt length	5 m	Residual tensile capacity	0
Bond strength	0.188 (MN/m)	Allow joints to shear Bolt	Yes

8.2.1 Excavation and construction sequences

The complete model in the study with dimensions is presented in Fig. 8.2. Three sets of discontinuities (one sub-horizontal bed with two sets of sub-vertical joint in Table 4.6) were modelled. Six stages of excavation with tunnel support systems were modelled as 13 steps as presented in the Fig. 8.3.

Fig. 8.2 RS² model of twin caverns with 3 sets of discontinuities (domain size 400 m × 300 m)

a. Left cavern TH excavation & support

b. Right cavern TH excavation & support

c. Left cavern B1 excavation & support

d. Right cavern B1 excavation & support

e. Left cavern B2 excavation & support

f. Right cavern B2 excavation & support

Fig. 8.3 Six stages of excavation with tunnel support system

8.2.2 Tunnel support system

Fig. 8.4a Liner (shotcrete) support system (Table 8.2) applied in RS²

Fig. 8.4b Rock bolt support system (Table 8.3) applied in RS²

80 mm thick shotcrete (Table 8.2) was sprayed on the excavated roof (TH only) and side walls (TH, B1 & B2) after mucking out of rock debris completed. GFRP (Glass Fibre Reinforced Polymer) bolts were installed after shotcrete spray. They were fully grouted bolts with 5 m embedded length (Table 8.3) inside rock mass. In this analysis, reinforced concrete type for shotcrete (Fig. 8.4a) and swellex-split type for rock bolts (Fig. 8.4b) are chosen with spacing of 2.2 m.

8.2.3 Field stress system

Gravity loading field stress type with actual ground surface, which is 145 m above the base level of twin caverns, was applied. Horizontal to vertical stress ratio of maximum (σ_H / σ_v) 2.2 and minimum (σ_h / σ_v) 1.8 (section 4.3.2) were put in the analysis as in Fig. 8.5.

Fig. 8.5 Field stress properties applied in RS²

8.2.4 Mesh setup

6 noded triangles element with graded mesh type was applied in the analysis as presented in Fig. 8.6. The mesh density was again increased as a window surrounding the both cavern. There are 70169 elements and 145805 nodes involved in the original mesh. The running time is about 17 minutes to be completed till 13 steps.

Fig 8.6 Mesh setup applied in RS²

8.2.5 Results of displacement and strength factor calculated by RS²

Maximum vertical displacement of about 8.8 mm (downward settlement, in negative values) was observed at roof centres at both left and right caverns. It is noted that about 90 % of the vertical displacement occurred at the roof during top heading excavation. The vertical displacement of cavern crown is stable with very minimal changes during the remaining 2 benches excavation. The similar vertical displacement is observed in the study by [Mandel et al. \(2013\)](#). Almost no upward vertical movement (heave, in positive values) was observed at the final floor of cavern as shown in [Fig. 8.7a](#).

In term of horizontal displacement, outer side walls of both caverns, i.e. left wall in left cavern and right wall in right cavern, showed around 18 mm (maximum) inward movement. However both inner walls of caverns, i.e. right wall of left cavern and left wall of right cavern, showed lesser horizontal movement about 8 mm ([Fig 8.7b](#)). The reason of higher horizontal displacement on outer walls is due to the higher stress movement of rock mass than those on inner walls.

The strength factors (the ratio of available rock mass strength to induced stress at given points) are presented in [Fig 8.7c](#) with all results are in the range higher than 3. The progressive changes of vertical settlements after each stage of excavation are shown in [Table 8.4](#).

Fig 8.7 a. Results of vertical displacement (m) throughout sequence of excavation and support application (Discontinuum approach)

Fig. 8.7 b Results of horizontal displacement (m) (Discontinuum approach)

Fig. 8.7 c Results of strength factor (Discontinuum approach)

Table 8.4 Progressive changes of vertical settlement of cavern roofs (Discontinuum finite element analysis) (Discontinuum approach)

	L heading		R heading		L bench1		R bench 1		L bench2		R bench2	
Sequence of excavation & support	excavation	support	excavation	support	excavation	support	excavation	support	excavation	support	excavation	support
Left cavern roof settlement (mm)	7.7	7.7	8.0	8.0	8.1	8.1	8.3	8.3	8.3	8.3	8.7	8.7
Right cavern roof settlement (mm)	(All in mm)		7.8	7.9	8.3	8.3	8.4	8.4	8.9	8.9	8.7	8.8

8.2.6 Calculated axial force of rock bolts, bending moment and shear force of shotcrete (liner)

The axial forces of all rock bolts installed were calculated in the range of 1.0 to 85 kN which is much lower than the maximum tensile capacity of rock bolts, 0.35 MN (350 kN specified by bolt manufacturer) as presented in [Table 8.5](#). The highest axial forces are expectedly observed at the tunnel roof where the rock bolts are installed in sub-vertical position. It is also at the tunnel crown where the highest vertical displacement with the lowest strength factors observed. The lowest values are observed at the final excavation stages where the bolts are installed in sub-horizontal position.

Table 8.5 Computed axial forces (kN) on rock bolts at different tunnel profile after final completion of caverns (Discontinuum approach)

	TH left	TH right	Left cavern		B2 left	B2 right
			B1 left	B1 right		
Max. bolt force (kN)	85.3	23.9	35.1	23.4	4.0	1.7
	TH left	TH right	Right cavern		B2 left	B2 right
			B1 left	B1 right		
Max. bolt force (kN)	23.7	19.0	28.2	45.8	0.4	0.4

Shotcrete bending moment was observed in the range between positive 0.015 kNm to negative 0.765 kNm whereas shotcrete shear force was observed in the range of negative 0.001 to negative 0.58 kN as presented in [Table 8.6](#). Positive bending moment causes top of shotcrete liner beam to undergo compression whereas negative bending moment cause to undergo tension. A shear force acting on a positive face is positive if it acts in the positive direction of an axis (upward direction of cavern) and negative if it acts in the negative direction of an axis (downward direction of cavern). Both of the predicted values are considerably low.

Similarly in rock bolt axial forces, the highest bending moment and shear forces are estimated on roof of the cavern. There was no yield capacity specification of bending moment and shear force for shotcrete adopted in the project. However on site observation there was no adverse effect on shotcrete surface such as cracking and

peeling off.

The low axial force of rock bolt and bending moment/shear force of shotcrete is due to the high horizontal stress ratio and good quality rock mass in the project.

Table 8.6 Computed axial bending moment (kNm) and shear force (kN) on shotcrete liner at different tunnel profile after final completion of caverns (Discontinuum approach)

	Left cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Bending moment (kNm)	-0.765	-0.408	-0.084	-0.010	-0.120	-0.034
Shear Force (kN)	-1.419	1.966	0.621	0.010	-0.055	0.016
	Right cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Bending moment (kNm)	-0.58	-0.084	-0.059	-0.018	0.015	0.021
Shear Force (kN)	0.121	-0.177	-0.044	-0.001	0.061	0.006

8.3 Continuum finite element analysis (Generalized Hoek-Brown rock mass properties, GSI = 55)

Among four equivalent rock mass properties of Jurong Rock Cavern Project presented in [Chapter 5](#), only wet mudstone properties shown in [Table 5.6 b](#) were used in this current analysis as it is the common rock mass in the project. The applied rock mass properties are presented again in [Table 8.7](#).

Table 8.7 - Generalised Hoek-Brown Rock Mass Properties of Wet Mudstone

Initial element loading		Field stress & Body Force	
Young's modulus (E_{rm})	11677 MPa	Poisson's ratio	0.3
Intact Compressive strength	143 MPa	Material unit weight	0.027 MN/m ³
m_i (intact rock)	7	GSI	55
Material Type	Plastic	Dilation parameter	0
m_b parameter	1.4	Residual m_b	1.4
s parameter	0.007	Residual s	0.007
a parameter	0.50	Residual a	0.50
Failure Criteria		Generalised Hoek-Brown	
Modulus Ratio (MR)		200	
Disturbance factor (D)		0	

In this analysis rock mass modulus (E_m) was calculated by [Hoek and Diederichs \(2006\)](#) proposed equation using GSI value of 55 and modulus ratio of 200. The input rock mass properties in RS² program is shown in [Table 8.7](#).

8.3.1 Excavation and construction sequences

The complete model in the study with dimensions is presented in [Fig. 8.8](#). Six stages of excavation with tunnel support systems were modelled as 13 steps which is similar to the discontinuum analysis.

8.3.2 Tunnel support system

The tunnel support system was applied similarly as in previous analysis.

8.3.3 Field stress system

The field stress system was applied similarly as in previous analysis.

Fig. 8.8 RS² model of twin caverns (domain size 400 m × 300 m)

8.3.4 Mesh setup

The mesh setup was applied similarly as in previous analysis

8.3.5 Results of displacement and strength factor calculated by RS²

Maximum vertical displacement of about 3.6 mm (downward settlement, in negative values) was observed at roof centre. Upward vertical movement (heave, in positive values) was observed in the range between 3 to 5 mm at the final floor of cavern as shown in [Fig. 8.9 a](#). There was no immediate support applied on the floor of cavern on site. There was also no negative impact on the stability of cavern floor before casting with 300 mm thick reinforced concrete in the later stage of construction work.

In term of horizontal displacement, outer side walls of both caverns, i.e. left wall in left cavern and right wall in right cavern, showed around 19 mm inward movement. However both inner walls of caverns, i.e. right wall of left cavern and left wall of right cavern, showed lesser horizontal movement about 10 mm ([Fig. 8.9 b](#)).

The strength factors (the ratio of available rock mass strength to induced stress at given points) are presented in [Fig 8.9c](#) with all results are in higher than 1. The strength factor is lower in this analysis as compared with the result of previous discontinuum analysis. The progressive changes of vertical settlements after each stage of excavation are shown in [Table 8.8](#).

Fig. 8.9 a Results of vertical displacement (m) in each sequential construction stages (Continuum approach, GSI=55)

Fig. 8.9 b Results of horizontal displacement (m) (Continuum approach, GSI=55)

Fig. 8.9 c Results of strength factor (Continuum approach, GSI=55)

Table 8.8 Progressive changes of vertical settlement of cavern roofs (Continuum approach, GSI=55)

Sequence of excavation & support	L heading		R heading		L bench1		R bench 1		L bench2		R bench2	
	excavation	support	excavation	support	excavation	support	excavation	support	excavation	support	excavation	support
Left cavern roof settlement	3.3	3.3	3.3	3.3	2.6	2.6	2.9	2.9	2.3	2.3	2.8	2.8
Right cavern roof settlement	(All in mm)		3.3	3.3	3.7	3.7	3.1	3.1	3.6	3.6	2.9	2.9

8.3.6 Calculated axial force of rock bolts, bending moment and shear force of shotcrete (liner)

The axial forces of all rock bolts installed were calculated in the range of 0.1 to 17.1 kN which is much lower than the maximum tensile capacity of rock bolts, 0.35 MN (350 kN specified by bolt manufacturer) as presented in [Table 8.9](#). The slightly higher axial forces are observed at the tunnel wall of bench 1 which is different from the previous analysis. The lowest values are observed at the final excavation stages where the bolts are installed in sub-horizontal position.

Table 8.9 Computed axial forces (kN) on rock bolts at different tunnel profile after final completion of caverns (Continuum approach, GSI=55)

	Left cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	15.3	14.7	11.1	16.1	1.0	1.1
	Right cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	13.9	11.3	17.1	14.3	0.1	0.1

Shotcrete bending moment was observed in the range between negative 0.002 kNm to negative 0.045 kNm whereas shotcrete shear force was observed in the range of negative 0.001 to negative 0.02 kN as presented in [Table 8.10](#). Both of the predicted values are considerably low.

Table 8.10 Computed axial bending moment (kNm) and shear force (kN) on shotcrete liner at different tunnel profile after final completion of caverns (Continuum approach, GSI=55)

	Left cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Bending moment (kNm)	-0.045	-0.034	-0.017	-0.029	-0.002	-0.006
Shear Force (kN)	-0.020	0.003	-0.006	0.008	-0.001	0.001
	Right cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Bending moment (kNm)	-0.045	-0.042	-0.025	-0.035	0.000	0.000
Shear Force (kN)	-0.013	0.010	-0.008	0.012	0.000	0.000

The highest bending moment and shear forces are expectedly estimated on roof of the cavern crown where the highest vertical displacement with the lowest strength factors observed. There was no yield capacity specification of bending moment and shear force for shotcrete adopted in the project. However on site observation there was no adverse effect on shotcrete surface such as cracking and peeling off.

