添付資料　1

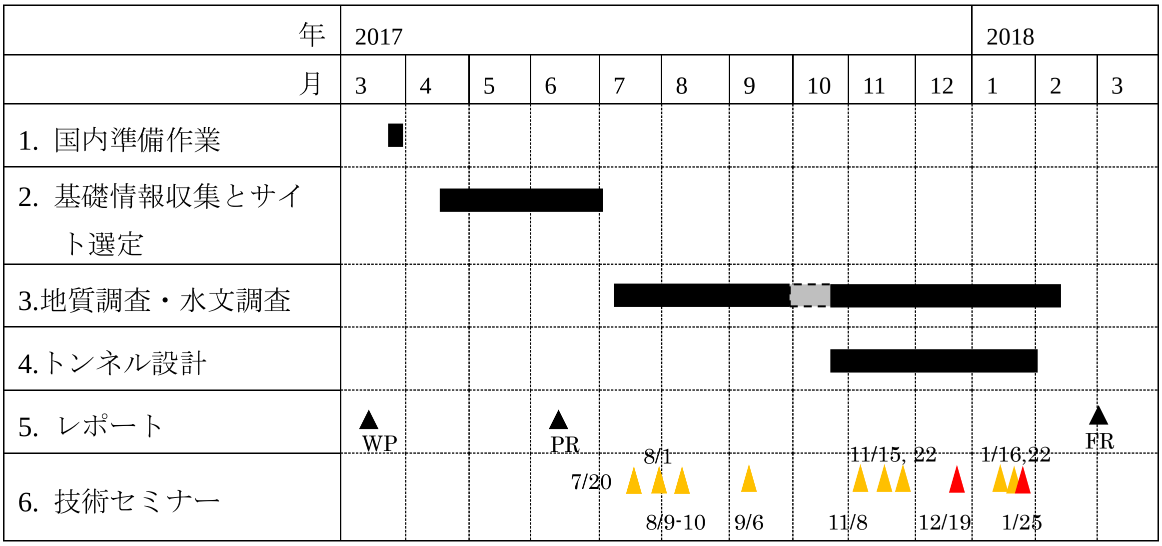
技術セミナー

セミナー写真
テクニカルノート

表 A1 セミナーの概要

技術セミナー	日付	セミナー内容	参加人数
1	7/20	1) トンネル地質調査 2) 弾性波探査	13
2	8/1	1) 水文地質 2) トンネル坑口斜面対策	14
3	8/9	1) 現場ワークショップの内容紹介, 2) 水文地質, 3) トンネル坑口斜面対策	25
4	8/10	現場ワークショップ 1) 弾性波探査, 2) コアボーリング	34
5	9/6	パイロットサイトにおける地質調査経過報告 1) 水文調査, 2) 弾性波探査, 3) コアボーリング	25
6	11/8	1) トンネル工法の紹介 (NATM), 2) 地山分類, 3) 空中写真地形判読, 4) 室内試験	25
7	11/15	室内試験場見学	13
8	11/22	室内試験場見学	8
9	12/19	**“Final Seminar Stage1”** 1) 道路トンネル建設のための弾性波探査, 2) 道路トンネル調査に関連する室内試験の紹介, 3) 道路トンネル建設のための地山分類, 4) 道路トンネルにかかる水文地質, 5) 道路トンネル工法, 6) キャンディトンネル計画	56
10	1/16	1) トンネル設計, 2) 地下水影響評価	24
11	1/22	1) トンネル設計, 2) 地下水影響評価	7
12	1/25	**“Final Seminar Stage2”** 1) 道路トンネル設計 -線形及び断面形状, 2) 道路トンネルに関連する地下水環境	31

表 A2 プロジェクトの活動およびセミナー



注: ▲“技術セミナー”、▲“Final Seminar”

表 A3 セミナー内容 (1/5)

	日　付	セミナーの実施内容
1	7 月 20 日	9:30 – 11:30 1) トンネル地質調査 2) 弾性波探査  
2	8 月 01 日	10.30 – 12:00 1) 水文地質 2) トンネル坑口斜面対策  

表 A3 セミナーの概要 (2/5)

	日　付	セミナーの実施内容
3	8 月 09 日	9:30 – 12:00 1) 現場ワークショップの内容紹介 2) 水文地質, 3) トンネル坑口斜面対策  
4	8 月 10 日	9:00 – 14:30 現場ワークショップ 1) 弾性波探査, 2) コアボーリング    

表 A3 セミナーの概要 (3/5)

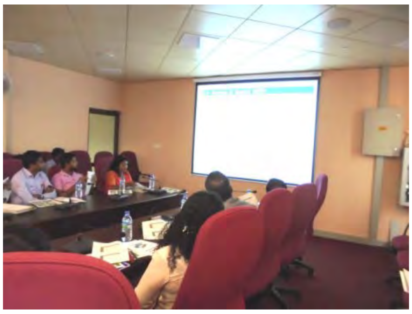
	Date	セミナーの実施内容
5	9 月 06 日	10:00 – 12:30 パイロットサイトにおける地質調査経過報告 1) 水文調査, 2) 弾性波探査, 3) コアボーリング 
6	11 月 08 日	10:00 – 12:30 1) トンネル工法の紹介 (NATM), 2) 地山分類 3) 空中写真地形判読, 4) 室内試験 
7	11 月 15 日	9.00 – 11:30 室内試験場見学 

表 A3 セミナーの概要 (4/5)

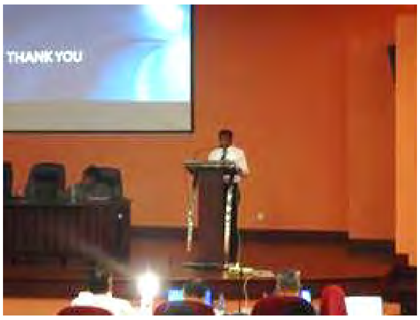
	日　付	セミナーの実施内容
8	11月22日	9.00 – 11:30 室内試験場見学  
9	12月 19日	9:00 – 14:00 **“Final Seminar Stage1”** 1) 道路トンネル建設のための弾性波探査 2) 道路トンネル調査に関連する室内試験の紹介 3) 道路トンネル建設のための地山分類 4) 道路トンネルにかかる水文地質 5) 道路トンネル工法 6) キャンディトンネル計画    

表 A3 セミナーの概要 (5/5)

	日　付	セミナーの実施内容
10	2018 1 月 16 日	9:00 – 12:00 1) トンネル設計 2) 地下水影響評価  
11	2018 1 月 22 日	9:00 – 11:30 1) トンネル設計 2) 地下水影響評価
12	2018 1 月 25 日	9:30 – 11:00 **"Final Seminar Stage2"** 1) 道路トンネル設計 -線形及び断面形状 2) 道路トンネルに関連する地下水環境    

JICA Technical Assistance Program on Road Tunnels -Final Seminar – Stage 1

Date : 19th Dec 2017
Venue : Auditorium, 11th floor, Sethsiripaya, stage II

1. 9.00 – 9.30 - Registration
2. 9.30 – 9.35 - Introductory Remarks by Chairman, RDA
3. 9.35-10.00 - Sub-surface exploration for road tunnels with specific reference to seismic refraction prospecting by Eng. (Mr.) B.H.T. Ariyarathne, R & D Division, RDA
4. 10.00-10.20 - Tea / Refreshment
5. 10.25 –10.50 - An introduction to laboratory tests in respect of road tunnels by. Eng (Ms.) Iromi Ranasoma, Deputy Director, R & D Division, RDA
6. 10.55 – 11.20 - Rock mass classification system for road tunneling by Eng.(Ms.) Dinuska Perera, Engineer, Highway Design Division.
7. 11.25-11.50 - Hydro - geological aspects of road tunnels by Eng (Ms.) B.G.N.Tharangani, Senior Bridge Design Engineer, RDA
8. 11.55-12.20 - Road Tunnels construction methods by Eng (Ms.) T.S.Silva, Deputy Project Director, Ruwanpura Expressway Project
9. 11.55-12.20 - Proposed Kandy Tunnel by Eng(Mr.) I.K. Baddegama, Engineer, MFAP Division
10. 12.55 – 13.20 - Questions and answers
11. 13.20 - Closer & Lunch

JICA Technical Assistance Program – Stage 2

Road Tunnel Final Seminar

25th January 2018

1. 9.00-9.30	Assembling of Participants
2. 9.30-9.35	Introductory Remarks by JICA, Colombo Office Representative
3. 9.35-10.00	Road Tunnel Design –Alignment Geometric (horizontal, vertical and CSS) by Eng(Ms) Shanika Wijerathne
4. 10.00	Tea/Refreshment (Served while presentation is on)
5. 10.00-10.25	Ground Water Environment around road tunnels by Eng(Ms)B.G.N.Tharangani
6. 10.30-10.55	Questions, Answers, Comments & Suggestions
7. 11.00	Announcements & Closer – Coordinator

MINISTRY OF HIGHWAYS PORTS & SHIPPING
ROAD DEVELOPMENT AUTHORITY

TECHNICAL ASSISTANCE FOR
IMPROVEMENT OF CAPACITY FOR
PLANNING OF ROAD TUNNEL

TECHNICAL NOTE

No. : TN-GS-001
Date : 2017.07.20

Geological and Geotechnical Investigation Program

By Dr. P. YANG, Geotechnical Expert

1. Introduction

This project, Technical Assistance for Improvement of Capacity for Planning of Road Tunnels, is carried out in the selected pilot tunnel site – Tunnel No. 2, to improve the road tunnel planning capacity of Road Development Authority (RDA) with the following outcomes:

1) Preparation of guidelines for rock mass classification system for road tunnel planning;
2) Preparation of manuals for geological and geotechnical investigation for rock mass classification system; and
3) Basic skills and information is shared with RDA for preparation of design document for road tunnel.

This technical note describes the purpose, scopes, types, quantities, methods and standards of the proposed geological and geotechnical investigations in the selected tunnel site. The investigation results are expected to provide reliable and sufficient geological, geotechnical and hydrological data and information for rock mass classification, and subsequent tunnel design and construction plan.

2. Purpose and Scope of this Investigation

2.1 Purpose

Every phase of a tunnel project, from its conception to construction, is generally influenced largely by the geology and geotechnical condition of the project area. Geology and geotechnical conditions of a tunnel project area significantly affect every major decision to be made in planning, designing and constructing such tunnel, determining its cost and even its operation performance and safety. An adequate geological and geotechnical investigation program and subsequent good understanding of the tunnel foundation geology, therefore, plays an important role in the success of the tunnel project.

The major purposes of this geological and geotechnical investigation are to explore and characterize the subsurface conditions along the selected tunnel alignment and then to develop geotechnical recommendations for the design of the tunnel support system including tunnel portal slope stabilization works. In more detail, the objectives of the investigation program are:

1) To explore and define the subsurface conditions including soil strata and rock weathering zone;
2) To observe groundwater levels and their configuration;
3) To identify and assess the stability of the portable slopes;
4) To geotechnically characterize or classify the ground or rock mass of the tunnel alignment;
5) To provide geotechnical design parameters for the design of the selected tunnel;
6) To recommend types of the tunnel support; and
7) To suggest further geotechnical investigation during tunnel construction, if necessary, in relation mainly to potential geological problems and geotechnical issues associated with tunnel construction.

The potential geological problems and geotechnical issues are those that can cause costly delays and disputes during tunnel construction, for examples, as listed below:

1) Thrust zones and shear zones. These zones are characterized by highly fractured, deformed and brecciated, water charged, and poor rock mass conditions,
2) Folded rock sequence. In Sri Lanka, a lot of rock sequences, especially metamorphic rocks have been folded and refolded, regionally and locally, due to polyphase tectonic movements.
3) In-situ stresses. In-situ stress generally varies from place to place. Accordingly, a major geotechnical uncertainty lies in forecasting the orientation and magnitude of the stresses in different sections of tunnel alignment.
4) Inflow of abundant water. Tunnelling through rock mass which is highly charged with ground water faces major problems, for example, a) heavy ingress of water in tunnel puts off the construction activities inside, and b) the high pore-water pressure behind the tunnel periphery adversely affects the support system resulting to distress.
5) Instability of tunnel portable slopes. Sri Lanka has experienced a large number of landslides and slope failures, especially in mountainous slopes. Landslides around the portal slopes, potential or old, might be reactivated by tunnel construction. Instability of a tunnel portal slope often causes a costly delay during construction, and even the safety of tunnel operation after tunnel completion.

2.2 Scope

The nature of a tunnel project and the complexity of the project site geology generally play a major role in determining the scope of the geotechnical investigation. A conventional or small project in uniform geology might require less investigation but a major or complex project in adverse geology might require much more investigation than the average.

It is generally difficult beforehand to set up a detailed plan for geological and geotechnical investigations and the investigation program should be adjusted and revised under progress per what is detected. It is more efficient to perform geological and geotechnical investigations in phases to focus the effort in the areas and depths that matter. Especially for a road tunnel through mountainous terrain, geological and geotechnical investigations are usually carried out at least in three to four phases to obtain the information necessary at each stage of the project in a more cost-efficient manner, as shown below:

1) Planning and feasibility study stage. At the planning and feasibility study stage, the investigation is usually limited and its purpose is mainly to identify and examine if the project is feasible technically and environmentally and can be completed within reasonable costs. The geological and geotechnical investigation includes existing data study and geological mapping, sometimes together with limited borings and geophysical survey.
2) Basic or preliminary design stage. Especially for a major tunnel project, basic design is often performed to further evaluate the project feasibility, to finally determine the tunnel location and alignment, and roughly calculate the project costs. At this stage, the geological and geotechnical investigation mainly includes detailed geological mappings, geophysical surveys, core borings, and in-situ and laboratory tests.
3) Detailed design stage. Following a detailed evaluation of the project area, the detailed design is carried out to prepare construction drawings and thereby accurately calculate project costs, and prepare the tender documents. The geological and geotechnical investigation at this stage focuses chiefly on individual zone, portal slope and some geological issues concerned, and generally includes geophysical survey, core boring, adit survey, in-situ rock stress measurement, etc.
4) Construction stage. In some major project with unfavourable geological conditions, the geological and geotechnical investigation at this stage might be required. This is geotechnical borings ahead of the tunnel face to explore rock overburden or weakness zones.

The geological and geotechnical investigation program for the design of the selected tunnel No. 2 project includes the following components (refer to Appendices A and B):

1) Existing data collection and desk study;
2) Site reconnaissance and geologic mapping;
3) Seismic refraction survey;

4) Core borings and sampling of soil and rock samples;
5) In-situ and laboratory tests; and
6) Observation and monitoring of groundwater levels at boreholes and existing wells.

The methods and standards of the above-mentioned geotechnical investigation program are given in the following sections.

3. Method and Standard of Geotechnical Investigation

3.1 Existing Data Collection and Desk Study

The first phase of an investigation program for a road tunnel project starts with collection and review of available information to develop an overall understanding of the site conditions and constraints at little cost. Existing data can help identify existing conditions and features that may impact the design and construction of the planned tunnel, and can assist in planning the scope and details of the geological and geotechnical investigation program.

Data to be collected mainly include published topographical, hydrological, geological, geotechnical, environmental maps and reports. Landslide maps, if available should be also collected to identify the potential for instability of the tunnel portal slopes. In Sri Lanka, the seismicity is active, historical seismic records should be collected and used to assess earthquake hazards especially for a major tunnel project.

In addition, case histories of underground works in the region are generally available from existing highway, railroad and water tunnels. Any geotechnical risk with the existing underground works will guide in planning geological geotechnical investigation program.

Around the selected tunnel project area, the following data have been collected and reviewed:

1) Geology of the Kandy-Hanguranketa, 1:100,000, Published by the Geological Survey and Mines Bureau of Sri Lanka (GSMB) (1996)
2) The National Atlas of Sri Lanka, Prepared by Survey Department, the Ministry of Lands and Land Development (2004)
3) Planning Drawings of the Tunnel No.2 Section

According to the above-collected geological maps and data, the geology of the project site is composed mainly of Quartzite and Quartz Schist and intercalated locally with Quartz-feldspar Gneiss. A thrust is inferred to pass through the planned tunnel alignment, which means that the underlying bedrocks along the tunnel line have a potential for fracturing, further indicating a possibility of overbreak or collapse during tunnel excavation.

These geological hazards shall be identified and confirmed following the geological and geotechnical investigation program.

3.2 Site Reconnaissance and Geological Mapping

Following a desk study, site reconnaissance shall be carried out over and around the tunnel alignment to check the features of the project site identified through desk study.

Especially, the following surface features should be observed at site reconnaissance and documented through geologic mapping:

1) Overburden soils (types, thickness, hardness, etc.)
2) Rock type and distribution (outcrops)
3) Rock weathering (weathering degrees, zones and their thickness)
4) Landslide and slope failure, new or old, particularly in the tunnel portal slopes

5) Faults and fold
6) Shear zones and joints (orientation, spacing, persistence, infilling, weathering, etc.)
7) Surface water and springs

For a major tunnel project with a complex geological and hydrological conditions, the site reconnaissance should cover the project vicinity, as well as a larger regional area so that regional geologic, hydrologic and seismic influences can be understood and examined.

3.3 Geophysical Survey – Seismic Refraction Survey

Geophysical surveys in underground engineering like tunnels include series of geophysical methods to determine geological-structural and physical-mechanical characteristics of the subsurface soils and rocks, for example, as listed below:

1) Gravity method
2) Magnetic method
3) Seismic refraction method
4) Seismic reflection method
5) Spectral analysis of surface waves
6) Borehole seismic method
7) Seismic tomography
8) Electrical resistivity method
9) Geo-radar
10) Transient electromagnetic method

Of the above-listed geophysical methods, the seismic refraction method has been proposed to be used for the selected tunnel site survey. The method is based on the analysis of artificially created seismic waves that are generated from the surface. Those waves travel to a particular depth and return to the surface after refraction at the boundaries of layers with different seismic velocities.

The survey is carried out to assist in characterizing the geotechnical conditions and to evaluate the dynamic properties of the subsurface soils and rocks along the tunnel alignment. Six seismic refraction lines shall be completed, three lines, each 300 m long are parallel to the tunnel alignment, and the others, each 100 m long, are perpendicular to the tunnel alignment (refer to Appendices A and B).

The survey results or records shall be plotted on time-distance graphs, and then interpreted into profiles of seismic wave velocity layers. The seismic wave velocity layers distinguished shall be geologically and geotechnically interpreted in correlation with the findings in the surface geological mappings, boreholes and test pits.

3.4 Core Boring and Sampling

Core borings shall be done to identify the subsurface stratigraphy, and to obtain disturbed and undisturbed soil samples and rock core samples for visual classification and laboratory tests.

1) Core borings, drilling: 2 boreholes×30m+1 borehole ×50m = 110 m
2) Undisturbed sampling (in soft soils): 3 boreholes×0 samples = 0 samples
3) Disturbed sampling (from SPT sampler): 3 boreholes×2 samples = 6 samples
4) Rock core samples: 10 samples in total

Soil samplings include disturbed and undisturbed ones. Disturbed soil samples should be collected at changes in strata and geological condition. Continuous sampling from one diameter above the tunnel crown to one diameter below the tunnel invert should be collected to better define the stratification and documented with detailed geological logging.

Below the surface of rock, continuous rock cores should be obtained with a minimum NX-size core (diameter of 54.7 mm). Double and triple tube core barrels should be used to obtain high-quality cores. The rock cores should be well logged soon after they are extracted from the core barrel. Generally, the following information should be recorded for each core run on the geological logs:

1) Depth of core run
2) Core recovery
3) Rock quality designation (RQD)
4) Rock type, including colour, texture, degree of weathering and hardness
5) Discontinuities, including joint spacing, orientation, roughness and alteration, joint infillings

Definitions and terminologies used in logging soil samples and rock cores are given in another technical note, No. TN-GS-002, Soil and Rock Logging Terminology.

Detailed quantity of boring and sampling together with in-situ tests is summarized in the following table.

Location	Borehole No.	Depth (m)	SPT (test)	WP (nos.)	DS (nos.)	RS (nos.)
Wester portal	BT-01	30	20	2	2	2
Top point	BT-02	50	20	6	2	6
Eastern portal	BT-03	30	20	2	2	2
Total	3 boreholes	110	60	10	6	10

Note: SPT=Standard penetration test, WP=Water pressure test, DS=Disturbed soil sample, RS=Rock core sample.

3.5 Test Pit and Sampling

Test pits are usually used to observe the shallow subsurface geological feature and continuous change, as well as top of bedrocks. The depth and size of test pits shall be determined by the depth and extent of the feature being exposed, generally 1 m log x 1 m wide x 1 m deep to 2 m long x 2 m wide x 3 m deep.

The conditions exposed in test pits, including the existing soil and rock materials, groundwater observations, and utility and other structure elements shall be documented by written records (or loggings) and photographs, and representative materials shall be sampled for future visual examination and laboratory tests.

Around the pilot project area, two test pits have been planned, and each one, 1 m log x 1 m wide x 2 m deep, shall be excavated on either portal area close to the borehole points, respectively, to observe the state and thickness of overburden soils, and the vertical change of rock weathering, and to collect undisturbed soil block samples (in soft soils) for laboratory tests.

3.6 In-situ Tests

In-situ tests are commonly used to obtain more correct engineering and index properties by testing the material in place to avoid the disturbance caused by sampling and handling of samples retrieved from boreholes.

The following in-situ tests shall be conducted in accordance with BS standards and methods:

1) Standard penetration tests (SPTs): BS1377-9
2) Water pressure test/lugeon test (WPs): BS5930-1990

The SPTs shall be done in overburden soils and underlying highly weathered rocks to roughly estimate their index and engineering properties through empirical correlations.

Water pressure test (packer or Lugeon test) shall be performed in bedrocks to measure the permeability of the tunnel surrounding rocks. The results of the test are usually expressed in terms of Lugeon units. A rock is said to have a permeability of 1 Lugeon if, under a head above groundwater level of 100 m, a 1 m length of borehole accepts 1 *l*/min of water. In general, 1 Lugeon unit is simply converted into a permeability of 10^{-7} m/s.

In addition, for a tunnel project one significant property of interest in rock is its in-situ stress condition. Horizontal stresses of geological origin are often locked within the rock masses, resulting in a stress ratio (K) often higher than the number predicted by elastic theory. Following the size and orientation of the tunnelling, high horizontal stresses may produce favourable compression in support and confinement, or induce popping or failure during and after excavation. Overcoring method has been widely used to measure the in-situ stress condition (orientation and magnitude). The method is to drill a small diameter borehole and then set into it an instrument to respond to changes in diameter. Rock stresses are determined indirectly from measurements of the dimensional changes of a borehole, occurring when the rock volume surrounding the hole is isolated from the stresses in the host rock.

However, because no local technology regarding the in-situ stress measurement is available, overcoring test has not been planned to be carried out in the pilot project.

3.7 Laboratory Tests

In the project, laboratory tests shall be conducted in accordance with BS standards and methods, as shown in the following table.

Sample Category	Test Item	Standard or Method	Tests (nos.)
Soils	1) Specific gravity	BS1377-2	6
	2) Unit weight	BS1377-2	6
	3) Grain size analysis	BS1377-2	8
	4) Atterberg Limit	BS1377-2	6
	5) Unconfined compressive strength	BS1377-7	2
	6) Direct shear strength	BS1377-7	2
Rock cores	1) Unit weight	BS812-2 [148]	3
	2) Water absorption	BS812-2 [150]	3
	3) Point load test	BS812-2 [153]	10
	4) Uniaxial compression strength	BS812-2 [153]	10
	5) Petrographic analysis	BS812-2 [149]	2

3.8 Hydrological Survey

Groundwater is always a critical issue for any tunnel project, because it may not only represent a large percentage of the loading on the final tunnel lining, but also it largely determines the ground behaviour and stability for soft ground tunnels; the inflow into rock tunnels; the method and equipment selected for tunnel construction; and the long-term performance of the completed underground structures.

Accordingly, for tunnel projects, special attention should be given to defining as follows:

1) Groundwater regime;
2) Aquifers and sources of water;
3) any perched conditions;
4) Groundwater temperature and smell;
5) Depth to groundwater and its configuration; and
6) Permeability of the various materials that may be encountered during tunnelling.

In the tunnel project, in addition to the above-mentioned Lugeon test, three boreholes and existing wells shall be monitored periodically over a rainy period of time to provide information on seasonal variations in groundwater levels.

In addition, monitoring of groundwater levels during construction should be also performed to evaluate the influence of tunnelling on groundwater levels.

4. Survey Period

The geological and geotechnical program was commenced in early July and shall be completed in early September with a work period of about 60 days, as shown roughly in the following table.

Description of works		Jul-17			Aug-17			Sept. 2017			Oct. 2017		
1	Preparation, Mobilization, etc.		■										
2	Topographical Survey			■	■	■							
3	Geotechnical Boring and In-situ Tests			■	■	■							
4	Seismic Refraction Survey				■								
5	Laboratory Tests				■	■	■						
6	Reporting						■	■					

5. Expected Investigation Results

The geological and geotechnical program shall be completed to obtain the following results:

1) Local geological map (geological plan)
2) Longitudinal geological profile
3) Longitudinal seismic refraction profile
4) Longitudinal detailed geotechnical profile with rock mass classification and recommended support system
5) Geological and geotechnical data report, which will be prepared by the Contractor – Engineering & Laboratory Services (Pvt) Ltd.
6) Geotechnical study (or interpretation) report, which will be prepared by JICA Study Team.

Appendix A Summary of Topographic, Geological and Geotechncial Surveys

No.	WORK ITEM AND SPECIFICATION	UNIT	QUANTITY
1	**Topographical Survey**		
1.1	Plan mapping (1:1,000, 2m interval contour)	m^2	30000
1.2	Cross section (Leveling 10m intervals spot height)	m	1,300
1.3	Setting up of temporary bench mark	site	2
1.4	Locating borehole points	hole	3
1.5	Identification of existing well position	point	10
2	**Geotechnical Drilling, In-situ Tests and Samplings**		
2.1	Core Borings (50m*1 hole + 30m*2 holes =110m)		
	1) Drilling (Overdurden soil)	m	15
	2) Drilling (Highly weathered/Soft rocks)	m	45
	3) Drilling (Hard rock)	m	50
2.2	In-situ tests		
	1) SPT @ every 1.0m interval	nos.	60
	2) Water pressure test (Parker or Lugeon test) in rocks	nos.	10
2.3	Samplings		
	1) Disturbed soils (from SPT sampler)	nos.	6
	2) Sampling of rock cores	nos.	10
3	**Test Pits and Samplings**		
3.1	Test pit excavatation (1m*1m*2m)	nos.	2
3.2	Sampling of undisturbed box soil samples from test pit bottom	nos.	2
4	**Laboratory Tests**		
4.1	Laboratory tests of soil samples		
	1) Particle size analysis by Sieve	nos.	8
	2) Atterberg limits test	nos.	6
	3) Specific gravity	nos.	6
	4) Unit weight	nos.	6
	5) Unconfined compression	nos.	2
	6) Direct shear test	nos.	2
4.2	Laboratory tests of rock core samples		
	1) Unit weight	nos.	3
	2) Water absorption	nos.	3
	3) Point load test	nos.	10
	4) Uniaxal compression	nos.	10
	5) Petrographic analysis (thin section observation)	nos.	2
5	**Groundwater Level Observation**		
5.1	Installing of piezometers and PVC pipes (3 bores)	m	110
5.2	Monitoring of groundwater level at boreholes (3bores*2 times)	nos.	6
4.3	Monitoring of groundwater level at existing wells	nos.	10
6	**Seismic Refraction Survey**		
6.1	Line clearing & section survey	m	1,200
6.2	Seismic refraction exploration	m	1,200

TECHNICAL NOTE

No. : TN-GS-002
Date : 2017.07.24

Soil and Rock Logging Terminology

By Dr. P. YANG, Geotechnical Expert

1.0 INTRODUCTION

1.01 Objectives

This objective of this Technical Note is to establish a consistent and standard method for RDA geologists and field staff to follow when completing the detailed and consistent description of soil and rock samples obtained from field sampling efforts and entry into geological logs.

Consistent and standardized description and presentation are important for understanding subsurface conditions and subsequent project design and construction activities mainly because:

(1) Soil and rock description is to a certain degree subjective and a standard terminology with a defined criterion should thus be used to reduce the subjective nature and variability of descriptions of soils and rocks encountered during the investigation, design and construction of an engineering project,
(2) Detailed and systemic soil and rock descriptions and classifications are an essential part of the subsurface information developed to support the design and construction processes of any project, and
(3) Many experts and engineers for different areas may be involved during an engineering project, such as geologist, geotechnical engineer, designer, construction material engineer, construction supervisor, etc., consistency with description is essential so that all experts and engineers of the project can properly and consistently understand and interpret the subsurface conditions for engineering projects.

1.02 Scope and Applicability

Besides boreholes and test pits in this project, this note is applicable to excavated trench, aidts, or other subsurface exposures such as construction excavation area, road cuts, or collapsed slopes for other road development and construction projects.

1.03 Reference Standards and Terminology

The format, methods and terminology used for all soil and rock logging are based on a combination of published international standards and references, as listed below, with relatively minor project-specific modifications and additions.

Reference No.	References
1	American Society for Testing and Materials (ASTM) D 420-98, Site Characterization for Engineering Design and Construction Purposes
2	American Society for Testing and Materials (ASTM) D 5434-97, Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock
3	American Society for Testing and Materials (ASTM) D 2488, Standard Practice for Description and Identification of Soils (Visual Manual Procedure)
4	British Standards Institute (BSI) BS 5930-1990, Code of Practice for Site Investigations
5	Suggested methods for the quantitative description of discontinuities in rock masses Published by ISRM (1981)

The terminology used in all logging preparations is defined and explained below. Some of the terms are only applicable to certain types of logs as indicated in Table 1.1 below:

Table 1.1 Description Sequence

Reference in Section	Item	Required	Optional
2.0	**General Information**		
2.01	Project and Site Information	✓	
2.02	Date of Work	✓	
2.03	Personnel	✓	
2.04	Location and Elevation of Borehole and Test Pit	✓	
2.05	Drilling and Sampling Equipment and Method	✓	
2.06	Groundwater	✓	
3.0	**Soil Description**		
3.01	Group name and group symbol	✓	
3.02	Percent or proportion of soils	✓	
3.03	Strength (fine-grained)	✓	
3.04	Density or Hardness (coarse-grained)	✓	
3.05	Colour	✓	
3.06	Moisture	✓	
3.07	Particle shape (coarse-grained)	✓	
3.08	Particle angularity (coarse-grained)	✓	
3.09	Grain size	✓	
3.10	Consistency (fine-grained)	✓	
3.11	Cementation (fine-grained)	✓	
3.12	Structure of (intact soils)		✓
3.13	Formation name		✓
3.14	Additional descriptions and other information		✓
4.0	**Rock Description**		
4.01	Rock name and type	✓	
4.02	Rock grain size	✓	
4.03	Colour	✓	
4.04	Texture	✓	
4.05	Weathering degree	✓	
4.06	Intact rock strength	✓	
4.07	Discontinuity type	✓	
4.08	Discontinuity direction (dip direction and dip angle)	✓	
4.09	Discontinuity condition (weathering, infilling, etc.)	✓	
4.10	Mechanical logs		✓
4.11	Additional description		✓

1.04 General Logging Process

The geological logs shall generally be prepared in three steps, as follows:

(1) Draft logging – Once the completion of a borehole, field sampling and descriptions of soil and rock shall be carried out based on visual observation. All significant observable properties of soils and rocks, as listed in Table 1.1 above, shall be described at natural conditions.

(2) Refinement of descriptions and classification of soils and rocks based on laboratory test results, if conducted. If the results of laboratory tests change the description and classification of the sample obtained by field observation, the classification and/or description resulting from the laboratory tests shall be used on the finalized logs. Disclosure of the tests on the logging makes it clear whether the samples are based on visual observation or on laboratory test results.

(3) Finalization of the borehole logs. The borehole logs shall be finalized base on the above-mentioned test results, and in some cases, comparison with nearby borehole logs.

A boring sample record is given in Appendix-A.

2.0 GENERAL INFORMATION

2.01 Project and Site Information

The following information shall be shown in Logs:

- Project Name
- RDA Local Office in Charge
- District and County
- Others

2.02 Date of Work

- Date of commence and completion of borehole drilling and test pit excavation
- Cause of termination (e.g., drilled to depth, refusal, early termination of local objection, etc.)
- Abandonment (e.g., encountered with underground utilities, drilling rig breakdown, etc.)

2.03 Personnel

- Logger/Geologist/Geotechnical Engineer
- Drillers

2.04 Location and Elevation of Borehole

- Location - Latitude and longitude shall be confirmed at site by GPS.
- Elevation is measured in metres from topographical survey in finalized logs.
- Depth of borehole is measured in metres relative to ground level.

2.05 Drilling and Sampling Equipment and Methods

- Drill rig (manufacturer, model, etc.)
- Drilling method (mud rotary, air rotary, solid auger, hollow stem auger. etc.) (refer to Table 2.1)
- Drill rod description (type, diameter)
- Drill bit description
- Casing (type, diameter) and installation depth
- SPT Hammer Type (Manufacturer & model, Safety/Automatic Hammer, Measured SPT energy efficiency ratio (if available)
- Type of sampler(s) and size (Undisturbed Shelby tube, Undisturbed Piston, Split spoon, Core (both rock and soil) (refer to Table 2.2)
- Other

Table 2.1 Drilling Methods and Drill Rig

Abbreviations	Definitions
A	Auger boring (hollow or solid stem, bucket)
R	Rotary drilled boring (both conventional and wire-line)

P	Rotary percussion boring (Air)
HD	Hand driven (1-inch soil tube)
P	Polymer mud flush
HA	Hand auger
BD	Borehole diameter (given in millimeters)
CD	Core Diameter (given in millimeters)
D	Double tube core barrel
SPT	Standard penetration test
O	Other

Table 2.2 Sample Type Abbreviations

Abbreviations	**Definitions**
BD	Bulk disturbed (large bag) sample from test pits or other purpose
BU	Bulk or box undisturbed sample for test pits
DS	Small disturbed soil sample from SPT sampler
US	Shelby undisturbed tube sample
RS	Rock core sample or block sample

Recoveries for the Piston and Tube samples are expressed as a percentage of the total drive length of the sampler.

2.06 Groundwater Level

- Method (observed during and after drilling, measured in borehole t, etc.)
- Date, time, and depth of each reading
- Depth of groundwater level readings is measured in metres relative to ground level.

3.0 SOIL DESCRIPTION

Soils are described on the basis of their visually assessed engineering behaviour, not strictly on the basis of the proportions of each soil type present. The following description sequence shall be used for describing each main soil stratum:

Table 3.1 Soil Description Sequence

Fine-Grained Soils (Clay or Silt)	**Coarse-Grained Soil (Sand or Gravel)**
Group name/symbol	Group name/symbol
Percent or proportion of soils	Percent or proportion of coarse-grained soils,
Strength	Density
Colour	Colour
Moisture	Moisture
Consistency	Particle angularity
Cementation	Particle shape
Formation name	Formation name
Other information	Other information

When describing and identifying soil, record the data in accordance with Tables 1.1 and 3.1 above, and present the information on the logs in the sequence shown. Items marked “required” must be used, when applicable, to describe the soil sample.

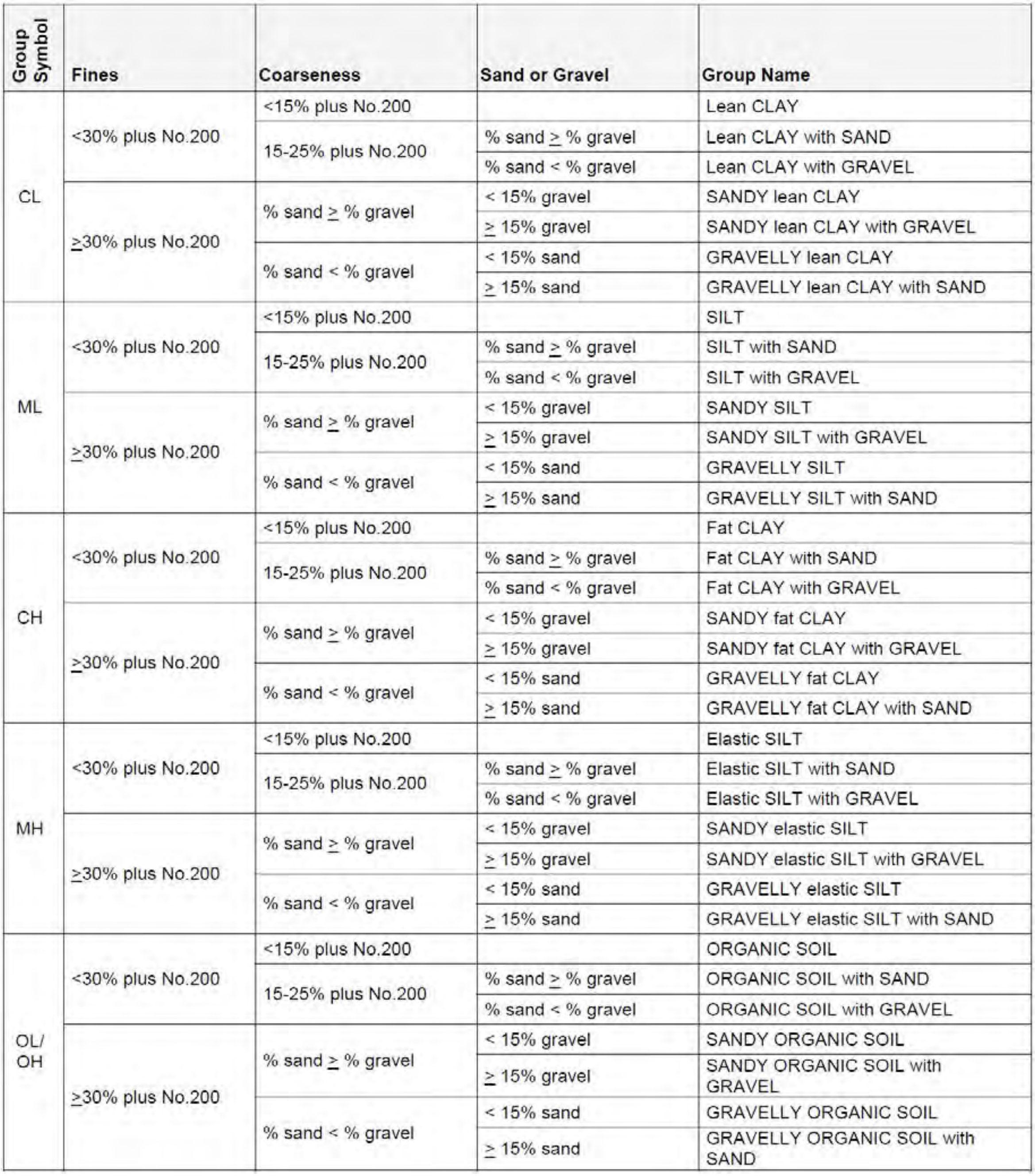
The following examples illustrate the application of the descriptive sequence based on field procedures: *Well-graded SAND with GRAVEL (SW), medium dense, brown and light gray, wet, mostly SAND, from coarse to fine, little coarse GRAVEL, trace fines, weak cementation.*

3.01 Group Name and Symbol

British Soil Classification System (BSCS) has been partially used In Sri Lanka. However, USCS (unified Soil Classification System) soil classification – ASTM D2487 has been most frequently and widely applied in most countries and is suggested to be used accordingly. The USCS soil classification may facilitate communication and understanding between engineers and geologists, especially for some international projects.

Identify a soil by assigning a group name and group symbol according to Table 3.2 for fine-grained soil and Table 3.3 for coarse-grained soil. The ASTM D2488 procedure for identifying and describing fine-grained and coarse-grained soil is only applicable to material passing the 3-inch sieve.

Table 3.2 Identification of Fine-Grained Soil (after ASTM D2488)



Group Symbol	Fines	Coarseness	Sand or Gravel	Group Name
CL	<30% plus No.200	<15% plus No.200		Lean CLAY
		15-25% plus No.200	% sand ≥ % gravel	Lean CLAY with SAND
			% sand < % gravel	Lean CLAY with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY lean CLAY
			≥ 15% gravel	SANDY lean CLAY with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY lean CLAY
			≥ 15% sand	GRAVELLY lean CLAY with SAND
ML	<30% plus No.200	<15% plus No.200		SILT
		15-25% plus No.200	% sand ≥ % gravel	SILT with SAND
			% sand < % gravel	SILT with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY SILT
			≥ 15% gravel	SANDY SILT with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY SILT
			≥ 15% sand	GRAVELLY SILT with SAND
CH	<30% plus No.200	<15% plus No.200		Fat CLAY
		15-25% plus No.200	% sand ≥ % gravel	Fat CLAY with SAND
			% sand < % gravel	Fat CLAY with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY fat CLAY
			≥ 15% gravel	SANDY fat CLAY with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY fat CLAY
			≥ 15% sand	GRAVELLY fat CLAY with SAND
MH	<30% plus No.200	<15% plus No.200		Elastic SILT
		15-25% plus No.200	% sand ≥ % gravel	Elastic SILT with SAND
			% sand < % gravel	Elastic SILT with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY elastic SILT
			≥ 15% gravel	SANDY elastic SILT with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY elastic SILT
			≥ 15% sand	GRAVELLY elastic SILT with SAND
OL/ OH	<30% plus No.200	<15% plus No.200		ORGANIC SOIL
		15-25% plus No.200	% sand ≥ % gravel	ORGANIC SOIL with SAND
			% sand < % gravel	ORGANIC SOIL with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY ORGANIC SOIL
			≥ 15% gravel	SANDY ORGANIC SOIL with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY ORGANIC SOIL
			≥ 15% sand	GRAVELLY ORGANIC SOIL with SAND

Fines are particles that pass through a Number 200 sieve (0.075mm). A soil is considered to be fine-grained if it contains 50% or more fines. On the other hand, a coarse-grained soil contains fewer than 50% fines. A coarse-grained soil is identified as gravel if the percentage of gravel is greater than the percentage of sand, or as sand if the percentage of gravel is equal to or less than the percentage of sand.

Table 3.3 Identification of Coarse-Grained Soil (ASTM D2488)

	Fines	Grade	Type of Fines	Group Symbol	Sand/Gravel	Group Name
Gravel	≤ 5%	Well		GW	< 15% sand	Well-graded GRAVEL
					≥ 15% sand	Well-graded GRAVEL with SAND
		Poorly		GP	< 15% sand	Poorly graded GRAVEL
					≥ 15% sand	Poorly graded GRAVEL with SAND
	10%	Well	ML or MH	GW-GM	< 15% sand	Well-graded GRAVEL with SILT
					≥ 15% sand	Well-graded GRAVEL with SILT and SAND
			CL or CH	GW-GC	< 15% sand	Well-graded GRAVEL with CLAY
					≥ 15% sand	Well-graded GRAVEL with CLAY and SAND
		Poorly	ML or MH	GP-GM	< 15% sand	Poorly graded GRAVEL with SILT
					≥ 15% sand	Poorly graded GRAVEL with SILT and SAND
			CL or CH	GP-GC	< 15% sand	Poorly graded GRAVEL with CLAY
					≥ 15% sand	Poorly graded GRAVEL with CLAY and SAND
	≥ 15%		ML or MH	GM	< 15% sand	SILTY GRAVEL
					≥ 15% sand	SILTY GRAVEL with SAND
			CL or CH	GC	< 15% sand	CLAYEY GRAVEL
					≥ 15% sand	CLAYEY GRAVEL with SAND
Sand	≤ 5%	Well		SW	< 15% gravel	Well-graded SAND
					≥ 15% gravel	Well-graded SAND with GRAVEL
		Poorly		SP	< 15% gravel	Poorly graded SAND
					≥ 15% gravel	Poorly graded SAND with GRAVEL
	10%	Well	ML or MH	SW-SM	< 15% gravel	Well-graded SAND with SILT
					≥ 15% gravel	Well-graded SAND with SILT and GRAVEL
			CL or CH	SW-SC	< 15% gravel	Well-graded SAND with CLAY
					≥ 15% gravel	Well-graded SAND with CLAY and GRAVEL
		Poorly	ML or MH	SP-SM	< 15% gravel	Poorly graded SAND with SILT
					≥ 15% gravel	Poorly graded SAND with SILT and GRAVEL
			CL or CH	SP-SC	< 15% gravel	Poorly graded SAND with CLAY
					≥ 15% gravel	Poorly graded SAND with CLAY and GRAVEL
	≥ 15%		ML or MH	SM	< 15% gravel	SILTY SAND
					≥ 15% gravel	SILTY SAND with GRAVEL
			CL or CH	SC	< 15% gravel	CLAYEY SAND
					≥ 15% gravel	CLAYEY SAND with GRAVEL

In addition, as shown in Tables 3.2 and 3.3, the identification and classification system consists of two characters to indicate a soil type, as follows:

- Character I: G – Gravel, S – Sand, M – Silt, C – Clay, O – Organic, Pt – Peat, and
- Character II: W – Well graded, P – Poorly Graded, M – Silty, C – Clayey, L – Low plasticity, H – High plasticity

3.02 Percent or Proportion of Soils

Report the percentage or proportion of gravel, sand, and fines, by weight of the total sample (excluding the cobbles/boulders), either by using a proportional descriptor (Table 3.4) or as a weight percentage (not a range), estimated to the nearest 5 %, of the total sample (excluding the cobbles and boulders). Report the percents or proportions in order of decreasing abundance. Percentages must add up to 100%.

Table 3.4 Percent or Proportion of Soil (after ASTM D 2488)

Term	Approximate percentage by mass (%)
Trace	<5
Few	5 to 10
Little	15 to 25
Some	30 to 45
Mostly	Over 50

For sands or gravels the secondary constituents have been described in two manners in accordance with Table 3.5.

Table 3.5 Secondary Constituents

Term before principal soil type	Term after principal soil type	Approximate percentage by mass
slightly (sandy)	with a little (sand)	<5
(sandy)	with some (sand)	5 to 30
very (sandy)	with much (sand)	30 to 50
	and sand	>50

3.03 Strength

The strength has been assessed on undisturbed samples or in situ and described in accordance with the following table.

Table 3.6 Fine-grained Soil Strength (after ISRM, 1981, AASHTO, 1988)

Term	Field Identification	UCS (KPa)	SPT N-Value (blows/300mm)
Very soft	Easily penetrated several inches by fist	<25	0-1
Soft	Easily penetrated several inches by thumb	25-50	2-4
Firm	Can be penetrated several inches by thumb with moderate effort	50-100	5-8
Stiff	Readily indented by thumb but penetrated only with great effort	100-250	9-15
Very stiff	Readily indented by thumbnail	250-500	16-30
Hard	Indented with difficulty by thumbnail	>500	31 or more

3.04 Density or Relative Density

An assessment of the relative density (or hardness) of coarse-grained soils (sands and gravels) shall be from the SPT N value using the following table.

Table 3.7 Relative Density (after AASHTO, 1988)

Term	SPT N_{60}-Value (blows/300mm)
Very loose	< 5
Loose	5 - 10
Medium dense	10 - 30

Dense	30 - 50
Very dense	>50

3.05 Colour

Table 3.8 Colour

Abbreviation	Definition
B	Brown, Brownish
C	Cream
D	Dark
G	Grey, Greyish
L	Light
O	Orange
P	Pink
R	Red, Reddish
Y	Yellow, Yellowish
GG	Greenish grey

3.06 Moisture

Table 3.9 Moisture (after ASTM D2488)

Term	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

3.07 Grain shape

Table 3.10 Grain Shape (after ASTM D2488)

Term	Criteria
Flat	Particles with width/thickness > 3
Elongated	Particles with length/width > 3
Flat and elongated	Particles meet criteria for both flat and elongated

3.08 Grain angularity

The grain angularity shall be described in terms of angularity, including angular, sub-angular, sub-rounded and rounded, as shown in the following table.

Table 3.11 Describing Angularity of Coarse-Grained Particles (after ASTM D2488)

Term	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

3.09 Grain Size

The grain size shall be described in accordance with the following table.

Table 3.12 Grain Size (after ASTM D2488)

Term	Sieve opening	Grain Size (mm)
Boulder	Greater than 12 inches	>300
Cobble	3 to 12 inches	75.0 to 300
Coarse Gravel	3/4 to 3 inches	19.0 to 75.0
Fine Gravel	No.4 to 3/4 inches	4.75 to 19.0
Coarse Sand	No.10 to No.4	2.00 to 4.75
Medium Sand	No.40 to No.10	0.425 to 2.00
Fine Sand	No.200 to No.40	0.075 to 0.425
Silt and Clay	Passing No.200	<0.075

In addition, soil grain size terminology and definition in BSCS and USCS are given in Table 2.13 below for comparative purpose.