8.4 Continuum finite element analysis (Generalized Hoek-Brown rock mass properties, GSI = 40)

Among four equivalent rock mass properties presented in [Chapter 5](#), only wet mudstone properties shown in [Table 5.6b](#) were used in this current analysis as it is the common rock mass in the project. The applied rock mass properties are presented again in [Table 8.11](#).

Table 8.11 - Generalised Hoek-Brown Rock Mass Properties of Wet Mudstone, GSI = 40

Initial element loading		Field stress & Body Force	
Young's modulus (E_{rm})	4566 MPa	Poisson's ratio	0.3
Intact Compressive strength	143 MPa	Material unit weight	0.027 MN/m ³
m_i (intact rock)	7.0	GSI	40
Material Type	Plastic	Dilation parameter	0
m_b parameter	0.82	Residual m_b	0.82
s parameter	0.0012	Residual s	0.0012
a parameter	0.51	Residual a	0.51
Failure Criteria		Generalised Hoek-Brown	
Modulus Ratio (MR)		200	
Disturbance factor (D)		0	

In this analysis rock mass modulus (E_{rm}) was calculated by [Hoek and Diederichs \(2006\)](#) proposed equation using GSI value of 40 and modulus ratio of 200. The input rock mass properties in RS² program is shown in [Table 8.11](#).

8.4.1 Excavation and construction sequences

The size of the model and excavation sequences are similar to the previous analysis using GSI = 55.

8.4.2 Tunnel support system

The tunnel support system was applied similarly as in previous analysis.

8.4.3 Field stress system

The field stress system was applied similarly as in previous analysis.

8.4.4 Mesh setup

The mesh set up was applied similarly as in previous analysis.

8.4.5 Results of displacement and strength factor calculated by RS²

Maximum vertical displacement of about 16 to 18 mm (downward settlement, in negative values) was observed at roof centre (Kar Winn, et al., 2017b). Upward vertical movement (heave, in positive values) was observed in the range between 17 to 31 mm at the final floor of cavern as shown in Fig. 8.10 a. There was no immediate support applied on the floor of cavern on site. There was also no negative impact on the stability of cavern floor before casting with 300 mm thick reinforced concrete in the later stage of construction work.

In term of horizontal displacement, outer side walls of both caverns, i.e. left wall in left cavern and right wall in right cavern, showed around 50 mm inward movement. However both inner walls of caverns, i.e. right wall of left cavern and left wall of right cavern, showed lesser horizontal movement about 27 mm (Fig. 8.10 b). The strength factors (the ratio of available rock mass strength to induced stress at given points) are presented in Fig 8.10c with all results are in higher than 1.

Fig. 8.10 a Results of vertical displacement (m) (Continuum approach, GSI=40)

Fig. 8.10 b Results of horizontal displacement (m) (Continuum approach, GSI=40)

Fig. 8.10 c Results of strength factor (Continuum approach, GSI=40)

8.4.6 Calculated axial force of rock bolts, bending moment and shear force of shotcrete (liner)

The axial forces of all rock bolts installed were calculated in the range of 3.0 to 46.2 kN which is much lower than the maximum tensile capacity of rock bolts, 0.35 MN (350 kN specified by bolt manufacturer) as presented in [Table 8.12](#). The highest axial forces are expectedly observed at the tunnel roof where the rock bolts are installed in sub-vertical position. The lowest values are observed at the final excavation stages where the bolts are installed in sub-horizontal position.

Table 8.12 Computed axial forces (kN) on rock bolts at different tunnel profile after final completion of caverns (Continuum approach, GSI=40)

	Left cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	46.2	44.2	13.8	21.1	3.0	25.6
	Right cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	36.9	43.7	25.9	22.1	4.4	3.5

Shotcrete bending moment was observed in the range between negative 0.005 kNm to negative 0.116 kNm whereas shotcrete shear force was observed in the range of negative 0.002 to negative 0.050 kN as presented in [Table 8.13](#). Both of the predicted values are considerably low.

Table 8.13 Computed axial bending moment (kNm) and shear force (kN) on shotcrete liner at different tunnel profile after final completion of caverns (Continuum approach, GSI=40)

	Left cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Bending moment (kNm)	-0.116	-0.068	-0.040	-0.058	-0.010	-0.017
Shear Force (kN)	-0.050	-0.046	-0.013	0.014	-0.002	0.009
	Right cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Bending moment (kNm)	-0.095	-0.074	-0.049	-0.061	-0.006	-0.005
Shear Force (kN)	0.029	-0.018	-0.013	0.014	-0.002	0.002

Similarly in rock bolt axial forces, the highest bending moment and shear forces are estimated on roof of the cavern where the highest vertical displacement and the lowest strength factors observed. There was no yield capacity specification of bending moment and shear force for shotcrete adopted in the project. However on site observation there was no adverse effect on shotcrete surface such as cracking and peeling off.

8.4.7 Parametric study on rock bolt spacing changes

The parametric study with wider rock bolt spacing of 3 m with other parameters remained unchanged was carried out to study the effect of spacing change. The calculated results of vertical & horizontal displacement, shear strength factor and rock bolt axial forces are presented in the following figures.

Fig 8.11a Results of vertical displacement (m) (Bolt spacing=3 m)

Fig. 8.11 b Results of horizontal displacement (m) (Bolt spacing=3 m)

Fig. 8.11 c Results of strength factor (Bolt spacing=3 m)

When rock bolts are installed in slightly wider spacing, the vertical displacements, horizontal displacement and strength factor do not change much. However, the axial forces of all rock bolts installed were calculated in the range of 1.1 to 77.2 kN (Table 8.14) and observed that maximum axial force is much higher when the rock bolts are installed in wider spacing.

Table 8.14 Computed axial forces (kN) on rock bolts at different tunnel profile after final completion of caverns (Bolt spacing=3 m)

	TH left	TH right	Left cavern			
			B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	18.3	77.2	13.4	22.2	1.9	1.1
			Right cavern			
			B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	30.8	47.4	20.8	23.6	2.0	2.1

8.4.8 Parametric study on spacing changes between caverns (pillar width 25 m)

The parametric study with 25 m pillar width (the width of rock mass between two caverns) with other parameters remained unchanged was carried out to study the effect of spacing change. All the cases under this study, the pillar width is designed as 40 m (two times of cavern width, 20 m). The calculated results of vertical & horizontal displacement, shear strength factor and rock bolt axial forces are

presented in the following figures.

Fig. 8.12 a Results of vertical displacement (m) (pillar width=25 m)

Fig. 8.12 b Results of horizontal displacement (m) (pillar width=25 m)

Fig. 8.12 c Results of strength factor (pillar width=25 m)

There is not much change in vertical displacements when pillar width reduced to 25 m. However the horizontal displacements on inner walls of two caverns become lower after lesser induced forces from the rock mass of narrow pillar. It is also observed that distributed major principal stress values slightly higher than those in larger pillar width. That increment of major principal stress may cause wall collapse in weaker rock mass.

The axial forces of all rock bolts installed were calculated in the range of 1.4 to 26.1 kN (Table 8.15) and it is observed that no significant difference as compared to other cases.

Table 8.15 Computed axial forces (kN) on rock bolts at different tunnel profile after final completion of caverns (Pillar width=25 m)

	Left cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	22.2	24.2	9.9	2.7	3.5	5.6
	Right cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	26.1	21.9	17.3	32.3	1.4	0.94

8.5 Continuum finite element analysis (layered rock mass properties with GSI=55)

Table 8.16 - Generalised Hoek-Brown Rock Mass Properties of Wet pyroclastic rock, GSI=55

Initial element loading		Field stress & Body Force	
Young's modulus	32580 MPa	Poisson's ratio	0.3
Intact Compressive strength	266 MPa	Material unit weight	0.027 MN/m ³
m _i (intact rock)	13	GSI	55
Material Type	Plastic	Dilation parameter	0
m _b parameter	2.6	Residual m _b	2.6
s parameter	0.007	Residual s	0.007
a parameter	0.50	Residual a	0.50
Failure Criteria	Generalised Hoek-Brown		
Modulus Ratio (MR)	300		

In this analysis, cavern excavation in interbedded mudstone and pyroclastic rock

mass was modelled. The wet mudstone properties shown in Table 5.6 b and wet pyroclastic properties described in Table 5.6 d were used in this current analysis. The applied rock mass properties are presented again in Table 8.16.

In this analysis rock mass modulus was calculated by Hoek and Diederichs (2006) proposed equation using GSI value of 55 and modulus ratio of 200 for mudstone and 300 for pyroclastic rock.

8.5.1 Excavation and construction sequences

The complete model in the study with dimensions is presented in Fig. 8.13. Six stages of excavation with tunnel support systems were modelled as 13 steps as in previous sections.

8.5.2 Tunnel support system

The tunnel support system was applied similarly as in previous analysis.

8.5.3 Field stress system

The field stress system was applied similarly as in previous analysis.

8.5.4 Mesh setup

The mesh setup was applied similarly as in previous analysis.

8.5.5 Results of displacement and strength factor calculated by RS²

Fig. 8.14 a Results of vertical displacement (m) (Layered rock mass)

Fig. 8.14 b Results of horizontal displacement (m) (Layered rock mass)

Maximum vertical displacement of about 3.0 mm (downward settlement, in negative values) was observed at roof centre. Upward vertical movement (heave, in positive values) was observed in the range between 4 to 6 mm at the final floor of cavern as shown in Fig. 8.14 a. There was no immediate support applied on the floor of cavern on site. There was also no negative impact on the stability of cavern floor before

casting with 300 mm thick reinforced concrete in the later stage of construction work.

In term of horizontal displacement, outer side walls of both caverns, i.e. left wall in left cavern and right wall in right cavern, showed around 15 mm inward movement. However both inner walls of caverns, i.e. right wall of left cavern and left wall of right cavern, showed lesser horizontal movement about 9 mm (Fig. 8.14 b).

The strength factors (the ratio of available rock mass strength to induced stress at given points) are presented in Fig 8.14 c with all results are in the range higher than one (1).

Fig. 8.14 c Results of strength factor (Layered rock mass)

8.5.6 Calculated axial force of rock bolts, bending moment and shear force of shotcrete (liner)

Table 8.17 Computed axial forces (kN) on rock bolts at different tunnel profile after final completion of caverns (Layered rock mass)

	Left cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	26.7	4.7	9.4	14.9	1.0	1.6
	Right cavern					
	TH left	TH right	B1 left	B1 right	B2 left	B2 right
Max. bolt force (kN)	5.0	7.1	14.9	14.0	0.1	0.2

The axial forces of all rock bolts installed were calculated in the range of 0.1 to 26.7 kN which is much lower than the maximum tensile capacity of rock bolts, 0.35 MN (350 kN specified by bolt manufacturer) as presented in [Table 8.17](#). The highest axial forces are expectedly observed at the tunnel roof where the rock bolts are installed in sub-vertical position. The lowest values are observed at the final excavation stages where the bolts are installed in sub-horizontal position.

Table 8.18 Computed axial bending moment (kNm) and shear force (kN) on shotcrete liner at different tunnel profile after final completion of caverns (Layered rock mass)

	TH left	TH right	Left cavern			
			B1 left	B1 right	B2 left	B2 right
Bending moment (kNm)	-0.041	-0.054	-0.014	-0.016	0.001	-0.001
Shear Force (kN)	-0.021	0.037	0.005	0.005	0.001	-0.001
			Right cavern			
Bending moment (kNm)	-0.084	-0.172	-0.028	-0.026	0.001	0.000
Shear Force (kN)	-0.154	1.236	-0.011	-0.116	0.000	0.000

Shotcrete bending moment was observed in the range between positive 0.001 kNm to negative 0.172 kNm whereas shotcrete shear force was observed in the range of positive 0.001 to positive 1.26 kN as presented in [Table 8.18](#). Both of the predicted values are considerably low.

Similarly in rock bolt axial forces, the highest bending moment and shear forces are estimated on roof of the cavern where the highest vertical displacement and the lowest strength factors observed. There was no yield capacity specification of bending moment and shear force for shotcrete adopted in the project. However on site observation there was no adverse effect on shotcrete surface such as cracking and peeling off.

8.6 3D numerical analysis using RS³ program (Rocscience Inc.)

In this study RS³ program ([Rocscience Inc., 2013c](#)) is applied to analyze the deformational and stress changes behavior along the tunnel route of sedimentary rock masses at JRC project.

RS³ is a program for 3D analysis of geotechnical structures for civil and mining applications for both rock and soil. It is a general purpose finite element analysis program for underground excavations, tunnel and support design, surface excavation, foundation design, embankments, consolidation, groundwater seepage and more.

3-dimensional model geometry is built up by creating a series of extruded 2-dimensional slices. The program offers complete flexibility for staging of excavations, support installation, loading and all other modeling aspects. Models with up to several hundred different stages can be analyzed with RS³. RS³ has material models for both soil and rock, including Mohr-Coulomb, Generalized Hoek-Brown, Cam Clay and Drucker-Prager strength models. Elastic models include isotropic, transversely isotropic, orthotropic, and Duncan Chang Hyperbolic. Any number of different material types can be defined and easily assigned to different regions.

In the design of geotechnical support systems, RS³ offers a wide variety of support systems such as bolts, liners, beams, and piles. These can be used in any combination for the support of excavations, slopes, foundations, retaining walls, and embankments. Bolt types consist of End Anchored, Tieback, Fully Bonded, Plain Strand Cable, Swellex, and Split Set. Liners can be represented as shotcrete or concrete support. Beams can be represented as steel sets or trusses. Composite liners with multiple layers and sliding interfaces can be defined.

Different types of loading can be modeled including in-situ field stress (constant or gravitational), distributed loads or surcharges, concentrated loads (line or point), seismic load (pseudo-static) and springs.

8.6.1 3-dimensional (3D) model of twin cavern excavation with rock support installation

Excavation was carried out in horizontal mode, in which Z-axis is perpendicular to gravity direction, Y-axis as presented in Fig. 8.15a. X-axis is in horizontal direction that perpendicular to Y-axis.

The dimension of model is 380 m by 300 m in X-Y plane. The model is 500 m long in Z direction in which excavation length up to 340 m.

a. 2D and 3D view of models used in the analysis

b. Bird eye view of twin caverns

c. Front view of twin caverns

d. Plan view of twin cavern with ramp in left cavern and connection tunnels

e. Total 11 segments used in the model (last one for unexcavated zone)

Fig. 8.15 3-dimesional model

One typical storage twin cavern was modelled for 3-dimensional analysis. There are three connection tunnels between left and right caverns, as shown in Fig. 8.15 b, c& d, which were used for construction access.

In left cavern there was traffic access ramp with sloping gradient of 13% which was left permanently. Total 11 slices of different length were used to model ramp and three connection tunnels as shown in Fig. 8.15 d & e.

8.6.2 Equivalent rock mass properties (Generalized Hoek-Brown parameters)

Only wet mudstone properties shown in Table 5.6 b were used in this current analysis as it is the common rock mass in the project. Those are described in Table 8.11 in section 8.4 in 2D analysis. In this analysis rock mass modulus was

calculated by [Hoek and Diederichs \(2006\)](#) proposed equation using GSI value of 40 and modulus ratio of 200.

8.6.3 Excavation and construction sequences

a. Top heading (TH) excavation in left & right caverns till 340m length

b. Bench 1 (B1) level excavation followed after TH completion till end

c. Bench 2 (B2) level excavation followed after B1 completion till end

d. Final completion of both caverns

Fig 8.16 Excavation and tunnel support sequences

Top heading (TH) of 9 m height excavation was started in left cavern with 5 m advance in each step. Shotcrete was sprayed on roof and two side walls as tunnel support after rock debris were mucked out and area was cleared. Installation of rock bolts were followed after one or two round, i.e. 5 to 10 m, of excavation and shotcrete spray. This similar excavation sequence was followed until 340 m length completion. TH of right cavern was excavated via connection tunnel from left cavern as shown in [Fig. 8.16a](#).

Next level Bench 1 (B1) was commenced in left cavern after full length of TH completion. The traffic accessible ramp, 5.5 m width with slope gradient 13%, was excavated downward from TH to B1 level inside left cavern. B1 of right cavern was excavated via connection tunnel as described in [Fig. 8.16 b](#).

Subsequent Bench 2 (B2) was started in left cavern after full length of B1 completion. The similar traffic access ramp was continued from B1 to B2 level in left cavern as presented in [Fig. 8.16c](#). B2 of right cavern was excavated via connection tunnel as in [Fig. 8.16 d](#).

8.6.4 Tunnel support system

Tunnel support systems were same as in 2-dimensional analysis that was presented in previous section.

8.6.5 Field stress system

Gravity loading system with horizontal to vertical stress ratios of 2.2 (maximum) and 1.8 (minimum) were used in the analysis as presented in [section 8.2.3](#) for 2D analysis.

8.6.6 Mesh setup

Fig. 8.17 Mesh setup

Four noded tetrahedron element types with graded mesh setup were applied as presented in [Fig. 8.17](#). The numbers of element is 165526 and numbers of node is 28342 in the model. The total computation time is about 4 hours.

8.6.7 Longitudinal displacement profile (LDP) for JRC

LDP for a tunnel is important to determine the appropriate timing for the installation of support or to optimize the installation of support with specific displacement capacity ([Song et al., 2016](#)).

Along the course of tunnel excavation, approximately 35-45% of the final tunnel wall deformation can be occurred before the support installation. This deformation can be estimated by using either (a) observed field values or (b) numerically from 3D finite element models or axisymmetric finite element models or (c) by using empirical relationships such as those proposed by [Panet and Guenot \(1982\)](#) or [Vlachopoulos and Diederichs \(2009\)](#).

a. Vertical displacement after TH excavation

b. Displacement ratio (displacement at point / maximum stable displacement)

Figure 8.18 Longitudinal displacement profile (LDP) of JRC cavern after 340 m length top heading excavation, displacement ratio 0.6

The vertical displacement at cavern roof centre, without tunnel support, along the 340 m length top heading excavation was plotted in [Fig. 8.18 a](#). The displacement ratio was calculated at 5 m behind the final excavated face, i.e. 335 m, since it is the location where full tunnel support can be installed practically. The computed displacement is about 0.6 (displacement @ 335 m / stable maximum displacement far behind the face).

8.6.8 Results of both caverns' tunnel roof vertical displacement at individual excavation stage

In Fig. 8.19 and 8.20, the vertical displacement before and after support application at different excavation stages in left and right caverns were plotted. It was noted that the change was very small amount between 0.1 to 0.2 mm.

a. After TH excavation

b. After B1 excavation

c. After B2 excavation

Fig 8.19 Comparison of vertical displacement of left cavern before and after tunnel support at each excavation sequence

a. After TH excavation

b. After B1 excavation

c. After B2 excavation

Fig 8.20 Comparison of vertical displacement of right cavern before and after tunnel support at each excavation sequence

8.6.9 Strength results of rock bolts calculated by RS³

The maximum computed axial force is about 0.02 MN (20 kN) which is lower than that calculated in 2D analysis. The reason is that the value is the residual force acted on rock bolts behind the progressively excavated face.

Fig. 8.21 demonstrates the axial force changes along the length of rock bolt at subsequent excavation stages. Fig 8.21a shows the rock bolt installed at roof centre of left cavern whereas Fig 8.21b shows the roof centre bolt at right cavern. The rock bolt at left cavern carries higher tensile load than the rock bolt at right cavern. The reason is that the left cavern is excavated before the right cavern. There is minimal increase of force after each stage of excavation.

8.7 Vertical displacement monitoring

Vertical displacement was monitored by installing glass prisms on the cavern crown and side walls at different locations along the alignment shown in Fig. 8.22 a. The measured results at cavern crown (points 2, 3 & 4) in a particular section over time are shown in Fig. 8.22 b. The maximum vertical displacement was measured around 5 mm, negative values in Fig. 8.22 b, which was observed to remain stable throughout the excavation period. These monitored settlements are lower than those obtained by RS² and RS³ analysis calculated in previous sections.

The monitored displacement results should be taken with attentions, as the accuracy of measurement depends on several factors such as the time of installation of the instruments after blasting, geological condition and damage caused by blasting. In addition, the calculated results are based on a simplified analysis using a simple

plasticity-based model. However, despite these restraints, comparisons between predictions and measurements can provide useful information.

Fig. 8.22 (a) Location of monitoring points for displacement measurements on cavern roof and walls in storage cavern, **(b)** Vertical displacement monitoring results at points 2, 3 and 4 (Kar Winn, et al., 2016)

The main factor that makes much difference between predicted and measured values is the time and place of monitoring device installed after face excavation. In JRC project, the instruments were installed a few tens of meters behind the excavated faces due to safety reasons. As discussed in section 8.6.7 and Fig. 8.18 (LDP), the calculated displacement ratio is about 0.6 (60% of total displacement) at 5 m behind the final excavated face. Hence the measured vertical displacement 5 mm (Fig. 8.22) would be equivalent to the remaining 40% of total displacement.

8.8 Summary and discussion

In this study, discontinuum analysis (intact rock and joint parameters applying Mohr-Coulomb failure criterion approach) in 2D and continuum analysis (equivalent rock mass properties applying Generalised Hoek-Brown failure criterion approach) in 2D and 3D are carried out to document the rock mass response after excavation work. The purpose of choosing FEM programs (RS² & RS³, Rocscience Inc.) is to prove the ability of codes in handling of analyses for mega-sized project and also to compare the output results by different approaches. Moreover the longitudinal displacement profile (LDP) of the tunnel which is suitably carried out with 3D program is presented to compare with the results of previous researchers.

Table 8.19 Calculated maximum vertical displacement by various analyses

Type of Analysis		Calculated maximum vertical displacement (mm)	
		Left cavern	Right cavern
		roof	roof
2D analysis	Discontinuum analysis (mudstone)	8.7	8.8
	Continuum analysis (GSI = 55) (mudstone)	3.6	3.6
	Continuum analysis (GSI = 40) (mudstone)	18.4	16.3
	Continuum analysis (GSI = 40) (mudstone)	15.9	15.4
	Rock bolt 3 m spacing		
	Continuum analysis (interbedded rock mass)		
	(mudstone and pyroclastic rocks interbedded; GSI = 55)	2.7	2.6
3D analysis	3 D continuum analysis (GSI = 40) (mudstone)	10.2	10.5

As summarized in [Table 8.19](#), the calculated results by different approaches gave a wide range of results. In 2D continuum analysis, the lowest displacement is obtained by applying rock mass parameters based on GSI of 55 (better rock mass property) whereas the highest values are from GSI 40 (weaker rock mass) based parameters

analysis.

Applying discontinuum analysis is to model bedded and jointed rock mass as in JRC project but the time for processing is longer than continuum analysis. On the other hand, the processing time in continuum approach is faster but the equivalent rock mass property is sometimes difficult to estimate due to the complex nature of rock mass.

As presented in [section 8.7](#), if the measured vertical displacement (about 5 mm) is taken as the remaining 40 %, the total displacement would be around 10 mm. That value is in the agreement range with the results by 2D discontinuum and 3D continuum analyses. GSI of 40 is the extreme lowest value and 55 is the highest recorded in the area. The measured vertical displacement might be the results of the instruments installed at the rock face having GSI between 40 and 55.

Applying mudstone and pyroclastic rock interbedded model in 2D analysis gave the least vertical settlements among all analyses. It is due to the higher strength of the rock mass medium which is provided by the pyroclastic rocks.

When conducting the study with 3 m spaced rock bolt installation and 25 m pillar width, the results of displacement, strength factor, axial forces of rock bolts and shotcrete do not change considerably and it is in safe working condition in each analysis. The 2.2 m spaced rock bolt installation design is based on the criterion accepted by local and international authorities and agencies. It is hardly to be approved by the authority if the alternative design is deviated from the accepted one. Based on the candidate's working experience in other cavern project, it is recommended that no tunnel support be required on the walls, from the invert level up to about 1.5 m high (around the shoulder height of people), of final Bench 2 zone. The impact on the safety is minimal even if the rock falls occur from those walls.