Table 3.13 Grain Size (after BS5930-1990 and ASTM D2488)

Term	USCS Grain Size (mm)	BSCS Grain Size (mm)
Boulder	>300	>200
Cobble	75.0 to 300	60.0 to 200
Coarse Gravel	19.0 to 75.0	20.0 to 60.0
Medium Gravel	-	6.0 to 20.0
Fine Gravel	4.75 to 19.0	2.0 to 6.0
Coarse Sand	2.00 to 4.75	0.6 to 2.0
Medium Sand	0.425 to 2.00	0.2 to 0.6
Fine Sand	0.075 to 0.425	0.06 to 0.2
Silt	0.005 to 0.075	0.002 to 0.06
Clay	<0.005	<0.002

3.10 Consistency (fine-grained)

Refer to Section 3.03 above.

Table 3.14 Describing Consistency of Fine-Grained Soils (after ASTM D2488)

Term	Criteria
Very soft	Thumb will penetrate soil more than 1 in. (25 mm)
Soft	Thumb will penetrate soil about 1 in. (25 mm)
Firm	Thumb will indent soil about 1⁄4in. (6 mm)
Hard	Thumb will not indent soil but readily indented with thumbnail
Very hard	Thumbnail will not indent soil

Table 3.15 Describing Consistency of Fine-Grained Soils (after AASHTO, 1988)

Term	SPT N-Value (blows/300mm)
Very soft	Less than 2
Soft	2 – 4
Firm	4 – 8
Stiff	8 – 15
Very Stiff	15 – 30
Hard	Over 30

3.11 Cementation (fine-grained)

Table 3.16 Describing Cementation of Fine-Grained Soils (after ASTM D2488)

Term	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

3.12 Structure (of Intact Soils)

Table 3.17 Describing Structure of Intact Soils (after ASTM D2488)

Term	Criteria
Stratified	Alternating layers of varying material or colour with layers at least ¼ in. thick; note thickness.
Laminated	Alternating layers of varying material or colour with the layers less than ¼ in. thick; note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.
Homogeneous	Same colour and appearance throughout.

3.13 Formation name

The geological formation names shall be determined in accordance with The National Atlas of Sri Lanka (Second Edition) prepared by the Survey Department, the Ministry of Lands and Land Development (2004), and/or Geological Map of Sri Lanka published by the Geological Survey and Mines Bureau of Sri Lanka (1996), or other published geological maps, if available.

3.14 Additional description

Any geological and geotechnical information, such as presence of roots or root holes, hardness and rock type of cobble and gravel, surface coatings on coarse-grained particles, caving or failure of borehole or test pit excavation, difficulty in drilling or test pit excavating, etc.

4.0 ROCK DESCRIPTION

The following description sequence shall be followed for each rock type description:

- Rock name and type
- Rock grain size
- Colour
- Texture
- Weathering degree
- Intact rock strength
- Discontinuity type
- Discontinuity direction (dip direction and dip amount)
- Discontinuity condition (infilling, etc.)
- Mechanical logs
- Additional description

4.01 Rock Name and Type

The rock name or type shall be determined in accordance with published maps (Refer to Section 3.13 above).

4.02 Rock Grain Size

Rock grain size shall be described as reference in Section 3.09 above.

4.03 Colour

Colour shall be described as reference in Section 3.05 above.

4.04 Texture

The texture and fabric shall be described in accordance with the following table.

Table 4.1 Fabric Spacing and Thickness Terms (ISRM, 1981)

Mean spacing (mm)	Spacing term	Planar fabric thickness term$^{1)}$
>6000	Extremely widely spaced	Extremely thickly bedded/banded/foliated
2000-6000	Very widely spaced	Very thickly bedded/banded/foliated
600 – 2000	Widely spaced	Thickly bedded/banded/foliated
200 – 600	Moderately widely spaced	Medium bedded/banded/foliated
60 - 200	Closely spaced	Thinly bedded/banded/foliated
20 - 60	Very closely spaced	Very thinly bedded/banded/foliated
<20	Extremely closely spaced	Thinly laminated (sedimentary and metamorphic rocks) Very narrowly banded/foliated (metamorphic/igneous rocks)

Note: Generally, the term Bedding has been applied to sedimentary rocks, and Foliation for metamorphic rocks

4.05 Weathering

The weathering has been described in accordance with the following table.

Table 4.2 Rock Mass Weathering (ISRM, 1981)

Term	Definition
Residual soil	All rock material is converted to soil. The mass structure and material fabric are completely destroyed. The soils have not been significantly transported.
Completely weathered	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.
Highly weathered	More than half of the rock material decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
Slightly weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker externally than in its fresh condition.
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surface.

4.06 Intact Rock Strength

The rock strength shall be estimated in the field or by the examination of fresh specimens, and described in accordance with the following table (Strength terms and ranges of unconfined compressive strength (UCS) as reference)

Table 4.3 Intact Rock Strength (ISRM, 1981)

Grade	Term	Field Identification	Approximate range of UCS (MPa)
R0	Extremely weak	Indented by thumbnail. Gravel size lumps can be broken between finger and thumb	<1
R1	Very weak	Crumbles under firm blows from point of geological hammer. Can be peeled by pocket knife. Gravel sized lumps can be broken in half by heavy hand pressure	1- 5
R2	Weak	Can be peeled by a pocket knife with difficulty. Shallow indentations made by firm blows with point of geological hammer.	5 -25
R3	Medium strong	Cannot be scrapped or peeled by a pocket knife. Rock can be broken by hammer blows when held in the hand	25 - 50
R4	Strong	Rock can be broken by hammer blows when resting on a solid surface	50 - 100
R5	Very strong	Rock chipped by heavy hammer blows	100 - 250
R6	Extremely strong	Rock rings on hammer blows. Only broken by sledge hammer	>250

4.07 Discontinuity Type

On detailed rock core logs, each set of discontinuities is described by using the abbreviations and terms given below. The descriptions apply over lengths of core which have a similar nature of discontinuities. A minimum thickness of 1.0m has been chosen for distinguishing between zones of different characteristics. Where important structural features, such as non-intact zones and shear zones, are identified which have a thickness below 1.0m the information is given in the detail in the geological logs.

The following abbreviations shall be used to describe the type of the discontinuity.

Table 4.4 Discontinuity Type

Abbreviation	Term
BE	Bedding discontinuity
CS	Cleavage/Schistocity discontinuity
FL	Foliation/Lamination discontinuity
JO	Joint
FA	Fault
FI	Fissure
VE	Vein
SZ	Shear/Shear Zone
BA	Banding discontinuity

To differentiate between discontinuity sets of the same type a number shall also be given, for example, J1, J2, and J3.

In addition, the following abbreviations shall be used to define the nature of the discontinuity and distinguish between drilling induced and natural discontinuities:

Table 4.5 Discontinuity Form

Abbreviation	Term
N	Natural discontinuity, continuous across core
D	Natural discontinuity, discontinuous across core
H	Healed, cemented discontinuity (intact core)
I	Drilling induced discontinuity along pre-existing weakness plane eg. foliation or lamination
B	Drilling induced discontinuity or artificial break unrelated to any plane of weakness

4.08 Discontinuity Direction

In rock outcrops, exposures and scanlines dip is measured as the angle in degrees between the discontinuity plane and the horizontal, in the range 0 - 90 deg. In core measured as the angle in degrees between the discontinuity plane and the perpendicular to the core axis, in the range 0 - 90 deg. The dip direction is measured in degrees and recorded as three digits relative to true north in the range 000 to 360 degrees. Where dip amount and dip direction are recorded together the format dip amount/dip direction shall be used, for example, 35/090.

4.09 Characteristics and Conditions of Discontinuity

4.09.01 Spacing

The following abbreviations have been used. The definition of the terms is given in Table 4.6.

Table 4.6 Discontinuity Spacing (ISRM, 1981)

Abbreviation	Term	Spacing (m)
EC	Extremely closely spaced	<0.02
VC	Very closely spaced	0.02 – 0.06
C	Closely spaced	0.06 – 0.2
MW	Moderately widely spaced	0.2 – 0.6
W	Widely spaced	0.6 – 2.0
VW	Very widely spaced	2.0 – 6.0
EW	Extremely widely spaced	< 6.0

In places, minimum, average and maximum spacing is given using the format minimum/average/maximum, eg. (20/50/100). The spacing is given in millimetres.

4.09.02 Persistence

The discontinuity trace length in the plane of the rock exposure is measured in metres (m) to one decimal place. The definition of the terms is given in Table 4.7.

Table 4.7 Discontinuity Persistence

Term	Surface Trace Length (m)
Very low persistence	< 1
Low persistence	1 – 3
Medium persistence	3 – 10
High persistence	10 – 20
Very high persistence	>20

4.09.03 Aperture

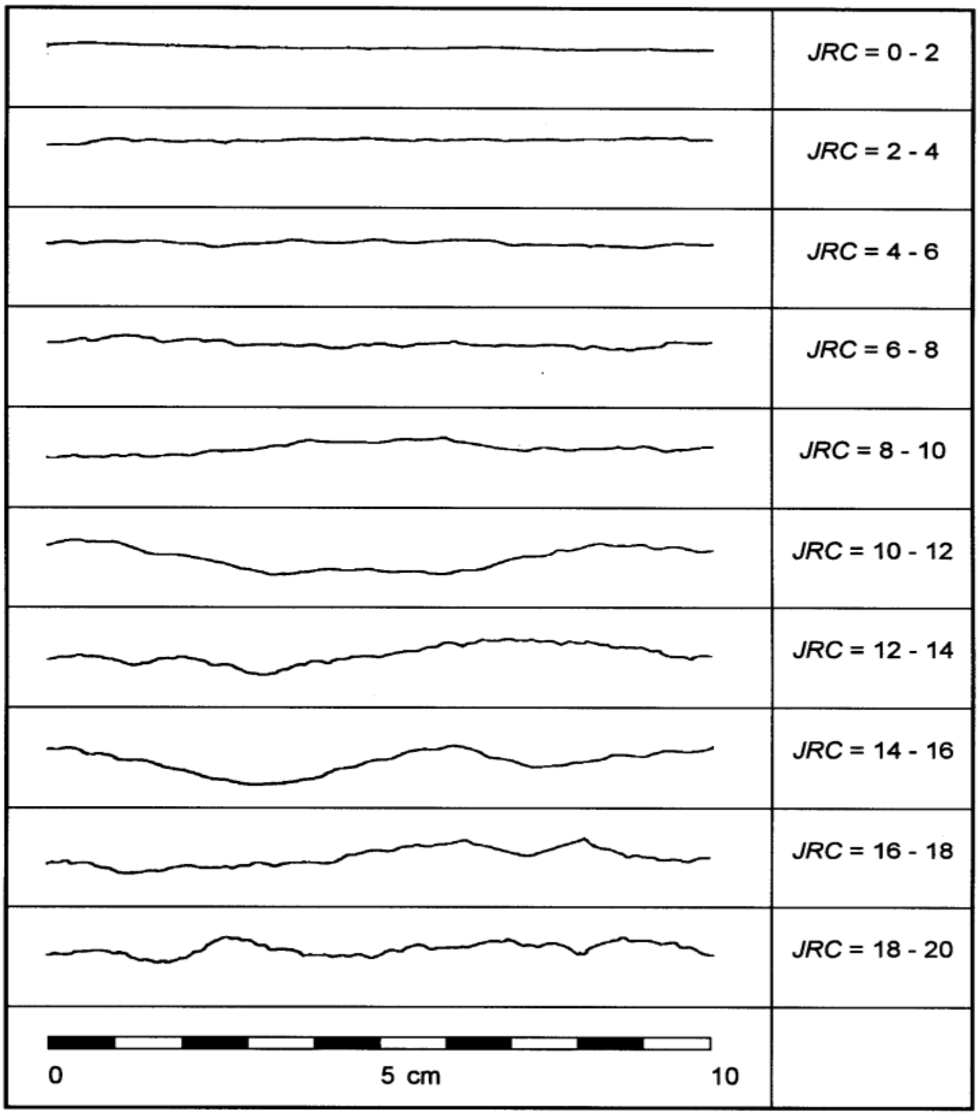
Discontinuity opening, recorded in millimetres. The definition of the terms is given in Table 4.8.

Table 4.8 Discontinuity Aperture (ISRM, 1981)

Aperture (mm)	Term	Feature
<0.01	Very tight	Closed feature
0.01 – 0.25	Tight	
0.25 – 050	Partly open	
0.50 – 2.5	Open	Capped feature
2.5 – 10.0	Widely open	
10 – 100	Very widely open	Open feature
100 – 1000	Extremely widely open	
< 1000	Cavernous	

4.09.04 Joint Roughness Coefficient (JRC)

The JRC value has been assessed in accordance with Figure 4.1, and at the scale shown in Figure 4.1.



Note: this must be printed so that the scale bar is 10cm long

Figure 4.1 Typical Roughness Profiles for JRC Range

4.09.05 Planarity

The planarity shall be described in accordance with Figure 4.2. The following abbreviations have been used.

Table 4.9 Planarity (after ISRM, 1981)

Abbreviation	Term
P	Planar
U	Undulating or curved
S	Stepped

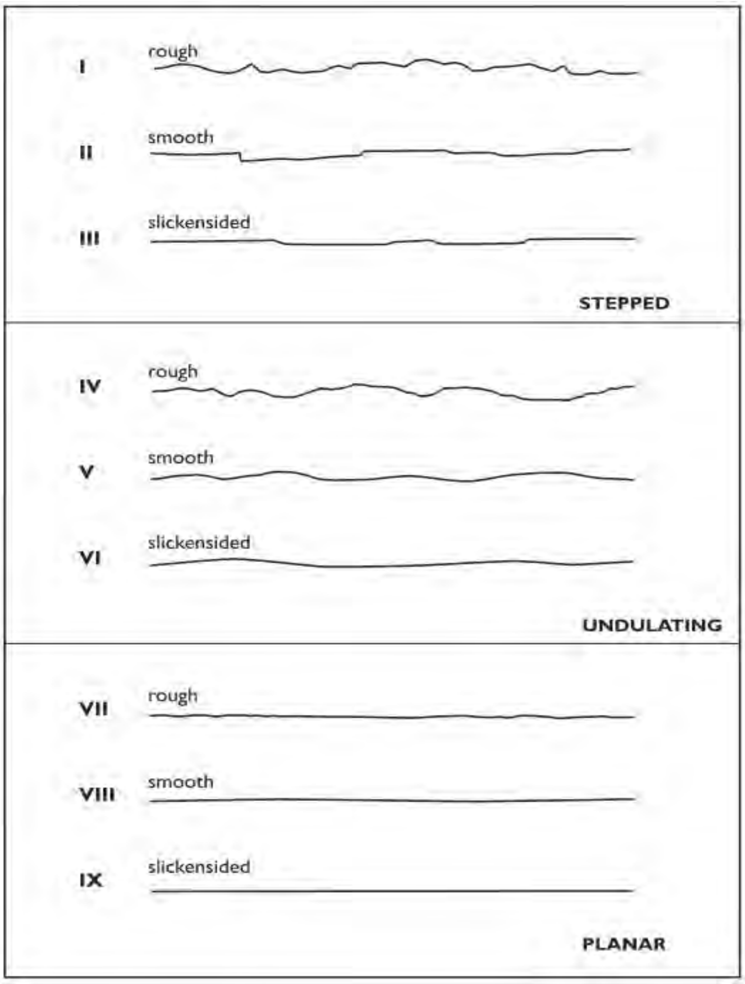


Figure 4.2: Roughness and Planarity Identification Profiles

4.09.06 Roughness

The roughness has been described in accordance with Figure 4.2 above. The following abbreviations have been used:

Table 4.10 Roughness

Abbreviation	Term
K	Slickensided (ie., polished and striated)
S	Smooth
R	Rough

4.09.07 Amplitude

The large scale (>0.1m) height of waviness, is measured in metres peak to trough or trough to peak.

4.09.08 Infill Thickness

Thickness of infill between the walls of a discontinuity, recorded in millimetres.

4.09.09 Infill Material

The following abbreviations have been used for the type of infill material:

Table 4.11 Infill Material

Abbreviation	Term
K	Clay
B	Chlorite
L	Limonite
I	Iron oxides
C	Carbonate
Q	Quartz/Silica
M	Matrix - rock fragments contained in a fine matrix eg recemented breccia, clay gouge
R	Breccia - uncemented angular fragments of wall rock
X	Silt
D	Sand
S	Iron sulphides
Py	Pyrite
Cm	Carbonaceous material
O	Serpentinites

4.09.10 Joint Wall Strength

Joint wall strength is described using the following abbreviations. The strengths are defined in Table 4.12.

Table 4.12 Wall Strength

Abbreviation	Term
EW	Extremely weak
VW	Very weak
W	Weak
MS	Medium strong
S	Strong
VS	Very strong
ES	Extremely strong

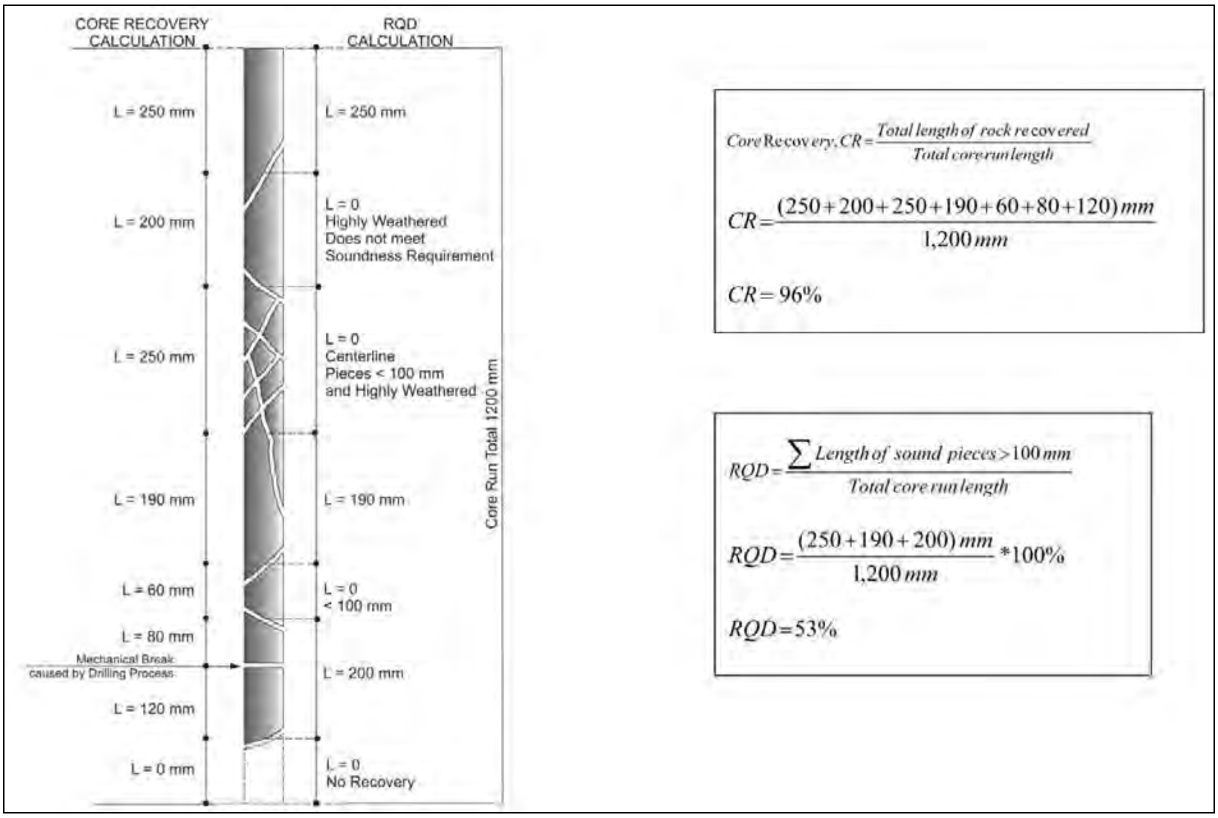
4.10 Mechanical Logs

All mechanical logging (discontinuity logging) is based only on those discontinuities assessed as natural. Any drilling induced discontinuities or artificial breaks (including post drilling stress relief, desiccation, transport damage and logging damage) are ignored. Any healed or incipient discontinuities are also ignored. If there is any doubt whether a discontinuity is natural or not it is assumed to be natural.

The terms Core Recovery (CR) and Rock Quality Designation (RQD) are applied only to rock core.

4.10.01 CR (Core Recovery)

Defined as the percentage ratio of solid core recovered to the total length of each core run. Solid core is defined as core with at least one full diameter (but not necessarily a full circumference) measured along the core axis between two natural discontinuities. The CR shall is measured for each core run (Figure 4.3).



$$Core\,Recovery, CR = \frac{Total\ length\ of\ rock\ recovered}{Total\ core\ run\ length}$$

$$CR = \frac{(250+200+250+190+60+80+120)\,mm}{1{,}200\,mm}$$

$$CR = 96\%$$

$$RQD = \frac{\sum Length\ of\ sound\ pieces > 100\,mm}{Total\ core\ run\ length}$$

$$RQD = \frac{(250+190+200)\,mm}{1{,}200\,mm} * 100\%$$

$$RQD = 53\%$$

Figure 4.3 Calculation of CR and RQD

By this definition core that contains a single set of inclined discontinuities would have a CR of 100%. Where there are two or more sets of non-parallel discontinuities, sections where two or more discontinuities intersect are not considered as solid core. Core broken by drilling induced discontinuities is considered as solid core. Core which is highly disturbed and non-intact is not considered as solid core, as it is not possible to make an assessment of the in-situ nature. Soil is not considered as solid core.

4.10.02 RQD (Rock Quality Designation)

Defined as the total length of solid core pieces each greater than 100 mm between natural (not drilling induced) discontinuities expressed as a percentage of the total length of each core run, measured along the core axis.

The RQD shall be measured for each core run (Figure 4.3 above).

4.11 Additional Description

Any comments and descriptions about the observed rock types.

Appendix A Geological Log Sample From

PROJECT						**DEPTH (m)**		**ELEVATION (m)**	
HOLE NO.		**COORDINATE**	**X:**	**Y :**		**INCLINATION (degree)**		**DRILLING RIG**	
DATE	**FROM:**			**TO:**		**DRILLER**		**LOGGED BY**	

ELEVATION (m)	**DEPTH (m)**	**SOIL OR ROCK TYPES OR FORMATION**	**MATERIAL GRAPHICS**	**DESCRIPTION**	**SAMPLE LOCATION**	**SAMPLE NUMBER**	**SPT N-VALUE (Blow per 15 cm)**	**WATER PRESSURE (cm/s)**	**RECOVERY (%)**	**RQD (%)**	**DISCONTINUITY**								**DEPTH (m)**
											TYPE	**FORM**	**DIP**	**SPACING**	**JRC**	**PLANARITY**	**ROUGHNESS**	**INFILLING MATERIAL**	
	1																		1
	2																		2
	3																		3
	4																		4
	5																		5
	6																		6
	7																		7
	8																		8
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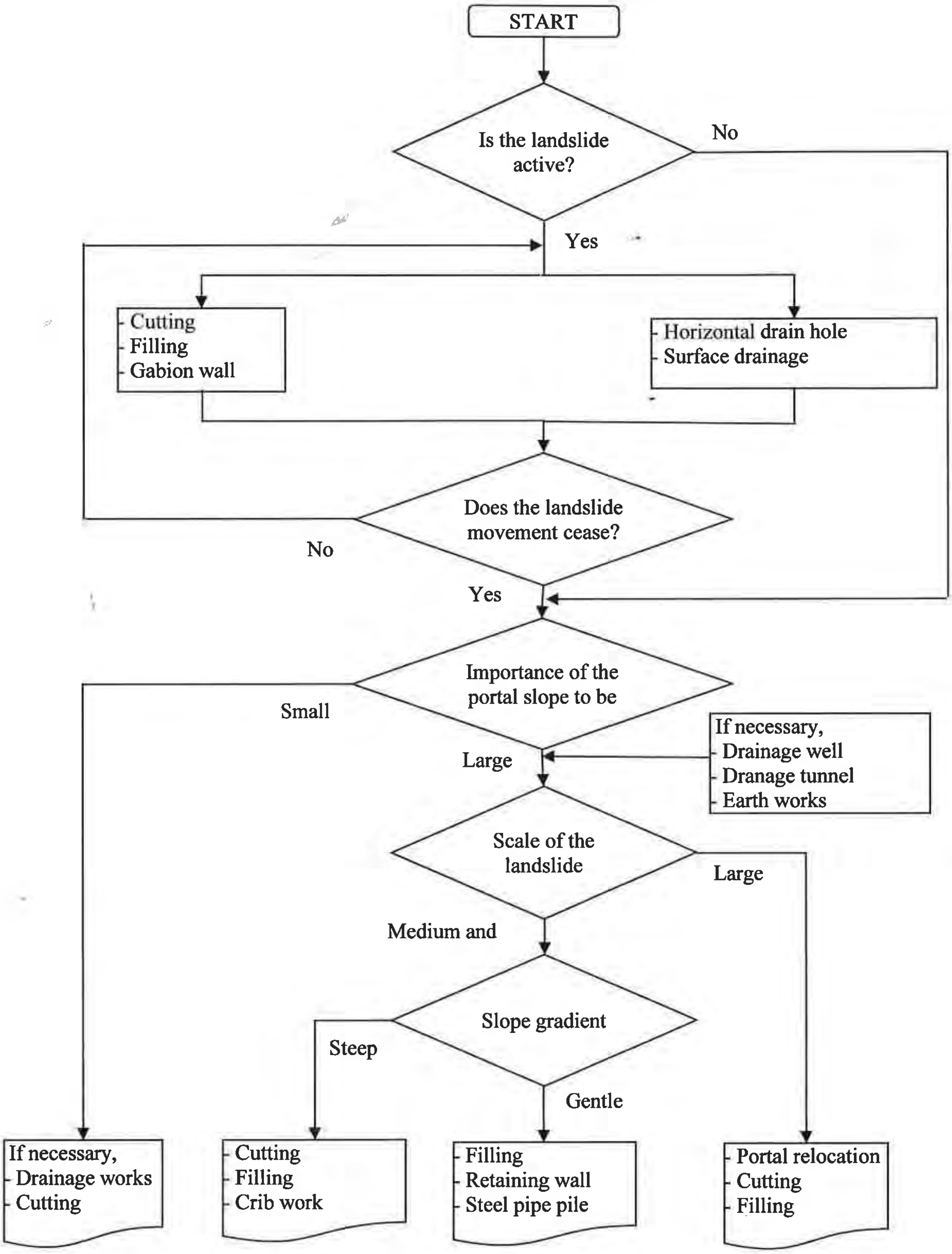


Selection Flowchart of Mitigation Methods for Landslide (Draft)

TECHNICAL NOTE

No. : TM-GS-003
Date : 30 Aug. 2017

Interpretation and Application of Lugeon Test Results

By Dr. P. YANG, Geotechnical Expert

1. Introduction

1.1 General

Lugeon Test, also called Packer Test or Water Pressure Test, is an in-situ, constant head permeability testing method commonly carried out in a portion of a borehole isolated by inflated packers. The test is named after Maurice Lugeon (1933), a Swiss geologist who first introduced the test.

The test, originally introduced and suggested as a test for assessing need for foundation grouting at dam sites, presently has been widely used to estimate the average hydraulic conductivity (or permeability) of rock mass. The test is a routine and common in-situ method of measuring hydraulic conductivity of rock mass for a tunnel project.

This technical note briefly presents general test procedure and test result interpretation method, which is based on some international standards, widely accepted methods and engineering experiences.

1.2 Purpose of Lugeon Test

The test has been widely used to estimate the permeability of the foundation rock mass for most of infrastructure projects. The purposes of the test for a tunnel project are as follows:

1) To provide in-situ permeability of the different rock formations in the region of the tunnel alignments and portal areas,
2) To aid design of rock excavation, tunnel support and lining, and dewatering systems for the proposed tunnel, and
3) To check the effectiveness of grouting through measuring of the impermeability of grouted rocks around the tunnel opening.

1.3 Definition

A Lugeon is defined as the loss of water in litres per minute and per metre borehole at an effective pressure of 1 MPa (10 bars or 10 kg/cm^2), equivalent to as follows:

1) 1 liter per minute per meter (l/min/m) at a pressure of 10 bars,
2) 0.0107 cubic feet per minute (ft^3/min) at 142 pounds per inch (psi),
3) 1 x 10^{-5} cm/sec = 1 x 10^{-7} m/sec, and
4) 10 ft/yr.

1.4 Limitations

The Lugeon test affects a limited volume of rock around the borehole to be tested. It has been reported that the effect of the Lugeon tests is generally restricted to an approximate radius of 30 feet around the borehole with a test section length of 10 feet (Bliss and Rushton, 1984). This indicates that the hydraulic conductivity obtained by the test method is only representative for a cylinder of rock delimited by the length of the test section and the radius as mentioned above.

In addition, when the test is conducted at high water pressure, the test result or the obtained Lugeon values could be misleading because high pressures would cause erosion or washout of fines from discontinuities as well as deformation of the rock mass and closure of discontinuities.

1.5 Related Standards and References

The following documents have been consulted and referred in preparation of this technical note.

1) ASTM Standard D4630-96, Standard Test Method for Determining Transmissivity and Storage Coefficient of Low-Permeability Rocks by In Situ Measurements Using the Constant Head Injection Test.
2) Japanese Geotechnical Society Standard JGS 1323-2012, Method for Lugeon Test.
3) British Standard BS5930-1990, Code of Practice for Site Investigations, Clause 25.5 Parker Test.
4) Quiñones-Rozo, Camilo (2010), Lugeon test interpretation, revisited. In: Collaborative Management of Integrated Watersheds, US Society of Dams, 30th Annual Conference, S. 405–414.

2. Test Equipment and Procedure

2.1 Test Equipment and Material

The Lugeon test is conducted in a portion of a borehole isolated by pneumatic packers as the borehole progresses. The water is injected into the isolated portion of the borehole using a slotted pipe which itself is bounded by the inflated packers. The packers can be inflated using a gas compressor on the ground surface, and so they can isolate and seal that portion of the borehole. A pressure transducer is also located in that portion to measure the pressure with a help of reading station on the surface.

Figure 1 illustrate the general arrangement of a Lugeon test and test equipment required.

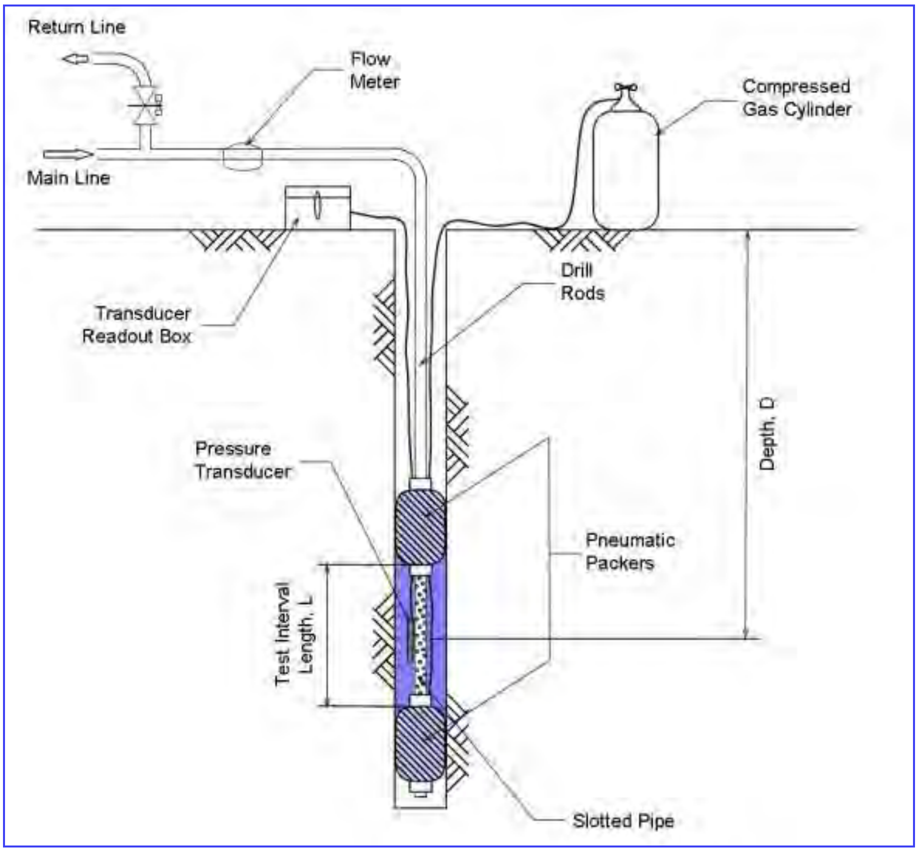


Figure 1 Lugeon Test Schematic (Quiñones-Rozo, 2010)

1) Parker System

Pneumatic packers are commonly used with the inflatable section at least 1m in length. Mechanical packers are generally not accepted because of leakage water between mechanical packer and borehole wall.

Both methods of single and double parkers are adaptable. The flow is confined between two packers in double packer test, or between one packer and the bottom of the borehole to be tested in single packer test. However, to minimize the leakage water from the spilt between packer and borehole wall or through cracks, single packer method is more recommendable.

2) Pump

The pump or pumps to be used for the Lugeon test should supply a range of flows and pressures of up to 200 litres per minute at a pressure of more than 15kg/cm^2 (15 bar). Controls shall be provided for adjusting the water flow and/or pressure.

In addition, the pump required should be a type of two cylinder and double action to avoid pulsation.

3) Pressure Gauge and Flow Meter

Several pressure gauges should be supplied with full scale deflections of approximately 1, 2.5, 5, 10 and 15kg/cm^2. The pressure gauges should be laboratory tested immediately before and after the fieldwork with test certificates.

Several flow meters should be supplied to measure flows at their maximum capacity of up to 2, 20 and 200 litres per minute. The flow meters should be calibrated over a range of flow rates on site prior to test.

4) Others

In general, a surge tank should be provided and installed between the pump and the flow and pressure gauges. The surge tank is used to supply sufficient water to sustain the maximum test flow for the required test duration.

2.2 General Requirements

The Lugeon test essentially comprises the measurement of the volume of water that can lose or escape from an uncased test section of borehole in a given time under a given pressure.

1) Maximum Test Pressure

Accordingly, a maximum test pressure (*Pmax*) is first defined and determined so that it does not exceed the in-situ minimum rock stress, thus avoiding hydraulic fracturing during the test.

As a rule of thumb, *Pmax*, is usually established using the following equation, where D is equal to the minimum ground coverage (in meter) – depth in the case of a vertical borehole in a flat site or minimum lateral coverage in the case of a test conducted in a hilly slope.

$$\begin{aligned} P_{\max} &= D \times \frac{1 psi}{1 ft} \\ &\approx 20 \times D(m) \quad \text{in } kPa \\ &\approx 0.02 \times D(m) \quad \text{in } kg/cm^2 \end{aligned}$$

In addition, according to ASTM D4630-96, the test pressure is typically determined between 300 and 600 *kPa* (50 to 100 *psi*).

2) Test Stage

The test is generally carried out at five stages or steps including increasing and decreasing pressure between zero and maximum pressure. Five loading and unloading stages form a pressure loop typically with the following pressure intervals (Table 1):

Table 1 Pressure Magnitudes Typically Used for Each Test Stage

Stage	Description	Pressure
1st	Low	0.50 Pmax
2nd	Medium	0.75 Pmax
3rd	Maximum (Peak)	1.00 Pmax
4th	Medium	0.75 Pmax
5th	Low	0.50 Pmax

In some cases, the test may involve only 3 pressure stages, in which case Pmax is at stage 2 and the stage 1 pressure should equal the stage 3 pressure.

3) Test Section Length

The Lugeon test is carried out as the borehole progresses, normally involving a continuous profile of measurements. The test section length (or packer spacing) generally depends on rock conditions. The wider spacing is used in good-quality rock and the closer spacing in poor-quality rock. The test section length is normally 1, 2, or 3 m, or at times 5 m, and typically between 3 m and 5 m.

2.3 Test Procedure

Two test procedures can be used for the test, depending on rock quality, as described below:

1) Test procedure I. The procedure is normally used in poor to moderately poor rock with borehole collapse problems, and involves drilling the borehole to required test depth and performing the test with a single packer. Casing is installed if necessary, and the borehole is advanced to the next test depth. The test is conducted as the borehole progresses.
2) Test procedure II. The procedure is generally used in good-quality rock where the borehole remains open, and involves drilling the borehole to the final depth, filling it with water, surging it to clean the walls of fines, and then bailing it. The test proceeds in sections from the bottom–up with two packers. The test is conducted after the completion of borings.

Test procedure I, which is commonly used for all rock mass conditions, is given as follows:

1) Wash the borehole with clean water

Once drilling the depth of bottom of a section to be tested in the bedrock, the borehole shall be washed inside by flushing clean water through the drill rod inserted to the bottom of the borehole until the water emerging from the top of the borehole is clear.

2) Expand the packer with air pressure.

A packer is installed at the top of the test section and then inflated to a pressure sufficient to ensure a water tight seal against the borehole wall, without damaging the rock formation.

After inflation of the packer the water level in the borehole above the packer is monitored with an electric dip meter when the first water pressure is applied and then at 5 minute intervals. If the water level rises during the initial stages of the test the packer should be re-seated and the test repeated.

3) Introduce water under a given pressure into the borehole

Water is injected into the borehole below the packer under a given pressure (refer to Table 1 above) – a constant water pressure for a duration of less than 15 minutes or until steady state flows are measured.

4) Record elapsed time and volume of water pumped

The flow at each pressure is measured over at least 3 intervals each of five minutes. Additional 5 minute intervals should be added until both the pressure and the flow are constant over two consecutive intervals. The flow readings may be recorded as Flux or Volume, and this generally depend on the meter type that is being used.

If the test pressure cannot be achieved due to large flows, and the water level in the borehole is not rising, then the test may be terminated after running the pump at the maximum flow rate for at least 5 minutes and recording the pressure and flow volume achieved.

5) Test at five pressures

Each test consists of measurements of water inflow at five pressures (refer to Table 1 above) measured at the surface pressure gauge.

3. Analysis and Interpretation

3.1 Calculation

The Lugeon is the conductivity required for a flow rate of 1 liter per minute per meter of the borehole interval under a constant pressure of 1 *MPa*. Accordingly, with the average values of water pressure and flow rate measured at each stage, the average hydraulic conductivity of the rock mass can be estimated in terms of Lugeon unit. The Lugeon value for each test is therefore calculated as follows and then an average representative value is selected for the tested rock mass.

Lugeon Value = (Q / L) x (P0 / P)

where,
Q = Flow rate [lit/min]
L = Length of the test section [m]
P_0 = Reference pressure of 1 MPa [MPa] (equivalent to 10 bar or 145 psi)
P = Test pressure [MPa] (at the specific stage)

In addition, the conversion of pressure (P) into injection head (H) is calculated as follows:

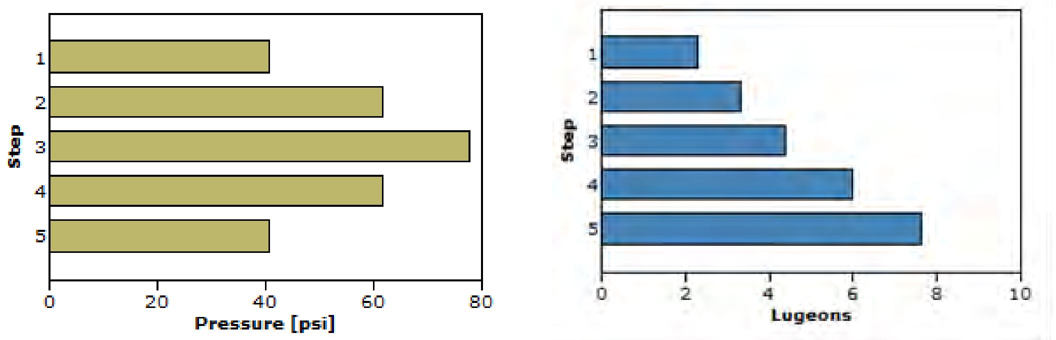
$$H = \frac{P}{pg}$$
$$P = H \times pg$$

where,
g = acceleration due to gravity ($g = 9.81\ m/s^2$)
p = density of water ($p = 999.7\ kg/m^3$)

3.2 Data Interpretation

1) Pressure and Lugeon Diagrams

The Gauge Pressure data are read and plotted on a simple Pressure vs. Step diagram as shown in Figure 2(a), while, for each step or stage, the Lugeon value is calculated using the equations described above and plotted on a simple bar chart as shown in Figure 2(b).



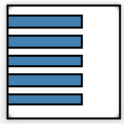
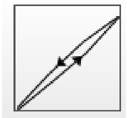
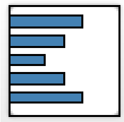
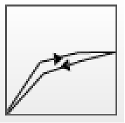
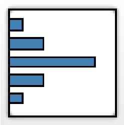
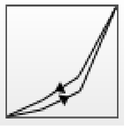
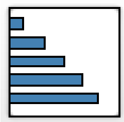
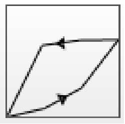
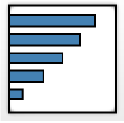
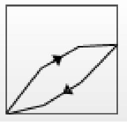
(a) Pressure Diagram (b) Lugeon Diagram

Figure 2 An Example of Pressure and Lugeon Diagrams

2) Typical Lugeon Behaviours

The trends from the Lugeon Diagram can be compared to the diagnostic plots as described in Table 2 below to identify typical behaviour and choose a suitable Lugeon value. A more detailed interpretation of Lugeon test results is described by Quiñones-Rozo (2010).

Table 2 Lugeon Interpretation Pressure Following Water Loss vs Pressure Pattern

Behaviour	Lugeon Pattern	Flow vs Pressure Pattern	Representative Lugeon Value
Laminar Flow			Average of Lugeon values for all stages
Turbulent Flow			Lugeon value corresponding to the highest water pressure (3rd stage)
Dilation			Lowest Lugeon value recorded, corresponding either to low or medium water pressures (1st, 2nd, 4th, 5th stages)
Wash-out			Highest Lugeon value recorded (5th stage)
Void Filling			Final Lugeon value (5th stage)

Source: Modified after Quiñones-Rozo, 2010

These typical Lugeon behaviours are summarized:

a) Laminar Flow: The hydraulic conductivity of the rock mass is independent of the water pressure employed. This behaviour is characteristic of rock masses with low hydraulic conductivities and relatively small seepage velocities.

b) Turbulent Flow: The hydraulic conductivity of the rock mass decreases as the water pressure increases. This behaviour is characteristic of rock masses exhibiting partly open to moderately wide cracks.

c) Dilation: Similar hydraulic conductivities are observed at low and medium pressures; however, a much greater value is recorded at the maximum pressure. This behaviour – which is sometimes also observed at medium pressures – occurs when the water pressure applied is greater than the minimum principal stress of the rock mass, thus causing a temporary dilatancy (hydro-jacking) of the fissures within the rock mass.

d) Wash-Out: Hydraulic conductivities increase as the test proceeds, regardless of the changes observed in water pressure. This behaviour indicates that seepage induces permanent and irrecoverable damage on the rock mass, usually due to infillings wash out and/or permanent rock movements.

e) Void Filling: Hydraulic conductivities decrease as the test proceeds, regardless of the changes observed in water pressure. This behaviour indicates that either: (1) water progressively fills isolated/non-persistent discontinuities, (2) swelling occurs in the discontinuities, or (3) fines flow slowly into the discontinuities building up a cake layer that clogs them.

4. Application and Correlation of Test Results

4.1 Effect of Discontinuities (or Joints) on Rock Mass Permeability

1) The permeability of intact rock is generally several orders less than in-situ permeability.
2) The permeability of rock mass is governed chiefly by discontinuity frequency, distribution, openness and infilling.
3) The likely permeability for various joints features would have most of the following characteristics (Tables 3 and 4).

Table 3 Effect of Discontinuity Characteristics on Rock Mass Permeability

Typical Discontinuity Characteristics				Permeability (m/s)
Opening	**Filling**	**Width**	**Joint Set**	
Open	Sands/Gravel	>20 mm	More than 3 interconnecting sets	$>10^{-5}$
Gapped	Non-plastic fines	2 - 20 mm	1 to 3 interconnecting set	10^{-5} - 10^{-7}
Closed	Plastic clays	<2 mm	Less than 1 joint set	$<10^{-7}$

Source: Bell, 1992

Table 4 Estimation of Rock Mass Permeability from Discontinuity Frequency

Discontinuity Pattern of Rock Mass	Term	Permeability(m/sec)
Very closely to extremely closely spaced discontinuities	Highly permeable	10^{-2} - 1
Closely to moderately widely spaced discontinuities	Moderately permeable	10^{-5} - 10^{-2}
Widely to very widely spaced discontinuities	Slightly permeable	10^{-9} - 10^{-5}
No discontinuities (same as intact rock)	Impermeable	$<10^{-9}$

Source: Bell, 1992

In addition, discontinuity spacing and opening are generally defined following Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses published by ISRM (1981), and given in Tables 5 and 6, respectively, for your reference.

Table 5 Discontinuity Spacing

Description/Term	Spacing (m)
Extremely closely spaced	<0.02
Very closely spaced	0.02 - 0.06
Closely spaced	0.06 - 0.2
Moderately widely spaced	0.2 - 0.6
Widely spaced	0.6 - 2.0
Very widely spaced	2.0 - 6.0
Extremely widely spaced	< 6.0

Source: Modified from ISRM, 1981

Table 6 Discontinuity Aperture (Opening)

Aperture (mm)	Description/Term	Feature
<0.01	Very tight	Closed feature
0.01 – 0.25	Tight	
0.25 – 0.50	Partly open	
0.50 – 2.5	Open	Capped feature
2.5 – 10.0	Widely open	
10 – 100	Very widely open	Open feature
100 – 1000	Extremely widely open	
< 1000	Cavernous	

Source: Modified from ISRM, 1981

4.2 Lugeon Test in Rock

As stated above, the hydraulic conductivity of rock mass is generally controlled chiefly by rock structure - discontinuities. Therefore, the Lugeon value could represent not only the conductivity but also the rock jointing condition. Typical range of Lugeon values and the corresponding rock condition is indicated in Table 7 below.

Table 7 Indicative Rock Mass Permeability from Lugeon Value

Lugeon Range	Classification	Hydraulic Conductivity (cm/sec)	Discontinuity Condition
<1	Very low	$<1 \times 10^{-5}$	Very tight
1-5	Low	1×10^{-5} - 6×10^{-5}	Tight
5-15	Moderate	6×10^{-5} - 2×10^{-4}	Few partly open
15-50	Medium	2×10^{-4} - 6×10^{-4}	Some open
50-100	High	6×10^{-4} - 1×10^{-3}	Many open
>100	Very high	$>1 \times 10^{-3}$	Open closely spaced or voids

Source: Modified after Quiñones-Rozo, 2010.

A general sense of the proportion of Lugeon units are as follows:

1) 1 Lugeon unit is the type of permeability consistent with sound bedrock.
2) 10 Lugeon units typically indicates a permeable formation in which seepage occurs.
3) 100 Lugeon units is the type of permeability typically observed in heavily jointed bedrock with relatively open joints, or in slightly to moderately jointed bedrock where joints are wide to very widely open.

No. : TN-GS-004
Date : 2017.09.04

Geotechnical Investigation Progress Report

By Dr. P. YANG, Geotechnical Expert

1. Introduction

This geotechnical investigation (the Investigation) commenced on July 7th, 2017 and at present is in progress by ELS (Engineering and Laboratory Services (Pvt) Ltd.), under the supervision of JICA Study Team (the Team). The Investigation has been conducted, as an example of geological and geotechnical surveys for a tunnel project, to provide reliable and necessary geological, geotechnical and geohydrological data and information for rock mass classification, and subsequent tunnel design and construction plan for the selected pilot tunnel No. 2.

This geotechnical investigation progress report (the Report) presents the investigation program and current progress, summarizes the investigation results so far, and provides some preliminary recommendations on geotechnical issues related to the design and construction of the selected tunnel based on the obtained investigation results.

2. Investigation Program and Progress

2.1 Investigation Program

In addition to 1) existing data collection and review, 2) Site reconnaissance and 3) existing well water table measurement, which are conducted by JICA Study Team, the Investigation, which is performed mainly by ELS, consists of as follows:

1) Core borings,
2) Sampling of soil and rock samples,
3) In-situ and Laboratory tests, and
4) Seismic refraction survey

Table 1 summarizes the item and quantity of the Investigation, while Figure 1 shows the plan and location of the Investigation.