The high horizontal displacements of about 50 mm inward movements were recorded in the analyses but there was no serious effect during excavation works. It was also noted that continuum analysis using rock mass parameters showed higher horizontal displacements than discontinuum analysis. The possible reason might be that the discontinuum analysis represents the real rock mass condition after shear

strength parameters (c and ϕ) of bedding and joint planes are applied in it.

After considering the wide-ranged predicted results from above, it is recommended to carry out the analyses on different approaches using varying parameters since the reliable rock mass properties are difficult to obtain due to the inherent complex nature of rock mass.

The strength factors after full face of excavation were calculated from all analyses. The calculated rock bolts and shotcrete strength are considerably very low. As the strength of the installed rock bolt were not measured in the project, the verification on predicted values cannot be made. It is also observed that rock bolt's axial force in 3D analysis is lower than the 2D analysis. The axial force of rock bolt by 3D is the remaining value due to stress relaxation behind the in-progress excavated face as similar to vertical displacement ratio. Hence, the calculated force and moment results by 2D analysis is considered to be correct.

The high horizontal to vertical stress ratio measured in JRC project also provided tunnel crown stability condition. Slight to moderate regional metamorphism by hot volcanic emplacement increase the strength of the rock mass in JRC project ([Chapter 3](#)). Thus such wide spanned cavern, i.e. 20 m width, could be excavated safely in bedded sedimentary rock at JRC project.

CHAPTER 9

Tunnel profiles controlled by geological structures

9.1 Observation of flat roofs in JRC project

Underground excavations are usually designed with an arched roof of either a horseshoe or near the circular profile since they provide optimal uniform stress distributions around the excavation which often result in a more stable condition. However, an arched roof design is occasionally not optimal in some geological settings consisting of low-angled bedded sedimentary rocks, horizontally-foliated metamorphic rocks and horizontal jointed and fractured rocks.

In JRC project, such flat-roofed tunnel profiles were distinctly encountered where excavation was carried out in flat to low-angled bedded sedimentary rocks with commonly dipping at 0 to 25 degrees.

9.2 Research approach for stability analyses of flat-roofed tunnels

In this thesis, three cases where flat-roofed profiles were distinctly encountered in the JRC project were studied. The storage caverns were excavated in flat to low-angled bedded sedimentary rocks with commonly dipping at 0 to 25 degrees at 130 m depth below existing ground level. The other two smaller galleries excavated in the same geological condition at around 100 m depth were checked for their crown stability as well. Three approaches based on analytical method, numerical analysis using finite element method and stability design chart application were performed to check the stability of the flat-roofed design.

With regard to the analytical approach, the Voussoir beam mechanics and shear support systems, which play essential roles in cavern stability, are briefly described in [section 2.5](#). The stability assessment of tunnel condition by the modified stability graph technique is also described in [section 2.6](#).

9.3 Roof stability analysis by Voussoir beam theory and 2D finite element program (RS²)

9.3.1 Area 1 (20 m spanned hydrocarbon storage cavern)

Liquid hydrocarbon storage caverns with dimensions of 340 m length, 20 m width and 27 m height were excavated at 135 m below ground level in rock masses of Ayer Chawan Facies, Jurong Formation having an average dip angle of 0 to 10 degrees with the observed average bed thickness of 0.40 m. A flat roof formed by those sub-horizontal bedding planes encountered in the cavern was shown in [Fig. 9.1](#). Two sub-vertical joint sets were observed in the area. The rectangular shaped cavern shoulder which was formed by the intersection of the vertical joint planes with the sub-horizontal beddings was shown in the upper right of the figure. The cavern was excavated in three stages with 9 m height in each stage. The 80 mm thick shotcrete and 5 m long rock bolts were applied as tunnel support after excavation.

Fig. 9.1 Area 1 – storage cavern showing horizontal bedding at the roof with right-angled corners at shoulders (20 m Width x 9 m Height at top heading excavation)

9.3.1.1 Analytical approach using Voussoir beam theory

Table 9.1 Sequential calculation with varying (N) value for Area 1

Parameters					
T (thickness), m	0.40	γ_e (corrected unit weight), kN/m ³			26.59
s (span width), m	20	E' (elastic modulus), kPa			20,000,000
γ (unit weight), kN/m ³	27	σ (UCS), kPa			143,000
α (dip angle), degree	10	ϕ (friction angle)			37
N	0.1	0.2	0.3	0.4	0.5
Z ₀ , m	0.373	0.347	0.32	0.293	0.267
L, m	20.019	20.016	20.014	20.011	20.009
ΔL m	0.000	0.228	0.000	0.009	0.017
Z _{check} , m	0.0186	-0.212	0.014	0.003	-0.008
Z, m	0.373	Negative	0.32	0.138	Negative
F _m (maximum), kPa	71222.7	Negative	27697.7	48146.4	Negative
F _{avg} , kPa	18201.4	Negative	8924.8	17118.7	Negative
$\Delta L_{\text{previous}} = \Delta L$	0.00	0.228	0.000	0.009	0.017
ΔL	0.018	Negative	0.009	0.017	Negative
End Loop ($\Delta L - \Delta L_{\text{previous}} = 0$)	0.018	Negative	0.009	0.008	Negative
<i>F.S</i> crushing	2.31	Negative	5.94	3.42	Negative
<i>F.S</i> sliding	2.43	Negative	2.84	6.58	Negative
Midspan deflection (Z ₀ – Z)	0.00	Negative	0.0	0.155	Negative
Deflection/thickness ratio (%)	0.0	Negative	0	36.8	Negative
N	0.6	0.7	0.8	0.9	1.0
Z ₀ , m	0.24	0.213	0.187	0.16	0.113
L, m	20.008	20.006	20.005	16.004	16.003
ΔL m	0.000	0.008	0.000	0.009	Negative
Z _{check} , m	0.008	-0.002	0.005	-0.004	Negative
Z, m	0.24	Negative	0.187	Negative	Negative
F _m (maximum), kPa	18465.1	Negative	17805.67	Negative	Negative
F _{avg} , kPa	7796.4	Negative	8704.997	Negative	Negative
$\Delta L_{\text{previous}} = \Delta L$	0.000	0.008	0.000	0.009	Negative
ΔL	0.008	Negative	0.009	Negative	Negative
End Loop ($\Delta L - \Delta L_{\text{previous}} = 0$)	0.008	Negative	0.009	Negative	Negative
<i>F.S</i> crushing	8.91	Negative	9.24	Negative	Negative
<i>F.S</i> sliding	3.79	Negative	4.87	Negative	Negative
Midspan deflection (Z ₀ – Z)	0.000	Negative	0.000	Negative	Negative
Deflection/thickness ratio (%)	0	Negative	0	Negative	Negative
Buckling Limit = 1-(N _{max} – N _{min})	1 – (0.4) = 0.6				

The analysis was carried out by the iteration steps proposed by [Diederichs & Kaiser \(1999\)](#) as in [Fig. 2.15 \(section 2.5\)](#). The results are shown in [Table 9.1](#). The sedimentary bed is taken as 0.40 m thick, dipping at 10 degrees. The beds apparently became thicker in the lower level where caverns were excavated than in upper level. The unit weight of rocks is corrected by multiplying cosine value of dip angle to the original unit weight. The strength properties of the rock mass are also shown in the same table. The results of mid-span deflection and safety factors against crushing ($F.S_{\text{crushing}}$) and sliding ($F.S_{\text{sliding}}$) are calculated. The sequential calculation starts at $N = 0.1$ with initial $\Delta L = 0$. For subsequent steps, N value is incremented by 0.1 per step.

In this calculation, the calculation steps are halted for a few times after a negative Z check is recorded. At $N = 0.5$, the calculation is terminated after a negative Z is recorded.

The next step is resumed at $N = 0.6$ with re-initialized $\Delta L = 0.0$ m. The calculation is again halted at $N = 0.7$ and 0.9 with negative Z check values. At the available lowest N value of 0.4 , the safety factor (crushing) is 3.42 and the safety factor (sliding) is 6.58 with displacement (mid-span deflection/bed thickness) ratio of 36.8% . The buckling limit, which is the percentage of values of N within the range of 0 to 1 , is calculated to be 0.6 .

The above displacement ratio and buckling limit are close to the collapse limit proposed by [Diederichs & Kaiser \(1999\)](#). They adopted yield limit at displacement ratio 10% and buckling limit at 0.35 whereas collapse limit at displacement ratio 25% and buckling limit between 0.8 and 1.0 . From the site observation, the flat crown after excavation (before rock support application) was stable without any sign of rock fall or collapse.

9.3.1.2 Numerical modeling using RS²

The numerical analysis on storage cavern is carried out as in [section 8.2](#) and maximum vertical displacement is calculated as 8.8 mm on the roof.

9.3.1.3 Vertical displacement monitoring

Vertical displacement monitoring in one cavern is presented in [section 8.7](#) with discussion on the comparison of calculated and measured results.

9.3.2 Area 2 (16 m spanned gallery)

In Area 2, a tunnel gallery was excavated at a depth of 100 m in rock masses of Ayer Chawan Facies, Jurong Formation having an average dip angle of 10 degrees. The observed average bed thickness is 0.35 m. The gallery was excavated in two stages. In stage 1, a 10.2 m wide and 12.3 m high operation access gallery was first excavated which was followed by rock support installation. In stage 2, the gallery was widened for housing pumping equipment. The final extended width and height were 16 m and 12.3 m respectively as shown in [Fig. 9.2 a & b](#).

Fig 9.2 (a) Area 2 – Tunnel section showing horizontal bedding at the roof with right angle cornered shoulders (16 m width $\times$ 12.3 m height), **(b)** The roof was secured with rock supports.

The flat roof was observed on the crown inside the pump house area due to the horizontally bedded structure. This structure can also be seen on all side walls. The flat roof was self-stable after excavation as shown in [Fig. 9.2 a](#). It was finally covered with 80 mm thick shotcrete and secured with rock bolts as shown in [Fig. 9.2b](#).

9.3.2.1 Analytical approach using Voussoir beam theory

The sedimentary bed is taken as 0.35 m thick, dipping at 10 degrees. The analysis was similarly carried out by the iteration steps as for Area 1 and the results are shown in [Table 9.2](#).

The sequential calculation starts at $N = 0.1$ with initial $\Delta L = 0$. For subsequent steps, N value is incremented by 0.1 per step. At $N = 0.4$, the calculation is halted after a negative Z is recorded. The next step is resumed at $N = 0.5$ with re-initialized $\Delta L = 0.0$ m. The calculation is again halted at $N = 0.7$ and 0.9 with negative Z check values. At the available lowest N value of 0.6 , the safety factor against crushing ($F.S_{\text{crushing}}$) is 7.53, safety factor against sliding ($F.S_{\text{sliding}}$) is 5.60 with displacement

(mid-span deflection/bed thickness) ratio of 22.9%. The buckling limit is calculated to be 0.7.