Table 1 Summary of the Planned Investigation Item and Quantity

	Investigation Item	Standard/Description	Unit	Quantity
1	Core borings	3 boreholes x 30 to 50 m	m	110
2	Samplings of soils and rocks	12 boreholes x 4 to 7 samples	nos.	64
	2.1 Undisturbed soils	2 test pit x 1 sample	nos.	2
	2.2 Disturbed soils	3 boreholes x 2 samples	nos.	6
	2.3 Intact rocks	3 boreholes x 2 to 6 samples	nos.	10
3	Field test			
	3.1 Standard penetration test	at about 1.0 m interval in soils	time	60
	3.2 Lugeon test	3 boreholes x 2 to 6 tests	test	10
4	Laboratory tests of soil samples			
	4.1 Specific gravity	BS1377-2	test	6
	4.2 Unit weight	BS1377-2	test	6
	4.3 Grain size analysis	BS1377-2	test	8
	4.4 Atterberg Limit	BS1377-2	test	6
	4.5 Unconfined compressive strength	BS1377-7	test	2

4.6 Direct shear strength	BS1377-7	test	2
5 Laboratory tests of rock core samples			
5.1 Unit weight	BS812-2 [148]	Test	3
5.2 Water absorption	BS812-2 [150]	Test	3
5.3 Point load test	BS812-2 [153]	Test	10
5.4 Uniaxial compression strength	BS812-2 [153]	Test	10
5.5 Petrographic analysis	BS812-2 [149]	test	2
6 Seismic refraction survey	3 lines x (100 m + 300 m)	m	1200

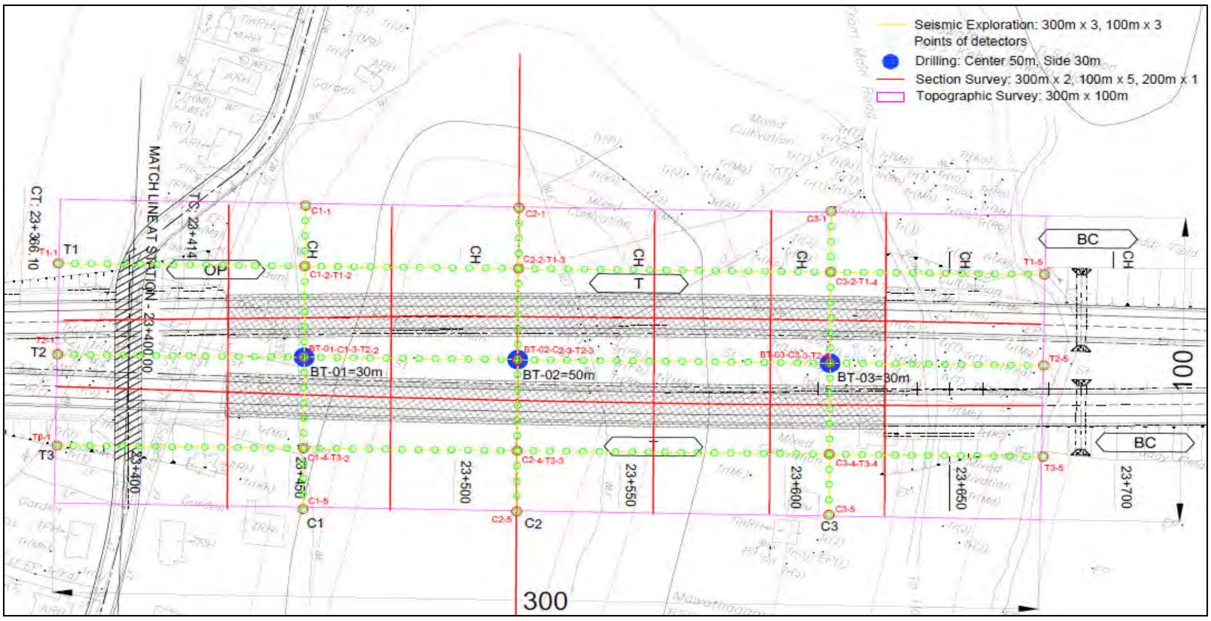


Figure 1 Location and Plan of the Investigation

2.2 Current Progress

The Investigation was commenced in early July and originally planned to be completed in mid September with a work period of about two to three months, as shown roughly in Table 2 below.

Table 2 Original Working Schedule of the Investigation

	Description of works	Jul-17			Aug-17			Sep-17			Oct-17		
1	Preparation, Mobilization, etc.		■										
2	Topographical Survey			■	■	■							
3	Geotechnical Boring and In-situ Tests			■	■	■							
4	Seismic Refraction Survey				■								
5	Laboratory Tests				■	■	■	■					
6	Reporting							■	■				

However, except for topographical survey, all items of the Investigation are much behind the original schedule. Table 3 gives roughly monthly progress for the Investigation as of August 31. The delay is due to lack of experience in core borings and poor management.

According to the current progress, the Investigation cannot be completed within the contract period and may be extended to the end of this October or more later. ELS shall be required to make much effort to improve the drilling progress and core quality.

Table 3 Monthly Progress for the Investigation

Geotechnical Investigation Item			Investigation Period (from 2017/7/12 to 2017/10/10)				
			2017/7/31	2017/8/31	2017/9/30	2017/10/31	2017/11/30
1. Core Boring	110	Actual Quantity	25	45			
	(m)	Progress (%)	23%	41%			
2. Lugeon Test	10	Actual Quantity	0	2			
	(test)	Progress (%)	0%	20%			
3. Lab Test	58	Actual Quantity	0	5			
	(test)	Progress (%)	0%	9%			
4. Seismic Refraction	1200	Actual Quantity	0	1200			
	(m)	Progress (%)	0%	100%			
5. Analysis and Report	1	Actual Quantity	0	0			
	(set)	Progress (%)	0%	0%			
6. Total Progress	**(set)**	**Progress (%)**	**5%**	**34%**	**0%**	**0%**	**0%**

3. Interim Results of the Investigation

3.1 Local Geology and Site Conditions

A geologic map of the Project site, which is extracted from Geology of the Kandy-Hanguranketa, 1:100,000, Published by the Geological Survey and Mines Bureau of Sri Lanka (GSMB) (1996), is shown in Figure 2 and summarized in Table 4 below. According to GSMB (1996), the selected tunnel No. 2 is crossing Quartzite, Quartz Schist and locally intercalated Quartz-feldspar Gneiss of the Proterozoic Metamorphic Rocks.

Table 4 Stratigraphic Units of the Project Site

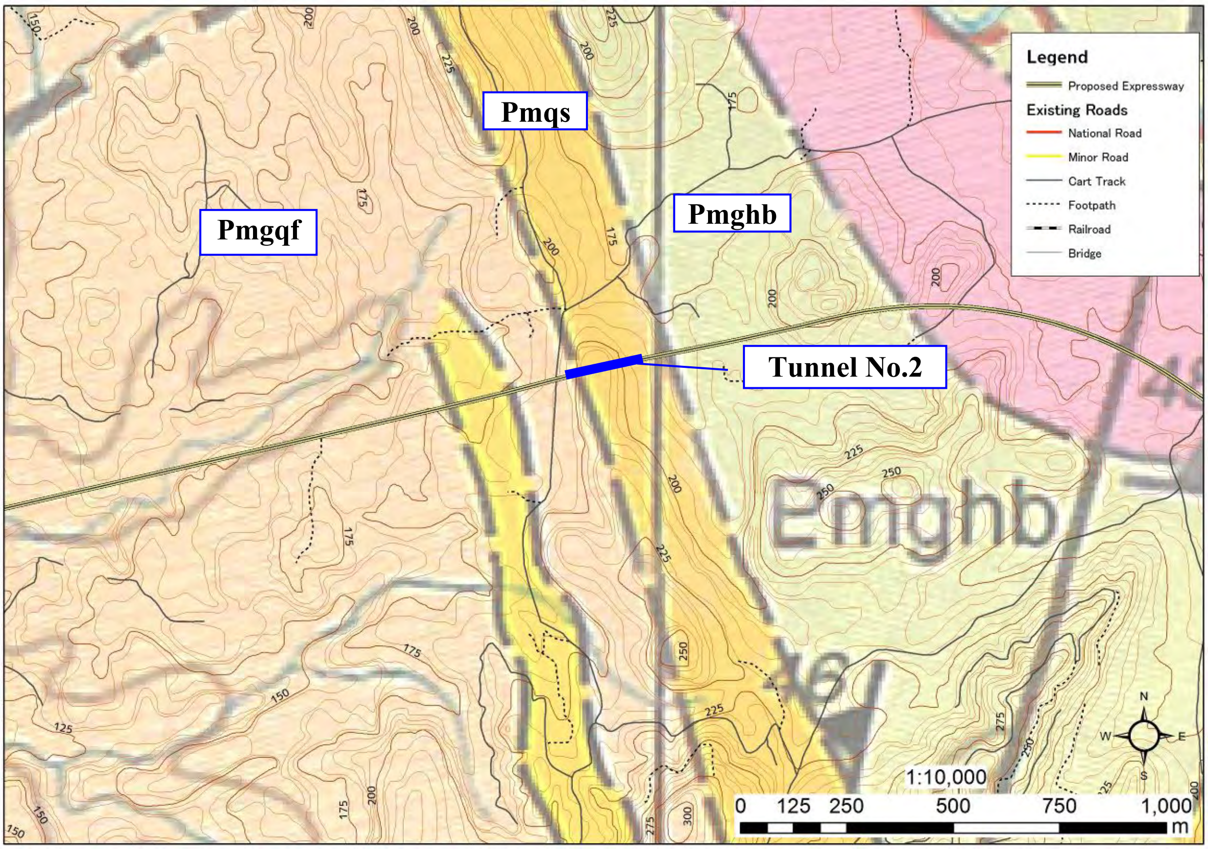
Geological Unit		Geological Description
Proterozoic Metamorphic Rocks	Pmghb	Hornblende-biotite migmatite: compositionally layered grey gneiss typically with white pegmatoid leucosomes
	Pmgqf	Quartzofeldsparthic gneiss: leucocratic, gneiss weakly compositionally layered, granoblastic, may include both para- and orthogneisses.
	Pmqs	Impure quartzites and quartz schists: with sillimanite, garnet, often interlayered with biotite-bearing quartz-rich quartzofeldspathic gneiss

Source: Modified from Geology of the Kandy-Hanguranketa, 1:100,000, Published by the Geological Survey and Mines Bureau of Sri Lanka (GSMB) (1996).

Some outcrops of quartz schist are observable around the project site. These exposures show a closely well-developed foliation within the rock mass that coincides with the plane of joint (Figure 3). The foliation joints generally dip toward east with a dip angle of 10 to 20 degrees. In addition, the foliation joints show some opening (Figure 3(b) at exposures due presumably to stress relief.

A thrust is inferred to pass through the planned tunnel alignment, which means that the underlying bedrocks along the tunnel line have a high potential for fracturing, further indicating a possibility of overbreak or collapse during tunnel excavation. However, no faulting movement evidences have been observed and identified from site reconnaissance.

In addition, no landslide scarps and potential landslides have been identified on the portal slopes and along the tunnel alignment from site reconnaissance. No spring water is observed around the tunnel alignment.



Note: Geological description and symbol are the same as those presented in Table 3 above

Figure 2 Site Geology together with Tunnel Alignment

(a) Closely Foliated Quartz Schist

(b) Quartz Schist with Open Foliation Joints

Figure 3 Photographs Showing Fracture Pattern in Outcrops of the Quartz Schist

3.2 Core Boring and In-situ Tests

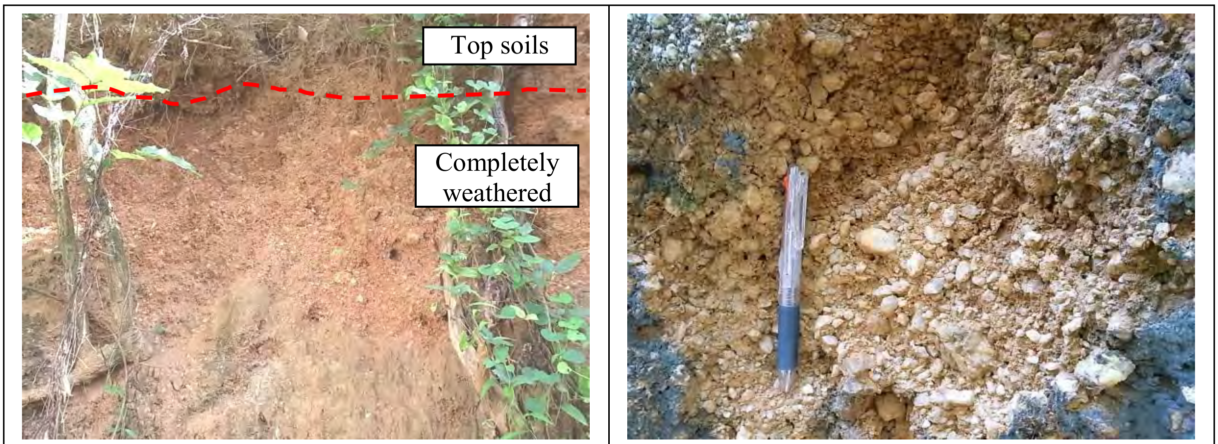
1) Rock Mass Weathering

Rock mass quality is generally affected significantly by weathering. Core observation on BT-03 shows that, below the top soil, the rock mass at drilled depth can be divided, following Suggested methods for the quantitative description of discontinuities in rock masses Published by ISRM (1981), into several weathered zones, as summarized in Table 5 below.

Table 5 Rock Mass Weathering Zone Identified at BT-03

Weathering Zone	Core Observation	Core Photo
Residual Soils (from 0.60 to 6.45 m)	a) All rock material is decomposed to soils and deposited without movement. b) The mass structure is completely destroyed. .	
Completely/highly weathered (from 6.45 to 16.00 m)	a) More than half of the rock material is decomposed and/or disintegrated to a soil. b) Fresh or discoloured rock is present as a corestone.	
Moderately weathered (from 16.00 to 19.00 m)	a) Less than half of the rock material is decomposed and/or disintegrated to a soil. b) Fresh or discoloured rock is present as a continuous framework	
Slightly weathered (from 19.00 to 22.35 m)	a) Discolouration indicates weathering of rock material and discontinuity surfaces. b) The rock material is partially discoloured by weathering.	
Completely/highly weathered (from 22.35 to 27.90 m)	a) More than half of the rock material is decomposed and/or softened to a soil.	
Slightly weathered (from 27.90 to 30.00 m)	a) Discolouration indicates weathering of rock material and discontinuity surfaces.	

The rock mass is generally completely weathered in outcrops. Field observation on cut exposures indicates that the bedrocks at ground surface are completely decomposed into sandy gravels – residual soils. The soils are very dense and can vertically stand at natural condition (Figure 4). However, the soils are generally susceptible to erosion by surface water because its composition of sand and gravel.



(a) Weathering Zone Observed in Cut Slope | (b) Close View of Completely Weathered Rock

Figure 4 Photographs Showing Surface Weathering of the Quartz Schist

2) Rock Mass Joint Conditions

From the drilled cores, one set of joint – foliation joint was identified, as shown in Figure 5. The joint is sub-horizontal, which obliques across the tunnel alignment at small angle. Except for mechanical break by drilling, the spacing of rock mass joints is mostly between 10 and 50 cm and locally between 5 and 10 cm.

As shown in Figure 6 below, the RQD is mainly between 0-10% and 70-100%, and mostly concentrated 0-10%. In addition, from core observation some joints and crack surfaces were very fresh and undulating, and these joints/cracks were thus considered to be formed due to drilling breaks. Accordingly, the RQD may be underestimated

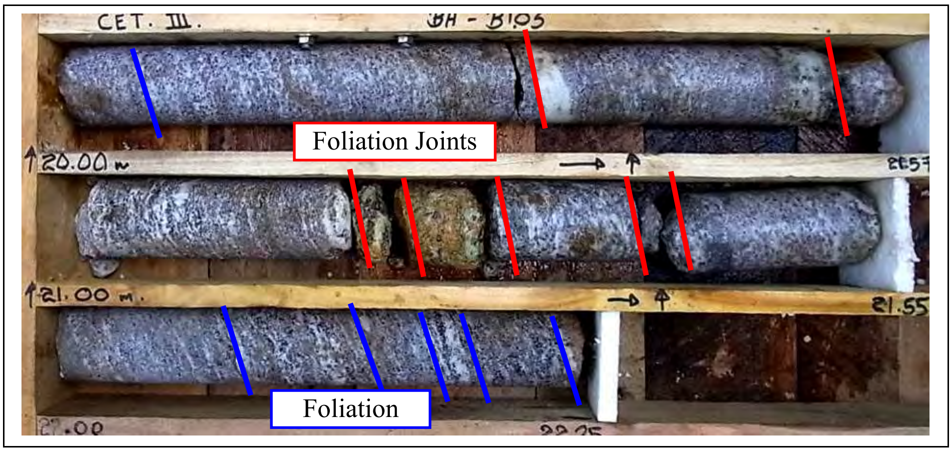


Figure 5 Photographs Showing Foliation Joints Formed within Quartz Schist

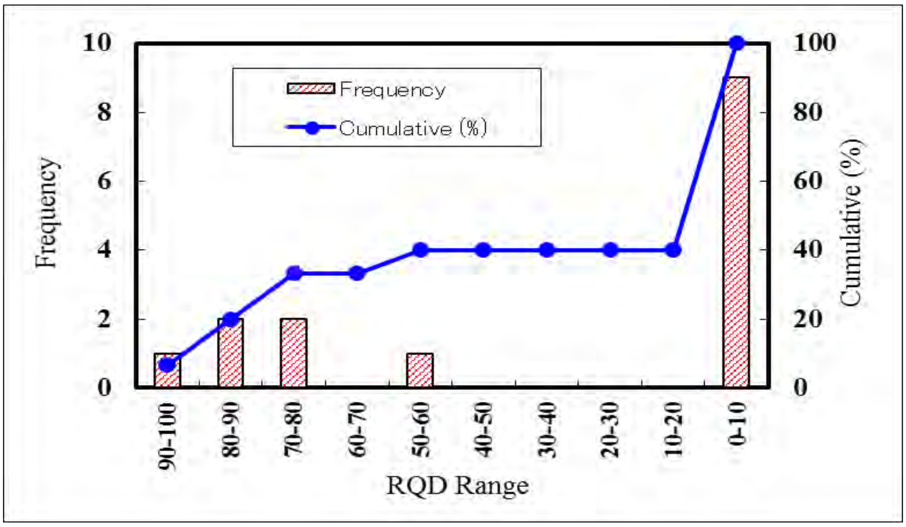


Figure 6 Histogram Distribution of the RQD Values in BT-03

3) Lugeon Test

The obtained results in BT-03 are around 1 to 3 Lugeons, which reveals low permeability in the tunnel portal area. In addition, according to Table 7, the rock mass permeability was at order of 1 to 6 x 10^{-5} cm/sec due to tight joints within the rock mass.

Table 6 Lugeon Tests Performed in BT-03

Test No.	Test Depth	Lugeon Value	Rock Mass Weathering Zone
1	From 20 m to 25 m	2.5	Moderately to slightly weathered
2	From 25 m to 30 m	1.0	Slightly to highly weathered

Table 7 Indicative Rock Mass Permeability from Lugeon Value

Lugeon Range	Classification	Hydraulic Conductivity (cm/sec)	Discontinuity Condition
<1	Very low	<1 x 10^{-5}	Very tight
1-5	Low	1 x 10^{-5} - 6 x 10^{-5}	Tight
5-15	Moderate	6 x 10^{-5} - 2 x 10^{-4}	Few partly open
15-50	Medium	2 x 10^{-4} - 6 x 10^{-4}	Some open
50-100	High	6 x 10^{-4} - 1 x 10^{-3}	Many open
>100	Very high	>1 x 10^{-3}	Open closely spaced or voids

Source: Modified after Quiñones-Rozo, 2010.

4) Groundwater Levels

Groundwater levels at borehole BT-03 were measured daily for two weeks. As seen from Table 8, the groundwater levels existed at a depth of 11.5 to 12.0 meters, approximately lying below the bottom (EL.180.615m) of the planned tunnel. After rainfall, the groundwater levels rose up to about 0.5 meter below the ground surface.

Table 8 Groundwater levels at Boreholes of BT-01 to BT-03

Measuring Date	BT-01		BT-02		BT-03	
	GL-m	EL-m	GL-m	EL-m	GL-m	EL-m
15-Aug					-11.58	178.87
16-Aug					-11.58	178.87
17-Aug					-12.00	178.45
18-Aug					-12.00	178.45
19-Aug					-11.58	178.87
20-Aug					-11.57	178.88
21-Aug					-11.43	179.02
22-Aug					-11.98	178.47
23-Aug					-11.98	178.47
24-Aug					-12.00	178.45
25-Aug					-12.00	178.45
26-Aug					-11.95	178.50
27-Aug					-11.96	178.49
28-Aug					-12.00	178.45
29-Aug					-12.01	178.44
30-Aug					-11.95	178.50
31-Aug					-11.98	178.47
Highest					-11.43	179.02
Lowest					-12.01	178.44
Average					-11.86	178.59

Note: GL-m = Ground level in meter, EL-m = Elevation level in meter.

5) Laboratory Tests

Laboratory tests, as listed in Table 9 below, are in progress.

Table 9 The Planned Items and Quantity of Laboratory Tests

Sample Category	Test Item	Standard or Method	Tests (nos.)
Soils	1) Specific gravity	BS1377-2	6
	2) Unit weight	BS1377-2	6
	3) Grain size analysis	BS1377-2	8
	4) Atterberg Limit	BS1377-2	6
	5) Unconfined compressive strength	BS1377-7	2
	6) Direct shear strength	BS1377-7	2
Rock cores	1) Unit weight	BS812-2 [148]	3
	2) Water absorption	BS812-2 [150]	3
	3) Point load test	BS812-2 [153]	10
	4) Uniaxial compression strength	BS812-2 [153]	10
	5) Petrographic analysis	BS812-2 [149]	2

6) Seismic Refraction Survey

All field works for the planned seismic refraction survey were completed on September 20th, 2017 and the result interpretation is in progress.

4. Summary and Geotechnical Recommendations

The conclusions and recommendations, which are based on the investigation results obtained so far, are draft and preliminary in nature, and shall be revised as the Investigation progresses.

1) The geology of the tunnel alignment is composed mainly of quartzite and quartz schist, and locally intercalated with quartz-feldspar gneiss. The rocks are characterized by a narrowly spaced foliation joints. When encountered in the highly or intensely foliated rocks, tunnel excavation would cause local cave-in failure.
2) No potential landslides were identified around the tunnel portal slopes, however, because the bedrocks surrounding the tunnel are completely to highly weathered to some depth, the portal slope excavation may lead to the potential for shallow slope collapses within the completely to highly weathered rocks. Stable cut portal slope should be designed following the weathering conditions of the bedrocks around the portal areas and cut slope protection works should be also provided to maintain the long-term stability of the tunnel portal slopes.
3) The bedrocks are low permeable and the groundwater level is below the tunnel bottom, and the water inflow into the tunnel during construction is expected to be limited.

MINISTRY OF HIGHER EDUCATION AND HIGHWAYS
ROAD DEVELOPMENT AUTHORITY

TECHNICAL ASSISTANCE FOR
IMPROVEMENT OF CAPACITY FOR
PLANNING OF ROAD TUNNEL

TECHNICAL NOTE

No. : TM-GS-005
Date : 18.12.2017

Consideration on rock mass classification in Sri Lanka

By Dinushka Perera RDA HWY Design, Kimihiko Kotoo

1 Introduction

Tunnelling work includes constructing a tunnel and associated temporary works. Design of tunnel is not similar to that of plant or structure and difficult to assume accurate geological conditions, properties and variability of rock mass along the tunnel. Tunnel design is generally carried out on the basis of less reliable geotechnical assumptions in comparison with other designs in the preliminary stage.

Implementing tasks to assess and manage changes in ground conditions and the adequacy of the tunnel design and ground support are required in order not to create a risk but to ensure safety. This may include suspension of tunnelling works, reassess the changed conditions and reviewing cost performance and safety control measures.

Mathematical solutions of empirical methods and numerical modelling can be applied to evaluate the stability of a tunnel. Rock mass classification system is chiefly composed of these mathematical solutions. Empirical methods are based on the experience obtained in the course of research works. Numerical modelling which uses various modelling techniques and computing power can be considered as another way of solving very complex problems and selecting the most suitable ground support system.

Both empirical methods and numerical modelling are recommended to conduct a stability analysis of the tunnel in each stage including the preliminary stage of the project. The results obtained from both methods can be compared for selection of the most suitable support system.

2. Geotechnical condition

Geology of Sri Lanka consists mainly of crystalline Precambrian metamorphic rocks and they are hard and compact in fresh condition, however top portion of the rock is cracky and relatively soft due to weathering especially above the groundwater table. Overburden of talus deposits, resudial soils, sand and gravels with clayey materials are distributed covering the bedrock. Hardness of the rock mass generally depends on the degree of weathering in the island.

Hydrogeological condition varies at places depending on localities of different topography and precipitation. Sufficient hydrogeological data shall be collected and analized in prior to any construction works.

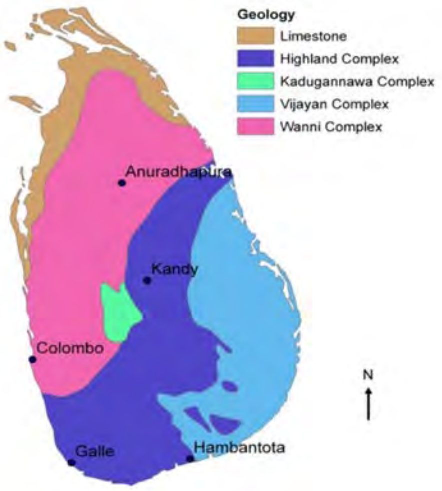


Figure 2.1 Geological Map of Sri-Lanka.
(ESDAC - European Soil Data Centre)

3. Proposed rock mass classification system in Sri Lanka

3.1 Concept

Rock mass is referred to an assemblage of rock material separated by rock discontinuities, mostly by joints, bedding planes, dyke intrusions and faults etc and joints are major factor of discontinuities in comparison with bedding planes, dyke intrusions and faults. According to Bieniawski (1989), Rock Mass Classification is the process of placing a rock mass into groups or classes on defined relationships, and the classification systems are not suitable for use in elaborated and final design, particularly for complex underground openings.

The rock mass classification systems were designed to act as an engineering design aid, and were not intended to substitute field observations, analytical considerations, measurements, and engineering judgment.

These systems provide a basis for understanding the characteristic behaviour and relate to experiences gained in rock conditions at one site to another. In the preliminary design stages of a project, comprehensive information related to the stress and hydrologic characteristics are mostly unavailable. Thus rock mass classification proves helpful at this stage for assessing rock mass behaviour. It not only gives information about the composition, strength, deformation properties and characteristics of a rock mass required for estimating the support requirements, but also shows which information is relevant and required. In practice, rock mass classification systems have provided a valuable systematic design aid on many engineering projects especially on underground constructions, tunnelling and other projects.

The rock mass classification systems of different ways have been proposed globally to design supports for tunnels and underground caverns. Similar design of supports are generally obtained in use of these systems, however some discrepancy of each system are indicated in poor rock conditions.

3.2 Parameters

Various parameters of great and different significance have to be considered in order to describe a rock mass satisfactorily for assuring stability of rock mass. The various important parameters used for description and classification of the rock mass are generally as follows.

(1) the strength of the intact rock material (compressive strength, modulus of elasticity);
(2) the rock quality designation (RQD) which is a measure of drill core quality or intensity of fracturing;
(3) parameters of rock joints such as orientation, spacing and condition (aperture, surface roughness, infilling and weathering);
(4) geological structures (folds and faults).
(5) in-situ stress
(6) Hydrogeological condition of groundwater pressure and flow

3.2.1 Rock grade

The behavior of intact rock material or blocks is continuous while that of highly fractured rock mass is discontinuous in nature. The engineering properties of rock material and discontinuities should be taken into consideration for any engineering design in rock mass.

The rock grade is determined on the basis of judgment of rock mass data mainly obtained through site geological investigations including core drillings. The rock mass classification systems are essential to form a part of design approaches of the empirical and numerical methods and increasingly used in both design approaches as computing power improves.

Rock grade is chiefly determined on the basis of the strength of cylindrical core sample obtained from the rock mass, and degree of weathering. Rock quality designation (RQD) can be applied to indicate spacing of the discontinuities due to weathering. Rock grade is classified into six states from A to F and described in detail in Table 3.1.

Table 3.1 Rock Grade

Class		Criteria for Judgment	Drilled Core sample	Weathering (Alteration)
A	□	Rock piece cannot be broken easily when struck by hammer, with metallic sound. No deterioration of rock-forming minerals.	Cylindrical *core* recovery and/or RQD are more than 90%. No fragmental piece of rock is recovered.	Fresh (No alteration observed.)
B	⬚	Metallic resonant sound when struck by hammer. Joints are adhered and fresh. Little trace of deterioration of minerals.	Cylindrical *core* recovery and/or RQD are more than 70%. Limited amount of fragmental pieces of rock is recovered.	Slightly (.Alteration of limited portion observed.)
C	▦	Rock often becomes broken when struck by hammer. Rock pieces keep almost intact when broken. Joints are slightly to moderately weathered in general.	Cylindrical *core* recovery and/or RQD are 40% to-70%. Some fragmental pieces of rock is recovered. Cranky in general.	Moderately (Ratio of discoloration is less than half)
D	◈	Broken by hand and slightly penetrated by hammer blow. Joints are generally not clear mainly due to highly weathered condition.	Cylindrical *core* recovery and/or RQD are less than 40%. Fragmental core with sand & clayey materials is recovered.	Highly (Ratio of discoloration is more than half)
E	▤	Broken and/or squeezed by finger, remaining particles of quartz and feldspar. Proportion of broken pieces is generally 30-50% and 20-50% in powder form.	Pebble, sand and clay samples recovered in general. Cylindrical *core* recovered mainly in clayey layer.	Completely (All discolored, Texture of rock can be confirmed)
F	▩	Generally in powder form when crushed by fingers. Proportion of broken pieces is less than 20 to-30% in general.	Cylindrical *core* recovered only in clayey layer.	Residual (No texture of rock confirmed)

3.2.2 Condition of joints

Table 3.2 Condition of joints

Class	Judgment of Criteria
a	Closely contact, no deterioration nor discolored.
b	Cracks are filled with limonite along cracks or very thin clay
c	Cracks are deteriorated and filled with 1 to 2cm thick clay.
d	Open crack

Condition of joints and/or cracks such as orientation, spacing, surface roughness, infilling and weathering is generally recognized as one of the parameters for rock mass classification system. Discontinuities distributed in the rock mass include faults and fractured zone including joints and cracks in general.

Spacing and/or frequencies of joints and cracks are assumed in the excavation surface of the tunnel.

The strength of intact rock mass without discontinuities is supposed to be equivalent to that of the rock constituting the rock mass. However, joints and cracks are generally distributed in the rock mass, and properties of the rock mass depend highly on the discontinuities like joints and cracks.

Condition of joints and cracks are classified into four categories of “a” to “d” from close contact to open, and five stages of their spacing as shown in Table 3.2 and 3.3.

Table 3.3 Spacing of joints

Class	Judgment of Criteria
I	More than 50 cm
II	30 to 50 cm
III	15 to 30 cm
IV	5 to 15 cm
V	Less than 5 cm

3.2.3 Classification of rock mass

The rock mass classification can be assumed on the basis of rock grade and condition of joints and cracks described in Table 3.1, 3.2 and 3.3. Rank S1 to S7 of rock mass classification is proposed on the basis of said parameters in this guideline as shown in Table 3.4 and properties of S1 to S7 are described in Table 3.5. This rock mass classification system is studied in further chapters considering other rock mass classification systems

Table 3.4 Classification of Rock Mass based on combination of Parameters

	A					B					C					D					E					F				
	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV	V
a	S 1	S1	S2	S2	(S2)	S2	S2	S2	S3	(S3)																				
b	S1	S2	S2	S2	(S2)	S2	S2	S3	S3	(S3)	S3	S3	S4	S4	S4	S4	S5	S5	S5	S5	(S5)	(S6)	(S6)	(S6)	(S6)					
c	S2	S2	S2	S2	(S3)	S2	S3	S3	S3	(S4)	S3	S4	S4	S4	S5	S5	S5	S5	S5	S6	S6	S6	S6	S6	S7	S6				
d											S4	S4	S4	S5	S5	S5	S5	S5	S6	S6	S6	S6	S6	S7	S7	S7	S7	S7	S7	S7

( ) Encountered in a limited occasion.

Table 3.5 Classification of Rock Mass and Parameters

Rock Mass Classification	Modulus of Deformation	Shear Strength	Friction Degree	Seismic velocity
	(MPa)	τ (MPa)	Φ (°)	Vp (km/s)
S1	More than 5,000	5.0	More than 50	More than 5.0
S2	3,000 – 5,000	2.5 – 5.0	50	3..5 – 5.0
S3	3,000	1.5 – 2.5	45	2.0 – 3.5
S4	1,000 – 2,000	1.0 – 1.5	40	1.5 – 3.5
S5	500 – 1,000	0.5 – 1.0	35	1.2 – 3.0
S6	250 - 500	0.2 – 0.5	30	0.8 – 2.5
S7	Less than 250	Less than 0.1	Less than 30	Less than 1.5

3.3 Regional geotechnical conditions

3.3.1 In-situ stress

The rock mass at a certain depth is said to be subjected to stresses resulting from the weight of the overlying strata with additional stress from locked in stresses of tectonic origin. However, in-situ stress at a certain depth can be simplified to the weight of the overlying strata.

The stress field is locally disrupted when an opening is excavated in the rock mass, and a new set of stresses are induced in the rock mass surrounding the opening.

Stability and potential failure mode of tunnels and underground rock caverns are directly related to the magnitude and orientation of the in-situ and induced rock stress. In some cases, the high horizontal in-situ stress is essential in maintaining cavern stability, whilst in other cases the high rock stress may cause additional difficulties in tunnel construction and rock support design.

A certain value of in-situ stress intensity (Ta) obtained in the calculation that “the weight of the overlying rock mass at a certain depth (Po)” is divided by “the uniaxial strength of the rock mass at the certain depth (σ)” is proposed to assume the condition of the stress of the rock mass at a certain depth of the rock mass.

Ta = Po /qu,

Po = γ x H

Where:

Ta = in-situ stress intensity

Po = in-situ stress (simplified)

γ = rock unit weight

H = tunnel depth below ground surface

qu = uniaxial compression strength of the rock mass at tunnel depth

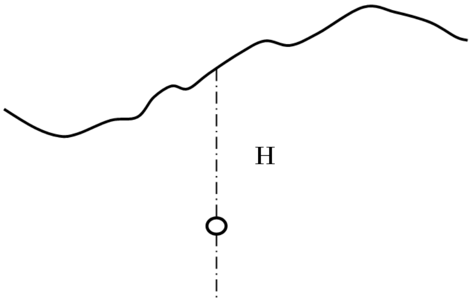


Figure 3.1 Tunnel and In-situ Stress

As a result of analysis based on accumulated data regarding this value of in-situ stress intensity, condition of the stress of rock mass is substantially changed from two (2) to four (4) of the value. Deformability of the underground opening due to in-situ stress appears to be less when the value exceeds four (4), however deformation and squeeze of the rock mass are often observed when the value is below two (2) as shown in Figure 3.2.

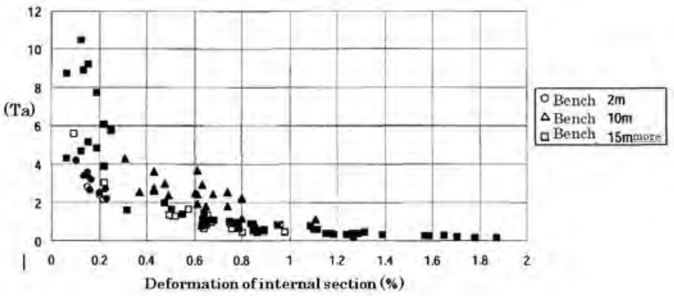


Figure 3.2 Relationship between In-situ Stress and Deformation

3.3.2 Hydrogeological condition

For conventional tunnels, the groundwater table is lowered during the excavation of the tunnel, because the tunnels act as a drain. When the undrained system is established after the final lining is placed, the groundwater table is supposed to be re-established its original position.

Inflow of groundwater to the tunnel is observed during and even after excavation of the tunnel. Natural groundwater level before the excavation is generally lowered after tunnel excavation started and some amount of groundwater flows into the tunnel. Gush of groundwater inflow to the tunnel is also observed in the course of the tunelling work. Steady inflows of groundwater are often recorded after construction of the tunnel.

These phenomena are analysed on the basis of hydrogeology and tunelling works. Some equation to assume quantity of groundwater inflows are introduced, however these equations can be applicable in the case that sufficient reliable geological, hydrogeological and geotechnical data is available.

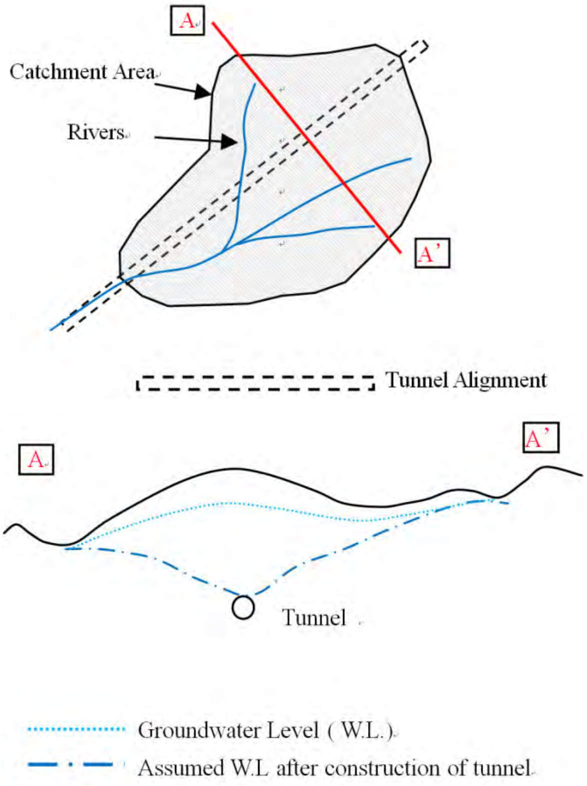


Figure 3.3 Hydrogeology around Tunnel

Hydrogeological condition around the tunnel is also an important factor to carry out a tunnel project. The hydrogeological factor is somehow included to clarify the conditions of joints and cracks which is related to water seepage in the rock mass in the proposed system. A tunnel is acting as drainage in the surrounding area along its alignment, therefore hydrogeological survey along the tunnel alignment is essential in prior to the construction.

3.4 Design of tunnel support

Design of tunnel support is performed generally on the basis of classification of rock mass. Tunnel support of classified rock mass of S1 to S7 is proposed in Table 3.4. This table is subject to change when the classification of rock mass is modified due to changes of actual condition of the site.

Table 3.6 Proposed classification system and Tunnel support

Rock Mass Classification	Shot Creting		Rock Bolting			Steel Support	
	Thickness (cm)	Area	Length (m)	Lateral	Longi -tudinal	Material	Pitch (m)
S1	5	Arch	3	1.5	At randam	---	---
S2	5	Arch	3	1.5	2.0	---	---
S3	10	Arch Wall	3	1.5	1.5	---	---
S4	15	Arch Wall	3	1.5	1.2	(125H)	1.2
S5	15	Arch Wall	4	1.2	1.0	(125H)	1.0
S6	20	Arch Wall	4	1.2	1.0	(150H)	1.0 or less
S7	More than 20	Arch Wall	4	1.0 or less	1.0 or less	(150H)	1.0 or less

4. Proposed system and other classification systems

4.1 Empirical classification system

Several systems are used for the rock mass classification to perform design of tunnels, and RMR and Q systems including other systems developed on the basis of these two systems are frequently used. The main rock mass classification systems of RMR and Q systems make use of similar rock mass parameters. Quality of the same ground is calculated differently in the two systems and several correlations are confirmed in different geology of rock mass. Granitic gneiss is predominantly distributed in the island; therefore the correlation of Diorite can be applied to the Sri Lanka including the project area. (Refer to Figure 4.1 ; RMR = 9 ln Q+ 44)

On the basis of the correlation mentioned above, relationship between the proposed rock mass classification and RMR rating & Q value is supposed to be shown in Table 4.1. Judgement of RMR rating and Q value is shown in Table 4.2.

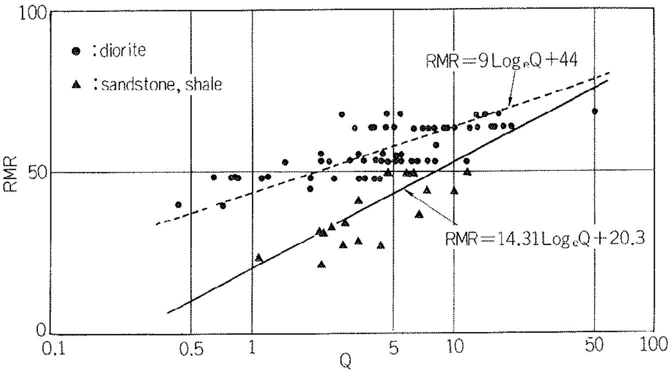


Figure 4.1 Correlation between RMR and Q-system (Diorite)

Table 4.1 Relationship between Proposed System and RMR & Q system

Rock Mass Classification	RMR rating	Q value
S1	85 – 100	More than 100
S2	75 – 85	40 – 100
S3	65 – 75	10 – 40
S4	55 – 65	4 – 10
S5	45 – 55	1 – 4
S6	20 – 45	0.1 – 4
S7	Less than 20	Less than 0.1

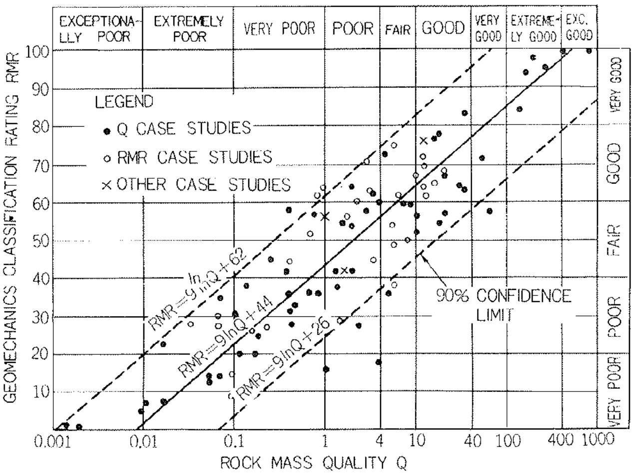


Figure 4.2 Correlation between RMR and Q system

Table 4.2 Judgement of rock mass classification system of RMR method and Q system

RMR Class	I	II	III	IV	V
	Very Good	Good	Fair	Poor	Very Poor
Rating	81 - 100	61 - 80	41 - 60	21 - 40	0 - 20

Q system Class	A			B	C	D	E	F	G
	Exceptionally Good	Extremely Good	Very Good	Good	Fair	Poor	Very Poor	Extremely Poor	Exceptionally Poor
Q value	400 - 1000	100 - 400	40 - 100	10 - 40	4 - 10	1 - 4	0.1 - 1	0.01 – 0.1	0.001 – 0.01

4.2 Numerical modelling method

Numerical analysis is often applied in these days and software products become easier to handle and find broad acceptance and use in geotechnical engineering practice.

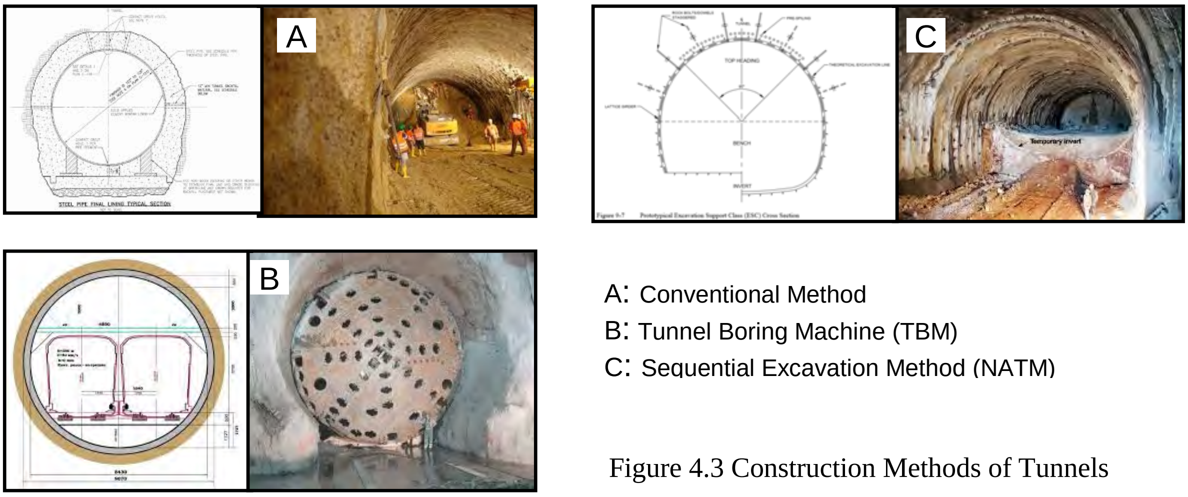
The finite element method (FEM) is a numerical method and commonly applied for solving problems of engineering and mathematical physics. The aim of the finite element method analysis is to verify the empirically evaluated tunnel support design. The FEM software module is often applied to evaluate induced stresses and maximum deformation for excavated portion of the tunnel and to examine the proposed tunnel support.

4.3. Construction Method

A new excavation method for an underground opening is proposed and applied to improve the progress, and safety of works since excavation of underground opening started. Significant progress of excavations of underground openings has been made since blasting method has been applied after dynamite invented.

A conventional method by using drilling and blasting is still applied in some project. Tunnel boring machine (TBM) and sequential excavation method of New Austrian Tunneling Method (NATM) are often applied as a result of latest concept of rock mechanics to improve the progress and safety of the tunneling works in these days. These construction methods are shown in Figure 4.3.

Supporting measures like pre-supports including spiles and forepiling are performed through and ahead of the tunnel face. These recently applied excavation methods are mainly proposed on the basis of latest concept of rock mechanics including rock mass classification system. The excavation method depends highly on the rock mass classification system developed on the basis of the concept of rock mechanics. The rock mass classification system might be developed as the excavation method improves.



A: Conventional Method
B: Tunnel Boring Machine (TBM)
C: Sequential Excavation Method (NATM)

Figure 4.3 Construction Methods of Tunnels

4.4 Survey and monitoring

Monitoring of ground deformations of the tunnel cavern is one of the principal means for selecting the appropriate excavation and support methods for ensuring safety and progress of tunnelling work.

Several types of ground deformation measurements often used in tunnelling works for obtaining the data of ground deformation and their subsequent evaluation. These procedures are essential for efficient tunnelling works. Survey and monitoring methods for tunnels are shown in Figure 4.4.

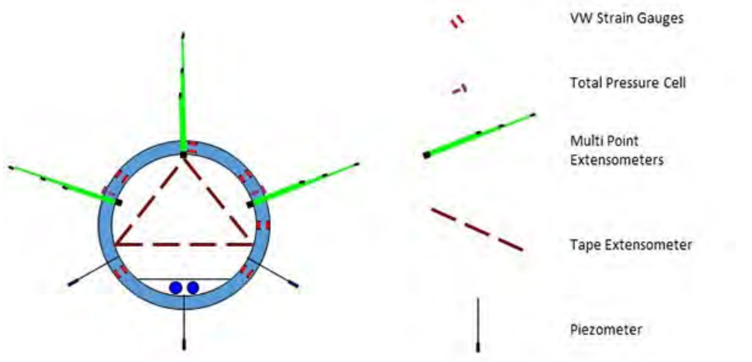


Figure 4.4 Survey and Monitoring of Tunnels

5 Conclusion and recommendation

5.1 Conclusion

The following conclusion can be obtained on the basis of the results of the research on the interaction between tunnel supports and rock mass under the geotechnical condition.

A methodology is proposed to determine the classification of rock mass in Sri Lanka. The classification system consists of the following steps:

- Determination of mechanical parameters of the rock mass
- Transformation of rock mass mechanical parameters into rock mass parameters by using rock mass classification systems
- Parameters shall be studied and optimized by choosing other different systems.
- Set-up of numerical model on the basis of the real geological situation, and perform the simulation by using the model and mechanical parameters.

Compared to other rock mass classification systems, like empirical correlations or simple analytical calculations, numerical modelling offers a much more detailed and physical based insight into the interaction between rock mass and tunnel supports.

Stability of openings of the tunnel and its support system can be confirmed by the applied numerical modelling approach. Realistic parameters can be obtained as s result of numerical modelling analysis for the tunnel and its support system. Consequently, numerical modelling can be used for detailed tunnel supports dimensioning, design and optimization.

5.2 Recommendation

- The proposed rock mass classification system shall be modified on the basis of actual conditions of the excavation surface and additional data obtained in other tunnel and related projects.
- Effective and comprehensive geological and geotechnical investigations are recommended to be carried out to determine the appropriate parameters.
- Numerical simulation of the interaction between rock mass and tunnel supports should be extended towards mathematical based sensitivity analysis and optimization. Further numerical simulations should include the more realistic tunnel supports models.
- In-situ stress of rock mass around the tunnel section shall be considered when design of the tunnel is implemented. The rock mass at a certain depth is said to be subjected to stresses resulting from the weight of the overlying strata with additional stress from locked in stresses of tectonic origin. Stability and potential failure mode of tunnels and underground rock caverns are directly related to the magnitude and orientation of the in-situ and induced rock stress.
- Hydrogeological conditions of the rock mass around the tunnel section often affect the progress of the tunnelling works in the area of thick overburden with abundant groundwater. Therefore, hydrogeological conditions shall be carefully surveyed when tunnelling work is planned to perform in these environment.
- The proposed rock mass classification system is recommended to be modified in the further stage to include parameters which are estimated on the basis of the in-situ stress of rock mass and hydrogeological conditions of the rock mass around the tunnel section.

TECHNICAL NOTE

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Laboratory Testing on Rock Samples for Rock Tunnel Design

By Iromi Ranasoma, Deputy Director, R & D, RDA, and
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1. Introduction

1.1 General

A rock mass is basically composed of intact rock separated by different discontinuities, and therefore its engineering behaviour depends not only on the properties of the intact material and the discontinuities separately, but also on the way they are combined. For this reason, rock properties can be divided into two categories: intact rock properties and rock mass properties.

Laboratory tests on intact rock samples are normally performed in a laboratory. These tests are conducted on relatively small rock core samples, and therefore, they do not consider the overall rock mass properties. Through laboratory tests do not adequately account for the overall properties of the rock mass, the results from these tests, when modified according to rock mass classification and some empirical correlations, provide a basis for engineering design. In addition, large-scale in-situ tests provide data on the properties of a rock mass, however these in-situ tests are often neither practical nor economically feasible.

Accordingly, laboratory testing on rock samples is a vital part of the site investigation and an efficient and accurate laboratory testing program should be developed to provide sufficient information for the completion of a safe and economical design.

This technical note provides a basic guideline for developing a laboratory rock test program relating mainly to rock tunnel projects, and briefly describes typical laboratory tests for rocks, and the interpretation and application of the data obtained from these tests.

1.2 Purpose of Laboratory Testing

Laboratory rock testing is generally conducted to obtain accurate information for engineering design and rock characterization purposes, as detailed below:

1) To identify and classify the rock type through mineralogical and petrographic tests;
2) To obtain the stratigraphy and the physical properties of the intact rocks;
3) To determine the strength and elastic properties of intact rock materials and the potential for the degradation and disintegration of the rock materials;
4) To provide data to estimate the mechanical properties of rock mass, such as rock mass shear strength and deformation modulus; and
5) To provide information for rock mass classification, for example, unconfined compressive strength of intact rocks for RMR and Q systems as well as Japanese ground classification.

1.3 Basic Definition

(1) Intact Rock

Intact rock or intact material refers to the unfractured blocks that exist between structural discontinuities. The intact rock may consist of only one type of mineral but more commonly it contains a variety of minerals. The intact rock blocks range from a few millimetres to several meters in size.

(2) Discontinuity

Discontinuity as a collective term is defined as the whole range of mechanical defects formed in rock mass, such as joints, bedding planes, foliation planes, fissures, shears, faults, etc. The mechanical properties of a discontinuity depend on the material properties (hardness, weathering) of the intact rock itself, its geometry (roughness), its genesis (tension or shear types), infilling materials, etc.

(3) Rock Mass

Rock mass is defined as the rock material together with the three-dimensional structure of discontinuities. The rock mass properties depend chiefly on the geometry (direction, length, spacing, position, etc.) and conditions (separation, weathering, infilling material, etc.) of discontinuities.

2. Planning a Laboratory-Testing Program

2.1 Laboratory Testing Program

The extent or scope (type and number) of laboratory testing for a tunnel project will vary, generally depending on as follows:

1) Tunnel type and function (road tunnel, motorway tunnel, railway tunnel, headrace tunnel, temporary tunnel, etc.),
2) Size and shape of the tunnel (length, height, width, or diameter),
3) Design requirements of the project structures,
4) Availability of pre-existing laboratory and in-situ tests data,
5) Further in-situ testing program,
6) Thickness of overburden above the planned tunnel alignments
7) Vertical and horizontal geological variations along the tunnel alignment
8) Known or suspected geological structures (i.e., fault, fold, shear zone, highly fractured zone, etc.)
9) Known or suspected geological hazards (landslide, slope instability, seepage zone, swell rock zone, etc.)
10) Project stage (for example, plan, preliminary design, detailed design, construction)
11) Project schedules and budgets
12) Property data needed for specific design procedures (i.e., nearby structures, etc.)
13) Critical tolerances for the proposed tunnel (e.g., tunnel invert settlement limitation, deformation limit, etc.)

Specifying unnecessary laboratory tests will add time and cost to the project and consume samples. Laboratory tests on rocks range from rock material index properties including density and mineralogy (thin-section analysis), to intact core mechanical properties including uniaxial compressive strength, tensile strength, static elastic constants, hardness, and abrasivity index.

Each project is unique; there is no fixed standard list that can be used to determine the scope of laboratory testing on rock. Table 1 lists common laboratory rock tests that may be applicable to tunnel designs, with priority as defined below:

1) High priority – these test items should be conducted as main, routine ones; and
2) Low priority – these test items should be conducted if budget and time is available.

In general, a laboratory testing program should be developed based on the discussion between geotechnical expert and tunnel designer on a case-by-case basis, including the type and item of testing methods, the quantity and quality of rock samples, and the index and mechanical parameters of rock materials required. Index property tests are not specifically used in tunnel design but are invaluable in establishing general conditions and assessing inherent variabilities of rock materials.

Table 1 Common Laboratory Tests for Rock

Parameter	Test Method	ASTM Standard	Priority
Index properties	Specific Gravity	C97	Low
	Density	D2216	High
	Moisture Content	D2216	High
	Slake Durability	D4644	Low
	Point Load Index	D5731	High
	Hardness	D5250	Low
Strength Properties	Uniaxial compressive strength	D2938	High
	Triaxial compressive strength	D2664	Low
	Tensile strength (Brazilian)	D3967	High
	Shear strength of joints	D5607	High
Deformability	Young's modulus, Poisson's ratio (uniaxial)	D7102	High
	Young's modulus, Poisson's ratio (triaxial)	D5407	Low
Time dependence	Creep characteristics in hard rock (uniaxial)	D4341	Low
	Creep characteristics in soft rock (uniaxial)	D4405	Low
Permeability	Coefficient of permeability	D4525	Low
Mineralogy	Thin section petrographic analysis	C295	High
	X-ray diffraction	D4926	Low

It should be noted that not every test listed in Table 1 above is applicable to every tunnel project. Engineering judgment should be used to properly set up a laboratory testing program to provide the information required for each individual tunnel project.

2.1 Core Sampling Requirements

It is necessary to select rock core samples for laboratory tests that will accurately characterize the project site. The frequency of rock core sampling depending on the size of the design tunnel and the complex of the project geology. In heterogenous areas, many samples may be required to obtain comprehensive parameters; in homogeneous areas, few samples may be required. A laboratory testing program should be performed on representative and critical specimens from all geologic layers across the project site. In addition, a sample diameter of more than 54.7 mm is recommended for rock mechanical laboratory tests on rock core samples.

3. Procedures and Methods of Routine Laboratory Rock Tests

Laboratory testing on rock samples is for determining intact material properties, such as strength, elasticity, deformation, etc. The following sections briefly describe procedures of some routine tests, as given in Table 1 above.

3.1 Absorption and Bulk Specific Gravity Tests

1) This test is performed in accordance with ASTM C 97.
2) Absorption is a measure of the amount of water a dry specimen can absorb during a 48-hour soaking period. The amount of absorbed water is indicative of the porosity of the sample.
3) Bulk specific gravity is used to calculate the unit weight of the material.

3.2 Unit Weight Test

1) This test is performed in accordance with ASTM D 2216.
2) This test is performed to measure the total/moist or oven-dried unit weight of a rock core sample.
3) Samples should be preserved and tested at the moisture content representative of field conditions.

3.3 Point Load Strength Index Test

1) This test is performed in accordance with ASTMD5731.
2) This test can be easily and inexpensively performed in the field or the laboratory in a short time
3) Rock core samples are typically tested diametrically (load applied along the axis of the diameter) preferably with a length-to-diameter ratio of 1.0. Irregular rock fragments and lumps can also be tested, with some preparation.
4) Tests can be performed both parallel and perpendicular to inherent planes of weakness within the rock mass. Size corrections are applied to obtain the point load strength index, $I_{s(50)}$, of a rock specimen.
5) The test results can be used to approximately estimate uniaxial compressive strengths (UCS or q_u) and determine strength classification of rock materials. It has been found that the UCS is about 20 to 24 times the $I_{s(50)}$, with a value of 24 commonly used, i.e., UCS = 24 $I_{s(50)}$.

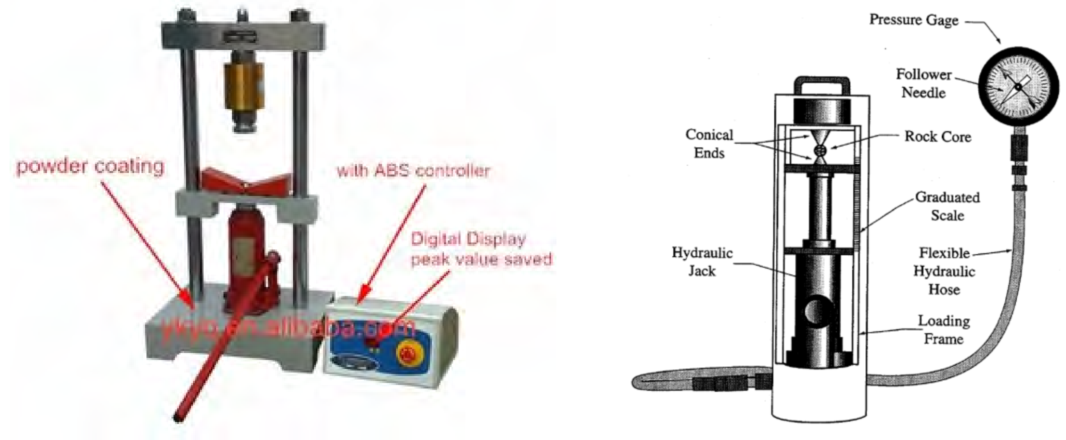


Figure 1 An Example of Laboratory Rock Point Load Apparatus

3.4 Unconfined Compression Test

1) This test is performed in accordance with ASTM D 2938.
2) This test is performed to obtain intact rock strength – an important parameter of intact material.
3) This test is performed on intact rock core specimens, preferably with L/D (length-to diameter) ratio of more than 2.0. The specimen is placed in the testing apparatus and loaded axially at an approximately constant rate such that failure occurs within 2 to 15 minutes.

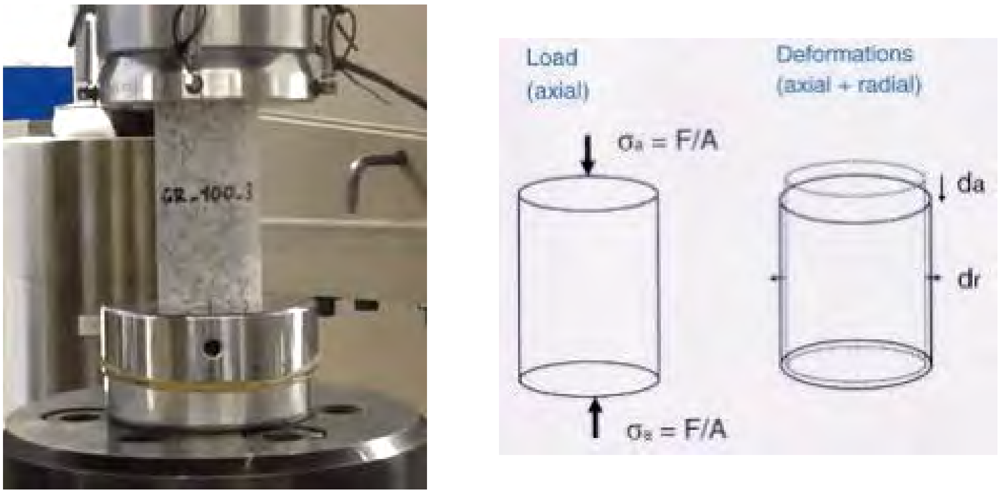


Figure 2 An Example of Rock Unconfined Compression Test Apparatus

3.5. Splitting Tensile Strength Test

1) This test is performed in accordance with ASTM D 3867.
2) This test is performed to obtain the indirect (Brazilian) tensile strength of intact material. Although rocks are much weaker in tension than in compression or shear, tensile failure also plays an important role in some engineering activities (e.g. drilling, cutting and blasting of rocks). Tensile behaviour of different rock formations can vary considerably, and neglecting such a parameter may overestimate the efficiency of the formation.
3) This test is an indirect tensile strength test like the point load test; however, the compressive loads are line loads applied parallel to the core's axis by steel bearing plates between which the specimen is placed horizontally. The specimen preferably has a thickness to diameter ratio between 0.2 to 0.75. Loading is applied continuously such that failure occurs within one to ten minutes.
4) The splitting tensile strength of the specimen is calculated from the results.

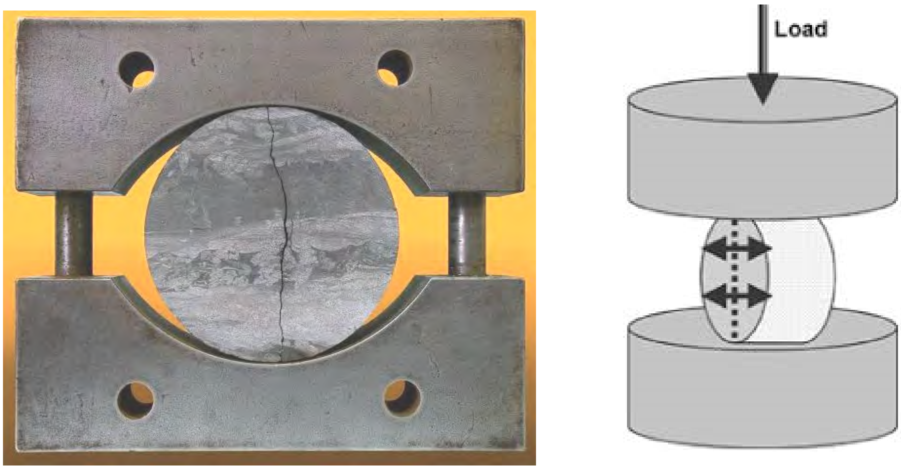


Figure 3 An Example of Splitting Tensile Strength Test Apparatus

3.6 Direct Shear Test

1) This test is performed in accordance with ASTM D 5607.
2) This test is performed to determine the shear strength of a rock discontinuity. Test results are used for geotechnical stability analysis of underground opening and portal slope.

Figure 4 An Example of Direct Shear Test Apparatus for Rock Discontinuity

3) The test is similar in concept to the direct shear test for soils. The rock sample is grouted in the lower platen of the shear box. The line of force is directed to act along the discontinuity in the rock. Irregularities in the surface of the discontinuity can create misleading results because the mode of failure may vary to that in the field.
4) An asperity in the discontinuity surface results in either shearing or overriding of the asperity, depending on the normal load applied.

3.7 Uniaxial Compressive Test (Elastic Moduli)

1) This test is performed in accordance with ASTM D 7102.
2) This test is performed to determine the stress-deformation characteristics of rock, calculate the elastic modulus, and to evaluate the suitability of the rock to support structure foundations.
3) In contrast to a conventional unconfined compression test (D2938), the strain for each loading step must be determined by using load and deformation transducers and then the elastic modulus is calculated.

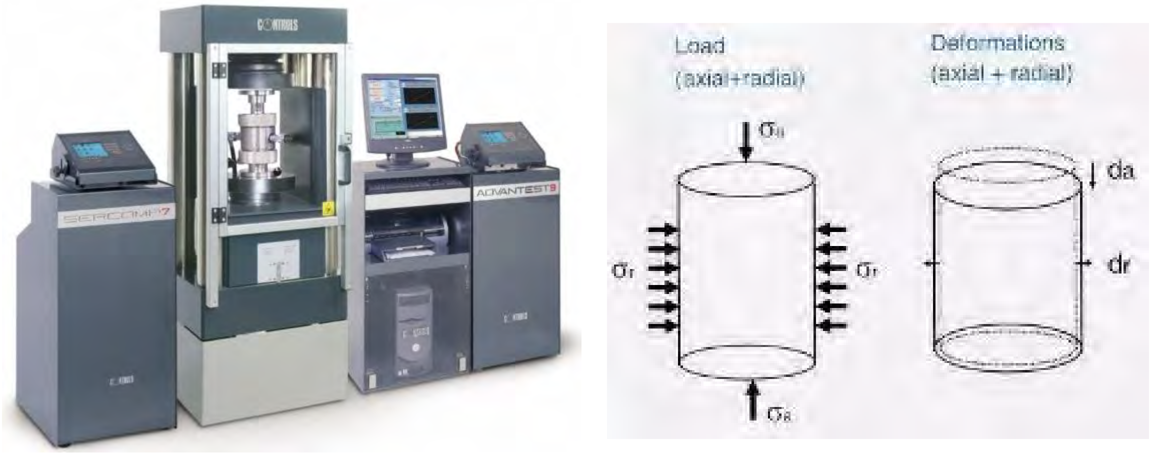


Figure 5 An Example of Stress Path Triaxial and Uniaxial Compression Test System

3.8 Thin Section Petrographic Analysis

1) This test is performed in accordance with ASTM C 952.
2) Thin section petrographic analysis is used to evaluate the mineralogy of the intact rock cores. A detailed analysis of minerals, by optical mineralogy in a thin section, is performed via a petrographic microscope. The micro-texture and structure reveal the origin of the rock.
3) In a petrographic analysis, a slice of rock is affixed to a microscope slide and then ground so thin that light can be transmitted through mineral grains. The extreme thinness of the section enables the various minerals to be distinguished according to their behaviour in transmitted light.
4) Polarizing filters within the petrological microscope produce crossed polarized light in which the crystals affect the light path to produce characteristic interference colors. This feature, together with other properties such as refractive index, crystal shape, and texture, enable almost all minerals and rock types to be identified.

4. Interpretation and Application of Laboratory Rock Testing

Interpretation of laboratory rock testing is how to use and assess the intact rock property data to establish the final rock mass parameters to be used for geotechnical design of a tunnel. The final rock mass properties required for design should be based on the results from the subsurface investigation, the in-situ testing, and the laboratory testing, used separately or in conjunction.

A combination of laboratory testing of small samples, field observations and empirical analysis should be performed to determine the engineering properties of rock mass for the design and construction of a tunnel.

4.1 Intact Rock Property Characterization

The intact properties of rock material are generally determined by either laboratory rock testing on small core samples or published results, mainly including below:

1) Specific gravity;
2) Unit weight;
3) Compressive strength (or unconfined compressive strength);
4) Elastic properties (e.g., ultrasonic velocity, modulus, Poisson's ratio); and
5) Tensile strength.

The rock material property data are commonly used to establish rock mass parameters for rock tunnel design.

(1) Specific Gravity

1) The specific gravity of solids (Gs) of different rock types depends upon the minerals present and their relative percentage of composition.
2) The bulk value of these together gives a representative average value of Gs =. 2.7 ± 0.1 for many rock types (refer to Figure 6).

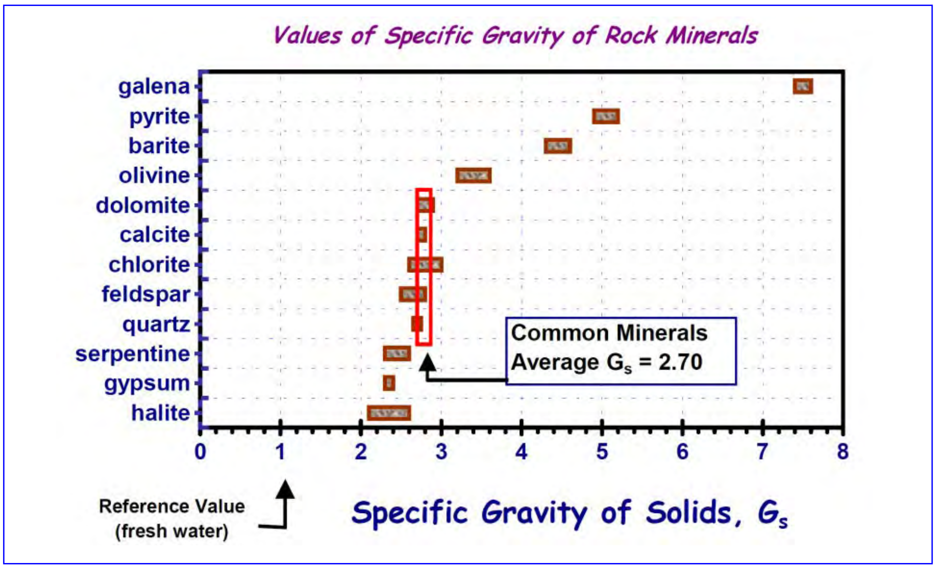


Figure 6 Specific Gravity of Solids for Selected Rock Minerals

(2) Unit Weight

1) The unit weight of a rock material is required in calculating overburden stress profiles related to rock slope stability and tunnel support design.
2) The unit weight of a rock material is an indicator of the degree of induration of the rock unit and is thus an indirect indicator of rock strength.
3) Strength of the intact rock material tends to increase proportionally to the increase in unit weight. Representative dry unit weights for different rock types are contained in Table 2.
4) The dry unit weight ((dry) is calculated from the bulk specific gravity of solids and porosity (n).

Table 2 Representative Range of Dry Rock Material Unit Weight

Rock Type	Unit Weight Range (kN/m^3)
Shale	20 - 25
Sandstone	18 - 26
Limestone	19 - 27
Schist	23 - 28
Gneiss	23 – 29
Granite	25 - 29
Basalt	20 - 30

(3) Compressive strength

1) The stress-strain-strength behaviour of intact rock specimens can be measured through either a uniaxial compression test (unconfined compression), or the more elaborate triaxial test.
2) The peak stress-strain curve during unconfined loading is the uniaxial compressive strength (designated q_u).
3) The value of q_u can be estimated from the point load index (Is) that is easily conducted in the field or laboratory.
4) Representative values of compression and tension strengths for a variety of intact rock specimens are listed in Table 3 (Goodman, 1989). For this database, the compressive strengths ranged from 11 to 355 MPa, showing a wide range in compressive strength.

Table 3 Representative Range of Unconfined Compressive and Tensile Strengths

Intact Rock Material	Unconfined Compression (MPa)	Tension Strength (MPa)
Baraboo Quartzite	320.0	11.0
Bedford Limestone	51.0	1.6
Solenhofen Limestone	245.0	4.0
Tavernalle Limestone	97.9	3.9
Cherokee Marble	66.9	1.8
Taconic Marble	62.0	1.2
Cedar City Tonalite	101.5	6.4
Dworshak Dam Gneiss	162.0	6.9
Lockport Dolomite	90.3	3.0
Oneota Dolomite	86.9	4.4
Palisades Diabase	241.0	11.4
Quartz Mica Schist	55.2	0.5
Nevada Granite	141.1	11.7
Pikes Peak Granite	226.0	11.9
John Day Basalt	355.0	14.5
Nevada Basalt	148.0	13.1
Nevada Tuff	11.3	1.1
Berea Sandstone	73.8	1.2
Navajo Sandstone	214.0	8.1
Hackensack Siltstone	122.7	3.0
Flaming Gorge Shale	35.2	0.2
Micaceous Shale	75.2	2.1

Source: Modified after Goodman, 1989.

(4) Elastic Modulus

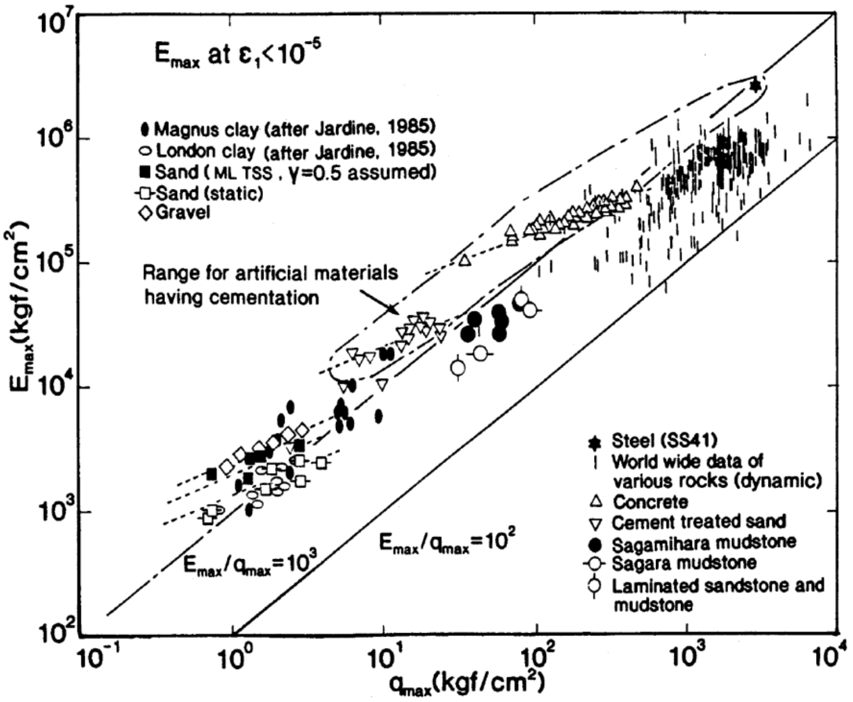
1) The Young's modulus (E_R) of intact rock is measured through either uniaxial compression or triaxial compression loading. Most common in engineering practice, the tangent value taken at 50% of ultimate strength is reported as the characteristic elastic modulus (E_{R50}).

2) Intact rock specimens can exhibit a wide range of elastic modulus. For many sedimentary and foliated metamorphic rocks, the modulus of elasticity is generally greater parallel to the bedding or foliation planes than perpendicular to them, due to closure of parallel weakness planes.
3) An intact rock classification system based on modulus ratio (E_t/q_u, where E_t is tangent modulus at 50% ultimate strength and $q_{a(ult)}$ is the uniaxial compressive strength) is given in Table 4.
4) A global database of E_{max} from small-strain measurements (ultrasonics, bender elements, resonant column) versus the compressive strength ($q_{max} = q_u$) shows a wide range of civil engineering materials ranging from soils to rocks, as well as concrete (Figure 7).

Table 4 Engineering Classification of Intact Materials Based on Modulus Ratio

Class	Description	Modulus Ratio
H	High modulus ratio	Over 500
M	Average (medium) ratio	200-500
L	Low modulus ratio	Less than 200

Source: Deere and Miller, 1966; Stagg and Zienkiewicz, 1968.



Source: Tatsuoka & Shibuya, 1992

Figure 7 Small-Strain Elastic Modulus versus Compressive Strength for Some Intact Materials

(5) Tensile strength

1) Rock is generally relatively weak in tension, and thus, the tensile strength of an intact rock is considerably less than its compressive value, as illustrated in Figure 8.
2) The tensile strength is usually obtained through indirect methods, including the split-tensile test (Brazilian test, or alternatively, a bending test to obtain the modulus of rupture.
3) A list of representative tensile strength values for various rocks is given in Table 3 above with a measured range from 0.2 to 14 MPa.
4) The tensile strength mostly averages only about 4% of the compressive strength for the same rock (refer to Figure 9).

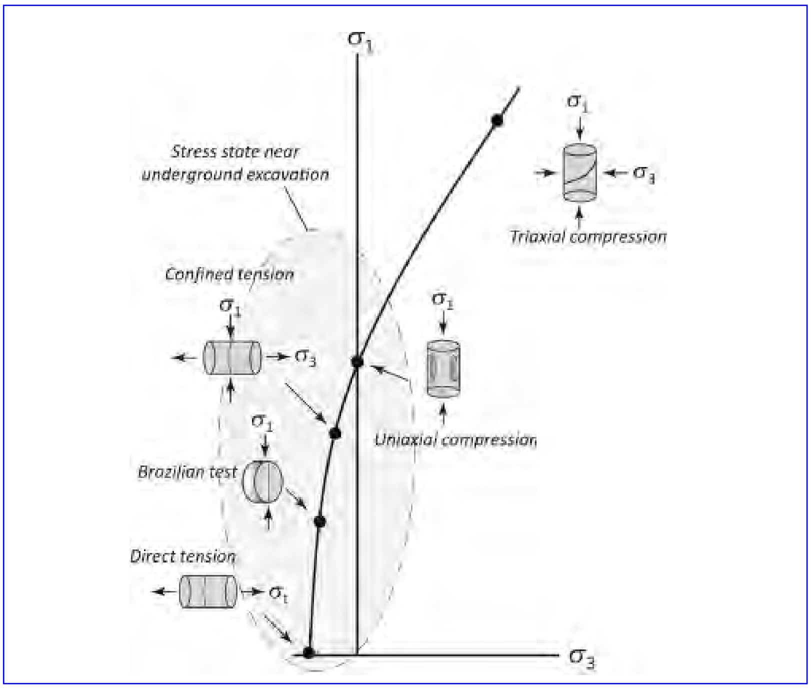


Figure 8 Interrelationship among Triaxial, Uniaxial Compression and Tensile Strength

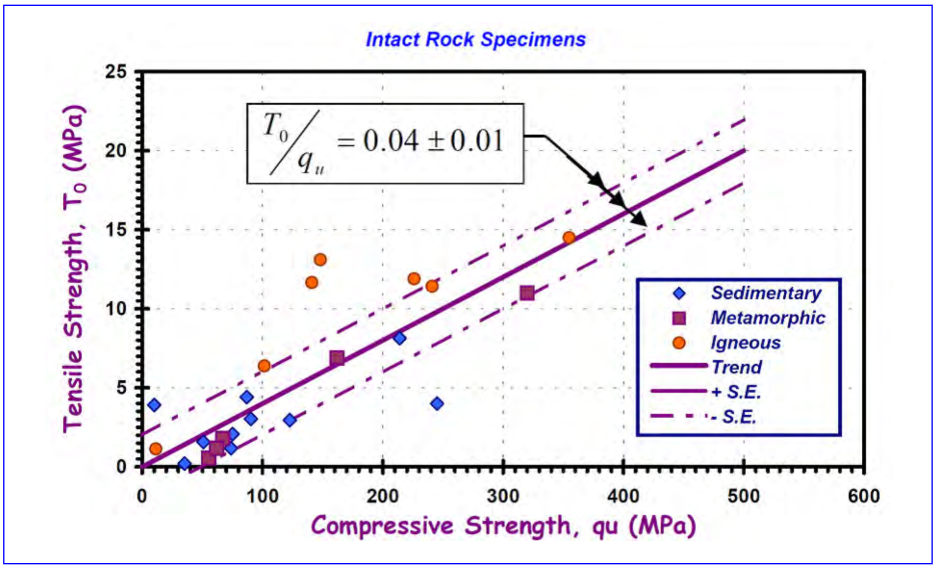


Figure 9 Comparison of Tensile vs. Compressive Strengths for Intact Rock Specimens

4.2 Intact Rock Strength

The strength of the intact rock plays a decisive role in the selection of excavation methods and has a strong bearing on stability of excavated opening. Three conditions of stability can be identified and classified following the competence factor, F_c=σ_{ci}/σ_v (where σ_{ci} is intact rock strength and σ_v is maximum vertical stress):

1) $F_c > 10$, the intact rock has a much greater strength than the vertical stress in the rock mass and the excavation is likely to be stable.
2) 10.>F_c >2, the Stability of opening is conditioned by time and rock properties, three types of deformation can be identified: elastic, plastic and brittle failures including the risk of rock burst.
3) $F_c < 2$, the excavation may be unstable as stresses exceed intact rock strength.

Estimating stability from F_c does not consider the presence of discontinuities and high horizontal stress. Although not common, this situation may occur in very homogenous rock masses, e.g., massive crystalline rock and rock salt, or in rock at great depth where the discontinuities are tightly closed.

4.3 Rock Mass Classification

The classification of rock mass is a process of rock mass characterization. Various classification systems have been developed. Several most widely used classifications include Rock Mass Rating (RMR) system, Rock Tunneling Quality Index (Q) system, and Japanese Ground Classification. These systems combine the effects of intact rock unconfined compressive strength (UCS), RQD, discontinuities conditions and groundwater conditions, for example, RMR is calculated according to the following:

RMR = C + RQD + Js + Jcd + JwR + Jo

C:	Rating (range: 1 to 15) of intact rock strength, which is a numerical value associated the point load test index and unconfined compressive strength of intact rock from laboratory test.
RQD:	Rating (range: 3 to 20) of RQD, which is a numerical value associated with the rock mass RQD (the rating is not equal to the RQD value).
Js:	Rating (range: 5 to 20) for joint set spacing (spacing <60 mm to >2 m), which is a numerical value associated with the fracture spacing of a given joint set.
Jcd:	Rating (range: 0 to 30) of joint condition, which is a numerical value associated with the condition of joints including the length, separation, roughness of joints, hardness and thickness of infilling materials, and weathering of joint wall.
JwR:	Rating (range: 0 to 15) of groundwater, which is a numerical value dependent on the inflow rate and pressure of groundwater.
Jo:	Rating (range: -12 to 0) for joint orientation, which is a numerical value associated with the orientation of critical joint from very unfavorable to very favorable joints relative to the orientation of tunnel and powerhouse axis.

4.4 Rock Mass Properties for Geotechnical Design

The rock mass properties are normally used for the design of tunnel supports rather than the intact properties of a rock sample. The two properties of primary interest for geotechnical design are rock mass strength and rock mass deformation modulus. A practical method of determining the two rock mass properties is empirical correlation - the Hoek-Brown Method (2002 Edition).

(1) General Form of Hoek-Brown Failure Criterion

The general form of Hoek-Brown failure criterion is:

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

σ_1', σ_3':	major and minor principal effective stresses
m_b:	Hoek-Brown constant for rock masses
s, a:	parameters describing rock mass properties
σ_{ci}:	uniaxial compressive strength of the intact rock

The rock mass parameters *mb*, *a* and *s* can be derived by means of the following parameters and shown as follows:

- Hoek-Brown constant for intact rock, m_i
- Geological Strength Index, *GSI*
- Disturbance Factor, *D*

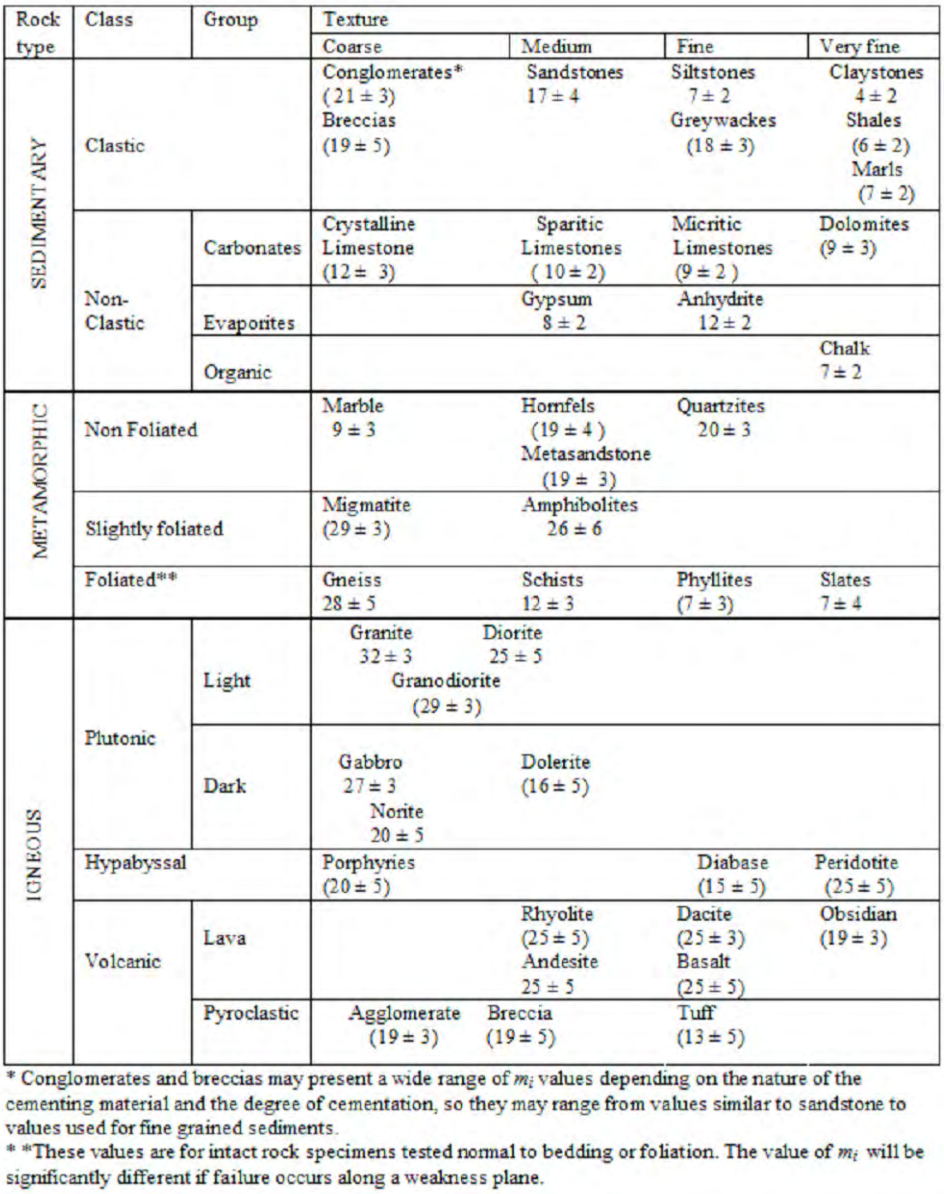
$$m_b = m_i \times \exp\left(\frac{GSI-100}{28-14D}\right)$$

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

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

(2) Intact Rock Hoek-Brown Parameters

The intact rock constant, *mi*, will be estimated from the best-fit curve fitting of the confined compression and tensile strength data, namely by using the relationship between the ratio of unconfined compression to tensile strength (R) and m_i. In case of no laboratory test results, the Hoek-Brown constant m_i is determined according to m_i Chart (Figure 10).



Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates* (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6		
	Foliated**		Gneiss 28 ± 5	Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	Obsidian (19 ± 3)
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

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

* *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.

Figure 10 Values of Constant m_i for Intact Rock (Hoek, 2002)

However, it has been reported that the published parameter, mi value, can be misleading as mi does not appear to be related to rock type, and that the Hoek-Brown criterion can be generalized by allowing the exponent to vary; and this change results in a better model of the experimental data (Mostyn1 G and Douglas K.G., 2000). In addition, Richards L. and Read S. (2011) pointed out that the most accurate method of assessing mi values remains as statistical analysis of data from a full set of laboratory test results, including triaxial and unconfined compression plus tensile tests within the recommended stress range (i.e. σ t < σ 3′ < 0.5 σci). In the absence of a full suite of laboratory tests, R, the ratio of unconfined compressive strength to tensile strength, is a useful indicator of mi values, particularly as the tests for both properties are relatively straightforward to perform.

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 he 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

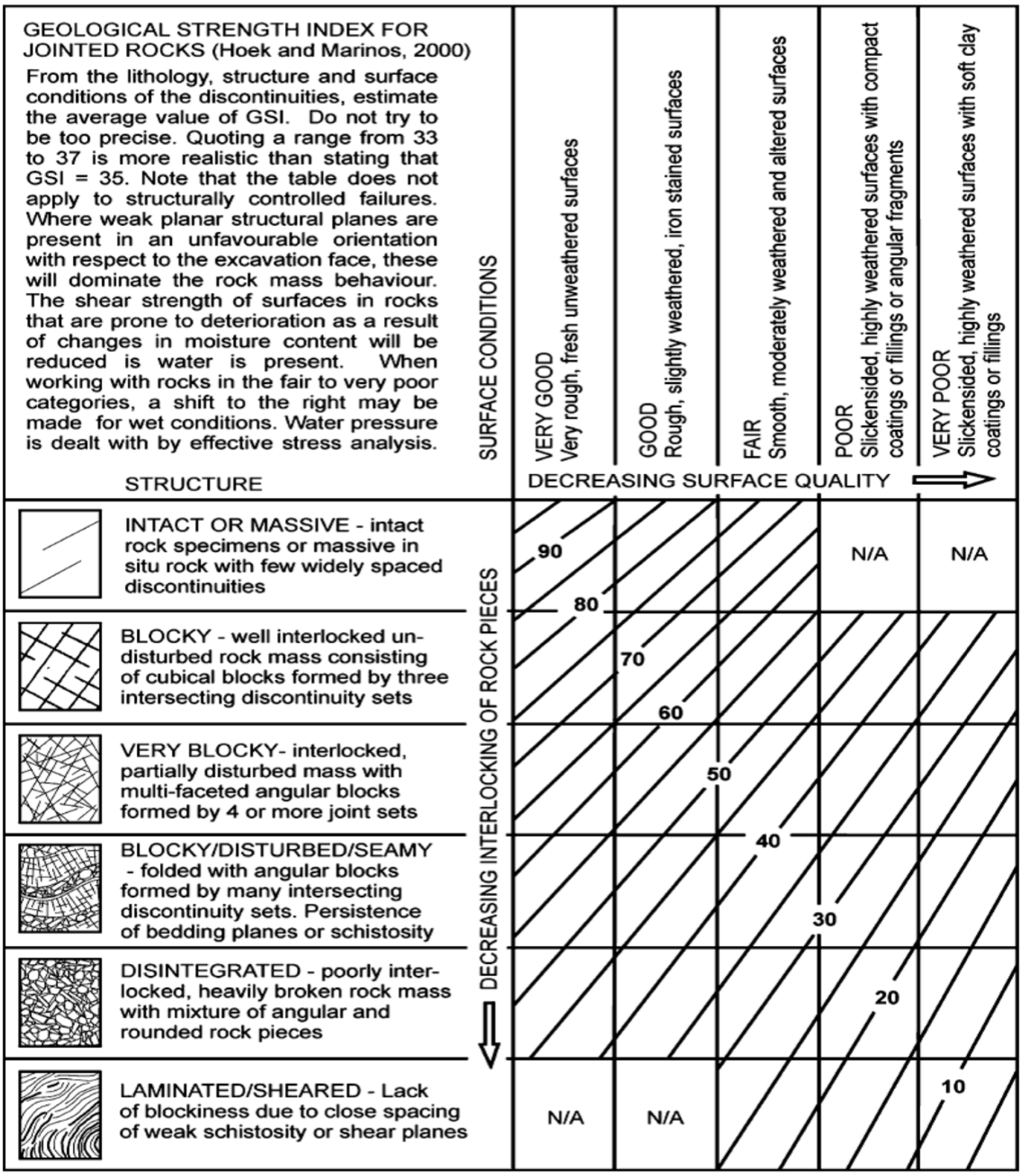
Figure 11 Guidelines for Estimating Disturbance Factor D (Hoek, 2002)

Furthermore, the Disturbance Factor, D, which depends upon the degree of disturbance to which the rock mass is subjected by blast damage and stress relaxation, varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses. In this project, the excavation programs will be carefully designed, and therefore the D values for the calculation of rock mass mechanical parameters are assumed to be 0 to 0.5 according to Guidelines for the selections of D (Figure 11 above).

(3) Geological Strength Index (GSI)

The Geological Strength Index (GSI) is an input parameter to derive the Hoek-Brown rock mass strength. The GSI, which provides a numerical rating of the rock masses based on the structure and surface of the rock mass, is obtained from the field observation, as shown in Figure 12 and Table 5 below.

In practice, the GSI is based mainly upon the visual impression of the rock mass structure (number of discontinuities, block geometry, lithological foliation and shear) and the surface condition of the rock discontinuity (roughness, infilling, weathering and alteration). The GSI is estimated from the contours in Figure 12; and an incremental range of 10 for the GSI estimate (e.g. 0-10, 10-20, 20-30…) is used as shown in Table 5.


GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek and Marinos, 2000)

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 is 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.

Figure 12 Diagram Estimation of GSI (Hoek and Brown, 1997)

Table 5 General Chart for GSI Estimate from Field Geological Observation

Geological Strength Index		**Surface Conditions of Discontinuities**				
		Very good	Good	Fair	Poor	Very poor
Rock Block Structure	Intact	90-80	80-70	70-60	none	none
	Blocky	80-70	70-60	60-50	50-40	40-30
	Very blocky	70-60	60-50	50-40	40-30	30-20
	Blocky/Disturbed	60-50	50-40	30-20	30-20	20-10
	Disintegrated	50-40	40-30	30-20	30-20	20-10
	Laminated/Sheared	none	none	30-20	20-10	10-0

(4) Rock Mass Peak Strength

The Hoek-Brown criterion serves to derive the Mohr-Coulomb parameters φ' and *c*' by the following equations.

$$\phi' = \sin^{-1}\left[\frac{6am_b(s+m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\sigma'_{3n})^{a-1}}\right]$$

$$c' = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\sigma'_{3n}](s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1+(6am_b(s+m_b\sigma'_{3n})^{a-1})/((1+a)(2+a))}}$$

Where, $\sigma'_{3n} = \sigma'_{3\max} / \sigma_{ci}$

The value of $\sigma_{3\max}$ is the upper limit of confining stress over which the relationship between the Hoek-Brown and the Mohr-Coulomb criteria is considered. In addition, Mohr-Coulomb failure criterion *c* (cohesion), and φ (friction angle), are dependent on the stress range over which the criterion is applied. The stress range selected will vary depending on the intended use of the data. In this project, according to the Hoek-Brown Method (2002 Edition), for underground structures Mohr-Coulomb failure criterion in the Hoek-Brown uses "tunnel" case where the stress range selected is between rock mass tensile strength and the maximum minor principal stress anticipated at the depth of tunnel or underground powerhouse.

(5) Rock Mass Deformation Modulus

Similarly, according to the Hoek-Brown Method (2002 Edition), the rock mass modulus of deformation is obtained by:

$$E_m = \left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \times 10^{(GSI-10)/40} \qquad \sigma_{ci} \leqq 100\ MPa)$$

$$E_m = \left(1-\frac{D}{2}\right) \times 10^{(GSI-10)/40} \qquad \sigma_{ci} > 100\ MPa$$

where E_m is the rock mass modulus of deformation in *GPa*.

(6) Rock Mass Residual Strength

Cai et al. (2007) extend Geological Strength Index (GSI) system for estimating the residual strength of a rock mass by adjusting the peak GSI to the Residual Geological Strength Index (GSIr) value in consideration of two major controlling factors in the GSI system — the residual block volume and the residual joint condition factor. The residual geological strength and Hock-Brown constants of a rock mass can be calculated as follows (Cai et al., 2007):

$$GSI_r = GSI \times e^{-0.0134GSI}$$

$$a_r = \frac{1}{2} + \frac{1}{6}(e^{-GSI_r/15} - e^{-20/3})$$

$$m_{br} = m_i \exp\left[\frac{GSI_r - 100}{28}\right]$$

$$s_r = \exp\left[\frac{GSI_r - 100}{9}\right]$$

Where,

GSI:	Peak GSI
GSI_r:	Residual GSI
m_i:	Hoek-Brown constant for intact rock
m_{br}:	Residual Hoek-Brown constant for rock mass
s_r, a_r:	Residual Hoek-Brown parameters for rock mass

As shown in the above equations, once GSIr is obtained, the residual Hoek-Brown parameters and constants for rock mass can be calculated using the above Equations.

5. Technical Recommendations

1) Laboratory testing program, as the vital portion of a site investigation should be intelligently planned but flexible enough to be modified as the site investigation and laboratory test progress. An ideal laboratory testing program will provide the tunnel engineer with sufficient data to complete an economical and safe design of the proposed, yet not tie up laboratory personnel and equipment with superfluous testing.

2) All laboratory tests on rocks should be conducted in accordance with international standard test methods, such as ASTM (American Society for Test Materials), AASHTO (American Association of State Highway and Transportation Officials System), and ISRM (the International Society for Rock Mechanics). Laboratory tests following international standard test methods can provide more accurate test results and can be used to establish some correlations between test results.

3) The geology along a tunnel alignment generally plays a dominant role in many of the major decisions that must be made in planning, designing, and constructing a tunnel. Site investigation program, mainly including core drilling, in-situ and laboratory tests, and hydrogeological survey, should begin very early in the conceptual planning of any tunnel project and continue through construction and even after construction to document the as-built conditions and the behaviour of the tunnel in operation.

4) US National Committee on Tunnelling Technology (USNC/TT) recommended that the cost of site investigations (or geotechnical investigations) be increased between 3% and 8% of the total cost of the tunnel construction. Site investigations including laboratory testing program should be carried out as possible as more and earlier. No matter what the final magnitude of cost, there is a need to fund more site investigations and laboratory testing program at earlier stages of a tunnel project. The cost for laboratory testing is always insignificant compared to the cost of an over-conservative design.

5) Geotechnical database, which are obtained through laboratory rock tests in site investigation of all tunnel project should be established to develop a variety of physical models, geotechnical model and hydrogeological model for the classification and characterization of rock masses around tunnels.