Table 9. 2 Sequential calculation with varying (N) value for Area 2

Parameters					
T (thickness), m	0.35	γ_e (corrected unit weight), kN/m ³			26.59
s (span width), m	16	E' (elastic modulus), kPa			20,000,000
γ (unit weight), kN/m ³	27	σ (UCS), kPa			143,000
α (dip angle), degree	10	ϕ (friction angle)			37
N	0.1	0.2	0.3	0.4	0.5
Z ₀ , m	0.33	0.3	0.28	0.26	0.23
L, m	16.018	16.015	16.013	16.011	16.009
ΔL m	0.000	0.0107	0.0118	0.0165	0.000
Z _{check} , m	0.0178	0.0047	0.0013	-0.0055	0.0091
Z, m	0.3267	0.1674	0.0887	Negative	0.2333
F _m (maximum), kPa	52094.0	50815.2	63947.4	Negative	14586.4
F _{avg} , kPa	13313.0	14680.0	20605.3	Negative	5672.5
$\Delta L_{\text{previous}} = \Delta L$	0.0000	0.0107	0.0118	0.0165	0.000
ΔL	0.0107	0.0118	0.0165	Negative	0.0045
End Loop ($\Delta L - \Delta L_{\text{previous}} = 0$)	0.0107	0.0011	0.0047	Negative	0.0045
<i>F.S</i> crushing	3.16	3.24	2.57	Negative	11.28
<i>F.S</i> sliding	2.23	4.34	8.19	Negative	3.12
Midspan deflection (Z ₀ – Z)	0.00	0.1359	0.1913	Negative	0.000
Deflection/thickness ratio (%)	0.000	38.8	54.7	Negative	0
N	0.6	0.7	0.8	0.9	1.0
Z ₀ , m	0.21	0.19	0.16	0.10	0.08
L, m	16.007	16.006	16.004	16.002	16.001
ΔL m	0.0045	0.0074	0.000	0.0051	Negative
Z _{check} , m	0.0028	-0.0016	0.0044	-0.0034	Negative
Z, m	0.1298	Negative	0.1633	Negative	Negative
F _m (maximum), kPa	21845.4	Negative	13023.6	Negative	Negative
F _{avg} , kPa	9223.6	Negative	6367.1	Negative	Negative
$\Delta L_{\text{previous}} = \Delta L$	0.0045	0.0074	0.000	0.0051	Negative
ΔL	0.0074	Negative	0.0051	Negative	Negative
End Loop ($\Delta L - \Delta L_{\text{previous}} = 0$)	0.0028	Negative	0.0051	Negative	Negative
<i>F.S</i> crushing	7.53	Negative	12.63	Negative	Negative
<i>F.S</i> sliding	5.6	Negative	4.45	Negative	Negative
Midspan deflection (Z ₀ – Z)	0.0802	Negative	0.000	Negative	Negative
Deflection/thickness ratio (%)	22.9	Negative	0	Negative	Negative
Buckling Limit = 1-(N _{max} – N _{min})	1 – (0.6 - 0.3) = 0.7				

The above displacement ratio and buckling limit are close to the collapse limit proposed by Diederichs & Kaiser. From the site observation, the flat crown after excavation (before rock support applied) was stable without any sign of rock fall or collapse.

9.3.2.2 Numerical modeling using RS²

The calculated vertical displacements after excavation with rock support application for stage 1 (operation access gallery) was about 3mm (Fig 9.3a) which was progressively increased to 6 mm after stage 2 pump room excavation with support application (Fig. 9.3b).

Fig. 9.3 (a) Area 2 – Vertical displacement (m) after excavation and support (10.2 m wide and 12.3 m high operation access gallery), **(b)** Vertical displacement (m) after excavation and support of pump room gallery (totally 16 m wide and 12.3 m high)

9.3.2.3 Vertical displacement monitoring

Fig. 9.4 (a). Location of monitoring points for displacement measurements on tunnel roof and wall, Area 2 **(b)** Vertical displacement monitoring results in Area 2 at main tunnel

The location of prism installed for convergence monitoring at the main gallery is shown in Fig. 9.4a whereas the results of vertical displacement were shown in Fig. 9.4b. The monitored results over one year period showed a stable trend with vertical displacement between 3 to 5 mm and it was reasonably comparable with the calculated results by RS^2 analysis.

9.3.3 Area 3 (7.9 m spanned gallery)

This third example (Area 3) was excavated at the same depth level and of similar geological condition as in Area 2. However, a smaller tunnel of 7.9 m span and 9.9 m height as shown in Fig. 9.5 a and b was excavated.

Fig. 9.5 (a) Area 3 – Tunnel section showing horizontal bedding at the roof with right-angled corners at shoulders (7.9 m Width $\times$ 9.9 m Height), **(b)** Tunnel section showing horizontal bedding at the roof with right-angled corners at shoulders (7.9 m Width $\times$ 9.9 m Height)

9.3.3.1 Analytical approach using Voussoir beam theory

Table 9.3 Sequential calculation with varying (N) value for Area 3

Parameters					
T (thickness), m	0.35	γ_e (corrected unit weight), kN/m ³			26.59
s (span width), m	7.9	E' (elastic modulus), kPa			20,000,000
γ (unit weight), kN/m ³	27	σ (UCS), kPa			143,000
α (dip angle), degree	10	ϕ (friction angle)			37
N	0.1	0.2	0.3	0.4	0.5
Z ₀ , m	0.3267	0.3033	0.28	0.2567	0.2333
L, m	7.936	7.9311	7.9265	7.9222	7.9184
ΔL m	0.0	0.0013	0.0008	0.0006	0.0006
Z _{check} , m	0.0360	0.0298	0.0257	0.0216	0.0178
Z, m	0.3267	0.297	0.2757	0.2529	0.2296
F _m (maximum), kPa	12700.0	6984.8	5015.3	4100.4	3613.2
F _{avg} , kPa	3245.56	2017.84	1616.04	1457.92	1405.15
$\Delta L_{\text{previous}} = \Delta L$	0.0	0.0013	0.0008	0.0006	0.0006
ΔL	0.0013	0.0008	0.0006	0.0006	0.0006
End Loop ($\Delta L - \Delta L_{\text{previous}} = 0$)	0.0013	-0.0005	-0.0002	-0.0001	0.00
<i>F.S</i> crushing	12.95	23.55	32.8	40.12	45.53
<i>F.S</i> sliding	1.1	1.21	1.3	1.42	1.56
Midspan deflection (Z ₀ – Z)	0.00	0.0064	0.0043	0.0037	0.0037
Deflection/thickness ratio (%)	0	1.82	1.22	1.06	1.06
N	0.6	0.7			
Z ₀ , m	0.21	0.1867			
L, m	7.9149	7.9118			
ΔL m	0.0006	0.0006			
Z _{check} , m	0.0143	0.0112			
Z, m	0.206	0.1822			
F _m (maximum), kPa	3355.9	3253.51			
F _{avg} , kPa	1416.94	1482.15			
$\Delta L_{\text{previous}} = \Delta L$	0.0006	0.0006			
ΔL	0.0006	0.0006			
End Loop ($\Delta L - \Delta L_{\text{previous}} = 0$)	0.000	0.000	Iteration stopped at N=0.5 after End Loop value at 0.00		
<i>F.S</i> crushing	49.02	50.56			
<i>F.S</i> sliding	1.74	1.97			
Midspan deflection (Z ₀ – Z)	0.004	0.0045			
Deflection/thickness ratio (%)	1.13	1.28			
Buckling Limit = 1-(N _{max} – N _{min})	1 – (0.5) = 0.5				

An analysis similar to previous two areas is carried out. The sequential calculation starts at $N = 0.1$ with $\Delta L = 0$. For subsequent steps, N value is incremented by 0.1 per step (Table 9.3). The iteration is successfully completed at $N=0.5$ after zero End Loop value was obtained. The safety factor against crushing ($F.S_{\text{crushing}}$) is 45.53, safety factor against sliding ($F.S_{\text{sliding}}$) is 1.56 with very low displacement (mid-span deflection/bed thickness) ratio of 1.06%. The buckling limit is calculated to be 0.5.

The above displacement ratio is much below the yield limit proposed by Diederichs & Kaiser, but buckling limit is above the yield limit. From the site observation, the flat crown after excavation (prior to rock support application) was stable without any sign of rock fall or collapse.

9.3.3.2 Numerical modeling using RS²

The calculated absolute vertical displacements before and after rock support application were shown in Fig. 9.6 a & b. The vertical displacement is considerably minimal in both analyses within 4 to 5 mm. There was no displacement measurement by prisms in this tunnel.

Fig 9.6 (a) Area 3 – Absolute vertical displacement (m) after excavation (7.9 m Width x 9.9 m Height), **b** Absolute vertical displacement (m) after rock support application

9.4 Assessment of tunnel condition by the modified stability graph technique in JRC

To determine the stability condition of JRC project, total 50 tunnel faces and roofs at different locations were assessed using the method proposed by Potvin (1988),

Nickson (1992) and Hutchison and Diederichs (1996). The necessary tunnel dimensions, joint orientation and Q' data were taken from the respective face geological mapping sheets. Subsequent stability parameters A, B and C were determined by applying the graphs shown in Fig. 2.20 (section 2.7) and hydraulic radius (HR) was calculated using Eq. 2.7. The stability numbers (N') was again calculated using Eq. 2.6.

In Table 9.4, required base-data and calculated HR and N' of some representative tunnel faces were presented. It was observed that all tunnel faces are in the stable zone when plotted on the stability graph of Fig. 2.22 (section 2.7.1). It was reproduced and presented in Fig. 9.7. The stability condition of tunnel faces is owing to good rock mass condition with high Q' values.

Table 9.4 Stability parameters and calculated HR and N' of some tunnel faces

Tunnel ID	X	Y	HR	RQD	Jn	Jr	Ja	A	B	C	N'
OT0/1D	9.4	8.4	2.2	90	6	3	1	1	0.2	2	18.0
OT01/F	12.4	7.6	2.4	80	9	3	1	1	0.2	2	10.7
OT02/B	12.4	11.4	3.0	80	12	1.5	1	1	0.2	2	4.0
OT0/5A	9.4	7.9	2.1	80	6	3	2	1	0.2	2	8.0
OT0/5E	9.4	7.9	2.1	95	4	3	1	1	0.2	2	28.5
WG1/3	5.8	5.9	1.5	90	12	1.5	1	1	0.2	2	4.5
WG1/4	5.7	6	1.5	90	12	1.5	1	1	0.2	2	4.5
WG1/5	5.2	6.1	1.4	90	12	3	1	1	0.2	2	9.0
AT1/1B	9.4	7.9	2.1	85	15	3	1	1	0.2	2	6.8
AT1/1A	9.4	7.9	2.1	80	15	3	1	1	0.2	2	6.4
CS1/1A	20.4	7.2	2.7	90	12	3	1	1	0.2	2	9.0
CS1/2A	20.4	7.2	2.7	80	9	3	1	1	0.2	2	10.7
CS1/3A	20.4	9.2	3.2	85	12	3	1	1	0.2	2	8.5
CS1/4A	20.4	9.2	3.2	90	12	3	1	1	0.2	2	9.0
CS1/5A	20.4	7.1	2.6	90	12	3	1	1	0.2	2	9.0

Fig. 9.7 Condition of tunnel faces of JRC project after plotted on stability graph of Potvin (1988), Nickson (1992), Hutchison & Diederichs (1996)

Similarly in Table 9.5, required base-data and calculated HR and N' of some representative tunnel roofs were presented. The required roof lengths were assigned with 2 round excavation depth at each roof. The obvious flat roofs encountered in three places were included in the table. It was observed that all tunnel roofs are in the stable zone when plotted on the stability graph of Fig. 2.20 (section 2.6). It was reproduced and presented in Fig. 9.8. The stability condition of tunnel faces is due to good rock mass condition with high Q' values.

Table 9.5 Stability parameters and calculated HR and N' of some tunnel roofs

Tunnel ID	X	Y	HR	RQD	Jn	Jr	Ja	A	B	C	N'
OT0/1D	9.4	8	2.2	90	6	3	1	1	0.2	2	18.0
OT01/F	12.4	8	2.4	80	9	3	1	1	0.2	2	10.7
OT02/B	12.4	8	2.4	80	12	1.5	1	1	0.2	2	4.0
OT0/5A	9.4	8	2.2	80	6	3	2	1	0.2	2	8.0
OT0/5E	9.4	8	2.2	95	4	3	1	1	0.2	2	28.5
WG1/3	5.8	8	1.7	90	12	1.5	1	1	0.2	2	4.5
WG1/4	5.7	8	1.7	90	12	1.5	1	1	0.2	2	4.5
WG1/5	5.2	8	1.6	90	12	3	1	1	0.2	2	9.0
AT1/1B	9.4	8	2.2	85	15	3	1	1	0.2	2	6.8
AT1/1A	9.4	8	2.2	80	15	3	1	1	0.2	2	6.4
CS1/1A	20.4	10	3.4	90	12	3	1	1	0.2	2	9.0
CS1/2A	20.4	10	3.4	80	9	3	1	1	0.2	2	10.7
CS1/3A	20.4	10	3.4	85	12	3	1	1	0.2	2	8.5
CS1/4A	20.4	10	3.4	90	12	3	1	1	0.2	2	9.0
CS1/5A	20.4	10	3.4	90	12	3	1	1	0.2	2	9.0
OT0/2A	21	7.5	2.8	85	12	1.5	1	1	0.2	2	4.3
OT1/0B	7.9	9.9	2.2	70	15	1.5	1	1	0.2	2	2.8

Fig. 9.8 Condition of tunnel roofs of JRC project after plotted on stability graph of Potvin (1988), Nickson (1992), Hutchison & Diederichs (1996)

9.5 Relaxation

Diederichs (1999) and Diederichs and Kaiser (1999) reported the important role of relaxation in the behavior of underground structures, highlighting that in complex geometry underground mines, excavations may affect each other in such a way that relaxation becomes a key parameter in controlling room and stope caving and instability. Due to the excavation of parallel tunnels, abutment relaxation (side walls displacement) in the tunnel excavated in stratified media may affect roof behavior in a similar way to convergence displacement of these walls. For the Voussoir analog approaches, this means an increase both in deflection and buckling tendencies in the roof beds. This relaxation could be more significant in distressed areas, as in the case of cross zones in rooms or faulted zones. A certain degree of relaxation may produce

a decrease in the compressive force of the beam, and consequently, an increase in the deflection necessary to attain equilibrium. In stable cases, the beam will be capable of supporting this increase in deflection, but for a beam already close to equilibrium, this relaxation could ultimately cause roof failure.

In JRC Project which is aimed for 50 years lifespan usage, the pillar width, rock mass medium between two adjacent excavations, is designed to balance the overstressing caused by the combination of rock mass strength and in situ stresses. A good rule of thumb is that the distance between two excavations should be approximately equal to the height of the larger tunnel (Hoek, 2007). The pillar width between twin storage caverns of JRC Project is 40 m which is more than 27 m, the height of cavern.

The stress distributions associated with the various major excavation sections (operation tunnels, access tunnels and storage caverns) and possible stress interactions between excavations were analyzed using computer codes Examnine2D (Rocscience, Inc. 2010). The code is a boundary element computer program for the elastic stress analyses of underground excavations.

Fig. 9.9 shows the major principal stress distribution of excavations for both two levels with some caverns and tunnels. The major and minor principal stresses around openings are in compression, and compressive stress levels are lower than the compressive strength (UCS) of the rock mass. There is very minimal stress interaction between the two storage caverns due to the pillar width between them is more than 27 m (height of cavern). Also, there is minimal stress interaction between the caverns and respective maintenance chambers due to the sufficient pillar width among them. Therefore there is unlikely to occur abutment relaxation (convergence displacement of walls) in JRC project.

9.6 Summary and discussion

In this study by analytical approach using Voussoir beam theory, the mid span deflection (vertical displacement at the crown) associated with safety factors for crushing and sliding of 0.35 to 0.40 m thick laminated beds are calculated at varying

N values using the Voussoir Beam theory for galleries and storage caverns of 3 different spans.

Fig. 9.9 Major principal stress distribution of excavations for both two levels with some caverns and tunnels

The 7.9 m wide span case could only be calculated successfully until the end loop ($\Delta L - \Delta L_{\text{previous}}$) value approached zero. The higher safety factor (crushing) and (sliding) with considerably low displacement (mid-span deflection/bed thickness) ratio are also obtained. The calculated mid-span deflection (vertical displacement) values match with those derived from the FEM RS² analysis.

Following [Diederichs & Kaiser \(1999\)](#), the calculated displacement ratio and buckling limit of the broader span of 16 m and 20 m cases are in the collapse regime. However, the respective flat roofs were observed to be stable. In the Voussoir Beam theory, only geometry, i.e., span (s) and bed thickness (T) of an individual layer are considered in the calculation of Z and Z_0 . The rock stiffness and bonding strength between layers are not considered in the displacement calculation.

The RS² program requires the input of rock mass modulus, normal and shear stiffness between layers for analysis. The high horizontal stress factor which is beneficial to the underground mines' roof stability is also considered ([Kar Winn, et al., 2016](#)). Thus the vertical displacement obtained from FEM is generally lower than that obtained by the analytical calculation.

Field vertical displacement monitoring by convergent points measurement at Areas 1 and 2 are comparable to the ranges predicted by RS² FEM program. Thus it is recommended to carry out the stability analysis using more than one approach during design stage before excavation.

From the above three case studies, the excavation of galleries and cavern in sub-horizontally bedded rock masses having an average thickness between 0.35 m and 0.40 m is permissible. However, rock support was still applied for the safety of subsequent excavations. By securing the rock layers by rock bolts, the multiple layers can act as one large beam of sufficient thickness.

The stability of tunnel roofs was checked by empirical approach after applying stability graph ([Potvin 1988](#), [Nickson 1992](#), [Hutchison & Diederichs 1996](#)). The stable condition of tunnel faces is due to good rock mass condition with high Q' values.

In the JRC Project which has a usage lifespan of 50 years, the pillar width is designed to balance the overstressing caused by the combination of rock mass strength and in situ stresses. Hence abutment relaxation is unlikely to occur.

This study shows that the geological structure plays an important role in controlling the tunnel profile of the excavation work.

CHAPTER 10

Summary and conclusions

This chapter summarizes the major discoveries and discussions in previous chapters and provides some recommendations for future research.

10.1 Summary on key findings of research work and recommendations for future application

10.1.1 Geological setting of the area

After seven southwestern islands were reclaimed and merged, the geology of the present research area in Jurong Island was first studied in 2004 for feasibility assessment to excavate and construct underground storage rock caverns namely Jurong Rock Cavern Project. Geologically speaking, based on constituted rocks, location and stratigraphic position it belongs to Ayer Chawan Facies of Jurong Formation.

The LA ICP-MS U-Pb zircon age of pyroclastic rocks of the Ayer Chawan Facies is determined to be 240.6 ± 1.2 Ma (Middle Triassic) from this study. It is slightly older but close to the age range assigned after fossil species collected by [Lim \(1974\)](#).

Dark grey to black mudstone were originated by turbidity sediments at the deep marine environment. Field evidence suggests that hot pyroclastic lava flowed concurrently over the muddy sediments while they were in wet and unconsolidated state. It gave rise to the formation of peperitic structures at the boundary between the two different rock types. Fluidal peperite having elongated globular, tendril and irregular shaped juvenile clasts were observed. In this study area fluid instabilities within vapour films, magma-sediment density contrasts and fluidisation of host sediment were likely the main mechanisms to form the fluidal juvenile clasts and mingling with host sediments.

The host sediments later experienced induration, contact metamorphism, local dewatering, cementation and compaction by simultaneous hot pyroclastic lava flows, leading eventually to the formation of strong mudstone. Having higher UCS results of mudstone in JRC (140 MPa) than similar types of typical sedimentary rocks

elsewhere on the world (40 MPa; [Waltham, 1994](#)) demonstrated the gain in strength after thermal effect on sedimentary rocks by lava emplacement. Hence, 20 m wide underground storage caverns could be successfully excavated safely in such rock mass.

This observation provides the source of metamorphism on sedimentary rock formation in Singapore. It is also the first ever observation of interbedded mudstone and lithic tuff sequence from deep underground project in Singapore. The finding also provides value added information to the modification of Singapore Geology.

10.1.2 Engineering geological properties of the rock masses of Ayer Chawan Facies, Jurong Formation in JRC Project

Measured discontinuities in one storage cavern (CS '1/1) were studied by using DIPS programme to define the major sets in the project. From the contour plot analysis, 1 set of bedding with varying dip angles (from 5° to 25°), 4 sets of joint and 2 sets of fault are grouped as prominent sets in the cavern. Those surveyed discontinuities data can be utilised in further evaluation to check for potential unstable wedge formation on the roof and walls of the cavern which are harmful during the works.

The results of measured horizontal stress of sedimentary rocks in JRC project were comparable with those of the Singapore granitic rock formation. Although the range of the stress ratios in Bukit Timah Granite is higher than those of the Jurong Formation, the general trend of maximum stress in both formations are nevertheless similar. It is also in the same general direction (i.e. NE-NNE) of maximum horizontal stress in the neighboring regions of Sumatra and southern part of Peninsular Malaya, The reason for the similarity is due to tectonic stresses developed at the convergent boundary (Sumatra-Java trench) between the Euroasian and the Indian Ocean Plate.

The understanding of the trend and magnitude of the horizontal stresses are beneficial in the tunnel alignment determination and the crown stability situation of the underground rock excavation projects. The higher horizontal stresses in both Singapore Formations provide safe and secured construction of large spanned storage caverns.

The cumulative distribution of the 79 coefficient of permeability data at water bearing zone indicates that k value is in the order of 1×10^{-6} m/s and the average value is 1.73×10^{-6} m/s. The calculated coefficient of permeability of recharging wells in one water curtain gallery is in the comparable range from 1×10^{-6} m/s to 1×10^{-7} m/s. Both results show that there is no high permeable water zone in the rock masses of the project.

Strength properties of intact rocks by point load tests, uniaxial compression test, triaxial compression tests and Brazilian tests were measured at laboratories in NTU and Beijing University. The measured Brazilian tensile strength is in the majority range between 6 and 10 MPa. Direct shear tests on rock joints of core samples were also carried out at the NTU laboratory. Those measured intact rock properties are fundamental data to determine the rock mass properties of the project area. The calculated factor of 20 from sedimentary rocks of JRC project is recommended as basic for conversion of point load results into uniaxial compression strength of the other rock units.

Two rock classification systems were determined independently on designated excavation faces in JRC. RMR was recorded between 60 and 80 while Q values scattered between 4 and 70. The general correlation equation between Q and RMR in JRC is comparable with that proposed by [Rutledge and Perston \(1978\)](#). This correlation factor can be used as reference for future projects.

The quantitative Geological Strength Index (GSI) of selected 420 excavated rock faces in JRC was determined by four different approaches available in the literature. The required rock mass parameters for calculation were based on the records of geology face mapping works carried out during the construction period.

Two basic parameters (blockiness of rock mass and surface conditions of discontinuities) necessary for the GSI determination were defined and calculated by four different approaches.

- [Hoek et al. \(2013\)](#) followed the joint condition rating of Rock Mass Rating (RMR) proposed by [Bieniawski \(1989\)](#) for surface condition and RQD for blockiness calculation.

- In Cai et al. (2004), the rock mass characterization system (RMI) proposed by Palmstrom (1995) was used for the rating of discontinuity surface and the individual rock block volume for blockiness determination.
- Sonmez and Ulusay (1999) followed the rating of Roughness R_r , Weathering R_w and Infilling R_f in Bieniawski (1989) RMR for surface condition rating and the volumetric joint counts for blockiness rating.
- Russo (2007 & 2009) applied the surface condition and blockiness rating similar to those by Cai et al. (2004) but he proposed a different GSI graph.

Although quantitative GSI was calculated by four different means, all the results are approximately in the same range from 45 to 55. Comparison was made with the qualitative GSI which were obtained based on assessment on the rock faces using the chart by Hoek et al. (1995). The quantitative GSI and qualitative GSI varied within a range of ± 10 .

As a modification to the Hoek et al. (2013) approach, a new GSI relationship was proposed for Ayer Chawan Facies by replacing $RQD/2$ with $RQD/3$ as the former was found to provide much higher GSI which do not match the qualitative field values.

The observed qualitative and quantitative GSI in JRC project is used as a guide for other sedimentary rocks in Singapore but some justification is suggested to suit the others. The field determination of GSI is suggested here to be carried out in every rock excavation project as it is the basic parameters for rock mass properties calculation.

The four approaches presented in this study for the quantitative GSI determination are all user friendly, and applicable to rock masses (Ayer Chawan Facies) in JRC Project. Thus it is recommended to choose any one of them, depending on the availability of rock mass parameters in the assigned project.

10.1.3 Rock mass properties determination

Set of data pairs ($\sigma_{1 \text{ i-n}}$ & $\sigma_{3 \text{ i-n}}$) are selected following the suggested procedures by Zhang et al. (2018) from the triaxial testing of 84 rock core specimens to determine the intact rock properties such as cohesion (c , MPa), friction angle (ϕ degree),

uniaxial compression strength (σ_{ci}) and Hoek-Brown parameter (m_i) applying RocData program. The triaxial tests were conducted for mudstone and pyroclastic rocks under dry and wet condition to explore the effect of water content on strength properties.

It is noted in the present study that the triaxial compressive strength generally increased with increasing confining pressures. Meanwhile, the water content affected the triaxial compressive strength and deformability of rocks. Based on the Mohr-Coulomb failure criterion, it was found that the cohesion increased and friction angle decreased from dry to wet conditions for both tested rock types (Li et al., 2012). Based on the Hoek-Brown failure criterion, the H-B constant of m_i was found to decrease obviously from dry to wet conditions for both tested rock types.