MINISTRY OF HIGHER EDUCATION AND HIGHWAYS
ROAD DEVELOPMENT AUTHORITY

TECHNICAL ASSISTANCE FOR
IMPROVEMENT OF CAPACITY FOR
PLANNING OF ROAD TUNNEL

TECHNICAL NOTE

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Hydro-geological Aspects of Road Tunnels
-Impact on groundwater environment by tunneling work-

by B.G.N. Tharanganie, Senior Design Engineer, Bridge Designs Office, RDA
and Dr. Shigeo Suizu, expert in Environmental Impact of Study (Groundwater)

1 Introduction

This project, Technical Assistance for Improvement of Capacity for Planning of Road Tunnels, is underway to improve the road tunnel planning capacity of Road Development Authority (RDA). "Guideline for Environmental Impact Study (Groundwater)" is one of the project output. The preparatory work for the guideline is currently taking place. This technical note describes the overview of the guideline, the investigation method and the forecasting method of the groundwater environment impacted by the tunnelling work.

2 Process of Environmental Impact Assessment (EIA)

This technical note is to provide how to investigate the groundwater environment and how to forecast the groundwater environment after the tunnelling work in order to take measures not to affect the environment. The purpose and the process of technical work are same as the process of Environmental Impact Assessment (EIA). The investigation and forecast methods are explained according to the work process of the EIA.
The work process of EIA is shown in Fig. 2.1. Scoping is the very important stage of planning the EIA, specifically to determine target areas, evaluation factors of EIA and the method of investigation and forecast in EIA. Therefore, prior the planning, project characteristics are understood well, regional outline investigation is carried out and the methods of investigation and forecast are deeply considered.
Consideration is made by the order of the method of evaluation, forecast and investigation in the scoping stage and the order is reverse of the actual EIA work.

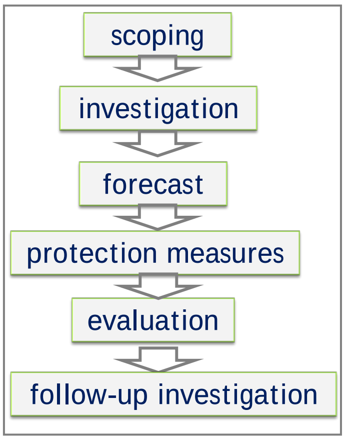


Fig. 2.1 Work process of EIA

3 Concept of Groundwater EIA

3.1 Classification of groundwater

Groundwater is classified into three types of pore water, fissure water and cavern water from the void structure of the ground. The first two types of schematic structures are shown in Fig. 3.1. Pore water fills between particles of gravel, sand, silt, weathered rock, and generally spreads horizontally. The more uniform and larger the particle size, the higher the fluidity. Fissure water is in the fissure or joint or crack opened in the rock. The flow of fissure water is local but it varies greatly depending on the place and the magnitude of fissures and their connected state.

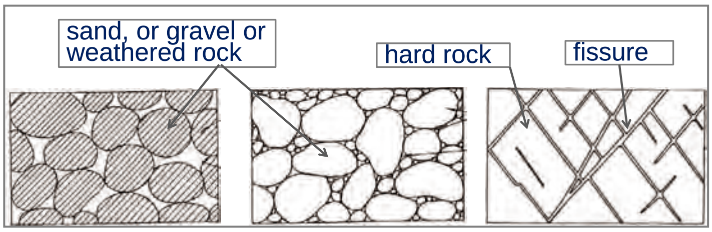


Fig. 3.1 Schematic structure of void containing groundwater

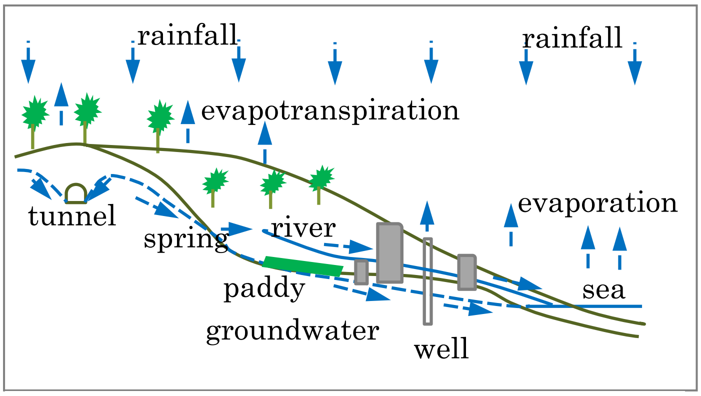


Fig. 3.2 Schematic diagram of hydrological cycle

3.2 Characteristics of groundwater

Water exists in various areas such as vapor, rain water, river water, lake water, groundwater, sea water etc. on the earth. They always circulate by processes such as evapotranspiration, precipitation, flow, infiltration, etc. to other areas as shown in Fig. 3.2. In order to grasp changes in groundwater, not changes in groundwater only but water balance terms such as rainfall, surface water and water use and their change processes must be analysed as a "system of the hydrological cycle". The excavation of a tunnel may cause the decrease of the groundwater, surface water and spring water as a direct impact as shown in Fig.3.3. Chain resulting from the decrease may cause land subsidence, surface displacement, vegetation change, etc.

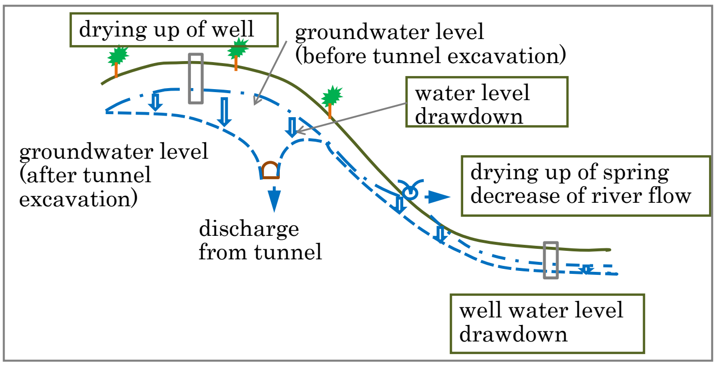


Fig. 3.3 Schematic diagram of influence on hydrological cycle by tunnel excavation

4 Forecast methods

The forecast method of groundwater environment is explained prior to the explanation of investigation method according to the work process in the scoping. The several methods to forecast the groundwater environment after tunnelling work are shown in this chapter. Careful consideration needs to choose the forecasting method because the required condition, the accuracy and the application conditions of the methods are greatly varied.

4.1 Forecasting by the use of existing similar case

This method is to collect many similar examples, arranging and analyzing them, and forecasting the groundwater discharge into the target tunnel. A part of the result in Japan summarizing the results of 273 tunnels is shown in table 4.1 as an example.

Table 4.1 Discharge per unit length into tunnel by geology
(unit: m3 / min / km, the value in brackets indicates fractured rock)

Geology		Range of specific discharge	Average of specific discharge
Volcanic rock, volcanic crash rock		0.035～0.9 (0.85～～10.0)	0.30 (3.71)
Plutonic rock including gneiss)		0.018～0.84 (0.17～3.80)	0.20 (1.38)
Paleozoic, Mesozoic		0.0～0.95 (0.10～4.50)	0.17 (0.79)
Tertiary ～ Pleistocene	Sand and gravel	0.02～3.6	0.87
	Sandstone, Shale, Tuff	0.014～0.95	0.25
	mudstone	0.0～0.26	0.07

This table cannot be applied to Sri Lanka because the environmental condition is different. It needs to be compiled similar information from tunnels in Sri Lanka. No source of the information has been found so far although the effort to gather the information is being made.

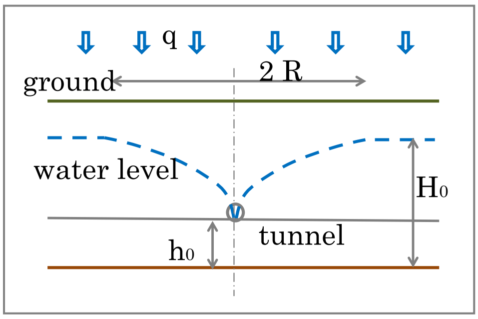


Fig. 4.1 Schematic diagram of groundwater draw down

4.2 Simplified calculation by using hydraulic formula

There are various types of discharge formulas into tunnels based on the hydraulic formulae but most are to estimate the discharge volume by inputting the range of groundwater drawdown. Those that can estimate groundwater drawdown ranges are shown here. Schematic diagram in equilibrium stage of groundwater drawdown is shown in Fig. 4.1 and the explanation of variables and the formulae are shown in Table 4.2.

Table 4.2 Groundwater drawdown range and discharge into tunnel in equilibrium stage based on hydraulic formulae

name	groundwater drawdown range (*R*)	discharge into tunnel in equilibrium stage (Q)
Bear	$R = \frac{1}{\sqrt{2}}\left(\frac{k}{q}\right)^{1/2} H0\left\{1-\left(\frac{h0}{H0}\right)^{2}\right\}^{1/2}$	$Q = \frac{k\,(H0^2 - h0^2)}{2R}$
Nishigaki et al.	$R = 1.22\left\{\left(\frac{k}{q}\right)^{1/2} - 1\right\} H0\left\{1-\left(\frac{h0}{H0}\right)^{2}\right\}$	$Q = \frac{0.72k(H0^2 - h0^2)\left(\frac{k}{q}\right)^{-0.35}}{H0}$
variables	*R:* range of draw down *k* : hydraulic conductivity of aquifer *q* : rain infiltration through ground *H0* : thickness of aquifer *h0* : thickness between bottom of tunnel and bottom of aquifer	

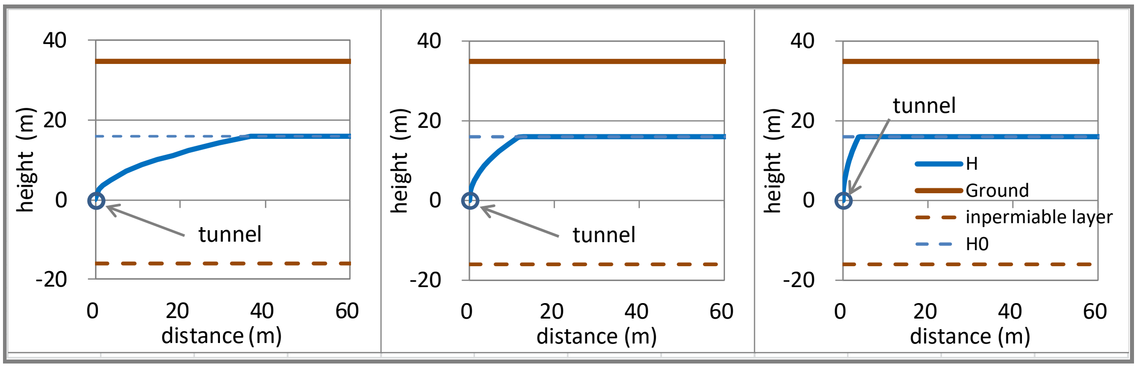


Fig. 4.2 calculation result of groundwater draw down by the use of Bear's equation

An example of the calculation result by the use of Bear's equation is shown in Fig. 4.2. The values of coefficient of permeability (hydraulic conductivity) are different, 0.5 x 10^{-3}, 0.5 x 10^{-4} and 0.5 x 10^{-5} m/min from the left. The drawdown ranges are 36.5, 11.6 and 3.7m and the discharges per unit length from the tunnel are 5 x 10-3, 16 x 10-3 and 52 x 10-3 m^2/min. It shows that the drawdown range and discharge from the tunnel vary according to the permeability of the groundwater.

4.3 Takahashi's method

Takahashi's method was devised for railroad tunnel excavation in Japan. Although it can be calculated by the use of easily available data, it is highly accurate. It has been used frequently in Japan. This method was developed focusing on the similarity of the river in the dry season and the tunnel as shown in Fig. 4.3. The groundwater above and around the tunnel under the equilibrium state is shown in the right figure. It is assumed that this equilibrium state of groundwater is the same as the water in the river in the dry season shown in the left figure.

The explanation of the calculation method is long, so it is not shown here. Necessary parameters for the calculation are as follows; discharge in the river near the tunnel in dry season, shape and elevation of the catchments above the tunnel (readable from topographic map). By adding geological information, the accuracies of tunnel discharge and groundwater drawdown range are improved.

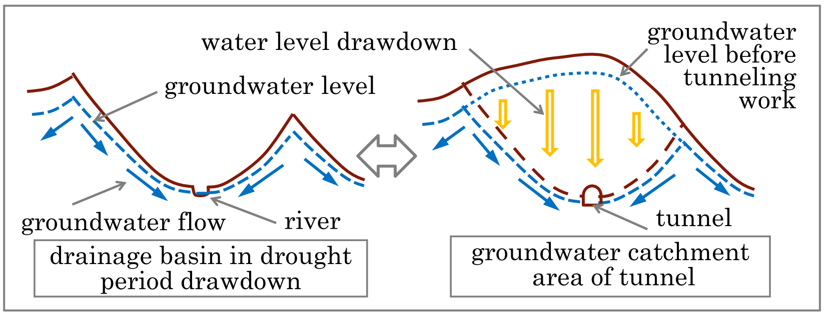


Fig. 4.3 Resemblance of drainage basin in dry season and groundwater catchment area of tunnel

4.4 Tank model

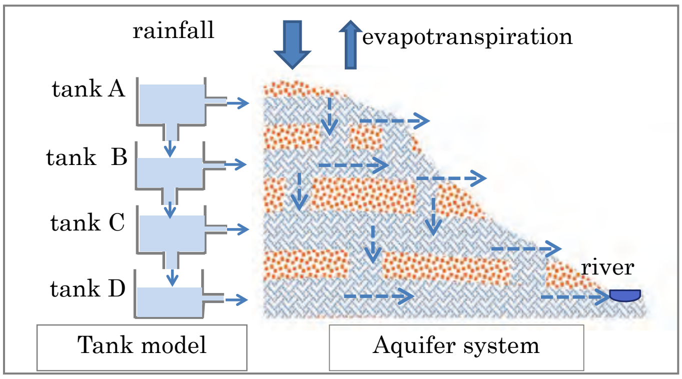


Fig. 4.4 Correspondence of aquifers and tanks in Tank Model

The tank model was developed for analysis of river runoff but it can be applied to the groundwater. The tank model has a structure in which several water storage tanks are arranged in a series in the vertical direction as shown in Fig. 4.4. Tanks are considered to correspond to the aquifer structure in the drainage basin and the groundwater infiltration is modelled by the flow down from the tank. Outlets as surface water discharge are formed on the side of the tanks and outlets indicating infiltration into the lower aquifer are formed on the bottom of the tanks. The outflow amount from the outlet is proportional to the depth of water above the outlet and the diameter of the outlet hole in the tank. Water entering into each tank flows out as surface water and flows into the lower tank as groundwater and the water balance is preserved.

The parameters of the tank model are the diameters and heights of the outlet holes. The data of the balance of daily rainfall and evapotranspiration are input and the parameters are adjusted to reduce the difference between calculated discharge and observed discharge in order to finalize the parameters.

After the parameters are finalized, the outlet for the tunnel with appropriate diameter is added to the appropriate tank with appropriate height as shown in Fig. 4.5. Then it can provide the surface discharge, discharge through the tunnel and groundwater levels after tunneling work. Fig. 4.6 and Fig. 4.7 show the difference of discharges and groundwater levels respectively as an example. They show that surface discharge and groundwater levels decrease after the tunneling work.

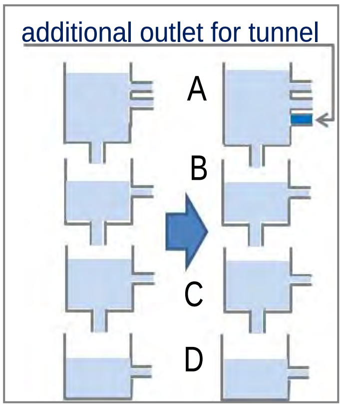


Fig. 4.5 Tank model for tunnel discharge

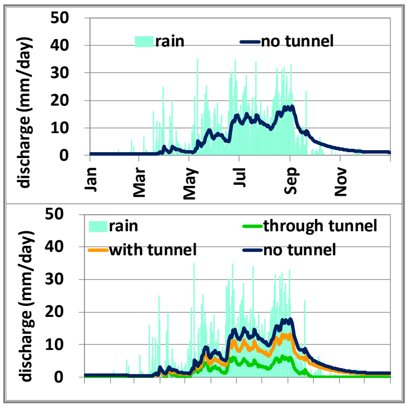


Fig. 4.6 Discharge difference without or with tunnel

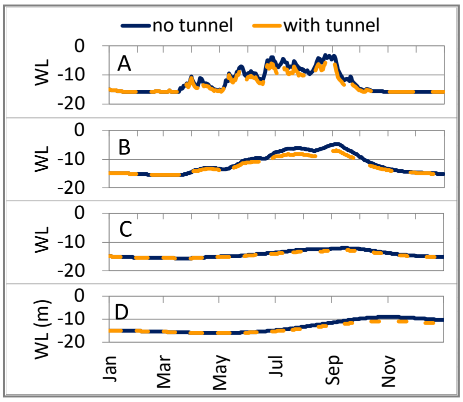


Fig. 4.7 Groundwater level difference without or with tunnel

4.5 Numerical analysis by the groundwater simulation

Due to accumulation of knowledge and improvement of computer performance in recent years, the possibility of application of groundwater flow simulation is increasing year by year. Various models and software have been developed and are available. Various parameters indicating characteristics of the aquifer such as permeability, storage coefficient, depth of aquifer are necessary in order to use the simulation model. Most of the data are obtained by drilling and aquifer capacity test and they are often costly.

In case of selecting the simulation, it is necessary to clarify the characteristics of the model, to examine the purpose of the simulation, available data and cost so on before selecting an appropriate model.

5 Investigation

There are three types of investigations in EIA process as explained in chapter 2. These are regional outline investigation, EIA investigation and follow-up investigation.

5.1 Regional outline investigation

The purpose of regional outline survey is to create the plan of the EIA, including the determination of target areas, method and evaluation factors of the EIA. Regional outline investigation is carried out mainly by gathering existing information, simple field investigation and interview. Investigation factors cover natural and social environment. Investigation factors and contents are shown in Table 5.1. This table is general, not necessarily to investigate all factors. Additional factors to be considered necessary are investigated.

Table 5.1 Factors and contents of outline regional investigation

Investigation factors		contents
natural environment	hydrogeology (groundwater)	water level, water flow, catchment area, water quality etc. (groundwater storage / flow situation)
	surface water, spring	location, water level, discharge, water quality etc.
	soil, ground	vegetation, covering condition, permeability etc.
	terrain, geology	topographic map, geological map, hydrogeological map, aerial photograph, satellite image etc.
	meteorology	Rainfall amount, air temperature, evapotranspiration amount etc.
	other	factors affected by hydrological cycle (animal and plant distribution, ecosystem)
social environment	population, industry	residential area, industries affecting groundwater etc.
	land use	land utilization related to vegetation and land cover etc.
	groundwater use	domestic water, agricultural water, industrial water etc.
	surface water use	intake water volume of intake facility etc.
	facilities easy to be affected	existing water source wells, existing water intake facilities, etc.
	laws and regulations	laws and regulations related to groundwater and surface water etc.
	other	artificial facilities related to hydrological cycle etc.

5.2 EIA investigation

EIA investigation is the main investigation in the EIA work. Its purpose is to clarify the existing environment quantitatively and to acquire the information for forecasting and setting of environmental protection measures. Investigation factors and contents of EIA are shown in Table 5.2.

Table 5.2 Factors and contents of EIA investigation

Investigation factors	contents
meteorology	rainfall amount, air temperature, evapotranspiration amount etc.
surface water, spring	location, water level, discharge, water use, water quality etc.
land use	land utilization related to vegetation and land cover etc.
geology	geological structure (sediment, rock quality, stratification, joint, fissure, fault etc.), soil, weathering of rock, etc.
soil	soil distribution
well	groundwater depth and the flow condition of groundwater, water use, water quality etc.
hydrogeology (groundwater)	extent and depth of aquifer, extent and depth of impermeable layer, location of water in fissure, hydraulic parameters of aquifer etc.
other factors affected by hydrological cycle	animal and plant distribution, ecosystem etc.

The hydrogeological information is the most important, because it directly shows the groundwater environment and is used to forecast the groundwater environment after the tunnelling work. Hydrogeological information is buried underground. We cannot see them and it is difficult to gather precise and accurate information basically. The hydrogeological information is obtained mainly by two methods. One is by the use of seismic prospecting or electric sounding. The other is drilling or the aquifer test in the borehole. Since seismic prospecting and electric sounding obtain data from a remote place through a medium, only average information on a certain volume can be obtained. Drilling and the aquifer test using borehole provide the accurate and high resolution information. The number of drilling is limited because of its cost and it cannot be guaranteed how far the same condition spreads. It is possible to advance hydrogeological investigation efficiently by compiling the results of geological investigation, seismic prospecting, electric sounding, etc. and determining the drilling sites.

5.3 Investigation during and after tunnel drilling

When tunnel drilling starts, the groundwater is affected. Therefore it is important to continue monitoring some factors of the EIA survey.

The purpose of investigation during and after tunnel drilling is as follows;

- to monitor directly how groundwater is impacted
- to improving the forecast
- to judge whether the predicted impact is within the forecast range
- to clarify whether environmental protection measures are fully functioning and showing effects
- to add and review environmental protection measures as necessary, in the case that a remarkable environmental impact exceeding the forecasting result is confirmed

Factors and contents of investigation during and after tunnel drilling are shown Table 5.4.

Table 5.4 Factors and contents of investigation during and after tunnel drilling

Investigation factors	contents
meteorology	rainfall amount, air temperature, evapotranspiration amount etc.
surface water, spring	location, water level, discharge, water use, water quality etc.
geology	geological characteristics in the tunnel
well	groundwater depth and the flow condition of groundwater, water use, water quality etc.
hydrogeology (groundwater)	discharge through tunnel wall, water level, water quality, water use
other factors affected by hydrological cycle	Vegetation, ground subsidence, animal, ecosystem etc.

6 Conclusion

By pursuing environmental protection, the value of projects that benefit people can be further increased. In order to protect the environment, it is necessary to have more accurate forecasts and effective investigations to indicate the environment accurately and to provide useful information for the forecast. Therefore, scoping, in which the investigation method and prediction method are determined, is important. EIA must be well planned in scoping stage after clarifying project characteristics and natural and social situations and considering the method of forecast and investigation deeply.

添付資料　2

地形測量結果

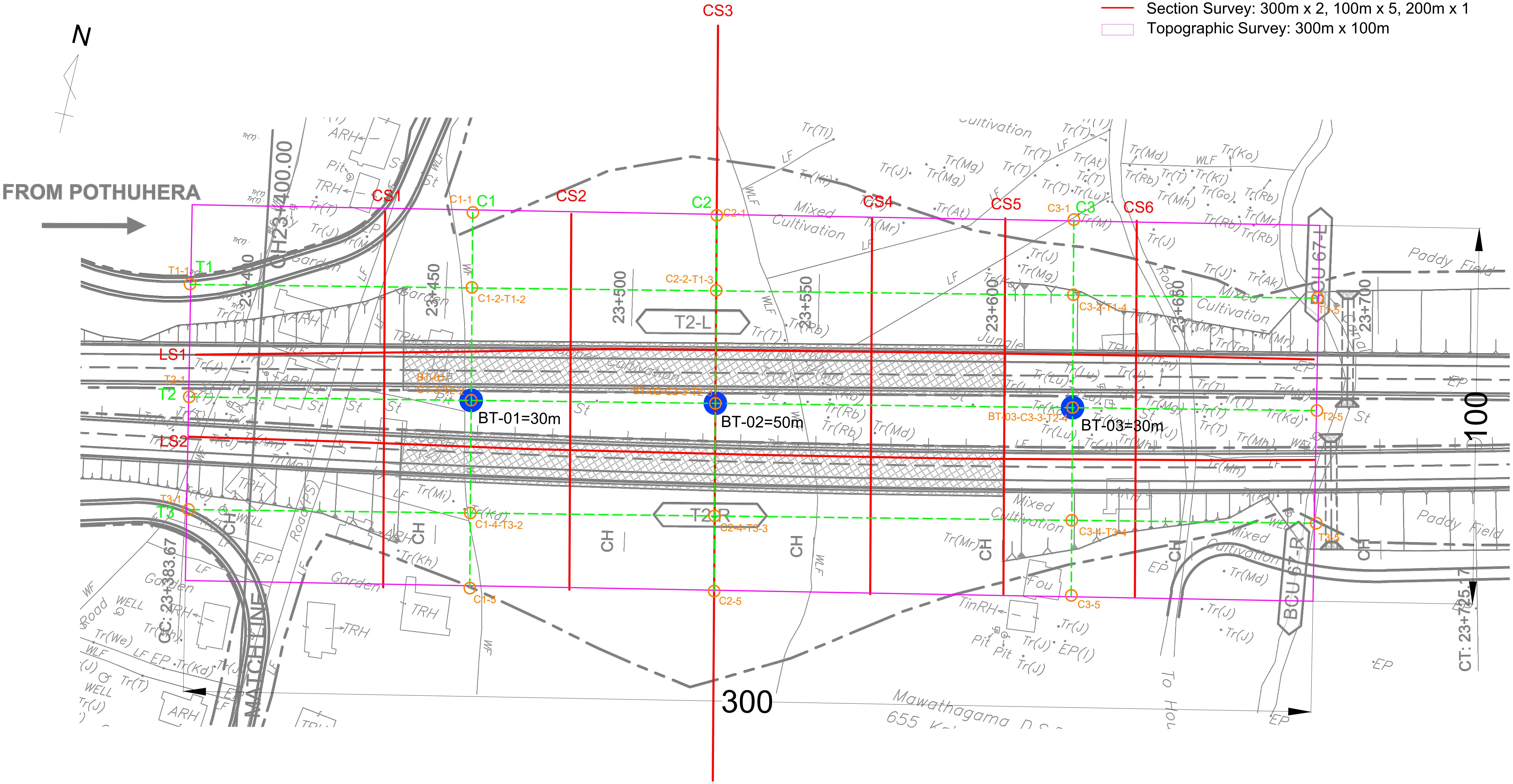
Arrangement of Survey Lines

🟩 Seismic Prospecting: 300m x 3, 100m x 3

● Drilling: Center 50m, Tunnel Portal 30m x 2

🟥 Section Survey: 300m x 2, 100m x 5, 200m x 1

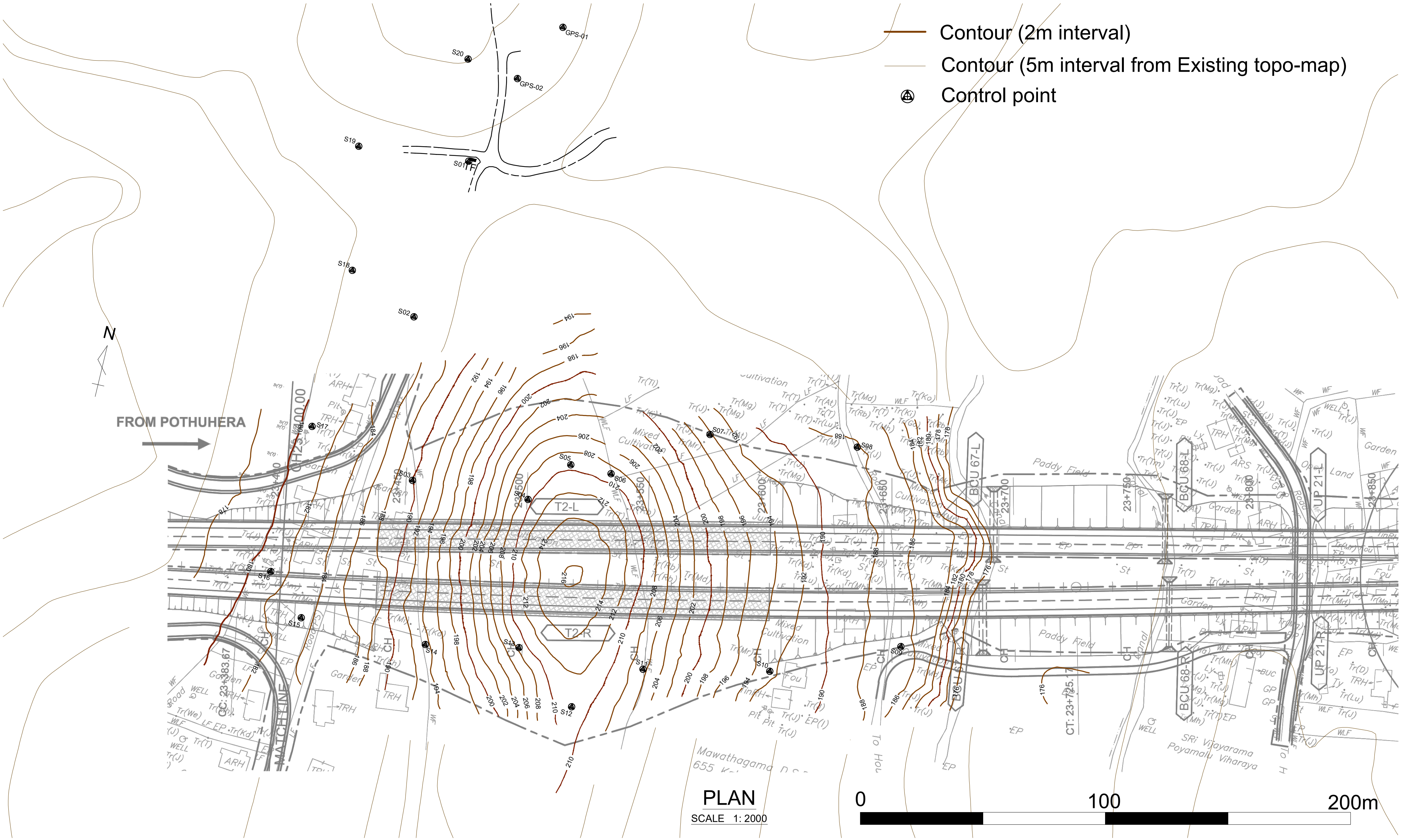
☐ Topographic Survey: 300m x 100m



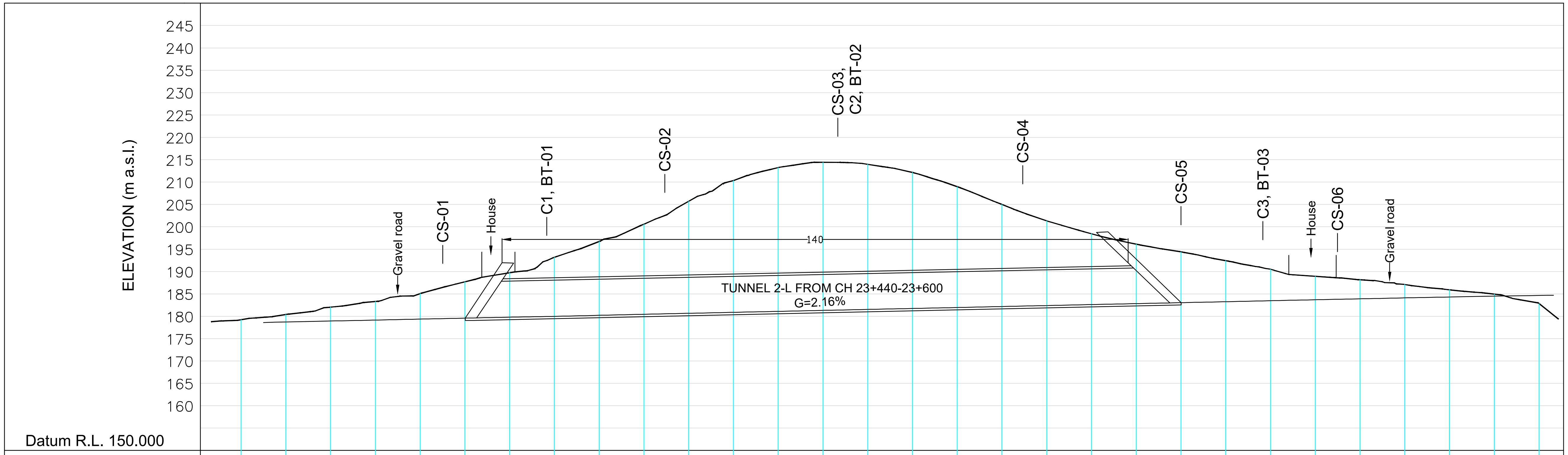
PLAN

SCALE 1: 1000

Topographic map and control points in pilot site



SECTION SURVEY (LS-1)



GROUND ELEVATION (m a.s.l.)	179.23	180.39	182.03	183.31	185.06	187.70	189.72	193.18	196.72	200.58	205.71	210.33	213.24	214.42	213.97	212.15	208.97	205.00	201.32	198.44	196.13	194.40	192.42	190.51	188.93	188.17	187.10	185.90	184.95	182.84
ROAD CHAINAGE	23+390.0	23+400.0	23+410.0	23+420.0	23+430.0	23+440.0	23+450.0	23+460.0	23+470.0	23+480.0	23+490.0	23+500.0	23+510.0	23+520.0	23+530.0	23+540.0	23+550.0	23+560.0	23+570.0	23+580.0	23+590.0	23+600.0	23+610.0	23+620.0	23+630.0	23+640.0	23+650.0	23+660.0	23+670.0	23+680.0

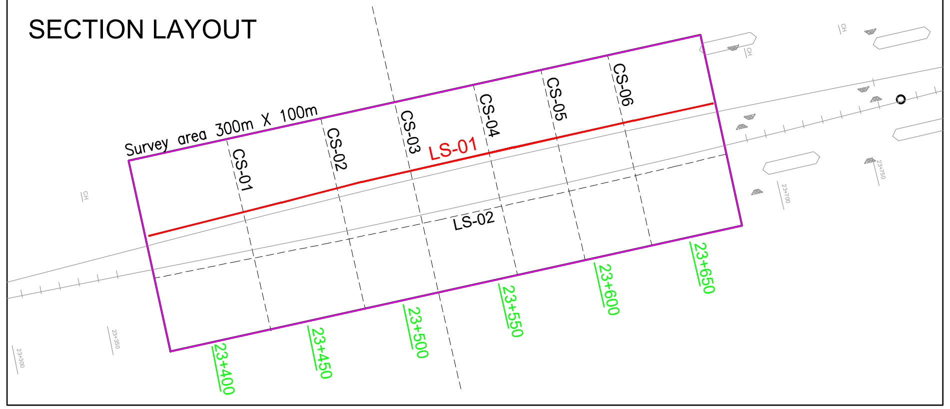
Horizontal Scale: 1:1000

Vertical Scale: 1:1000

LS - 01

LONGITUDINAL SECTION

SECTION LAYOUT

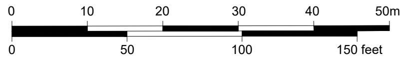


Surveyed from 14th to 25th July 2017.

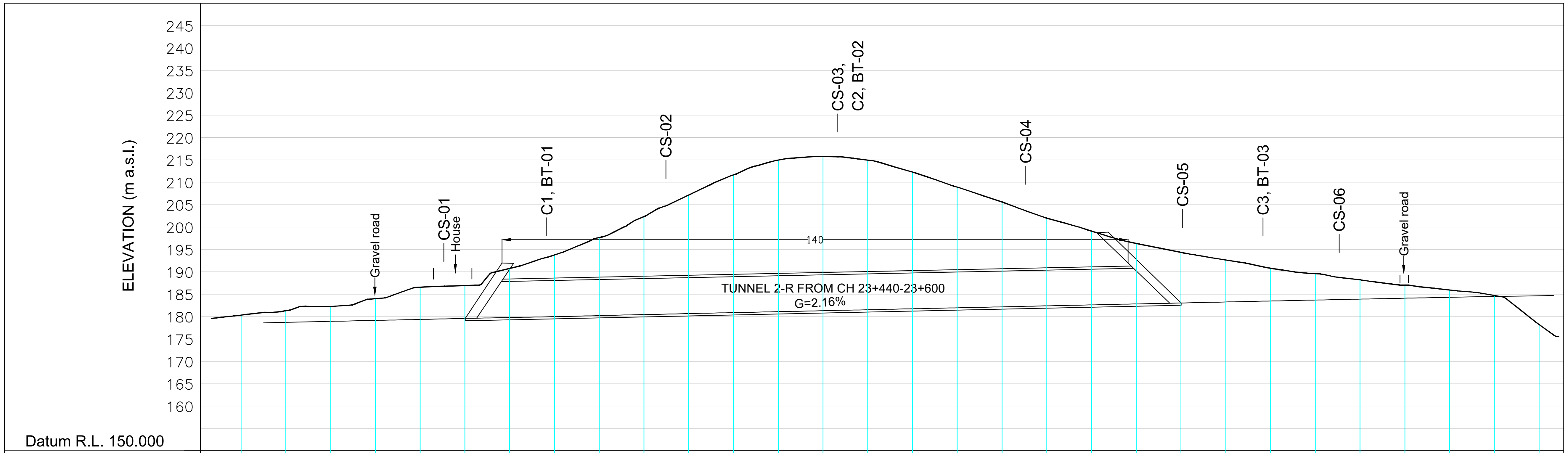
All the elevations are referred to Mean Sea Level (MSL)

CENTRAL EXPRESSWAY PROJECT - SECTION 3
TOPOGRAPHIC SURVEY FOR
PLANNING OF ROAD TUNNEL
AT MAWATHAGAMA, KURUNAGALA DISTRICT

SCALE 1 : 1000



SECTION SURVEY (LS-2)



GROUND ELEVATION (m a.s.l.)	180.33	181.31	182.26	184.00	186.55	186.93	190.76	193.77	197.66	202.34	208.30	211.65	214.96	215.77	214.95
ROAD CHAINAGE	23+390.0	23+400.0	23+410.0	23+420.0	23+430.0	23+440.0	23+450.0	23+460.0	23+470.0	23+480.0	23+490.0	23+500.0	23+510.0	23+520.0	23+530.0

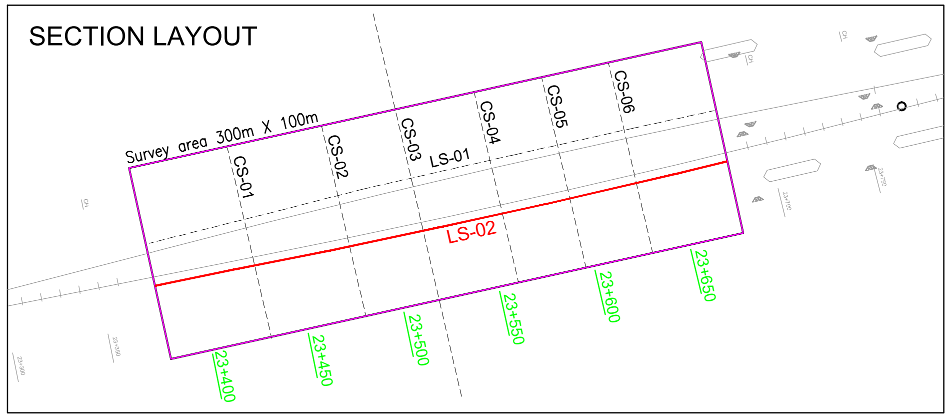
GROUND ELEVATION (m a.s.l.)	212.28	208.93	205.60	201.99	198.35	196.33	194.33	192.65	190.80	189.58	188.21	187.03	185.93	184.72	178.23
ROAD CHAINAGE	23+540.0	23+550.0	23+560.0	23+570.0	23+580.0	23+590.0	23+600.0	23+610.0	23+620.0	23+630.0	23+640.0	23+650.0	23+660.0	23+670.0	23+680.0

LS - 02

LONGITUDINAL SECTION

Horizontal Scale: 1:1000
Vertical Scale: 1:1000

SECTION LAYOUT

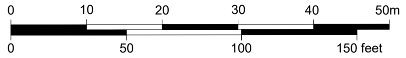


Surveyed from 14th to 25th July 2017.

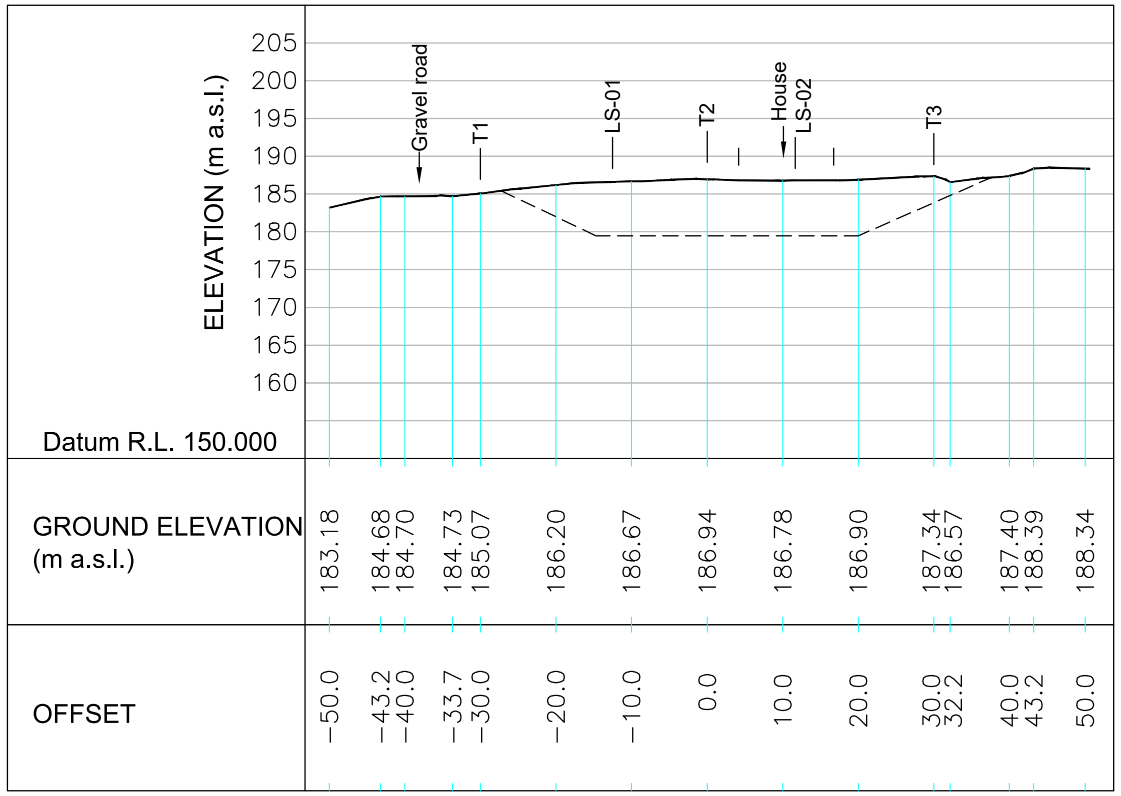
All the elevations are referred to Mean Sea Level (MSL)

CENTRAL EXPRESSWAY PROJECT - SECTION 3
TOPOGRAPHIC SURVEY FOR
PLANNING OF ROAD TUNNEL
AT MAWATHAGAMA, KURUNAGALA DISTRICT

SCALE 1 : 1000

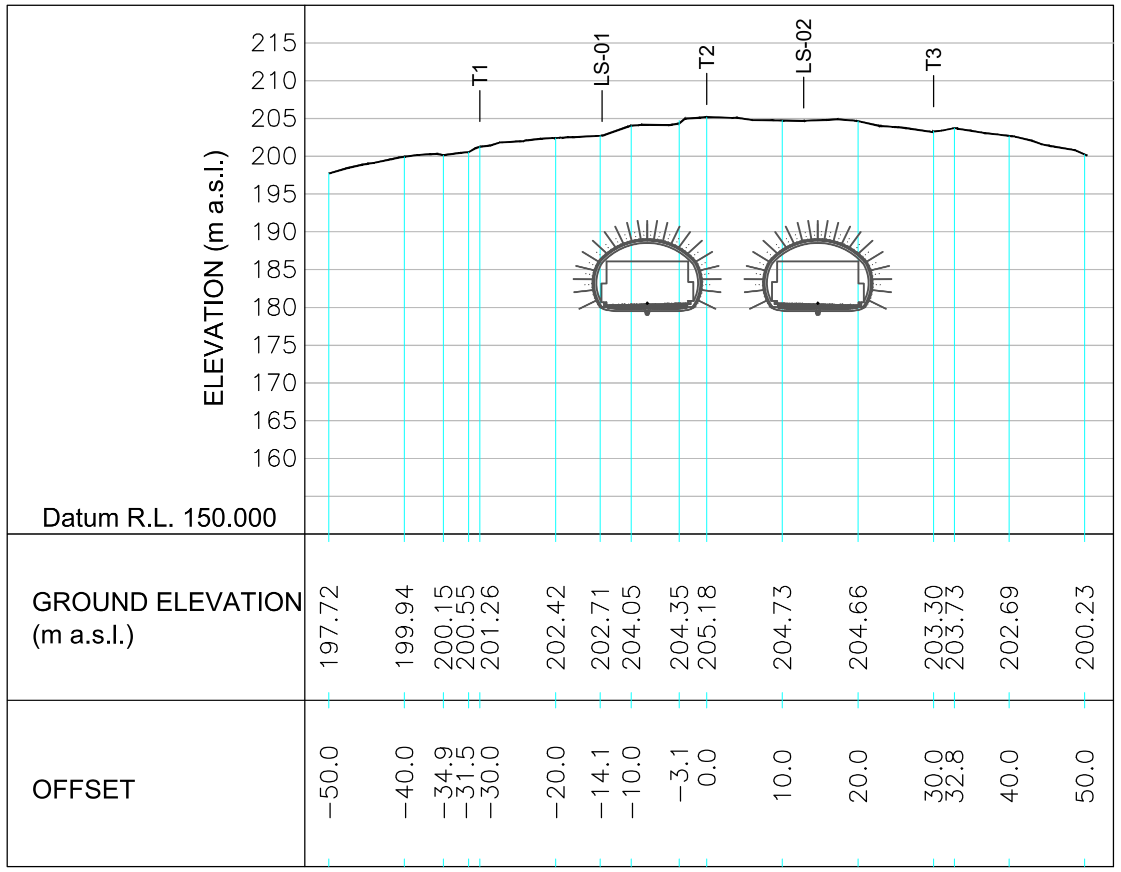


SECTION SURVEY



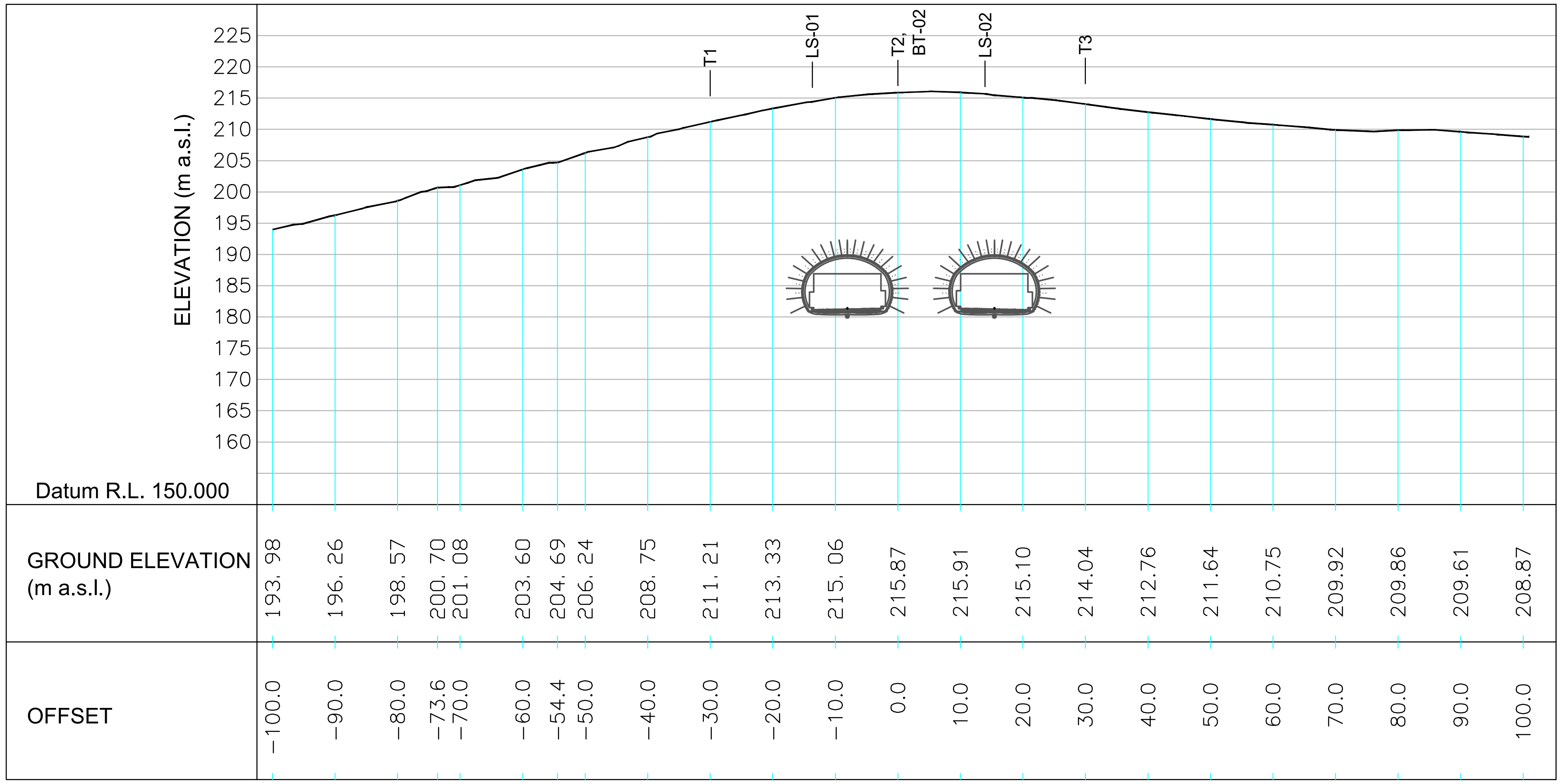
GROUND ELEVATION (m a.s.l.)	183.18	184.68	184.70	184.73	185.07	186.20	186.67	186.94	186.78	186.90	187.34	186.57	187.40	188.39	188.34
OFFSET	−50.0	−43.2	−40.0	−33.7	−30.0	−20.0	−10.0	0.0	10.0	20.0	30.0	32.2	40.0	43.2	50.0

CROSS SECTION No. CS-01 at Ch: 23+435.04



GROUND ELEVATION (m a.s.l.)	197.72	199.94	200.15	200.55	201.26	202.42	202.71	204.05	204.35	205.18	204.73	204.66	203.30	203.73	202.69	200.23
OFFSET	−50.0	−40.0	−34.9	−31.5	−30.0	−20.0	−14.1	−10.0	−3.1	0.0	10.0	20.0	30.0	32.8	40.0	50.0

CROSS SECTION No. CS-02 at Ch: 23+484.67



GROUND ELEVATION (m a.s.l.)	193. 98	196. 26	198. 57	200. 70	201. 08	203. 60	204. 69	206. 24	208. 75	211. 21	213. 33	215. 06	215.87	215.91	215.10	214.04	212.76	211.64	210.75	209.92	209.86	209.61	208.87
OFFSET	−100.0	−90.0	−80.0	−73.6	−70.0	−60.0	−54.4	−50.0	−40.0	−30.0	−20.0	−10.0	0.0	10.0	20.0	30.0	40.0	50.0	60.0	70.0	80.0	90.0	100.0

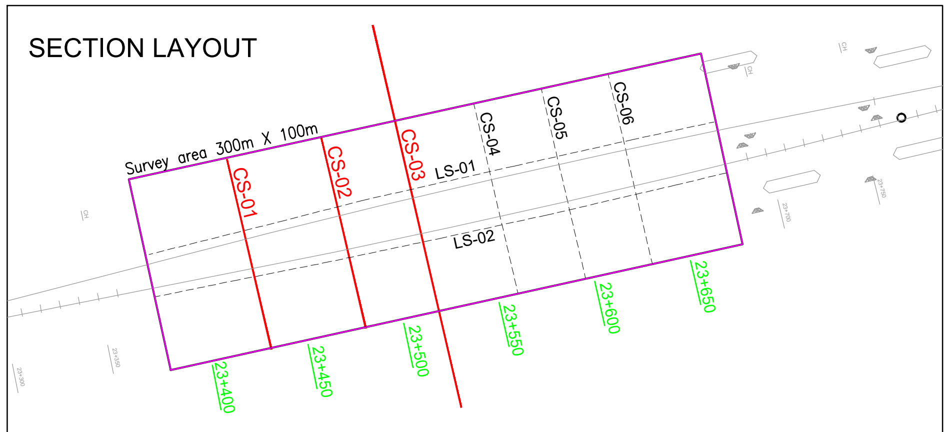
Horizontal Scale: 1:1000
Vertical Scale: 1:1000

CROSS SECTION No. CS-03 at Ch: 23+523.3

Surveyed from 14th to 25th July 2017.