The uniaxial compressive strength (σ_{ci}) was observed to decrease in mudstone from dry to wet conditions but increase in pyroclastic rocks. It is interpreted that the bonding strength between individual rock fragments in pyroclastic rocks becomes weaker when they are oven dried under 105° C.

Subsequently, the rock mass properties such as Hoek-Brown criterion m_b , s , a and deformation modulus (E_{rm}) were determined from intact properties with actual field input data such as GSI and D . The existing established equations are used in the determination. The rock mass deformation modulus (E_{rm}) decreased from dry to wet conditions for both rock types.

Among the three calculated results, E_{rm} by Hoek and Diederichs (2006) (Eq. 5.15) approach is observed reliable since the actual measured GSI, D and σ_{ci} are used as input parameters in it. Thus the results obtained by this approach are used in further numerical analyses in this study.

The present work will not only benefit the future underground space development in the Jurong Formation of Singapore, but also enrich the literature with regard to the mechanical properties of low porosity sedimentary rocks.

10.1.4 Soft computing and regression analysis

In this study, an intelligent approach based on the Mamdani inference mechanism was applied to predict UCS of the rock masses of Ayer Chawan Facies in JRC Project. To predict the output of UCS, bulk density, porosity and point load index test results were taken as input parameters. To validate the results from Mamdani fuzzy analysis, the acquired results are validated with those from the statistical model applying the coefficient of correlation (R^2) and root mean square error (RMSE). For the fuzzy model, R^2 and RMSE were calculated 0.8529 and 14.1 whereas for the statistical model, these indices were determined as 0.6334 and 14.4 respectively. These results proved that the performance of fuzzy model is significantly better than the statistical model. Also, fuzzy model results provided comparable range of results with the laboratory-measured UCS.

As the intelligence approach is proved to be competent in predicting UCS, this practice should be applied in preliminary UCS determination in the works related to mining engineering and engineering geology. Being the first ever effort using this approach but with limited test results, it is advisable to update the current model when additional UCS results are available in local civil and mining engineering projects.

The fuzzy set theory is also applied in quantitative GSI determination following the Mamdani fuzzy algorithm. At first, joint surface condition (J_c) was determined by the first fuzzy algorithm using a total of 210 "if-then" rules from three input functions namely joint waviness, small-scale smoothness and joint alteration factors as proposed by [Palmstrom, 1995](#).

In the second fuzzy algorithm, the fundamental parameters (block volume and joint surface condition) which were used in Cai's quantitative GSI chart (2004) are used as two input membership functions in the determination of GSI after setting 22 "if-then" rules.

The coefficient of correlation, R^2 between quantitative GSI and fuzzy-based predicted GSI is calculated as 0.8477 whereas qualitative GSI and fuzzy-based predicted GSI is 0.7945. Hence it can be concluded that GSI determined by fuzzy analysis approach for Singapore's rock masses in JRC project is comparable with quantitative GSI ([Cai](#)

et al., 2004 approach) and on-site qualitatively assessed results (field assessment using Hoek et al., 1992).

10.1.5 Engineering geological classification of storage cavern

13 geological homogeneous zones were categorised in one storage cavern, as a sample approach, based on the assessments of recorded field geological data. Each zone is again examined to evaluate the relative potential wedge formation by combining geological information, such as joint roughness, orientation, persistence, aperture and number of joint set. 4 types of zones (none, low, medium and high potential) were classed based on the possibility to form wedges.

The prominent discontinuity sets (Table 4.1) are again used with UnWedge program to evaluate the possibility of wedges formation during subsequent bench excavation. UnWedge program is applicable to quickly create a model, perform a safety factor analysis, apply reinforcement and interpret the output results.

Two different cases were studied; one case using bedding and two major joint sets, the other using bedding with all 4 joints. The results show that no additional unsafe wedges can be formed beyond using with those investigated discontinuity sets encountered in the cavern.

It is advisable to study and differentiate the engineering geological zones in the excavated area and explore any possible wedge formation on roof and walls. If the hazardous wedges on the roof are not properly secured, they may endanger those working there.

10.1.6 2D and 3D numerical analysis for the stability of excavation

In this study, discontinuum analysis (intact rock and joint parameters under Mohr-Coulomb failure criterion approach) in 2D and continuum analysis (equivalent rock mass properties under Generalised Hoek-Brown failure criterion approach) in 2D and 3D are carried out to keep a record of the results of rock mass response after excavation work. The aim of choosing FEM programs (RS² & RS³, Rocscience Inc.) is to prove the ability of programs in handling analyses of large-scale projects and also to compare the output results with approaches using different parameters. In

addition, the longitudinal displacement profile (LDP profile) of the tunnel was established to compare with the results of previous researchers.

As summarized in [Table 8.19](#), the calculated results by different approaches gave a wide range in vertical settlement. In 2D continuum analysis, the lowest displacement is obtained by applying rock mass parameters based on GSI of 55 whereas the highest values are from GSI 40 based parameters analysis. About 90 % of total vertical displacements have already occurred on the roofs after top heading excavation.

As presented in [section 8.7](#), if the measured vertical displacement (about 5 mm) is taken to be the remaining 40 %, the total displacement will be around 10 mm. In the project, the convergence monitoring instruments are usually installed on the faces after a few rounds of excavation completion to prevent instrument damage by blasting. From the study of LDP profile of JRC, the remaining 40 % of total displacement can be captured at about 5 m behind the current excavated face.

That range of measured value is agreeable with the results by 2D discontinuum and 3D continuum analyses. GSI of 40 is the extreme lowest value and 55 is the highest recorded in the area. The measured vertical displacement might be the result of the instruments installed at the rock face having GSI between 40 and 55.

Applying mudstone and pyroclastic rock interbedded model in 2D analysis gave the least vertical settlement among all analyses. It is due to the higher strength of the pyroclastic rocks.

When conducting the study with 3 m spaced rock bolt installation, the results of displacement, strength factor, axial forces of rock bolts and shotcrete do not change considerably and it is in safe working condition in each analysis. However it is hard for the authorities to accept if the alternative design deviates from the approved code of 2.2 m spacing. Based on working experience in other cavern projects, it is recommended that both sides of the walls, from the invert level up to about 1.5 m high (around the shoulder height of people), of final Bench 2 zone, can be left without support measures. The impact on the safety is minimal even if the rock falls occur on those walls.

There is not much change in vertical displacements when pillar width reduced to 25 m. However the horizontal displacements on inner walls of two caverns become lower after lesser induced forces from the rock mass of narrow pillar. It is also observed that distributed major principal stress values slightly higher than those in larger pillar width. That increment of major principal stress may cause wall collapse in weaker rock mass.

The high horizontal displacements of about 50 mm inward movements were recorded in the analyses but there was no serious effect during excavation works. It was also noted that continuum analysis using rock mass parameters showed higher horizontal displacements than discontinuum analysis. A possible reason might be that the discontinuum analysis represents the real rock mass condition after shear strength parameters (c and ϕ) of bedding and joint planes are applied in the calculation.

After considering the wide-ranged predicted results from above, it is recommended to carry out the analyses using different approaches with varying parameters since the reliable rock mass properties are difficult to obtain due to the inherent complex nature of rock mass.

The strength factors after full face of excavation were calculated from all analyses. The calculated rock bolts and shotcrete strength are considerably very low. Unsurprisingly the highest axial force of rock bolts and moments and shear forces of shotcrete are recorded on the cavern roofs. As the strength of the installed rock bolt were not monitored in the project, the verification on predicted values cannot be made.

It is also observed that rock bolt's axial force in 3D analysis is lower than the 2D analysis. The axial force by 3D is the remaining value due to stress relaxation behind the excavated face in progress as similar to vertical displacement ratio. Hence, the calculated force and moment results by 2D analysis is considered to be correct.

The high horizontal to vertical stress ratio measured in JRC project also provided tunnel crown stability condition. Slight to moderate regional metamorphism by hot volcanic emplacement increased the strength of the rock mass in JRC project. Thus such wide-spanned cavern, i.e. 20 m width, could be excavated safely in bedded

sedimentary rock at JRC project.

10.1.7 Tunnel profiles that controlled by geological structures

In the project, the unexpected flat-roofed profiles are observed in some area where excavation is done in low-angle bedded competent sedimentary beds which are intersected by prominent vertical joints at the crown. To prove the stability of such flat-roofed profile, the analytical, numerical and empirical analyses are performed on 3 areas having different span widths.

In the analytical approach the iteration steps proposed by [Diederichs and Kaiser \(1999\)](#) is applied but only the 7.9 m wide span case could be calculated successfully with high safety (crushing and sliding) with considerably low vertical displacement (mid-span deflection / bed thickness) ratio. The calculated results of the wider span of 16 m and 20 m cases are in the collapse regime. However, the flat roofs of those wider spanned tunnel are observed stable after excavation.

The numerical analysis by RS² showed stable condition for all three cases and predicted vertical displacements are in the comparable ranges with measured values in 16 m and 20 m wide tunnel and cavern.

In the [Diederichs & Kaiser \(1999\)](#) proposed calculation steps which are based on Voussoir Beam theory, only geometry, i.e., span (s) and bed thickness (T) of an individual layer are considered. The RS² program requires the input parameters such as rock mass modulus, normal and shear stiffness between layers for analysis. The high horizontal stress factor which is beneficial to the underground mines' roof stability is also considered in the calculation. Thus the vertical displacement obtained from FEM is generally lower than that obtained by the analytical calculation.

The tunnel roofs and faces are found to be stable by applying empirical approach using stability graph ([Potvin 1988](#), [Nickson 1992](#), [Hutchison & Diederichs 1996](#)). The stable condition of tunnel faces are due to good rock mass condition with high Q' values.

In the JRC Project which has a usage lifespan of 50 years, the pillar width is designed to balance the overstressing caused by the combination of rock mass strength and in situ stresses. Therefore abutment relaxation is unlikely to occur.

From this site observation and study, the shape of tunnel profiles are largely controlled by some major geological structures such as bedding planes, foliation planes and discontinuities. It is also recommended to use more than one approach for the stability analysis of unforeseen tunnel profiles during excavation work.

This study shows that the geological structure plays an important role in controlling the configuration of the tunnels and galleries in the excavation work.

10.2 Recommended works for future research in rock excavation projects

Based on the current study and observation the following is recommended for carrying out in future rock excavation projects and related research works:

- The detailed engineering geological properties of intact rocks are to be measured to understand the properties of rock masses;
- The shear strength properties of discontinuity surfaces are to be measured for determining failure potential along the plane;
- The in situ stresses are to be measured provided the budget for the project allows so that proper tunnel support design can be determined;
- Both tunnelling rock mass quality "Q" and rock mass rating "R" should be measured concurrently so as to make correlation between Q and RMR;
- Geological Strength Index "GSI" is to be determined on rock faces. It is useful in the determination of the rock mass properties. Based on the present study, the four approaches for the quantitative GSI determination are user-friendly and are applicable to Singapore's sedimentary rock masses. Thus the user may choose any one of them, depending on the availability of rock mass parameters. Either qualitative or quantitative GSI determinations are suggested to be carried out in other rock excavation projects in Singapore;
- Hydrogeological property of rock mass is to be measured inside drilled boreholes, probe holes or water-bearing zones. It can determine the rate of water injection rate inside water curtain galleries for the hydrodynamic containment system for oil and gas storage;

- Based on the longitudinal profile determined from a 3D analysis, about 60 % of a total vertical settlement are relaxed at about 5 m behind the excavated face in the project. Thus installation of convergence monitoring apparatuses should not be too far behind the excavated rock face;
- Instruments to measure rock bolt loads are to be used to check the load-bearing capacity of bolts;
- Installation of rock bolts should not be too far behind the excavated rock face to prevent potential rock fall;
- The strike and dip of discontinuity planes are to be measured to observe the geological structures of the area. It is to be used for wedge stability analysis;
- The soft computing approach (Mamdani fuzzy algorithm) is recommended to provide the preliminary estimate of UCS for similar sedimentary rocks in other parts of Singapore. As the current study is the first attempt of such with limited data, it is advisable to update the current model when additional UCS results are available in local civil and mining engineering projects;
- During the preliminary stage, both analytical and numerical (2D) analyses are recommended to be performed for safe and economical design. The 3D numerical analysis should also be involved during the construction stage by using detailed engineering properties to counter check the earlier proposed design;

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Fig. A.1 Routine mineral determination (Raith et al., 2012)

Fig. A.2 World stress map of South Asia showing Singapore and Java Trench
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Fig. A.4 Portable point load equipment and typical failure mode of valid tests (NTU, 2013)

Fig. A.5 Typical failure modes; (a) axial test (b) diametric test (NTU, 2013)

Fig. A.6 Correlation between the RMR and the Q index (Bieniawski, 1976)

MODIFIED HOEK-BROWN FAILURE CRITERION												
$\sigma'_1 = \sigma'_3 + \sigma_c \left(m_b \frac{\sigma'_3}{\sigma_c} \right)^\alpha$ <p>σ'_1 = major principal effective stress at failure σ'_3 = minor principal effective stress at failure σ_c = uniaxial compressive strength of <i>intact</i> pieces in the rock mass m_b and α are constants which depend on the composition, structure and surface conditions of the rock mass</p>												
STRUCTURE			SURFACE CONDITION		VERY GOOD		GOOD		FAIR		POOR	
					Unweathered, discontinuous, very tight aperture, very rough surface, no infilling		Slightly weathered, continuous, tight aperture, rough surface, iron staining to no infilling		Moderately weathered, continuous, extremely narrow, smooth surfaces, hard infilling		Highly weathered, continuous, very narrow, polished/slickensided surfaces, hard infilling	

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek and Marinos, 2000)						
<p>From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.</p>		SURFACE CONDITIONS				
		VERY GOOD Very rough, fresh unweathered surfaces				
		GOOD Rough, slightly weathered, iron stained surfaces				
		FAIR Smooth, moderately weathered and altered surfaces				
		POOR Slackensided, highly weathered surfaces with compact coatings or fillings or angular fragments				
VERY POOR Slackensided, highly weathered surfaces with soft clay coatings or fillings						
STRUCTURE		DECREASING SURFACE QUALITY →
INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities	DECREASING INTERLOCKING OF ROCK PIECES ↓	90			N/A
BLOCKY - well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets		80			N/A
VERY BLOCKY- interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets		70
BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity		60
DISINTEGRATED - poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces		50
LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes		40			
			30			
			20			
			10			
			N/A	N/A		

Fig. A.8 Characterization of blocky rock masses on the basis of interlocking and joint condition for GSI determination (Marinos and Hoek, 2000, modified after Hoek, et al., 1995)

Fig. A.9 Estimate of Geological Strength Index, GSI, for heterogeneous rock masses such as flysch (Marinos and Hoek, 2000)

Fig. A.10 Quantitative GSI determination by joint condition and RQD (Hoek et al., 2013)

Fig. A.11 Quantitative GSI determination by joint condition and block volume (Cai et al., 2004)

Fig. A.12 Block volume (V_b) determination by three joint sets (S_1, S_2, S_3 = spacing between discontinuity sets; $\gamma_1, \gamma_2, \gamma_3$ = angles between discontinuity sets) ([Palmstrom, 1995](#))

Fig A.13 Quantitative GSI determination by surface condition and structure rating (Sonmez and Ulusay, 1999)

Fig. A.14 Determination of structure rating from J_v (Sonmez and Ulusay, 1999)

Fig. A.15 Quantitative GSI determination by joint condition factor and block volume (Russo, 2009)

Appearance of rock mass	Description of rock mass	Suggested value of D
Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	$D = 0$
Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	$D = 0$ $D = 0.5$ No invert
Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	$D = 0.8$
Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

Fig. A.16 Guidelines for estimating disturbance factor D (Hoek, et al., 2002)

Fig. A.17 Plot of normalized in situ rock mass deformation modulus from China and Taiwan against Hoek and Diederichs Equation (Eq. 5.15). Each data point denotes the average of multiple tests at the same site and the rock mass (Hoek and Diederichs, 2006)

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Table A.1: Rating of JCond₈₉ after Bieniawski (1989)

Condition of discontinuities	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
Guidelines for classification of discontinuity conditions					
Discontinuity length (persistence) Rating	<1 m 6	1 to 3 m 4	3 to 10 m 2	10 to 20 m 1	More than 20 m 0
Separation (aperture) Rating	None 6	<0.1 m 5	0.1-1.0 mm 4	1 – 5 mm 1	More than 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard infilling < 5 mm 4	Hard filling > 5 mm 2	Soft infilling < 5 mm 2	Soft infilling > 5 mm 0
Weathering Rating	Unweathered 6	Slightly weathered 5	Moderate weathered 3	Highly weathered 1	Decomposed 0

Table A.2: Rating of JCond₇₆ after Bieniawski (1976)

Condition of discontinuities	Very rough surfaces, Not continuous, No separation, Hard wall rock	Slightly rough surfaces, separation < 1 mm, Hard joint wall rock	Slightly rough surfaces, Separation < 1 mm, Soft joint wall rock	Slickensided surfaces or Gouge < 5 mm thick or Joints open 1 – 5 mm, Continuous joints	Soft gouge > 5 mm thick or Joints open > 5 mm, Continuous joints
Rating	25	20	12	6	0

Table A.3: Rating of J_r and J_a for rock wall contact (no pre-shearing) after [Barton et al. \(1974\)](#)

Joint roughness number, J_r	Rating	Joint alteration number, J_a	Rating
Discontinuous joints	4	Tightly healed, hard, non-softening, impermeable filling	0.75
Rough and irregular, undulating	3	Unaltered joint walls, surface staining only	1.0
Smooth, undulating	2	Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0
Slickensided, undulating	1.5	Silty or sandy-clay coatings, small clay fraction (non-softening)	3.0
Rough or irregular planar	1.5	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 in thickness)	4.0
Smooth, planar	1.0		
Slickensided, planar	0.5		

Table A.4: Rating of joint waviness, J_w after [Palmstrom \(1995\)](#)

Waviness terms	Undulation	Rating for waviness J_w
Interlocking (large scale)		3
Stepped		2.5
Large undulation	>3%	2
Small to moderate undulation	0.3 – 3%	1.5
Planar	<0.3%	1

Undulation = a/D
D - length between maximum amplitudes

Table A.5: Rating of small-scale smoothness, J_s after Palmstrom (1995)

Smoothness terms	Description	Rating for smoothness J_s
Very rough	Near vertical steps and ridges occur with interlocking effect on the joint surface	3
Rough	Some ridge and side-angle are evident: asperities are clearly visible; discontinuity surface feels very abrasive (rougher than sandpaper grade 30)	2
Slightly rough	Asperities on the discontinuity surface are distinguishable and can be felt (like sandpaper grade 30-300)	1.5
Smooth	Surface appear smooth and feels so tough (smoother than sandpaper grade 300)	1
Polished	Visual evidence of polishing exists. This is often seen in coating of chlorite and specially talc	0.75
Slicken sided	Polished and striated surface that results from sliding along a fault surface or other movement surface	0.6-1.5

Table A.6 Rating of joint alteration factor, J_a after Palmstrom (1995)

	Term	Description	J_a
Rock wall contact	<i>Clear joints</i>		
	Healed or welded joints (unweathered)	Softening, impermeable filling (quartz, epidote etc.)	0.75
	Fresh rock walls (unweathered)	No coating or filling on joint surface, except for staining	1
	Alteration of joint wall: slightly to moderately weathered	The joint surface exhibits one class higher alteration than the rock	2
	Alteration of joint wall: highly weathered	The joint surface exhibits two classes higher alteration than the rock	4
	<i>Coating or thin filling</i>		
	Sand, silt, calcite etc.	Coating of frictional material without clay	3
	Clay, chlorite, talc, etc.	Coating of softening and cohesive minerals	4
	Filled joints with partial or no contact between the rock wall surfaces	Filling of frictional material without clay	4
	Compacted clay materials	“Hard” filling of softening and cohesive materials	6
	Soft clay materials	Medium to low over-consolidation of filling	8
	Swelling clay materials	Filling material exhibits swelling properties	8-12

Table A.7 Surface condition rating, SCR after [Sonmez and Ulusay \(1999\)](#)

Roughness rating (R_r)	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Weathering rating (R_w)	None 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0
Infilling rating (R_f)	None 6	Hard (<5 mm) 4	Hard (>5 mm) 2	Soft (<5 mm) 2	Soft (>5 mm) 0
SCR = R_r + R_w + R_f					

Table A.8 Rating of joint size factor, J_L after [Palmstrom \(1995\)](#)

Joint length (m)	Type	Continuous joints+	Discontinuous joints
<0.5	Crack	4	8
<1	Bedding or foliation parting	3	6
0.1-1	Joint (small)	2	4
1-10	Joint (medium)	1	2
10-30	Joint (long or large)	0.75	1.5
>30	(Filled) joint, seam or shear++	0.5	1

+ Discontinuous joints end in massive rock

++ Often a singular discontinuity with significant impact and should in these cases be evaluated separately

Table A.9 Field estimates of uniaxial compressive strength ([Brown, 1981](#))

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, limestone, marble, rhyolite, tuff
R4	Strong	50 - 100	2 - 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, phyllite, sandstone, schist, shale
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	Claystone, coal, concrete, schist, shale, 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, rocksalt, potash
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
R0	Extremely weak	0.25 - 1	**	Indented by thumbnail	Stiff fault gouge

*Grade

according to [Brown \(1981\)](#)

** Point load tests on rock with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results

Table A.10 Values of the constant m_i for intact rock, by rock group. Note that values in parenthesis are estimates (Hoek and Brown, 1980a)

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (22)	Sandstone 19 —— Greywacke (18)	Siltstone 9	Claystone 4
	Non-Clastic	Organic		—— Chalk 7 —— Coal (8-21)		
		Carbonate	Breccia (20)	Sparitic Limestone (10)	Micritic Limestone 8	
		Chemical		Gypstone 16	Anhydrite 13	
METAMORPHIC	Non Foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 25 - 31	Mylonites (6)	
	Foliated*		Gneiss 33	Schists 4 - 8	Phyllites (10)	Slate 9
IGNEOUS	Light		Granite 33		Rhyolite (16)	Obsidian (19)
			Granodiorite (30)		Dacite (17)	
	Dark		Diorite (28)		Andesite 19	
			Gabbro 27	Dolerite (19)	Basalt (17)	
			Norite 22			
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

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

Table A.11 Guidelines for the selection of modulus ratio (MR) values in Eq. (5.14) – based on Deere (1968) and Palmstrom and Singh (2001)

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
Sedimentary	Clastic		Conglomerates 300–400 Breccias 230–350	Sandstones 200–350	Siltstones 350–400 Greywackes 350	Claystones 200–300 Shales 150–250 ^a Marls 150–200
			Crystalline limestones 400–600	Sparitic limestones 600–800	Micritic Limestones 800–1000	Dolomites 350–500
	Non-clastic	Carbonates		Gypsum (350) ^b	Anhydrite (350) ^b	
		Organic				Chalk 1000 +
Metamorphic	Non-foliated		Marble 700–1000	Hornfels 400–700 Metasandstone 200–300	Quartzites 300–450	
	Slightly foliated		Migmatite 350–400	Amphibolites 400–500	Gneiss 300–750 ^a	
	Foliated ^a			Schists 250–1100 ^a	Phyllites/Mica Schist 300–800 ^a	Slates 400–600 ^a
Igneous	Plutonic	Light	Granite ^c 300–550 Granodiorite ^c 400–450	Diorite ^c 300–350		
		Dark	Gabbro 400–500 Norite 350–400	Dolerite 300–400		
	Hypabyssal			Porphyries (400) ^b	Diabase 300–350	Peridotite 250–300
	Volcanic	Lava		Rhyolite 300–500 Andesite 300–500	Dacite 350–450 Basalt 250–450	
		Pyroclastic	Agglomerate 400–600	Volcanic breccia (500) ^b	Tuff 200–400	

^aHighly anisotropic rocks: the value of MR will be significantly different if normal strain and/or loading occurs parallel (high MR) or perpendicular (low MR) to a weakness plane. Uniaxial test loading direction should be equivalent to field application.

^bNo data available, estimated on the basis of geological logic.

^cFelsic Granitoids: coarse grained or altered (high MR), fined grained (low MR).