All the elevations are referred to Mean Sea Level (MSL)

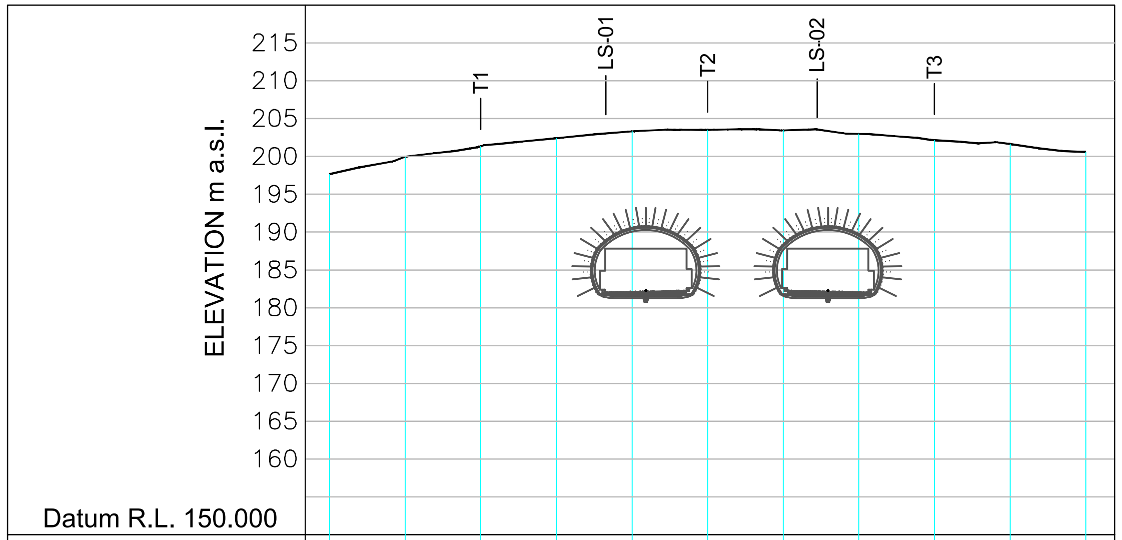
SECTION LAYOUT



CENTRAL EXPRESSWAY PROJECT - SECTION 3
TOPOGRAPHIC SURVEY FOR
PLANNING OF ROAD TUNNEL
AT MAWATHAGAMA, KURUNAGALA DISTRICT

SCALE 1 : 1000
0 10 20 30 40 50m
0 50 100 150 feet

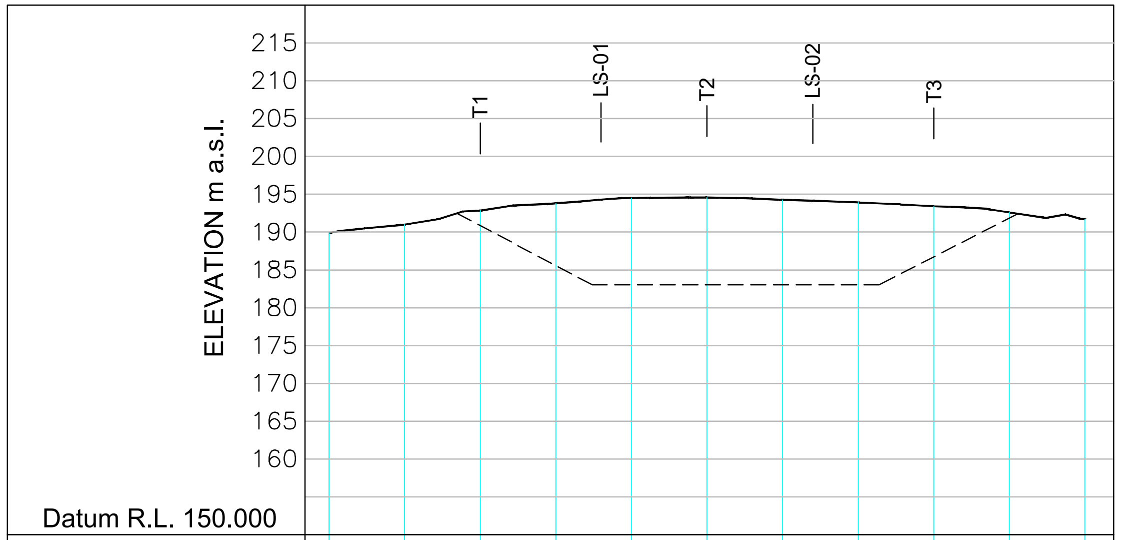
SECTION SURVEY



Datum R.L. 150.000

GROUND ELEVATION m a.s.l.	197.66	199.97	201.32	202.41	203.31	203.50	203.44	202.98	202.15	201.63	200.57
OFFSET	-50.0	-40.0	-30.0	-20.0	-10.0	0.0	10.0	20.0	30.0	40.0	50.0

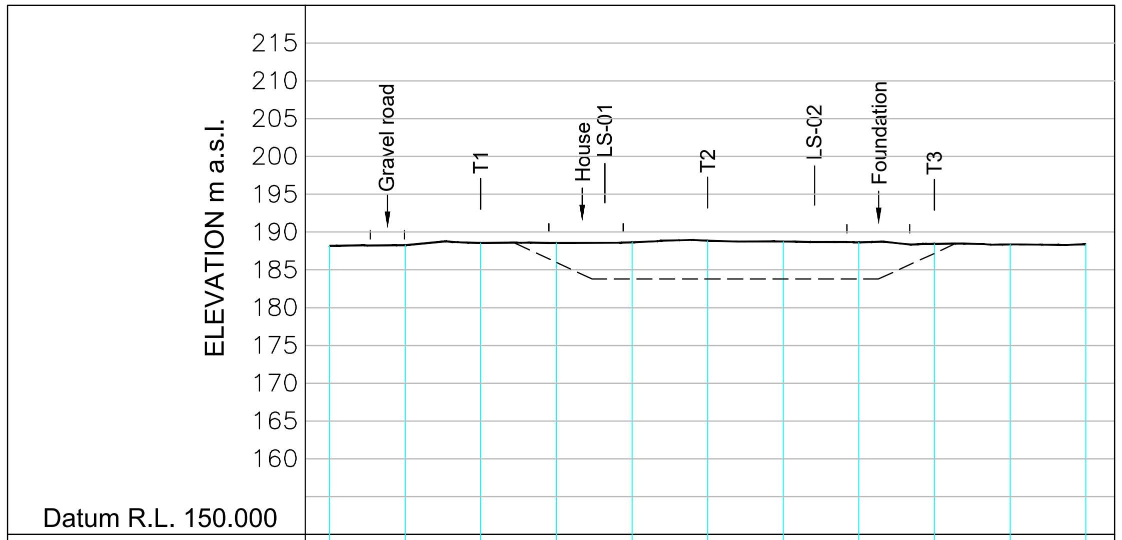
CROSS SECTION No. CS-04 at Ch:23+564.63



Datum R.L. 150.000

GROUND ELEVATION m a.s.l.	189.88	191.00	192.85	193.80	194.50	194.58	194.27	193.92	193.41	192.60	191.69
OFFSET	-50.0	-40.0	-30.0	-20.0	-10.0	0.0	10.0	20.0	30.0	40.0	50.0

CROSS SECTION No. CS-05 at Ch:23+600.00



Datum R.L. 150.000

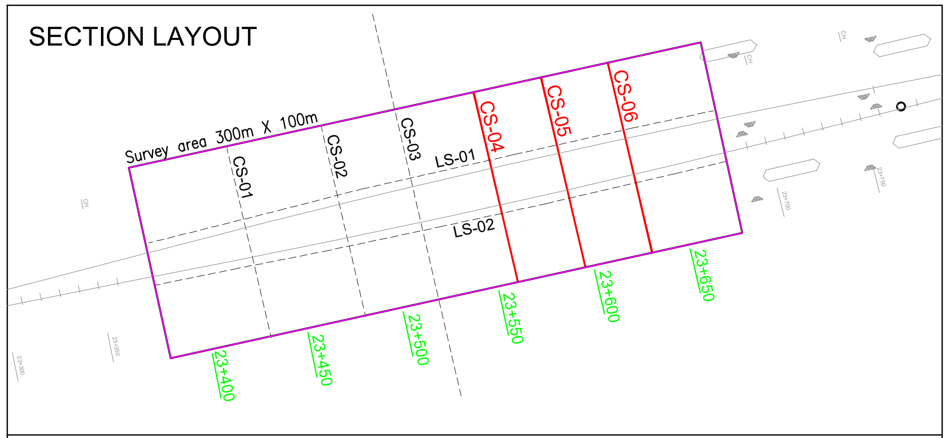
GROUND ELEVATION m a.s.l.	188.16	188.27	188.54	188.56	188.64	188.84	188.75	188.65	188.43	188.36	188.37
OFFSET	-50.0	-40.0	-30.0	-20.0	-10.0	0.0	10.0	20.0	30.0	40.0	50.0

CROSS SECTION No. CS-06 at Ch:23+634.97

Surveyed from 14th to 25th July 2017.

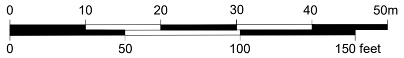
All the elevations are referred to Mean Sea Level (MSL)

SECTION LAYOUT



CENTRAL EXPRESSWAY PROJECT - SECTION 3
TOPOGRAPHIC SURVEY FOR
PLANNING OF ROAD TUNNEL
AT MAWATHAGAMA, KURUNAGALA DISTRICT

SCALE 1 : 1000



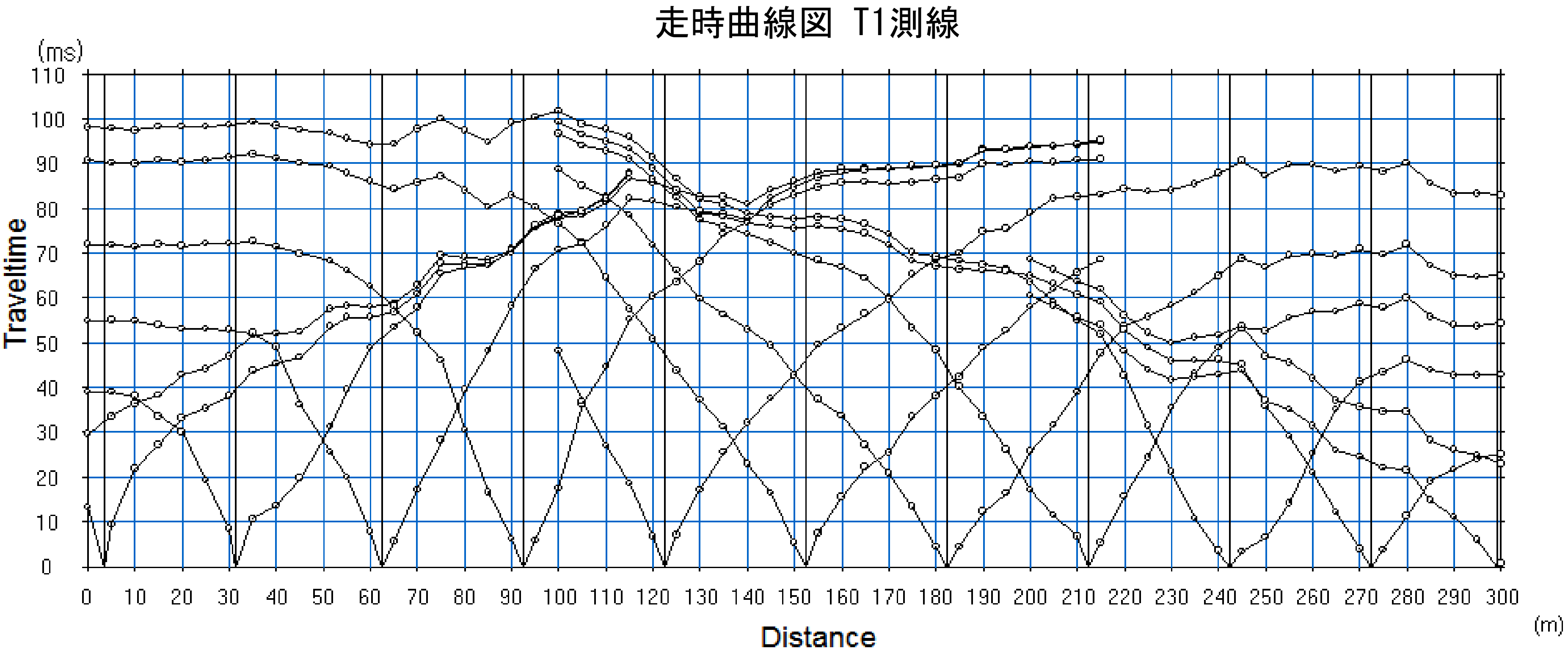
CONTROL TRAVERSE CO-ORDINALES (SLD 99)

No.	Easting (m)	Northing (m)	Elevation (Z) MSL
GPS-01	**464802.524**	**544050.996**	**186.954**
GPS-02	**464789.305**	**544026.336**	**187.124**
S1	464777.825	543988.883	182.782
S2	464770.806	543921.969	183.632
S3	464785.736	543856.934	189.931
S4	464833.377	543860.271	209.653
S5	464847.040	543878.130	209.358
S6	464863.945	543878.347	209.06
S7	464899.487	543903.295	194.225
S8	464959.121	543912.288	188.321
S9	464995.368	543837.253	186.567
S10	464945.631	543815.047	192.652
S11	464895.149	543803.848	206.47
S12	464870.361	543782.130	210.879
S13	464843.796	543800.598	208.213
S14	464806.353	543792.972	193.561
S15	464754.565	543791.848	182.286
S16	464738.124	543807.350	180.809
S17	464740.751	543868.824	180.675
S18	464741.968	543934.609	179.581
S19	464732.721	543984.372	181.436
S20	464767.853	544029.325	190.135

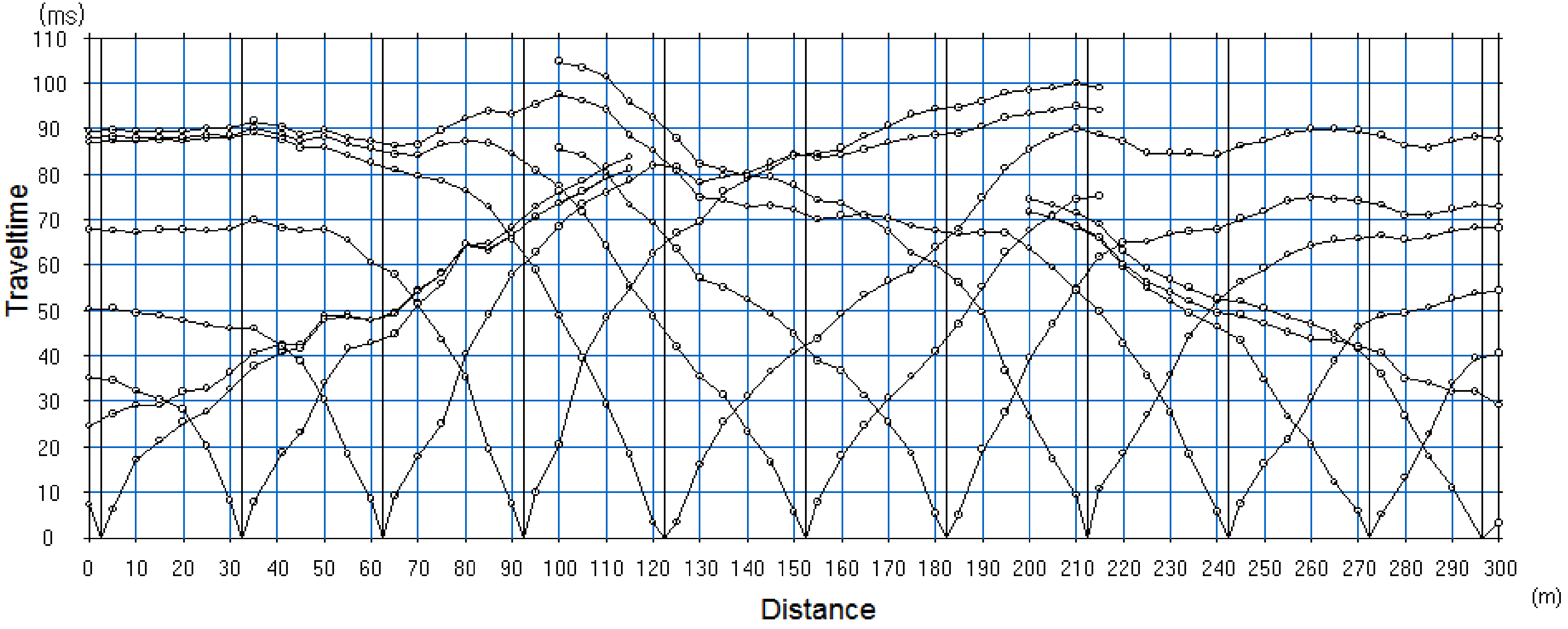
添付資料　3

弾性波探査結果

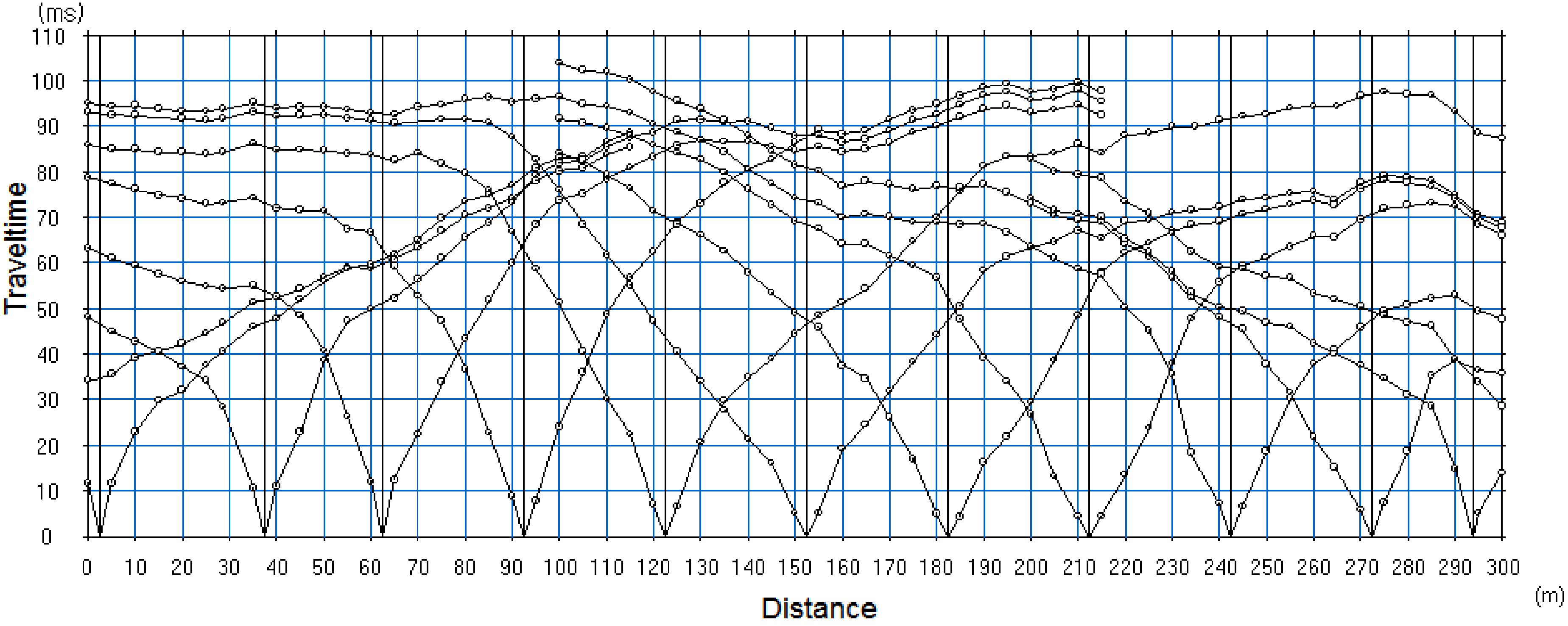
走時曲線図


走時曲線図 T1測線
(ms)
110
100
90
80
70
60
50
40
30
20
10
0
Traveltime
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
280
290
300
(m)
Distance

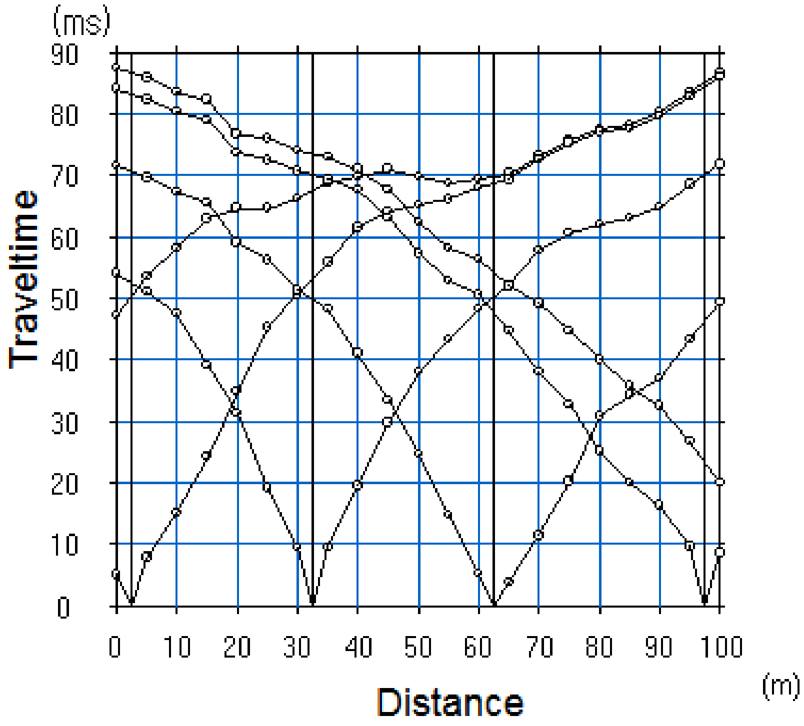
走時曲線図 T2測線


(ms)
110
100
90
80
70
60
50
40
30
20
10
0
Traveltime
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
280
290
300
(m)
Distance

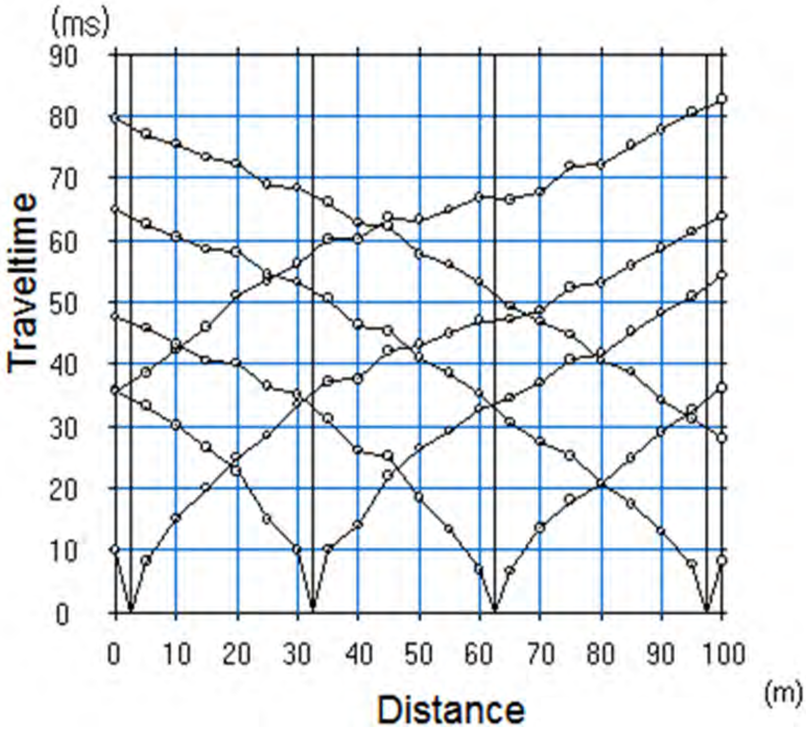
走時曲線図 T3測線


(ms)
110
100
90
80
70
60
50
40
30
20
10
0
Traveltime
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
280
290
300
Distance
(m)

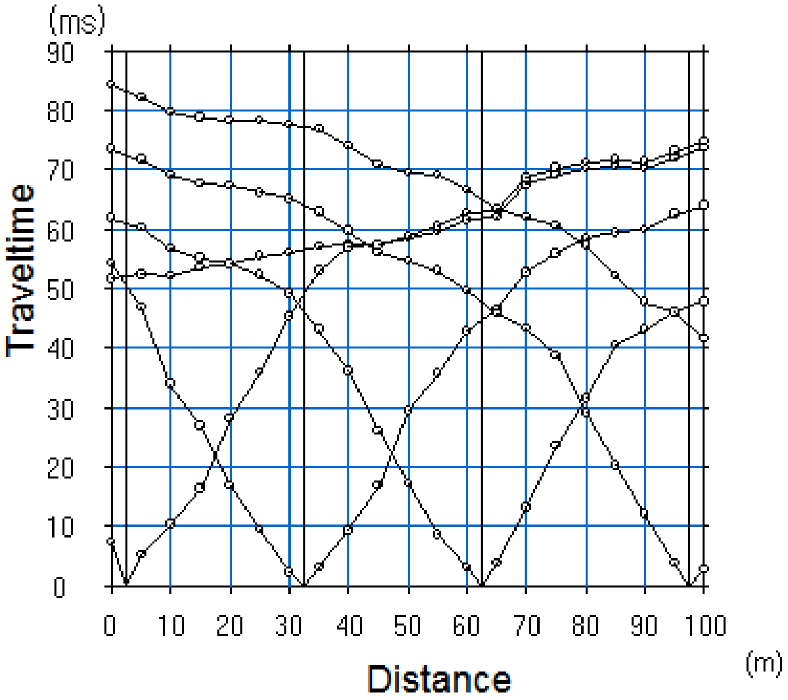
走時曲線図 C1測線



走時曲線図 C2測線

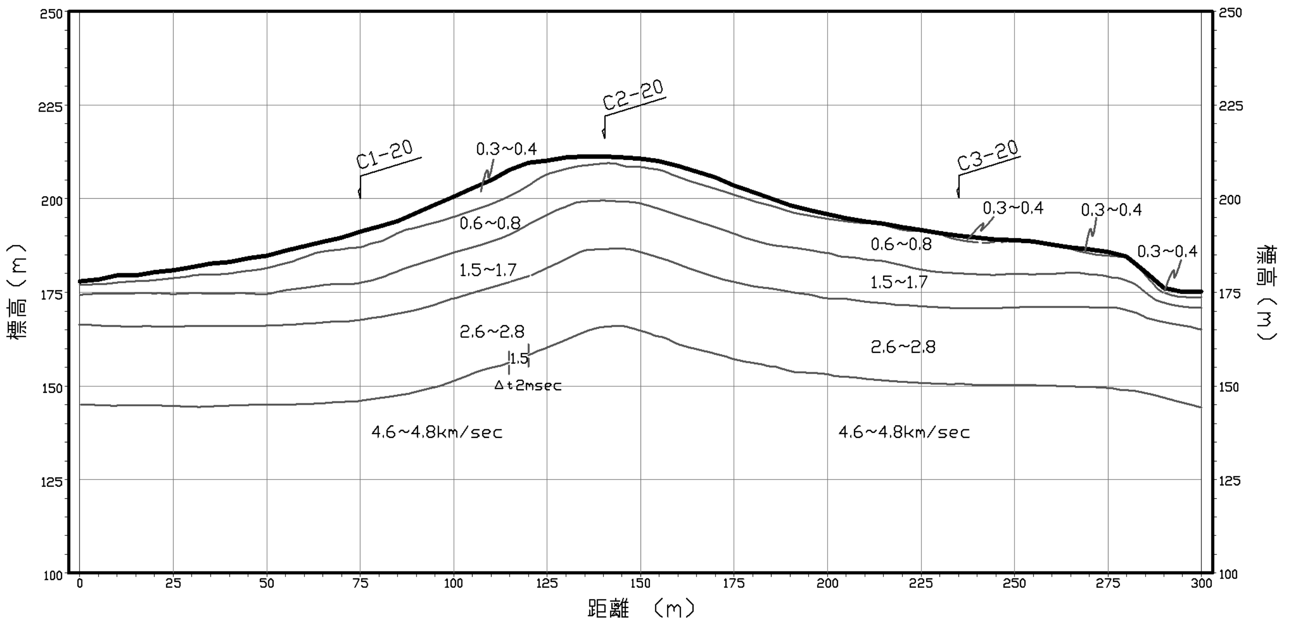


走時曲線図　C3測線

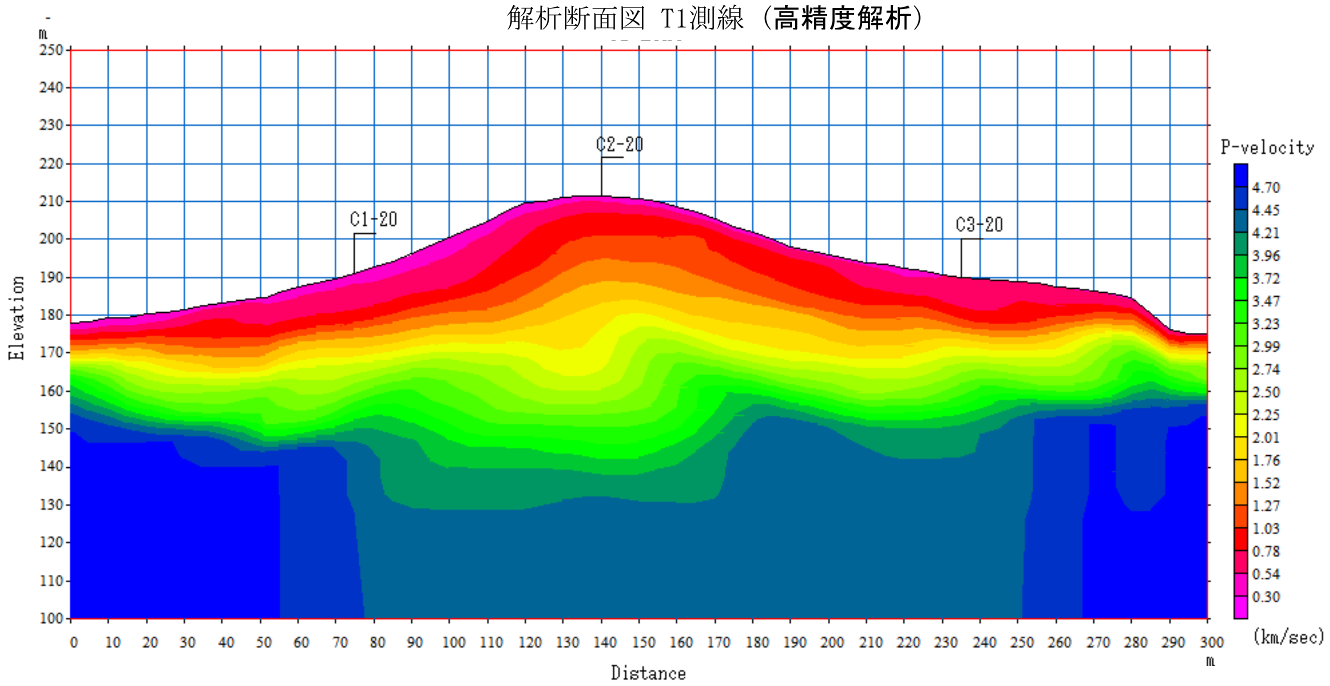

(ms)
90
80
70
60
50
40
30
20
10
0
Traveltime
0
10
20
30
40
50
60
70
80
90
100
(m)
Distance

解析断面図

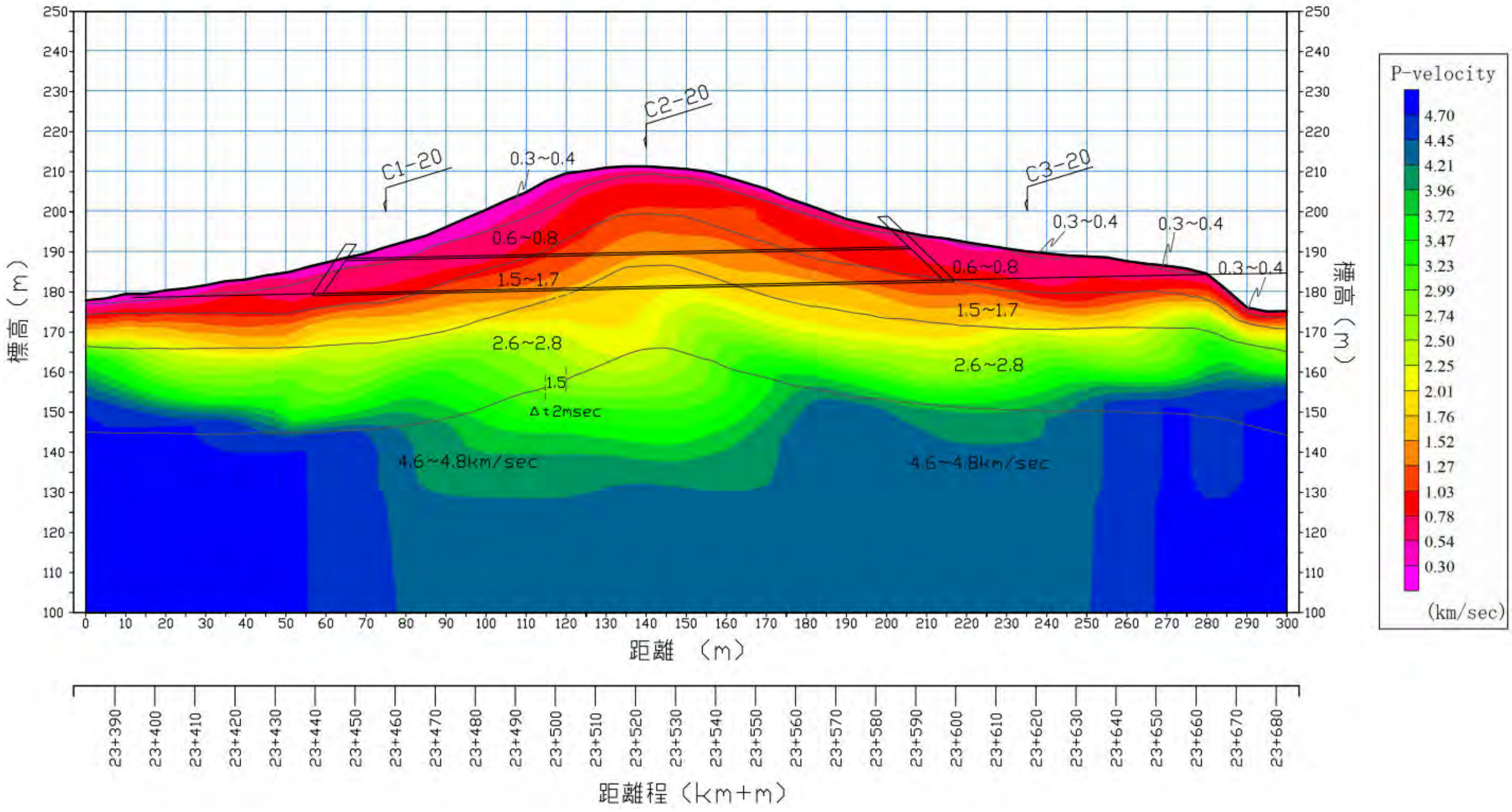
解析断面図 T1測線（萩原法）



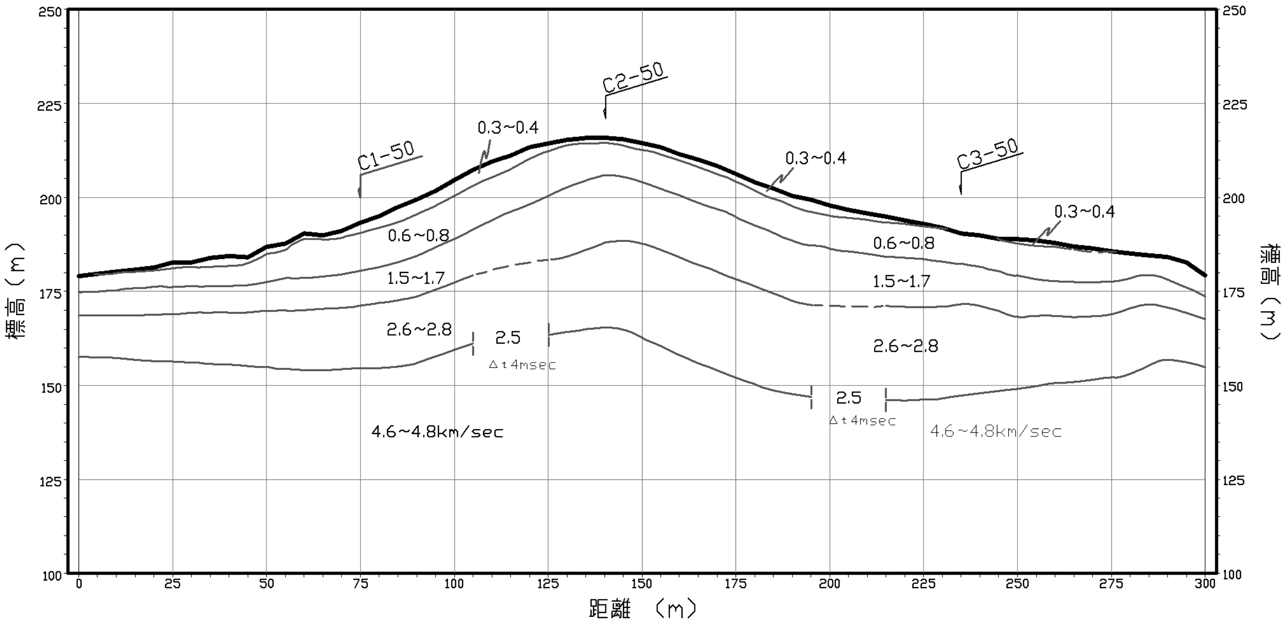
解析断面図 T1測線（高精度解析）



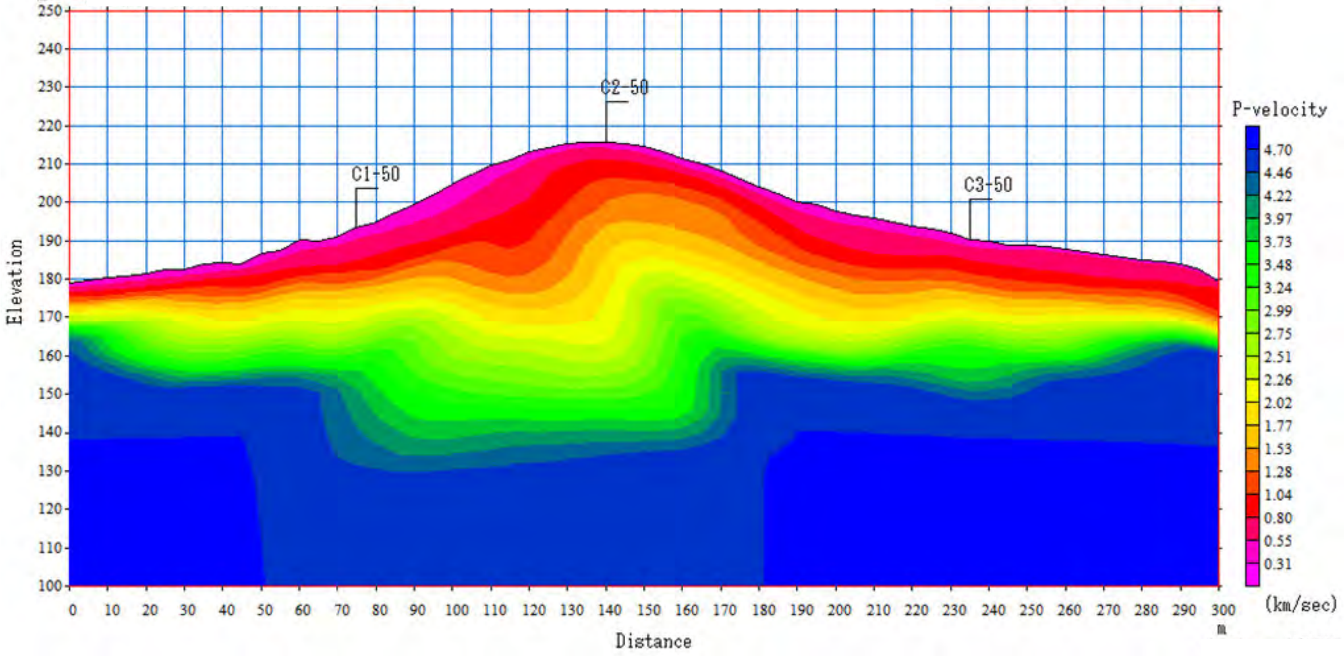
解析断面図 T1


250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
標高（m）
C1-20
C2-20
C3-20
0.3~0.4
0.6~0.8
1.5~1.7
2.6~2.8
1.5
Δt2msec
4.6~4.8km/sec
0.3~0.4
0.6~0.8
1.5~1.7
2.6~2.8
4.6~4.8km/sec
0.3~0.4
0.3~0.4
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250 260 270 280 290 300
距離 （m）
23+390 23+400 23+410 23+420 23+430 23+440 23+450 23+460 23+470 23+480 23+490 23+500 23+510 23+520 23+530 23+540 23+550 23+560 23+570 23+580 23+590 23+600 23+610 23+620 23+630 23+640 23+650 23+660 23+670 23+680
距離程（km+m）
P-velocity
4.70
4.45
4.21
3.96
3.72
3.47
3.23
2.99
2.74
2.50
2.25
2.01
1.76
1.52
1.27
1.03
0.78
0.54
0.30
(km/sec)

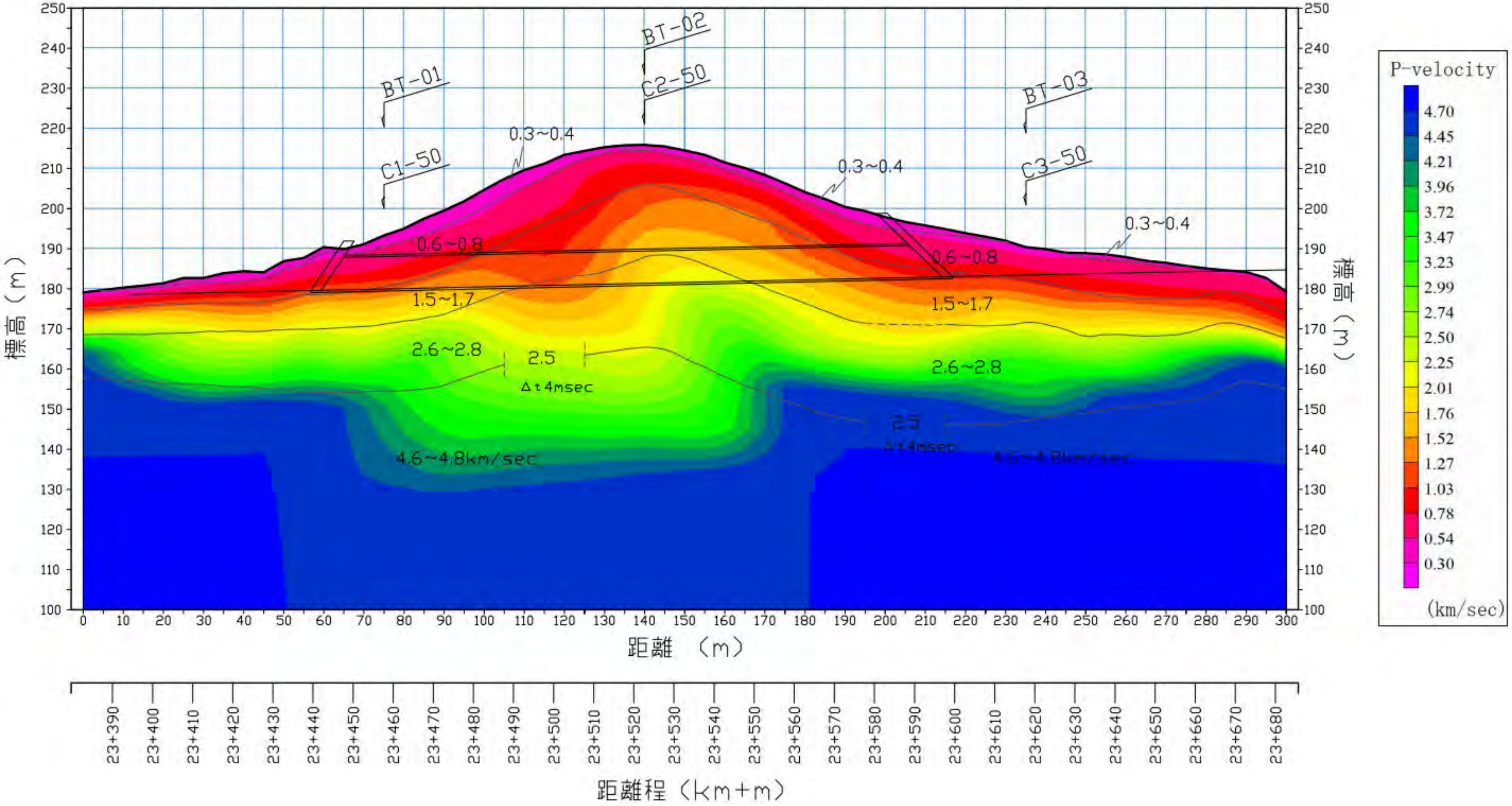
解析断面図 T2測線（萩原法）



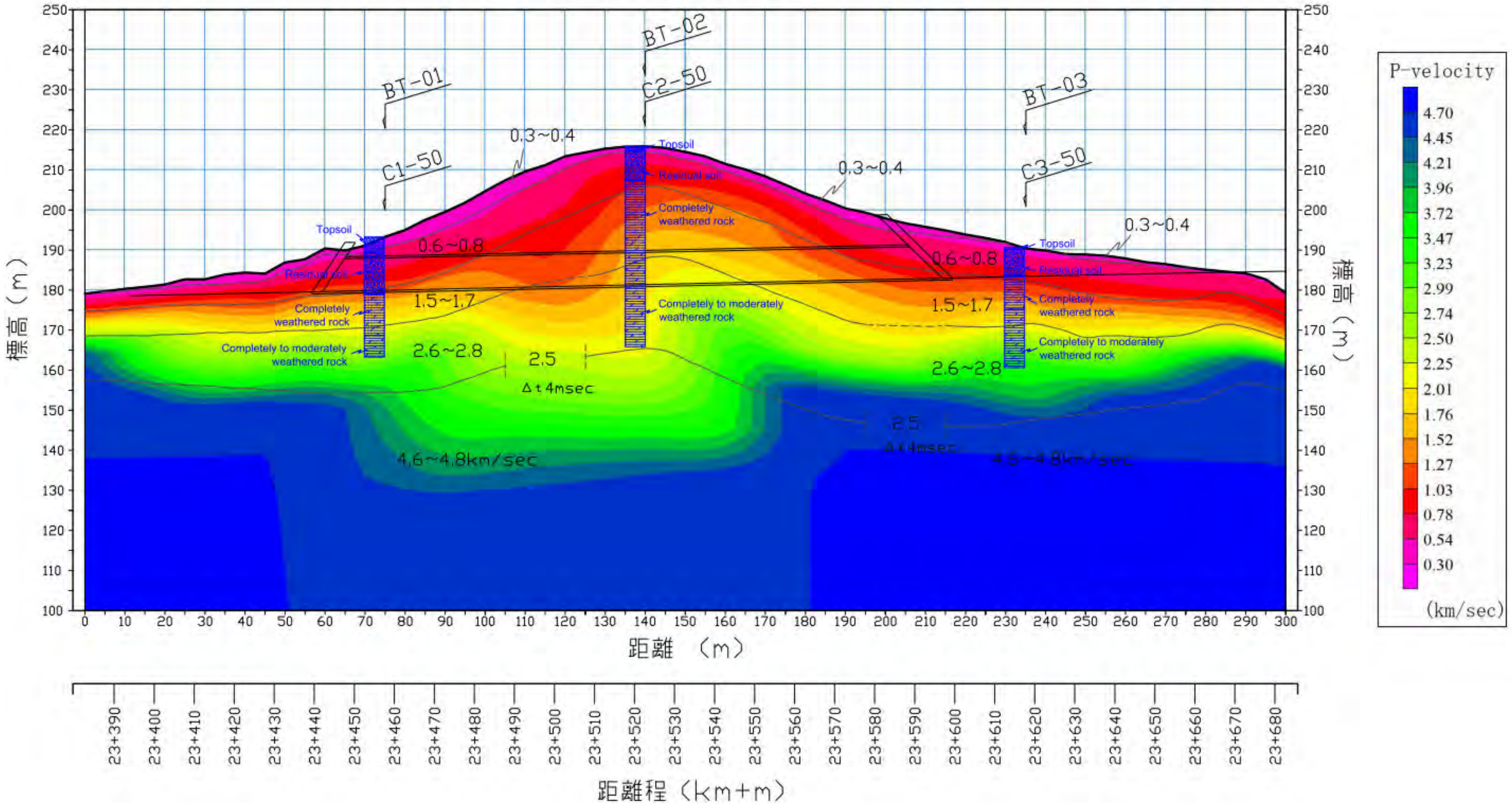
解析断面図 T2測線（高精度解析）



解析断面図 T2

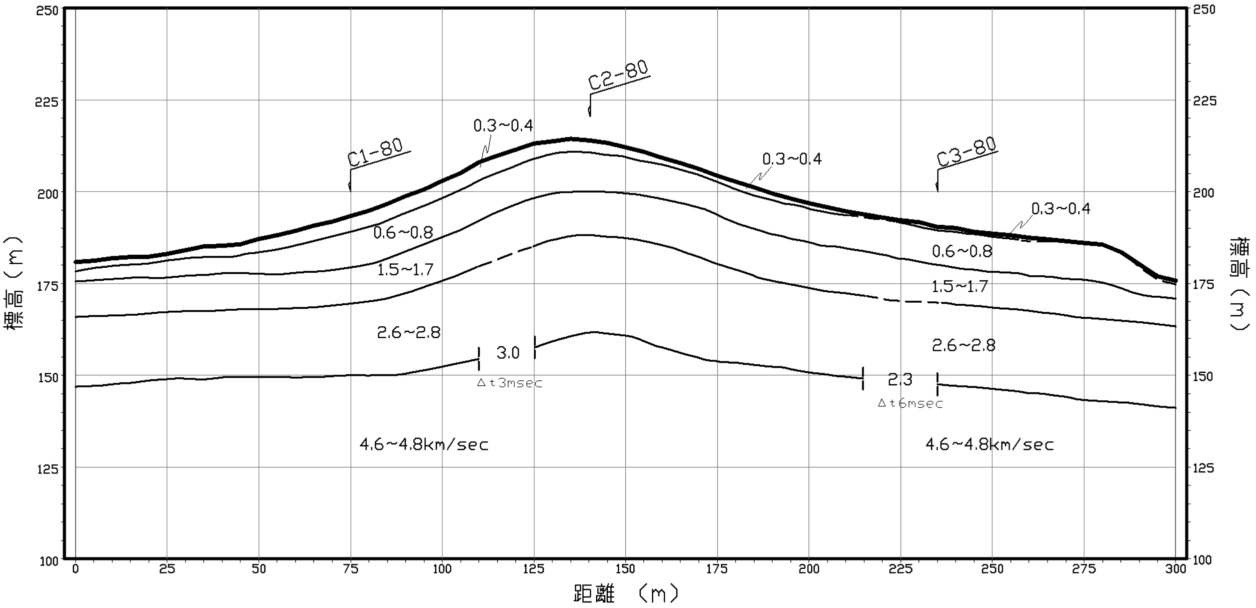

BT-01
C1-50
BT-02
C2-50
BT-03
C3-50
0.3~0.4
0.3~0.4
0.3~0.4
0.6~0.8
0.6~0.8
1.5~1.7
1.5~1.7
2.6~2.8
2.5
Δt4msec
2.6~2.8
2.5
Δt4msec
4.6~4.8km/sec
4.6~4.8km/sec
標高（m）
標高（m）
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
280
290
300
距離 （m）
23+390
23+400
23+410
23+420
23+430
23+440
23+450
23+460
23+470
23+480
23+490
23+500
23+510
23+520
23+530
23+540
23+550
23+560
23+570
23+580
23+590
23+600
23+610
23+620
23+630
23+640
23+650
23+660
23+670
23+680
距離程（km+m）
P-velocity
4.70
4.45
4.21
3.96
3.72
3.47
3.23
2.99
2.74
2.50
2.25
2.01
1.76
1.52
1.27
1.03
0.78
0.54
0.30
(km/sec)

解析断面図 T2

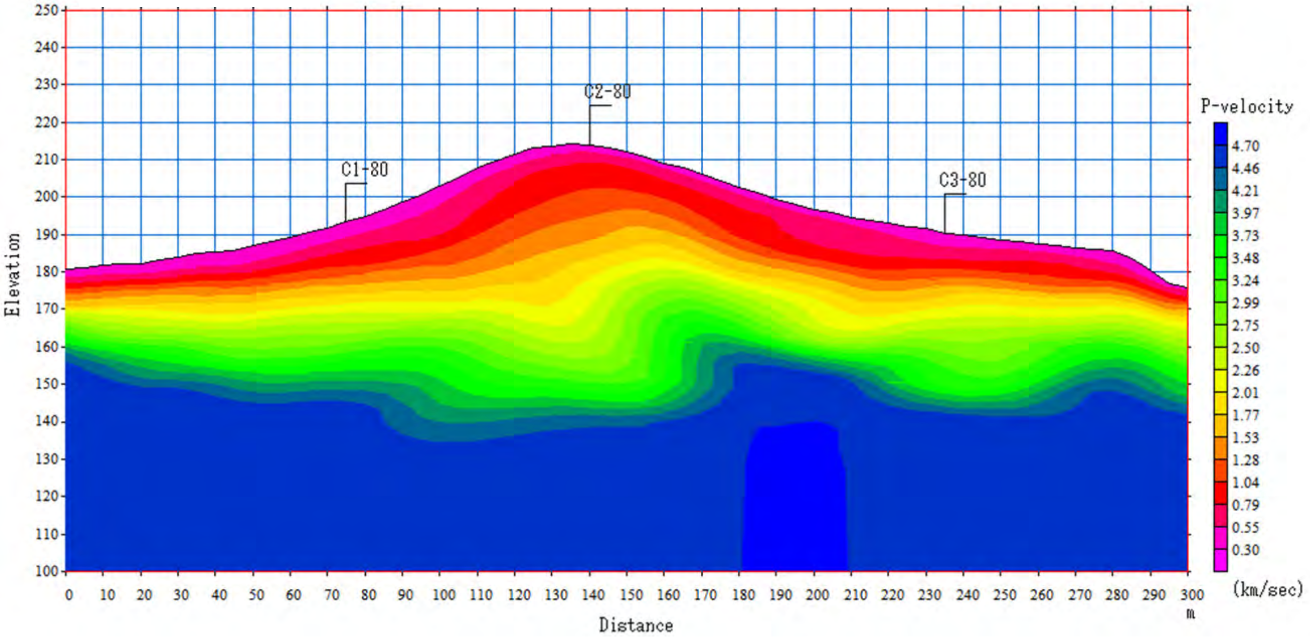


BT-01
C1-50
BT-02
C2-50
BT-03
C3-50
0.3~0.4
0.3~0.4
0.3~0.4
0.6~0.8
0.6~0.8
1.5~1.7
1.5~1.7
2.6~2.8
2.6~2.8
2.5
Δt 4msec
2.5
Δt 4msec
4.6~4.8km/sec
4.6~4.8km/sec
Topsoil
Residual soil
Completely weathered rock
Completely to moderately weathered rock
Topsoil
Residual soil
Completely weathered rock
Completely to moderately weathered rock
Topsoil
Residual soil
Completely weathered rock
Completely to moderately weathered rock
標高（m）
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
280
290
300
距離 （m）
23+390
23+400
23+410
23+420
23+430
23+440
23+450
23+460
23+470
23+480
23+490
23+500
23+510
23+520
23+530
23+540
23+550
23+560
23+570
23+580
23+590
23+600
23+610
23+620
23+630
23+640
23+650
23+660
23+670
23+680
距離程（km+m）
P-velocity
4.70
4.45
4.21
3.96
3.72
3.47
3.23
2.99
2.74
2.50
2.25
2.01
1.76
1.52
1.27
1.03
0.78
0.54
0.30
(km/sec)

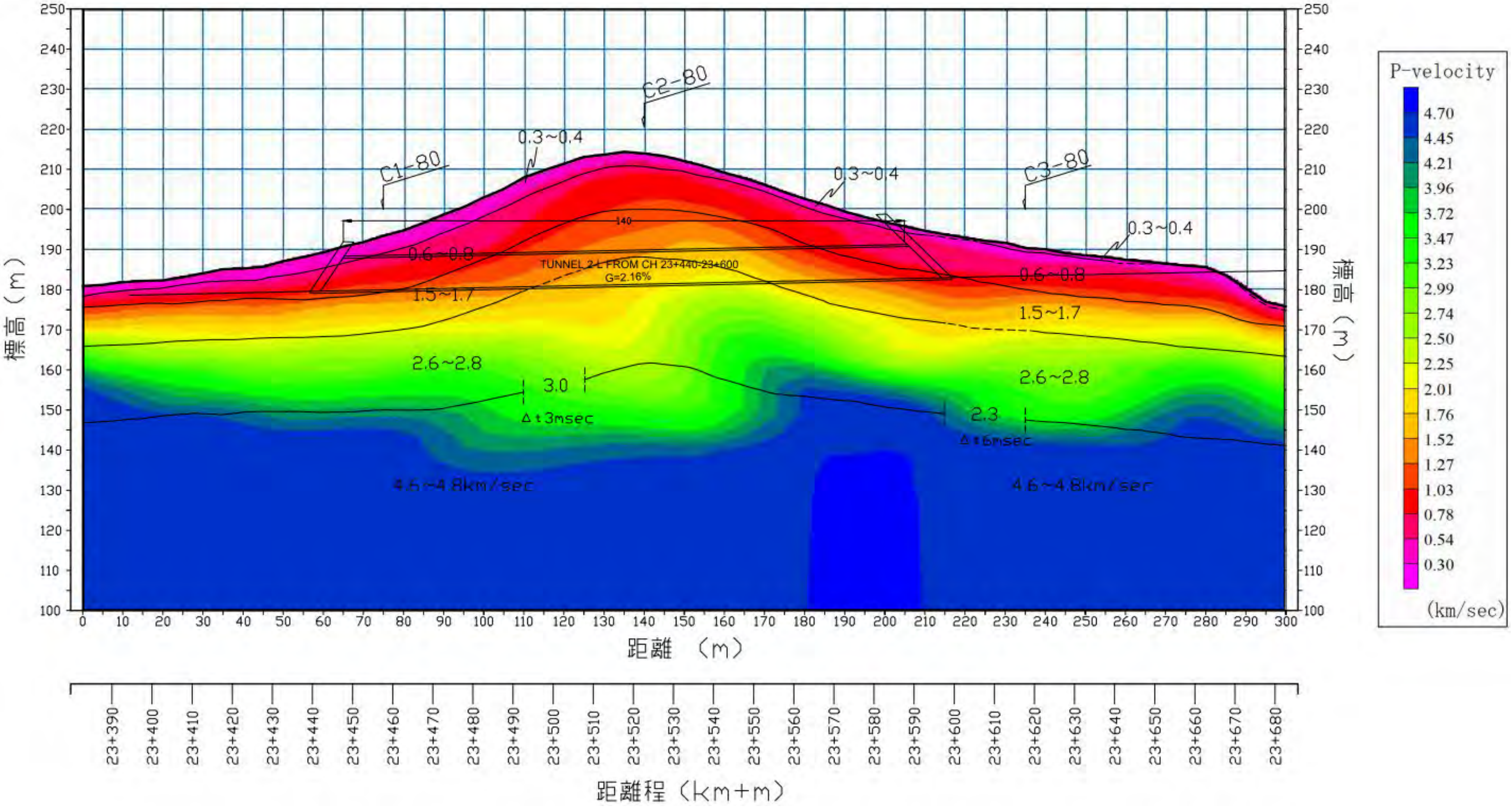
解析断面図 T3測線（萩原法）



解析断面図 T3測線（高精度解析）

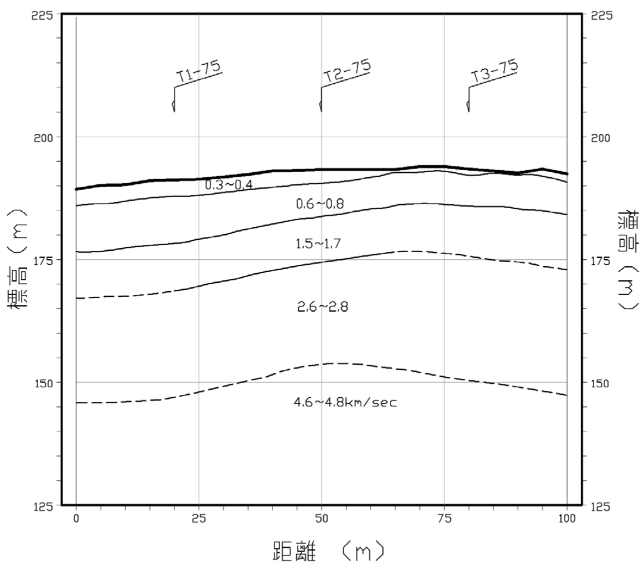


解析断面図 T3

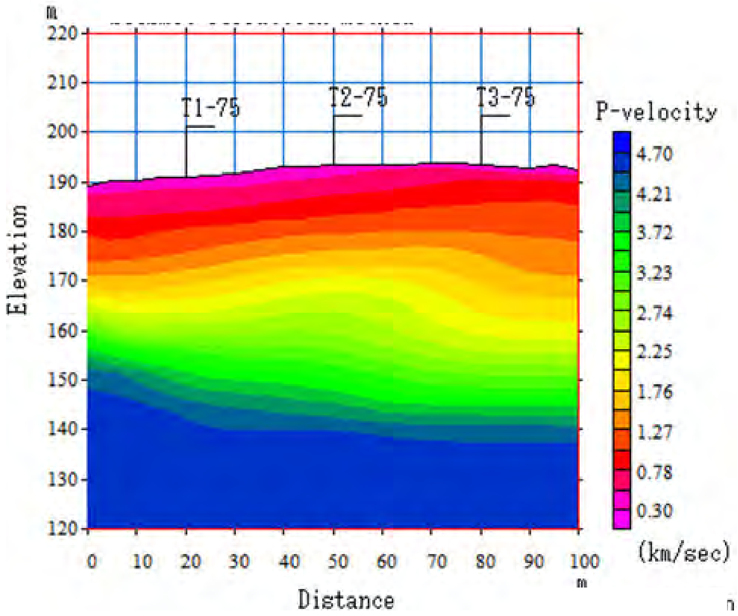


標高（m）
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
C1-80
C2-80
C3-80
0.3~0.4
0.3~0.4
0.3~0.4
140
0.6~0.8
0.6~0.8
TUNNEL 2-L FROM CH 23+440-23+600
G=2.16%
1.5~1.7
1.5~1.7
2.6~2.8
2.6~2.8
3.0
2.3
Δt3msec
Δt6msec
4.6~4.8km/sec
4.6~4.8km/sec
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
280
290
300
距離 （m）
23+390
23+400
23+410
23+420
23+430
23+440
23+450
23+460
23+470
23+480
23+490
23+500
23+510
23+520
23+530
23+540
23+550
23+560
23+570
23+580
23+590
23+600
23+610
23+620
23+630
23+640
23+650
23+660
23+670
23+680
距離程（km+m）
P-velocity
4.70
4.45
4.21
3.96
3.72
3.47
3.23
2.99
2.74
2.50
2.25
2.01
1.76
1.52
1.27
1.03
0.78
0.54
0.30
(km/sec)

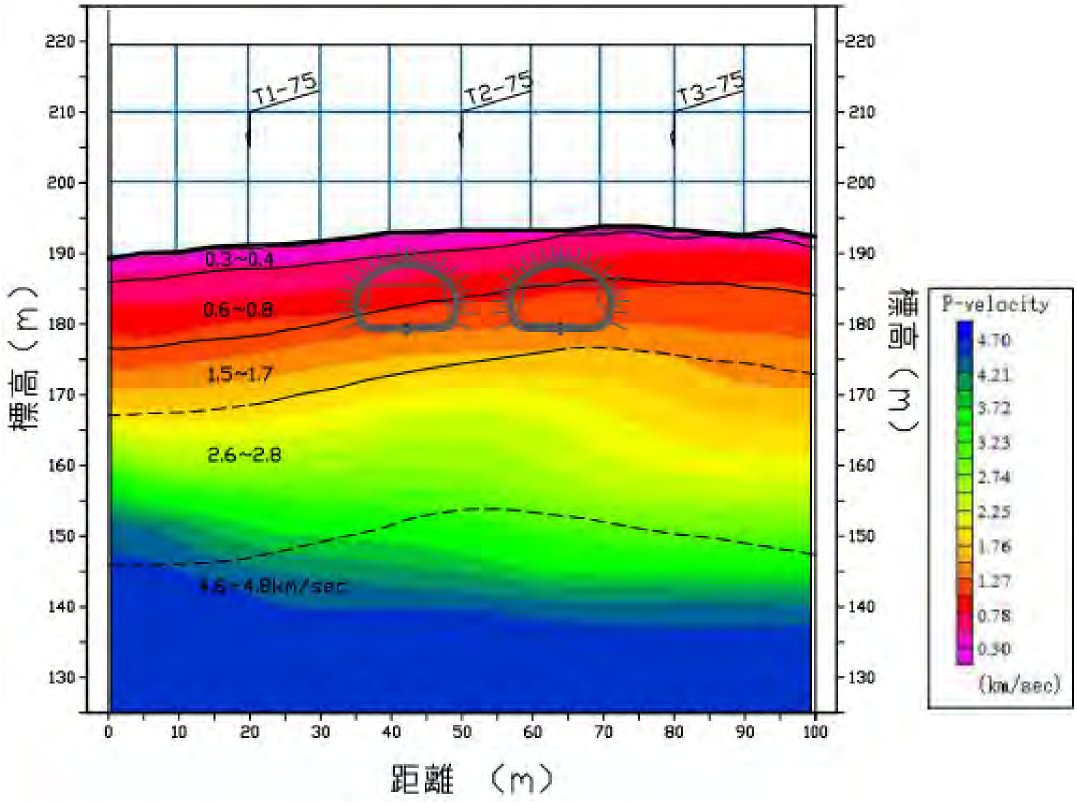
解析断面図 C1測線（萩原法）



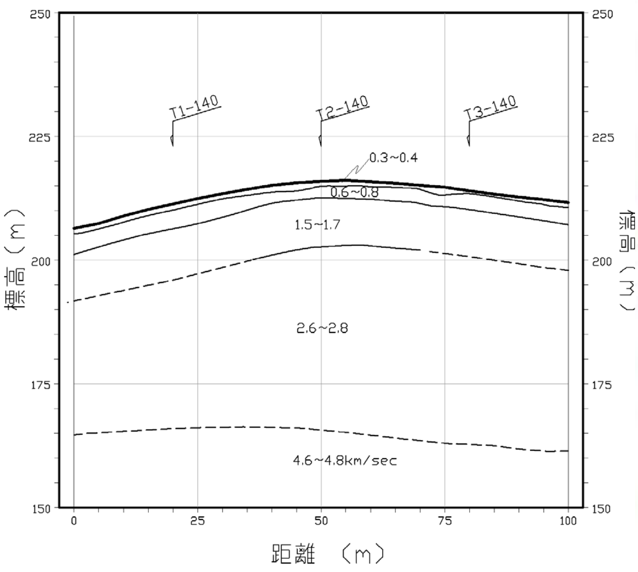
解析断面図 C1測線（高精度解析）



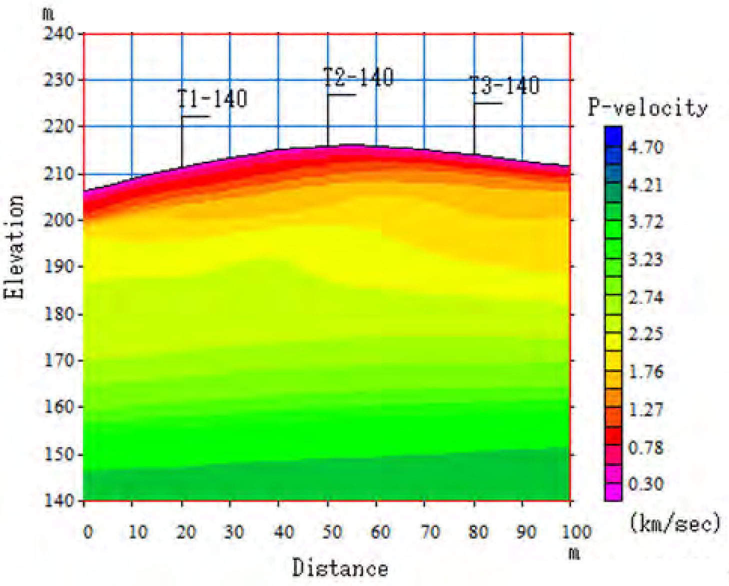
解析断面図 C1（CH23+458.3）



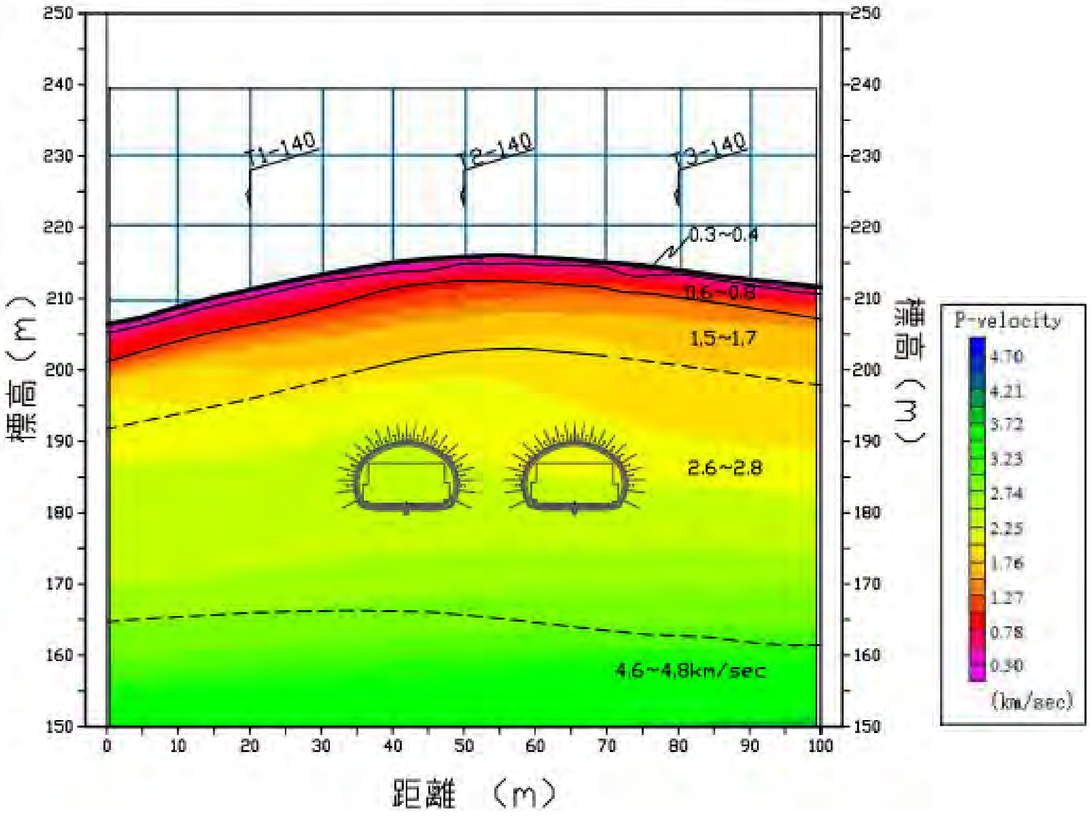
解析断面図 C2測線（萩原法）



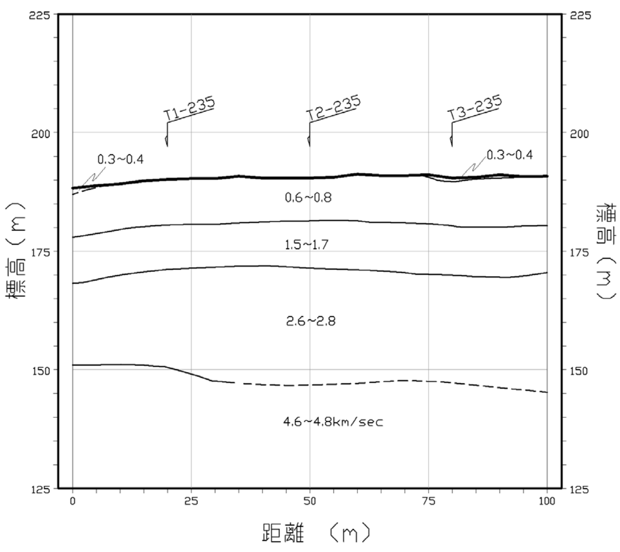
解析断面図 C2測線（高精度解析）



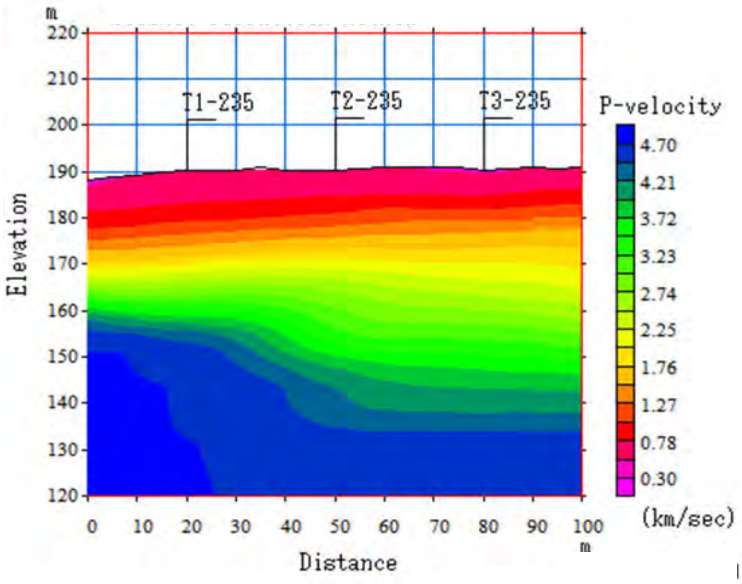
解析断面図 C2 (CH23+523.3)



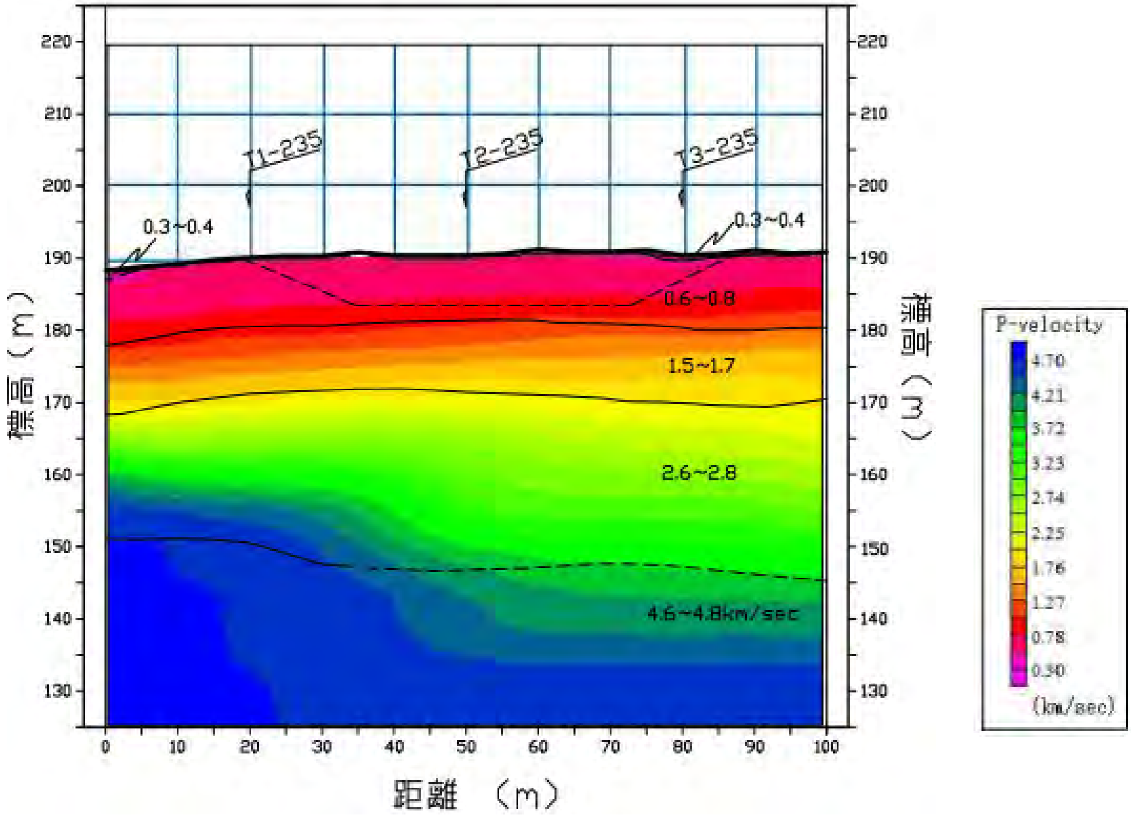
解析断面図 C3測線（萩原法）



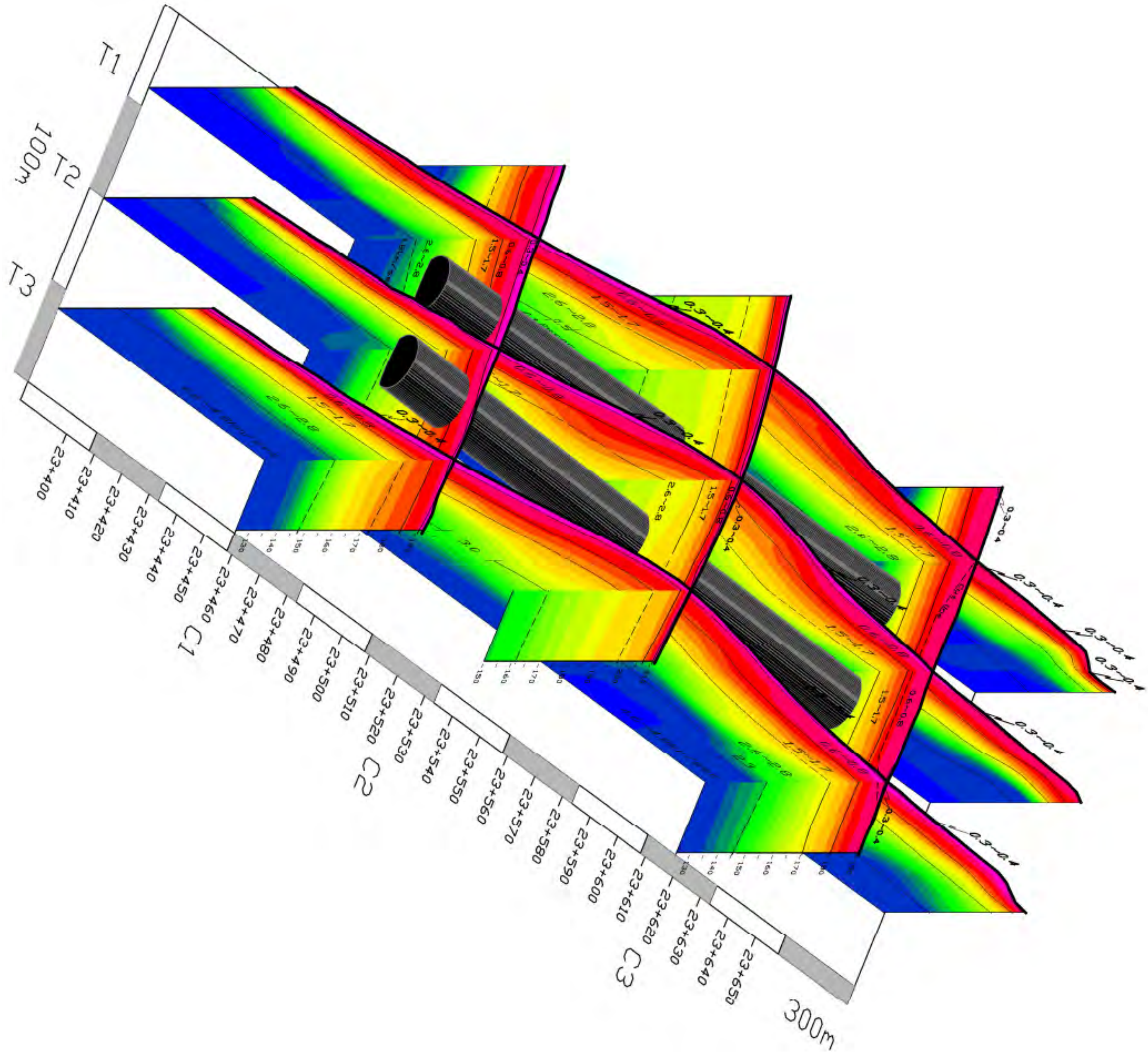
解析断面図 C3測線（高精度解析）



解析断面図 C3（CH23+618.3）



3D 解析断面図


T1
T2
T3
100m
300m
C1
C2
C3
23+400
23+410
23+420
23+430
23+440
23+450
23+460
23+470
23+480
23+490
23+500
23+510
23+520
23+530
23+540
23+550
23+560
23+570
23+580
23+590
23+600
23+610
23+620
23+630
23+640
23+650
0.3~0.4
0.6~0.8
1.5~1.7
2.6~2.8
130
140
150
160
170

添付資料　4

ボーリング調査結果

ボーリング柱状図
ボーリングコア写真
地質断面図
室内試験結果

ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

NO 62/3, Neelammahara Road,
Katuwawala, Sri Lanka.
Tel: 0114 309 494

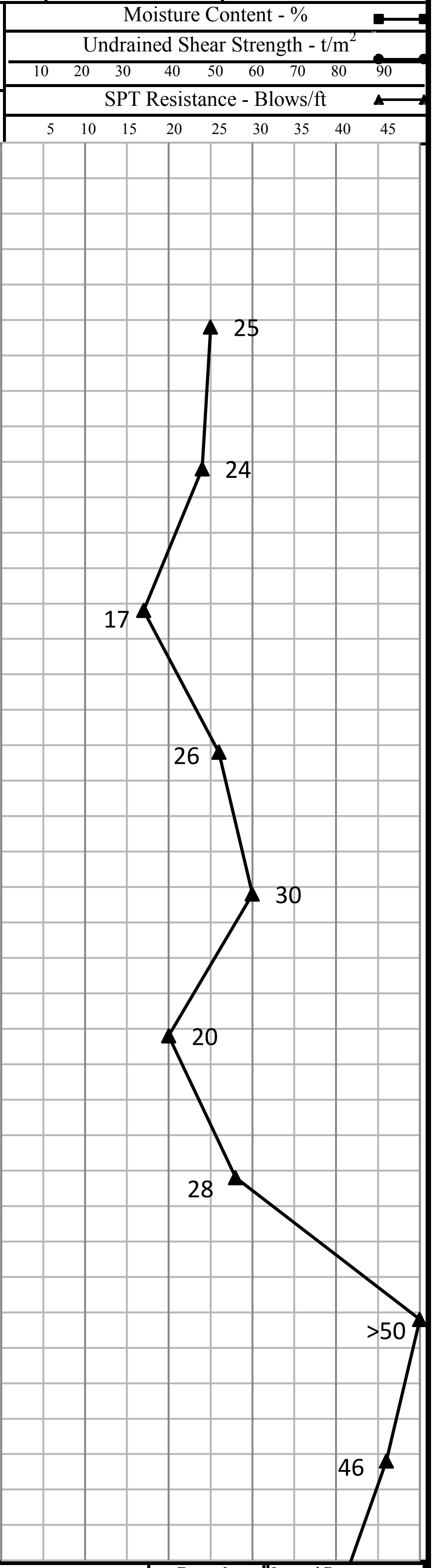
Format No:
ELS-SI-02

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-01
Client	**M/s. Earth System Science Co., Ltd**			Sheet	1 of 3
Location	Mawathagama	Rig	4005-0016	Core Diameter	54.00mm
Date of Started	18.09.2017	Drilling Method	Rotary	Casing depth	14.00m
Date of Finished	27.09.2017	Casing Diameter	76.00mm	Elevation (m)	193.343

Ground Water level 14.00m

Coordinates 464796.950E
543826.986N

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
0.00							Ground level				
	☒	D1	DS	193.3	0.00		Clayey sand with gravels, reddish,moise, mostly sand with gravels, angular to sub angular, weak cementation				
			DS								
1.00				192.6	0.70		Poorly graded, clayey GRAVELS with SAND, medium dense, Reddish brown, moist, Mostly gravels, fine, weak cementation				
	☒	D2	SS					6	14	11	**25**
			DS								
2.00	☒	D3	SS					7	10	14	**24**
			DS								
3.00	☒	D4	SS					6	7	10	**17**
			DS								
4.00	☒	D5	SS	189.3	4.00		SAND with gravels, medium dense,traces of clay, medium dense, light brown, moist, fine gravels	6	10	16	**26**
			DS	188.9	4.45		Clayey sand reddish,moise, mostly sand Slightly clayey sand reddish,moise, sand				
5.00	☒	D6	SS	188.3	5.00		Poorly graded, slightly clayey GRAVELS with SAND, medium dense, Reddish brown, moist.	8	13	17	**30**
			DS	187.7	5.65		Slightly clayey SAND,medium dense reddish,moise,sub angular to angular, sand				
6.00	☒	D7	SS	187.3	6.00		Silty SAND, medium dense,reddish brown, moist,sub anular to angular sand	5	7	13	**20**
			DS	186.9	6.45		Silty SAND, orange, reddish brown, fine to coarse, angular to sub angular sand				
7.00	☒	D8	SS	186.3	7.00		Silty SAND, medium dense, reddish brown, fine to coarse, angular to sub angular sand	8	12	16	**28**
			WS								
8.00	☒	D9	SS	185.3	8.00		Silty SAND,very dense, reddish brown, fine to coarse, angular to sub angular, lateritic SAND	13	17	HB	**>50**
			WS								
9.00	☒	D10	SS	184.3	9.00		Clayey SAND,dense, reddish brown, fine to coarse, angular to sub angular, lateritic SAND	11	19	27	**46**
			WS	183.9	9.45		Clayey SAND with gravels, dense, light brown, yellowish brown, fine to coarse, angular to sub angular, lateritic SAND with gravels				
10.00											



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wash Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks

Existing ground level considered as the zero level

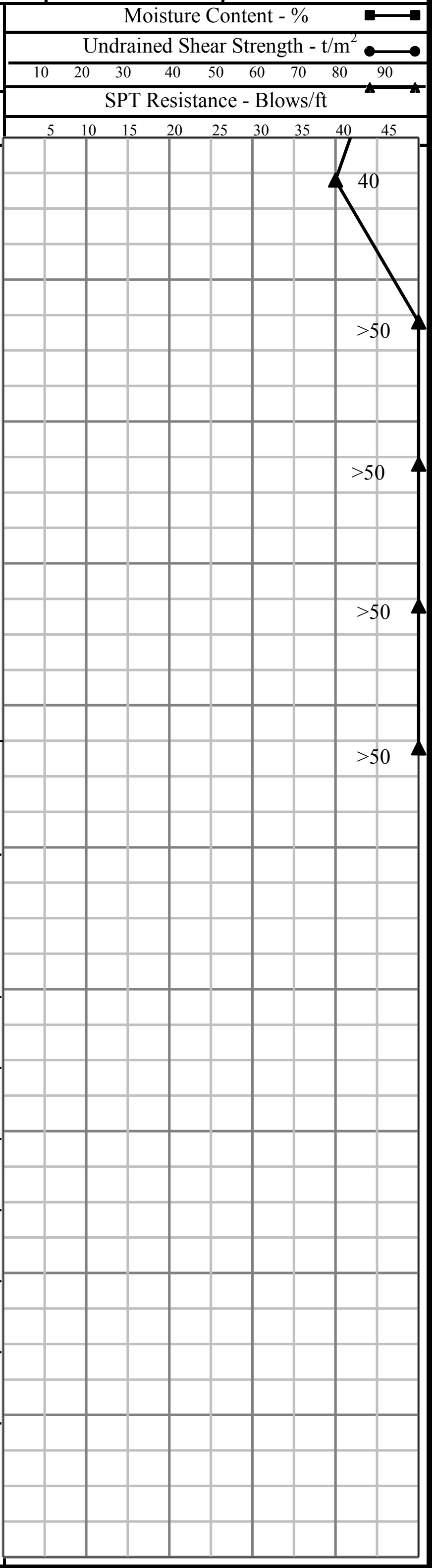
Logged By :
Dimuthu
Supervised By:
Lakshitha
Drilled By:
Saman

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	Fresh Rock

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-01
Client	**M/s. Earth System Science Co., Ltd**			Sheet	2 of 3
Location	Mawathagama	Rig	4005-0016	Core Diameter	54.00mm
Date of Started	18.09.2017	Drilling Method	Rotary	Casing depth	14.00m
Date of Finished	27.09.2017	Casing Diameter	76.00mm	Elevation (m)	193.343

Ground Water level	14.00m
Coordinates	464796.950E 543826.986N

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	Field Records (SPT) 15cm	15cm	15cm	N
10.00							Continue from Page 1				
	☒	D11	SS	183.34	10.00		Clayey SAND, light brown, angular to sub angular, fine to coarse, lateritic sand	14	17	23	**40**
11.00	☒	D12	SS	182.8	10.50		Clayey SAND with gravels, light brown, angular to sub angular, fine to coarse, lateritic sand with few gravels	21	HB		>50
				182.14	11.20		Slightly clayey SAND, light brown, angular to sub angular, fine to coarse, lateritic sand				
12.00							Clayey SAND, brown, gray, reddish brown, fine to coarse, angular to sub-angular				
	☒	D13	SS	181.34	12.00			13cm/ HB			**>50**
13.00	☒	D14	SS	181.213	12.13		Silty SAND, brown, gray, black, fine to coarse, sub-rounded to rounded sand with mica traces	10cm/ HB			**>50**
14.00	☒	D15	SS	179.343	14.00		Silty SAND, black, brown, fine to coarse, subrounded to rounded, sand with traces of mica	14cm/ HB			>50
				179.143	14.20		QUARTZITE, moderately weathered, light brown, offwhite, strong, undulating smooth QUARTZITE rock	**CR=39%**		**RQD=0%**	
15.00			CS	178.343	15.00		QUARTZITE, moderately weathered, light brown, offwhite, strong, surface staining brown undulating smooth, QUARTZITE rock	**CR=80%**		**RQD=40%**	
16.00			CS	177.343	16.00		QUARTZITE, moderately weathered, light brown, offwhite, strong, surface staining brown undulating smooth, QUARTZITE rock	**CR=35%**		**RQD=0%**	
			CS	176.843	16.50		QUARZITE, completely weathered, reddish brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments				
17.00			CS	176.343	17.00		QUARZITE, completely weathered, reddish brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments				
			CS	175.843	17.50		QUARZITE, completely weathered, reddish brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments				
18.00			CS	175.343	18.00		QUARZITE, completely weathered, reddish brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments				
			CS	174.843	18.50		QUARZITE, highly weathered, medium strong, gray, offwhite, QUARTZITE	**CR=60%**		**RQD=0%**	
19.00			CS	174.343	19.00		QUARZITE, completely weathered, reddish brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments				
20.00											



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)

GWL : Ground Water Level observed inside the Borehole, after the saturation

NE Not Encountered

HB -Hammer Bounce

FD - Free Down

D - Disturbed Sample
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Cr - Core Recovery (%)
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L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks

Existing ground level considered as the zero level

Logged By :
Dimuthu

Supervised By:
Lakshitha

Drilled By:
Saman

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock
				Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD. SITE INVESTIGATIONS DIVISION	NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494	Format No: ELS-SI-02

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-01
Client	**M/s. Earth System Science Co., Ltd**			Sheet	3 of 3
Location	Mawathagama	Rig	4005-0016	Core Diameter	54.00mm
Date of Started	18.09.2017	Drilling Method	Rotary	Casing depth	14.00m
Date of Finished	27.09.2017	Casing Diameter	76.00mm	Elevation (m)	193.343

Ground Water level 14.00m

Coordinates: 464796.950E, 543826.986N

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	Field Records (SPT) 15cm	15cm	15cm	N	Moisture Content - % ■—■; Undrained Shear Strength - t/m^2 ●—●; 10 20 30 40 50 60 70 80 90; SPT Resistance - Blows/ft ▲—▲; 5 10 15 20 25 30 35 40 45
20.00							Continue from Page 1					
			CS	173.343	20.00		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
21.00			CS	172.84	20.50		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
				172.34	21.00		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
22.00			CS	171.84	21.50		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
				171.34	22.00		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
23.00			CS	170.84	22.50		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
			CS	170.34	23.00		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
24.00				169.84	23.50		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
			CS	169.34	24.00		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
25.00			CS	168.84	24.50		Silty SAND, completely weathered rock, reddish brown, moist,gray, silty SAND with mica traces					
			CS	168.34	25.00		QUARZITE, completely weathered, reddish brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					
26.00			CS	167.84	25.50		QUARZITE, completely weathered, reddish brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					
			CS	167.34	26.00		QUARZITE, completely weathered, reddish brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					
27.00			CS	166.84	26.50		QUARZITE, completely weathered, light brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					
			CS	166.34	27.00		QUARZITE, completely weathered, light brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					
28.00			CS	165.84	27.50		QUARZITE, completely weathered, light brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					
			CS	165.34	28.00		QUARZITE, completely weathered, light brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					
29.00			CS	164.84	28.50		QUARZITE, completely weathered, light brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					
			CS	164.34	29.00		QUARZITE, completely weathered, light brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					
30.00			CS	163.84	29.50		QUARZITE, completely weathered, light brown, light brown, slightly clayey, medium to coarse, sand with fine gravel size quartzite rock fragments					

Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks: Existing ground level considered as the zero level

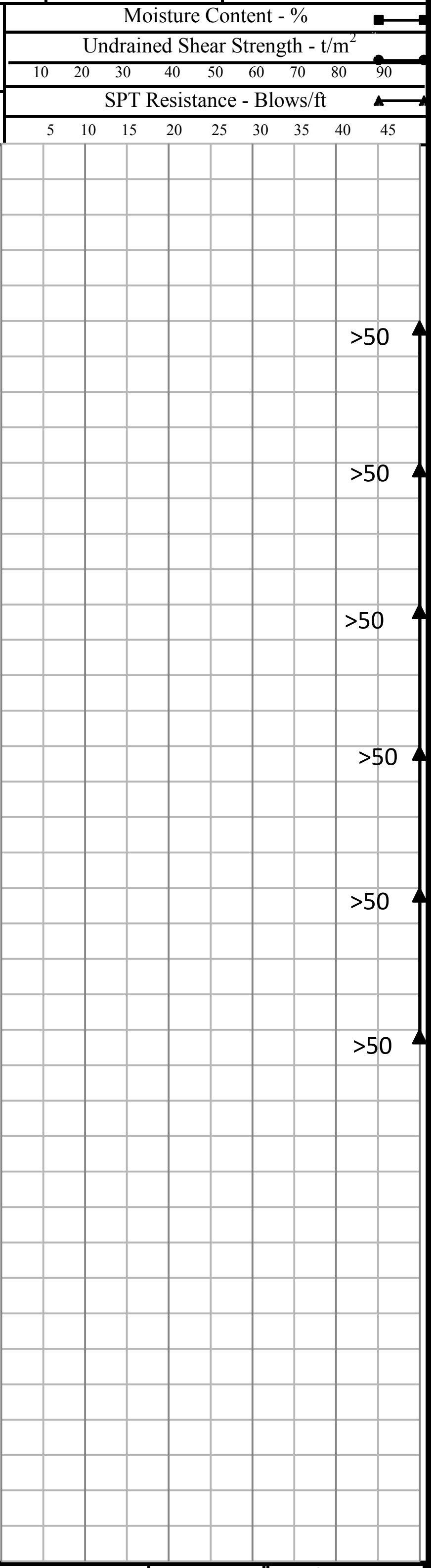
Logged By : Dimuthu
Supervised By: Lakshitha
Drilled By: Saman

Made Ground
Clay
Silt
Sand
Gravel
Organic Matter
Laterite Nodules
Silty Sand
Completely Weathered Rock
Highly Weathered Rock
Fresh Rock

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-02
Client	**M/s. Earth System Science Co., Ltd**			Sheet	1 of 5
Location	Mawathagama	Rig	4005-0016	Core Diameter	54.00mm
Date of Started	20.08.2017	Drilling Method	Rotary	Casing depth	50.00m
Date of Finished	28.08.2017	Casing Diameter	76.00mm	Elevation (m)	215.906

Ground Water level	14.00m
Coordinates	464860.359E 543841.280N

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	Field Records (SPT) 15cm	15cm	15cm	N
0.00							Ground level				
		D1	DS	215.9	0.00		SAND with Gravels, brown, fine to coarse, angular to sub-angular, slightly clayey SAND with gravels				
1.00		D2	SS	214.9	1.00		SAND with Gravels,very dense, brown, fine to coarse, angular to sub-angular, slightly clayey SAND with gravels	22	46	71	>50
				214.7	1.20						
			WS								
2.00		D3	SS				SAND with Gravels, pink,brown,offwhite fine to coarse, angular to sub-angular, slightly clayey SAND with gravels	6	23	HB	>50
			WS								
3.00		D4	SS	212.9	3.00		SAND with Gravels,very dense, pink brown, fine to coarse, angular to sub-angular, slightly clayey SAND with gravels	17	HB		>50
			WS	212.5	3.45		SAND with Gravels,very dense,brown,yellowish brown fine to coarse, angular to sub-angular, slightly clayey SAND with gravels				
4.00		D5	SS	211.9	4.00		QUARTZITE, completely weathered, brown, pink, fine to coarse, angular SAND with GRAVELS	8/HB			>50
			WS	211.5	4.45		QUARTZITE, completely weathered, brown, pink, fine to coarse, angular SAND with GRAVELS				
5.00		D6	SS					24	42	HB	>50
				210.7	5.25		QUARTZITE, completely weathered, brown,reddish brown, fine to coarse, angular SAND with GRAVELS				
			WS								
6.00		D7	SS				QUARTZITE, completely weathered, brown, light brown, fine to coarse, angular SAND with GRAVELS	40	HB		>50
				209.8	6.15						
			CS				SAND, brown, fine to coarse sand with gravels				
7.00				208.9	7.00						
			CS				SAND, pink, light brown, fine to coarse, angular to sub angular SAND with gravels				
8.00			CS	207.9	8.00		Silty SAND, Light pink, fine to coarse, sub-angular to sub rounded sand				
			CS	207.4	8.50		QUARTZITE, completely weathered, brown, light brown, fine to coarse, angular SAND with GRAVELS				
9.00											
			CS								
			CS								
10.00											



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
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FD - Free Down

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SS -SPT Sample
W - Water Sample
WS-Wash Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks

Existing ground level considered as the zero level

Logged By : Dimuthu
Supervised By: Lakshitha
Drilled By: Saman

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	Fresh Rock

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-02
Client	**M/s. Earth System Science Co., Ltd**			Sheet	2 of 5
Location	Mawathagama	Rig	4005-0016	Core Diameter	54.00mm
Ground Water level	14.00m				
Date of Started	20.08.2017	Drilling Method	Rotary	Casing depth	50.00m
Date of Finished	28.08.2017	Casing Diameter	76.00mm	Elevation (m)	215.906
Coordinates	464860.359E 543841.280N				

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
10.00							Continue from Page 1				
			CS				QUARTZITE, completely weathered, brown, light brown, fine to coarse, angular SAND with GRAVELS				
			CS								
11.00			CS								
			CS								
12.00			CS								
			CS								
13.00			CS								
			CS	202.4	13.50		Silty SAND with gravels, brown, light brown, fine to coarse, angular to sub angular silty SAND				
14.00			CS	▼ G.W.L. at 201.9m							
			CS								
15.00			CS								
			CS								
16.00			CS								
			CS								
17.00			CS								
			CS	198.4	17.50		SAND, brown, light brown, fine to coarse, angular to sub-angular, slightly clayey SAND				
18.00			CS								
			CS								
19.00			CS								
			CS								
20.00											

Field Records (SPT)

Moisture Content - % ■—■

Undrained Shear Strength - t/m^2 ●—●

10 20 30 40 50 60 70 80 90

SPT Resistance - Blows/ft ▲—▲

5 10 15 20 25 30 35 40 45

Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
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UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

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UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks

Existing ground level considered as the zero level

Logged By : Dimuthu
Supervised By: Lakshitha
Drilled By: Saman

▨ Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	Fresh Rock
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-02
Client	**M/s. Earth System Science Co., Ltd**			Sheet	3 of 5
Location	Mawathagama	Rig	4005-0016	Core Diameter	54.00mm
Date of Started	20.08.2017	Drilling Method	Rotary	Casing depth	50.00m
Date of Finished	28.08.2017	Casing Diameter	76.00mm	Elevation (m)	215.906

Ground Water level	14.00m
Coordinates	464860.359E 543841.280N

Moisture Content - % ■—■
Undrained Shear Strength - t/m² ●—●
10 20 30 40 50 60 70 80 90
SPT Resistance - Blows/ft ▲—▲
5 10 15 20 25 30 35 40 45

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
20.00							Continue from Page 1				
21.00			CS CS	195.9	20.00		Clayey SAND, brown, light brown, fine to coarse, angular to sub-angular, clayey SAND				
22.00 23.00 24.00 25.00 26.00			CS CS CS CS CS CS CS CS CS CS CS	194.9	21.00		QUARTZITE, completely weathered, pink, light brown, fine to coarse, angular SAND with GRAVELS				
27.00			CS	189.4	26.50		QUARTZITE,highly weathered, weak, pink, brown, QUARTZITE rock fragments				
28.00 29.00 30.00			CS CS CS CS	188.9	27.00		QUARTZITE, completely weathered, pink, light brown, fine to coarse, angular SAND with GRAVELS				

Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
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SS -SPT Sample
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WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks
Existing ground level considered as the zero level

Logged By :
Dimuthu
Supervised By:
Lakshitha
Drilled By:
Saman

▨ Made Ground	▨ Silt	○ Gravel	▲ Laterite Nodules	▨ Completely Weathered Rock	▨ Fresh Rock
▨ Clay	▨ Sand	▨ Organic Matter	▨ Silty Sand	▨ Highly Weathered Rock	

ENGINEERING & LABORATORY SERVICES (PVT) LTD.

SITE INVESTIGATIONS DIVISION

NO 62/3, Neelammahara Road,
Katuwawala, Sri Lanka.
Tel: 0114 309 494

Format No:
ELS-SI-02

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-02
Client	**M/s. Earth System Science Co., Ltd**			Sheet	4 of 5
Location	Mawathagama	Rig		Core Diameter	54.00mm
Date of Started	20.08.2017	Drilling Method	Rotary	Casing depth	50.00m
Date of Finished	28.08.2017	Casing Diameter	76.00mm	Elevation (m)	215.906

Ground Water level 14.00m

Coordinates: 464860.359E, 543841.280N

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	Field Records (SPT) 15cm	15cm	15cm	N
30.00							Continue from Page 1				
			CS				Same as previous description				
			CS								
31.00											
			CS								
32.00			CS								
				183.9	32.00		QUARTZITE,highly weathered, weak, pink, brown, QUARTZITE rock fragments				
			CS								
				183.4	32.50		QUARTZITE, completely weathered, pink, light brown, fine to coarse, angular SAND with GRAVELS				
33.00			CS								
			CS								
34.00			CS								
			CS								
35.00			CS								
				180.9	35.00		Silty SAND, completely weathered rock, compacted, fine to coarse, subrounded to rounded, SAND with silt				
			CS								
36.00			CS								
			CS								
37.00											
			CS								
38.00			CS								
				177.9	38.00		Well graded SAND, completely weathered rock to highly weathered rock, brown, fine to coarse, SAND with weathered rock fragments				
			CS								
39.00											
				176.9	39.00		Biotite Gneiss rock fragment with well graded SAND, highly weathered rock, brown, fine to coarse, SAND with weathered rock fragments				
			CS								
40.00											

Moisture Content - % ■—■

Undrained Shear Strength - t/m^2 ●—●

10 20 30 40 50 60 70 80 90

SPT Resistance - Blows/ft ▲—▲

5 10 15 20 25 30 35 40 45

Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
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Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

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UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks

Existing ground level considered as the zero level

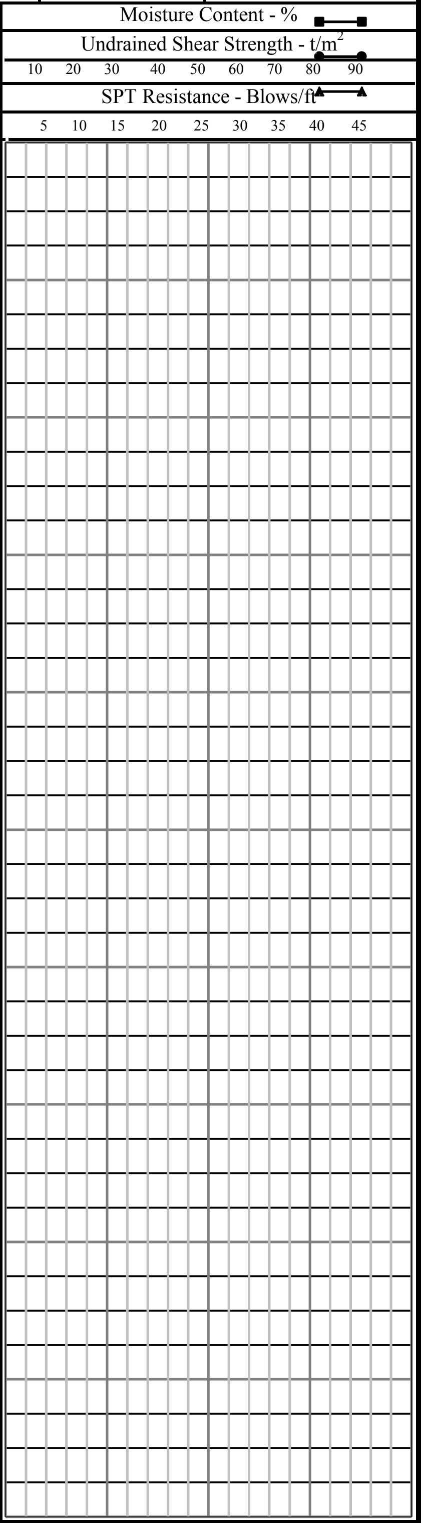
Logged By : Dimuthu
Supervised By: Indunil
Drilled By: Dhanushka

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	Fresh Rock

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-02
Client	**M/s. Earth System Science Co., Ltd**			Sheet	5 of 5
Location	Mawathagama	Rig		Core Diameter	54.00mm
Date of Started	20.08.2017	Drilling Method	Rotary	Casing depth	50.00m
Date of Finished	28.08.2017	Casing Diameter	76.00mm	Elevation (m)	215.906

Ground Water level	14.00m
Coordinates	464860.359E 543841.280N

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	Field Records (SPT) 15cm	15cm	15cm	N
40.00							Continue from Page 1				
				175.9	40.00		Well graded SAND, completely weathered rock brown, fine to coarse, SAND with weathered rock fragments				
		CS									
41.00											
		CS									
42.00											
		CS									
43.00											
			CS								
44.00											
			CS								
45.00											
			CS								
46.00											
				169.9	46.00		Biotite gniess rock fragments, highly weathered rock to moderately, brown, fine to coarse, SAND with weathered rock fragments				
			CS								
47.00											
			CS								
48.00			CS								
			CS								
49.00			CS								
			CS				The borehole was terminated at the depth of 50.00m				
50.00											



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
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NE Not Encountered
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CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

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pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks

Existing ground level considered as the zero level

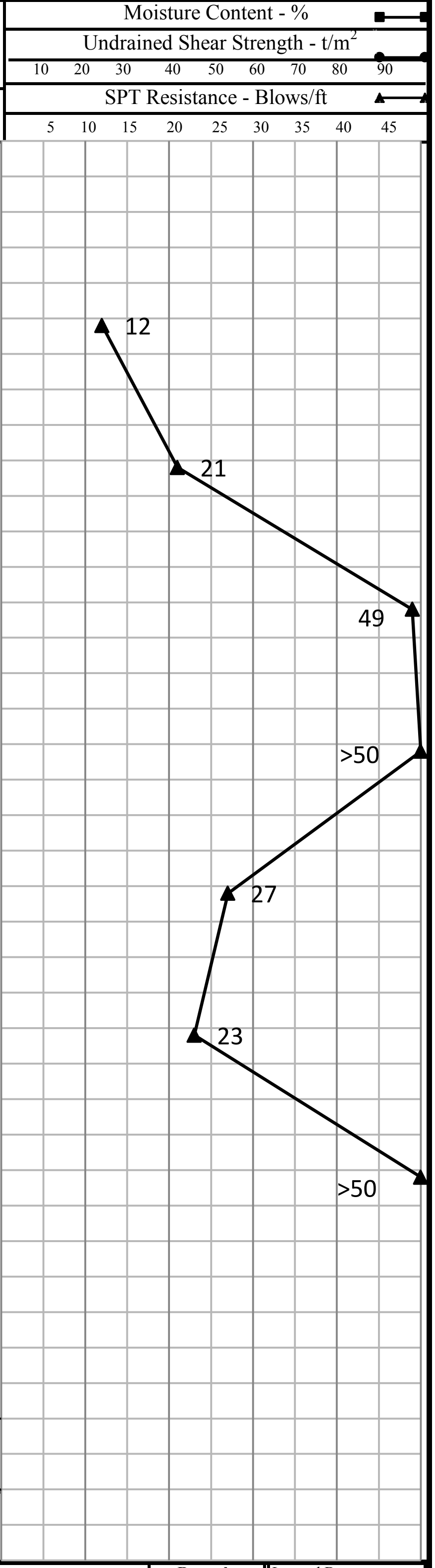
Logged By : Dimuthu
Supervised By: Indunil
Drilled By: Dhanushka

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	Fresh Rock

	ENGINEERING & LABORATORY SERVICES (PVT) LTD. **SITE INVESTIGATIONS DIVISION**	NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494	Format No: ELS-SI-02

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**					Borehole No	BT-03
Client	**M/s. Earth System Science Co., Ltd**					Sheet	1 of 3
Location	Mawathagama	Rig	4005-0016	Core Diameter	54.00mm	Ground Water level	12.00m
Date of Started	24.07.2017	Drilling Method	Rotary	Casing depth	15.00m	Coordinates	464953.016E
Date of Finished	02.08.2017	Casing Diameter	76.00mm	Elevation (m)	190.447		543862.250N

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	Field Records (SPT) 15cm	15cm	15cm	N
0.00							Ground level				
	☒	D1	DS	190.4	0.00		CLAYEY SAND(SC), brown, slightly moist, angular to sub angular, fine to coarse, SAND with few gravels				
			DS	189.8	0.60		CLAYEY SAND(SC), brown, slightly moist, angular to sub angular, fine to coarse, SAND with few gravels				
1.00											
	☒	D2	SS	189.4	1.00		Poorly graded SAND(SP-SM) with silt, fine to coarse, medium dense, brown, moist, sub angular to sub rounded, po0rly graded sand with few gravels	4	5	7	**12**
			DS								
2.00											
	☒	D3	SS					6	7	14	**21**
			DS								
3.00											
	☒	D4	SS	187.4	3.00		Poorly graded SAND(SP-SM) with silt, fine to coarse, Dense to very dense, brown, moist, sub angular to sub rounded, poorly graded sand with few gravels	18	25	24	**49**
			DS								
4.00				186.7	3.70		Poorly graded SAND(SP-SM) with silt, fine to coarse, Very dense, brown, gray moist, angular to sub angular, SAND				
	☒	D5	SS					HB			**>50**
			DS	186.0	4.45		Poorly graded SAND(SP), fine to coarse, medium dense, pink, light brown, moist, angular to sub angular, SAND				
5.00											
	☒	D6	SS					5	12	15	**27**
			DS								
6.00											
	☒	D7	SS					6	11	12	**23**
			DS								
7.00											
	☒	D8	SS	183.4	7.00		Poorly graded SAND(SP), fine to coarse, very dense, pink, light brown, moist, sub angular to sub angular, SAND	14	36	HB	**>50**
8.00											
			CS	182.2	8.25		Poorly graded SAND (SP) fine to coarse,very dense, brown, moist, angular to sub angular SAND with few gravels COMPLETELYWEATHERED ROCK				
9.00											
				181.4	9.00		Garnet bearing QUARTZITE, brown, red, pink, highly weathered,medium strong,natural discotinous dip angle 34 degrees	CR=31%		RQD=31%	
				181.0	9.45		COMPLETELYWEATHERED ROCK				
10.00											



Sample Key / Test Key

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pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks

Existing ground level considered as the zero level

Logged By : Dimuthu
Supervised By: Indunil
Drilled By: Dhanushka

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock
				Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

NO 62/3, Neelammahara Road,
Katuwawala, Sri Lanka.
Tel: 0114 309 494

Format No:
ELS-SI-02

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-03
Client	**M/s. Earth System Science Co., Ltd**			Sheet	2 of 3
Location	Mawathagama	Rig	4005-0016	Core Diameter	54.00mm
Date of Started	24.07.2017	Drilling Method	Rotary	Casing depth	15.00m
Date of Finished	02.08.2017	Casing Diameter	76.00mm	Elevation (m)	190.447

Ground Water level	12.00m
Coordinates	464953.016E 543862.250N

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	Field Records (SPT) 15cm	15cm	15cm	N
10.00							Continue from Page 1				
				180.4	10.00	Completely Weathered Rock	Poorly graded SAND with SILT(SP-SM), fine to coarse,brown, light brown, moist, sub angular to sub rounded SAND with traces of mica				
			DS								
11.00											
			DS								
12.00				G.W.L at 178.5m			COMPLETELY WEATHERED ROCK				
			DS								
13.00											
			DS				Poorly graded SAND with SILT(SP-SM), fine to coarse,brown, , moist, sub angular to sub rounded SAND with traces of mica and weathered rock particles				
14.00											
			DS								
15.00											
				174.9	15.50		Biotite GNEISS, brown,light brown, highly weathered,medium strong, undulating rough surfaces of rock	CR=66%		RQD=0%	
16.00			CS								
			CS	174.4	16.00		Biotite GNEISS, brown,light brown, highly weathered,medium strong, undulating rough surfaces of rock	CR=68%		RQD=0%	
			CS	173.9	16.50		Biotite GNEISS, brown,light brown, highly weathered,medium strong, undulating rough surfaces of rock	CR=62%		RQD=0%	
17.00											
			CS	173.4	17.00		Biotite GNEISS, brown,light brown, highly weathered,medium strong, undulating rough surfaces of rock	CR=90%		RQD=0%	
			CS	172.9	17.50		Biotite GNEISS, gray, light gray, black,moderately weathered, strong,natural discotinuous dip angle 0-35	CR=98%		RQD=54%	
18.00				172.4	18.00		(18.00-18.29)m CORE LOSS				
			CS	172.2	18.29		Biotite GNEISS, gray, light gray, black,moderately weathered ,strong,natural discotinuous dip angle 0-35	CR=99%		RQD=99%	
			CS	171.9	18.50		Biotite GNEISS, gray, light gray, black,moderately weathered,strong,natural discotinuous dip angle 0-35	CR=90%		RQD=86%	
19.00			CS	171.4	19.00		Biotite GNEISS, gray, light gray, black,moderately weathered weathered,medium strong,natural discotinuous dip angle 0-35	CR=90%		RQD=82%	
			CS	170.9	19.50		Biotite GNEISS, gray, light gray, black,moderately weathered ,strong,natural discotinuous dip angle 0-35	CR=78%		RQD=0%	
20.00											

Moisture Content - %
Undrained Shear Strength - t/m^2
10 20 30 40 50 60 70 80 90
SPT Resistance - Blows/ft
5 10 15 20 25 30 35 40 45

Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl$^-$ - Cloride Content

Remarks
Existing ground level considered as the zero level

Logged By :
Dimuthu
Supervised By:
Indunil
Drilled By:
Dhanushka

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	Fresh Rock

Project	**Technical Assitance for Improvement of Capacity for Planning of Road Tunnel**			Borehole No	BT-03
Client	**M/s. Earth System Science Co., Ltd**			Sheet	3 of 3
Location	Mawathagama	Rig	4005-0016	Core Diameter	54.00mm
Date of Started	24.07.2017	Drilling Method	Rotary	Casing depth	15.00m
Date of Finished	02.08.2017	Casing Diameter	76.00mm	Elevation (m)	190.447

Ground Water level 12.00m

Coordinates 464953.016E 543862.250N

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	Field Records (SPT) 15cm	15cm	15cm	N
20.00							Continue from Page 1				
			CS	170.4	20.00		Biotite GNEISS, gray, light gray, black,moderately weathered, strong,natural discotinuous dip angle 0-35	CR=80%		RQD=75%	
21.00			CS	169.4	21.00		Biotite GNEISS, gray, light gray, black,moderately weathered ,strong,natural discotinuous dip angle 0-35	CR=79%		RQD=71%	
22.00				168.4	22.00		Biotite GNEISS, gray, light gray, black,moderately weathered weathered,medium strong,natural discotinuous dip angle 0-35	CR=27%		RQD=0%	
			CS	167.9	22.50		(22.35-23.00)m core loss				
23.00 24.00			CS CS	167.4	23.00		Well graded SAND with clay (SP-SC), fine to coarse, black, gray, offwhite, sub clay COMPLETELY WEATHERED ROCK				
25.00 26.00				165.4	25.00		Well graded SAND with clay (SP-SC), fine to coarse, brown, offwhite, sub angular to sub rounded, SAND with clay COMPLETELY WEATHERED ROCK				
27.00				163.9	26.50		SANDY lean CLAY(CL), brown, very stiff, brown, light brown, moist, moderately strong sandy CLAY				
				163.4	27.00		CORE LOSS				
28.00			CS CS	163.1	27.33		SANDY lean CLAY(CL), brown, very stiff, brown, light brown, moist, moderately strong sandy CLAY, last 5cm sample is QUARTZITE rock sample				
			CS	162.4	28.00		QUARTZITE, white, light brown, moderately weathered, strong, rough QUARTZITE	CR=70%		RQD=0%	
29.00			CS	161.9	28.50		QUARTZITE, white, light brown, moderately weathered, strong, rough QUARTZITE	CR=40%		RQD=0%	
			CS	161.4	29.00		QUARTZITE, white, light brown, moderately weathered, strong, rough QUARTZITE	CR=44%		RQD=0%	
30.00			CS	160.9	29.50		QUARTZITE, white, light brown, moderately weathered, strong, rough QUARTZITE	CR=40%		RQD=0%	

Moisture Content - % ■—■

Undrained Shear Strength - t/m^2 ●—●

10 20 30 40 50 60 70 80 90

SPT Resistance - Blows/ft ▲—▲

5 10 15 20 25 30 35 40 45

Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)

GWL : Ground Water Level observed inside the Borehole, after the saturation

NE Not Encountered

HB -Hammer Bounce

FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks

Existing ground level considered as the zero level

Logged By :
Dimuthu

Supervised By:
Indunil

Drilled By:
Dhanushka

▨ Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	Fresh Rock

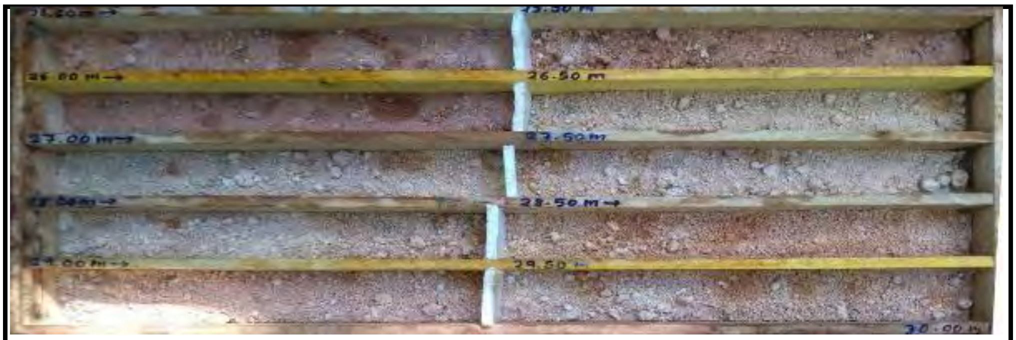
PHOTOS OF BORE-CORES

HOLE NO.	**BT-01**	ELEVATION	
LOCATION		DEPTH (m)	0.0 to 25.0

Depth (m)	Formation	Rock Grade
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		
21		
22		
23		
24		
25		

PHOTOS OF BORE-CORES

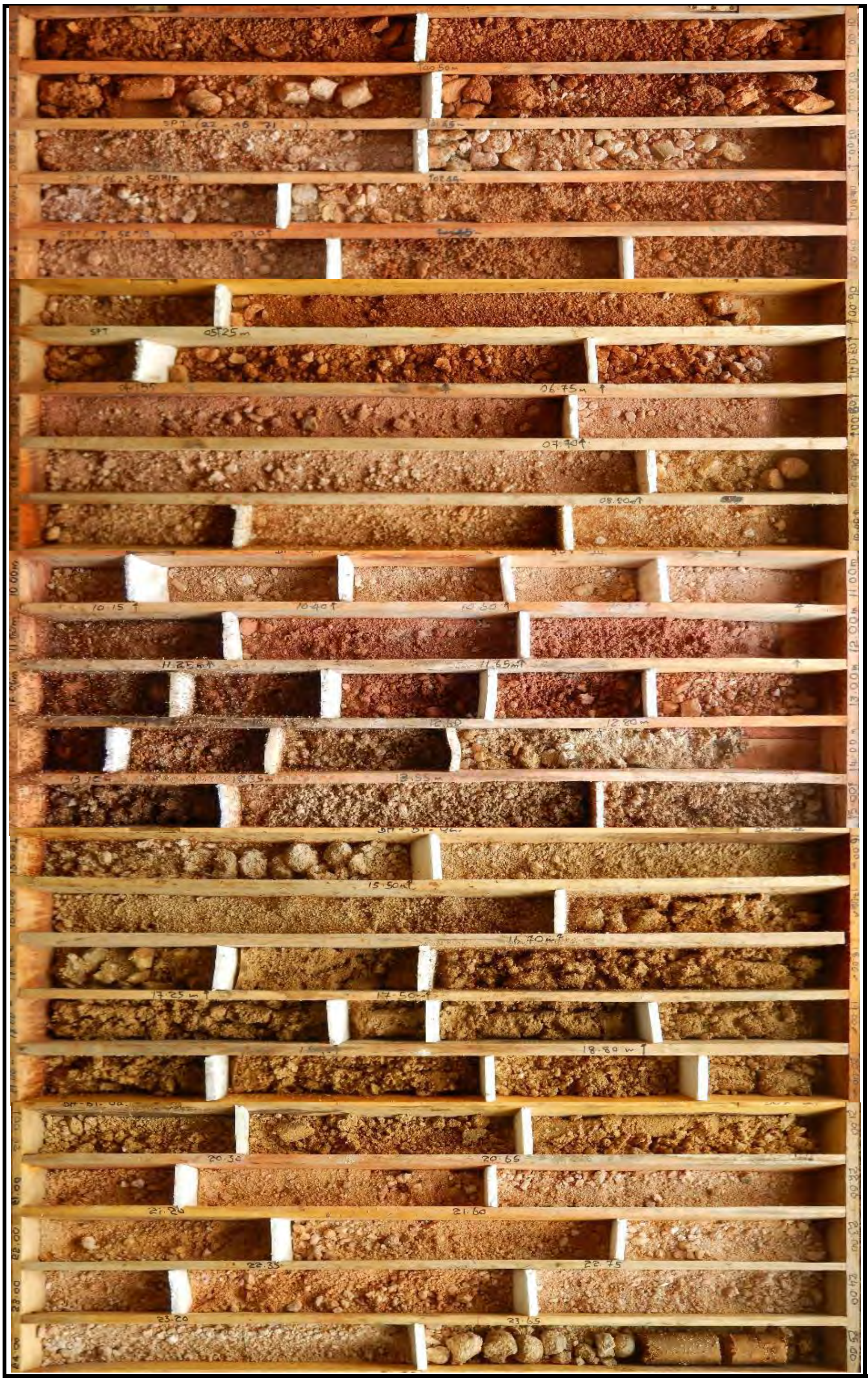
HOLE NO.	**BT-01**	ELEVATION	
LOCATION		DEPTH (m)	26.0 to 30.0



Depth (m)	Formation	Rock Grade
26		
27		
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		
39		
40		
41		
42		
43		
44		
45		
46		
47		
48		
49		
50		

PHOTOS OF BORE-CORES

HOLE NO.	**BT-02**	ELEVATION	
LOCATION		DEPTH (m)	0.0 to 25.0



Depth (m)	**Formation**	**Rock Grade**
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		
21		
22		
23		
24		
25		

PHOTOS OF BORE-CORES

HOLE NO.	**BT-02**	ELEVATION	
LOCATION		DEPTH (m)	26.0 to 50.0

Depth (m)	Formation	Rock Grade
26		
27		
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		
39		
40		
41		
42		
43		
44		
45		
46		
47		
48		
49		
50		

PHOTOS OF BORE-CORES

HOLE NO.	**BT-03**	ELEVATION	
LOCATION		DEPTH (m)	0.0 to 25.0

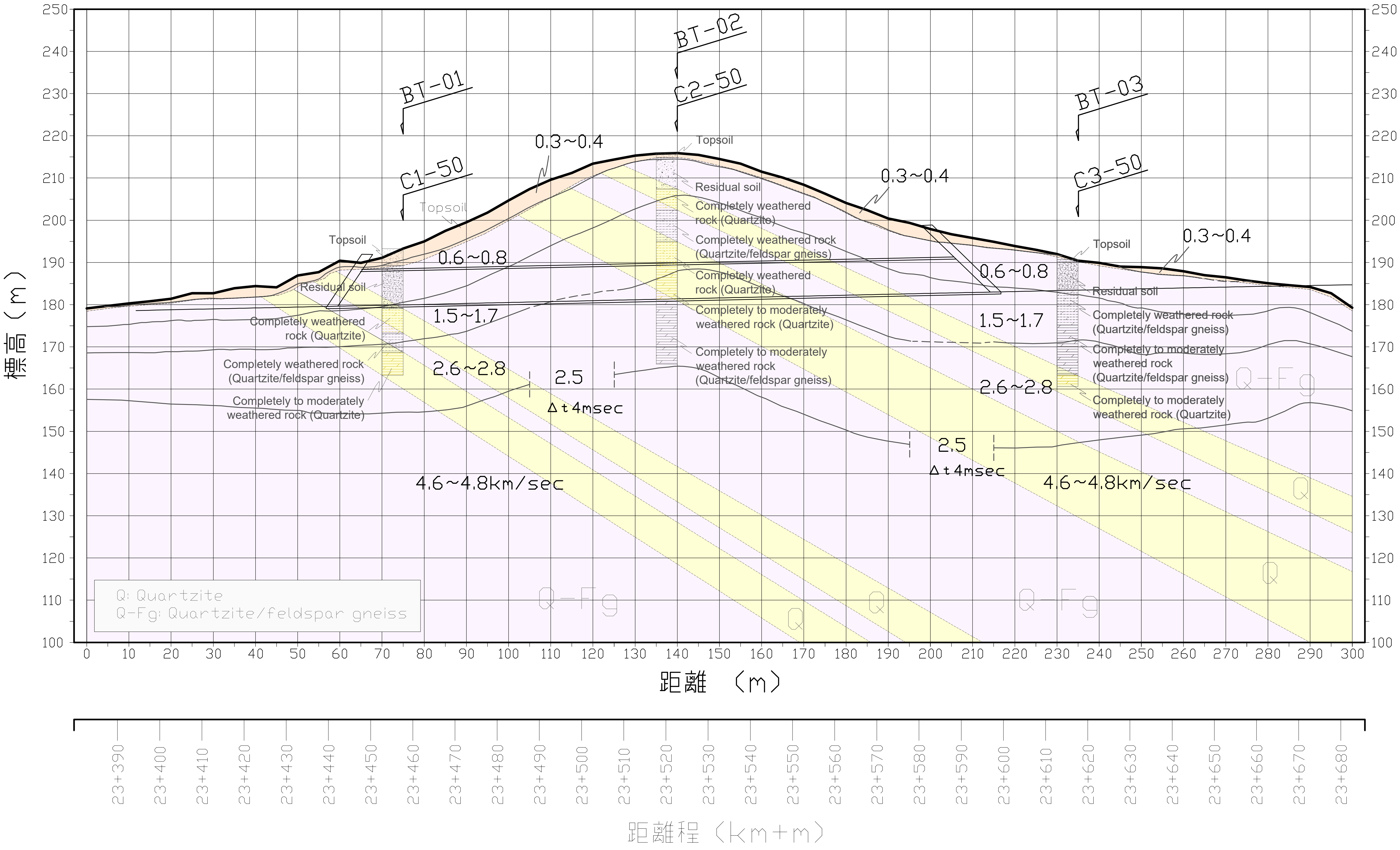
Depth (m)	**Formation**	**Rock Grade**
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		
21		
22		
23		
24		
25		

PHOTOS OF BORE-CORES

HOLE NO.	**BT-03**	ELEVATION	
LOCATION		DEPTH (m)	26.0 to 30.0

Depth (m)	Formation	Rock Grade
26		
27		
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		
39		
40		
41		
42		
43		
44		
45		
46		
47		
48		
49		
50		

解析断面図 T2



BT-01
C1-50
BT-02
C2-50
BT-03
C3-50
Topsoil
Residual soil
Completely weathered rock (Quartzite)
Completely weathered rock (Quartzite/feldspar gneiss)
Completely to moderately weathered rock (Quartzite)
Completely to moderately weathered rock (Quartzite/feldspar gneiss)
0.3~0.4
0.6~0.8
1.5~1.7
2.6~2.8
2.5
Δt4msec
4.6~4.8km/sec
Q-Fg
Q
Q: Quartzite
Q-Fg: Quartzite/feldspar gneiss
標高（m）
距離 （m）
距離程（km+m）
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
280
290
300
23+390
23+400
23+410
23+420
23+430
23+440
23+450
23+460
23+470
23+480
23+490
23+500
23+510
23+520
23+530
23+540
23+550
23+560
23+570
23+580
23+590
23+600
23+610
23+620
23+630
23+640
23+650
23+660
23+670
23+680

Summary of Laboratory Soil Test Results

Borehole No.	Sample No.	Sample depth (m)	Index and Mechanical Properties of Soils													Descriptions of Soil Samples
			Grain size distribution (%)							Water content	Natural density	Specific gravity	Liquid limit	Plastic limit	Plasticity index	
			Gravel			Sand			Silt and Clay							
			20 - 60	6.0 - 20	2.0 - 6.0	0.6 - 2.0	0.20 - 0.60	0.060 - 0.20	< 0.060	W (%)	g (kN/m^3)	G_s	W_L (%)	W_P (%)	I_P (%)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
BT-01		6.00 - 6.45		1.8	3.3	35.2	25.5	6.6	27.6			2.70	35	23	12	
		8.00 - 8.45			6.5	31.1	31.3	8.5	22.6			2.63	59	36	23	
		9.00 - 9.45		15.6	8.4	26.9	19.9	5.7	23.6			2.61				
		11.00 - 11.45		9.4	31.8	32.7	7.5	1.7	17.0			2.74				
		17.00 - 17.20		14.4	34.7	29.3	4.9	2.4	14.3			2.65				
BT-02		1.00 - 1.20		5.9	23.5	41.4	20.4	2.9	5.9			2.68				
		1.20 - 1.45		1.8	7.9	42.3	25.4	4.2	18.4			2.65				
		3.00 - 3.30		0.2	10.5	43.2	28.8	4.7	12.7			2.63				
		14.25 - 14.50		9.2	10.2	31.1	21.1	5.0	23.5			2.72				
		20.00 - 20.30		17.7	13.6	33.0	16.1	2.7	17.0			2.69				
		35.00 - 35.35			2.2	8.9	49.9	13.4	25.6			2.68	42	23	19	
BT-03		5.00 - 5.45			33.0	23.6	28.1	13.8	1.4			2.68				
		7.00 - 7.45			13.0	19.9	43.1	22.4	1.6			2.69				
Count												13	3	3	3	
Maximum value												2.74	59	36	23	
Minmum value												2.61	35	23	12	
Mean value												2.67	45	27	18	
Median mean value												2.68	42	23	19	

Summary of Laboratory Rock Test Results

Borehole No.	Sample No.	Sample depth (m)	Physical and Mechanical Properties of Rocks					Descriptions of Rock Samples
			Specific gravity	Water absorption	Dry Density	Point Load	Uniaxial compression	
			G_s	(%)	g (g/cm^3)	$I_{s(50)}$ (MPa)	UCS (MPa)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
BT-01		15.78 - 16.00	2.64	0.25	2.72	2.81	61.2	
BT-02		39.52 - 39.60				0.81		
		46.11 - 46.20				0.31		
BT-03		17.50 - 17.57				0.99		
		18.29 - 18.36				0.31		
		19.02 - 19.16	2.90	0.86	2.77		23.0	
		19.16 - 19.30			2.93		34.9	
		19.75 - 20.00			3.01		63.2	
		20.19 - 20.30	2.79	0.34	3.04	2.48	66.1	
		20.91 - 21.00				0.93		
		21.00 - 21.20			2.98	1.07	36.3	
		21.36 - 21.44				2.23		
		21.44 - 21.55			2.77		59.4	
		21.30 - 21.50			3.01		53.0	
		21.87 - 22.00			2.91	2.91	23.7	
		22.00 - 22.11	2.74	0.39	2.97	2.33	44.7	
		29.92 - 30.00	2.64	0.38		3.06		
Count			5	5	10	12	10	
Maximum value			2.90	0.86	3.04	3.06	66.1	
Minmum value			2.64	0.25	2.72	0.31	23.0	
Mean value			2.74	0.44	2.91	1.69	46.5	
Median mean value			2.74	0.38	2.95	1.65	48.8	

添付資料　5

水文観測結果

Hydrological Observation Data

1. Tunnel No.1 Area

1.1 Water Levels (unit: m.a.s.l.) and Discharges (L/min) from Springs in Tunnel No.1 Area

Spring No.		T1-S1
Date	GWL(m a.s.l.)	Discharge(L/min)
29/08/2017	116.27	0.5
16/11/2017	116.27	1.0
30/11/2017	116.27	5.0
20/12/2017	116.27	0.0
31/01/2018	116.27	0.0
06/02/2018	116.27	0.0

Spring No.		T1-S4
Date	GWL(m a.s.l.)	Discharge(L/min)
30/08/2017	109.12	0.0
16/11/2017	109.42	1.0
30/11/2017	109.42	30.0
31/01/2018	109.38	0.0
06/02/2018	109.32	0.0

Spring No.		T1-S5
Date	GWL(m a.s.l.)	Discharge(L/min)
30/11/2017	123.18	5.0
31/01/2018	122.83	0.0
06/02/2018	122.73	0.0

Spring No.		T1-S6
Date	GWL(m a.s.l.)	Discharge(L/min)
30/11/2017		120.0
31/01/2018		6.0
06/02/2018		0.0

1.2 Water Levels (unit: m.a.s.l.) and Discharges (L/min) from Springs in Tunnel No.1 Area

River No.	T1-R1
Date	Discharge(L/min)
18/05/2017	0.0
30/08/2017	1.0
18/09/2017	10.0
16/11/2017	30.0
30/11/2017	60.0
20/12/2017	21.6
31/01/2018	8.4
06/02/2018	2.4

River No.	T1-R2
Date	Discharge(L/min)
30/08/2017	2.0
18/09/2017	20.0
16/11/2017	40.0
30/11/2017	60.0
20/12/2017	14.4
31/01/2018	4.8
06/02/2018	2.4

River No.	T1-R3
Date	Discharge(L/min)
30/08/2017	0.0
16/11/2017	0.1
30/11/2017	0.1
31/01/2018	0.0
06/02/2018	0.0

River No.	T1-R4
Date	Discharge(L/min)
30/08/2017	0.0
18/09/2017	0.0
16/11/2017	10.0
30/11/2017	30.0
20/12/2017	6.0
31/01/2018	0.0

River No.	T1-R5
Date	Discharge(L/min)
30/08/2017	0.0
18/09/2017	1.0
16/11/2017	20.0
30/11/2017	30.0
31/01/2018	3.6
06/02/2018	0.0

1.3 Observed Water Levels of Wells in Tunnel No.1 Area (unit: m.a.s.l.)

Well No.	T1-W1
Date	GWL(m a.s.l.)
29/08/2017	123.07
01/12/2017	125.64
20/12/2017	125.64
31/01/2018	124.94
06/02/2018	124.82

Well No.	T1-W2
Date	GWL(m a.s.l.)
29/08/2017	120.19
06/02/2018	121.83

Well No.	T1-W4
Date	GWL(m a.s.l.)
29/08/2017	118.25
16/11/2017	118.45
20/12/2017	118.40
31/01/2018	118.48
06/02/2018	118.35

Well No.	T1-W5
Date	WL(m, depth from the ground.)
29/08/2017	−1.56
06/02/2018	−1.46

Well No.	T1-W6
Date	GWL(m a.s.l.)
29/08/2017	115.81
16/11/2017	117.57
30/11/2017	118.12
20/12/2017	117.45
31/01/2018	116.11
06/02/2018	116.07

Well No.	T1-W7
Date	GWL(m a.s.l.)
29/08/2017	114.69
16/11/2017	115.55
30/11/2017	115.55
20/12/2017	115.39
31/01/2018	115.04
06/02/2018	114.76

Well No.	T1-W8
Date	GWL(m a.s.l.)
29/08/2017	116.65
16/11/2017	117.53
30/11/2017	117.97
20/12/2017	117.54
31/01/2018	117.16
06/02/2018	117.10

Well No.	T1-W9
Date	GWL(m a.s.l.)
29/08/2017	121.09
16/11/2017	124.11
30/11/2017	124.41
20/12/2017	124.87
31/01/2018	123.70
06/02/2018	123.16

Well No.	T1-W10
Date	GWL(m a.s.l.)
29/08/2017	131.88
01/12/2017	133.96
20/12/2017	133.11
31/01/2018	131.51
06/02/2018	131.65

Well No.	T1-W11
Date	GWL(m a.s.l.)
29/08/2017	109.28
30/11/2017	109.88
20/12/2017	109.69
31/01/2018	109.58
06/02/2018	109.48

Well No.:	T1-W13
Date	GWL(m a.s.l.)
30/08/2017	129.51
20/12/2017	130.71
31/01/2018	130.08
06/02/2018	130.14

Well No.	T1-S3
Date	GWL(m a.s.l.)
16/11/2017	128.20
30/11/2017	128.57
31/01/2018	128.01
06/02/2018	127.97

Well No.	T1-W18
Date	GWL(m a.s.l.)
30/08/2017	126.25
16/11/2017	127.23
30/11/2017	127.43
20/12/2017	127.34
31/01/2018	126.93
06/02/2018	126.86

Well No.	T1-W19
Date	GWL(m a.s.l.)
30/08/2017	124.88
16/11/2017	125.28
30/11/2017	125.28
20/12/2017	125.23
31/01/2018	124.93
06/02/2018	124.96

Well No.	T1-W20
Date	GWL(m a.s.l.)
30/08/2017	121.52
30/11/2017	123.02
31/01/2018	122.19
06/02/2018	122.06

Well No.	T1-W21
Date	GWL(m a.s.l.)
16/11/2017	121.89
30/11/2017	122.04
20/12/2017	121.92
31/01/2018	121.58
06/02/2018	121.40

Well No.	T1-W23
Date	GWL(m a.s.l.)
30/08/2017	108.97
30/11/2017	110.38
20/12/2017	110.09
31/01/2018	109.08
06/02/2018	109.12

Well No.	T1-W24
Date	GWL(m a.s.l.)
30/08/2017	111.46
30/11/2017	113.40
20/12/2017	113.29
31/01/2018	113.18
06/02/2018	112.95

2. Tunnel No.2 Area

2.1 Observed Discharges of Rivers in Tunnel No.2 Area (unit: Litre/minute.)

River No.	T2-R1
Date	Discharge(L/min)
10/08/2017	0.0
22/09/2017	0.0
16/11/2017	10.0
01/12/2017	12.0
20/12/2017	18.0
30/01/2018	8.4
05/02/2018	1.0

River No.	T2-R54
Date	Discharge(L/min)
26/07/2017	2.7
10/08/2017	1.8
14/08/2017	2.9
18/08/2017	11.2
19/08/2017	23.7
20/08/2017	25.5
22/08/2017	10.3
23/08/2017	14.3
29/08/2017	9.9
30/08/2017	5.7
16/09/2017	55.5
19/09/2017	46.3
22/09/2017	42.6
16/11/2017	154.8
17/11/2017	220.0
01/12/2017	214.0
20/12/2017	189.0
30/01/2018	140.0
05/02/2018	80.0

River No.	T2-R55
Date	Discharge(L/min)
25/07/2017	1.0
10/08/2017	1.0
14/08/2017	1.0
19/08/2017	8.6
20/08/2017	16.8
22/08/2017	8.7
23/08/2017	10.6
29/08/2017	11.7
30/08/2017	7.9
16/09/2017	0.8
19/09/2017	2.4
22/09/2017	1.8
17/11/2017	130.0
17/11/2017	130.0
01/12/2017	308.0
20/12/2017	130.0
30/01/2018	19.2
05/02/2018	39.6

River No.	T2-R56
Date	Discharge(L/min)
29/08/2017	10.7
16/09/2017	30.6
19/09/2017	38.8
22/09/2017	50.4
17/11/2017	294.0
01/12/2017	256.0
20/12/2017	160.0
30/01/2018	102.0
05/02/2018	56.0

2.2 Observed Water Levels of Wells and Springs in Tunnel No.2 Area (unit: m.a.s.l.)

Well No.	T2-BT-01
Date	GWL(m a.s.l.)
28/09/2017	179.34
16/11/2017	180.75
01/12/2017	181.19
20/12/2017	181.51
30/01/2018	181.62
05/02/2018	181.60

Well No.	T2-BT-02
Date	GWL(m a.s.l.)
16/11/2017	181.09
01/12/2017	181.75
20/12/2017	182.21
30/01/2018	182.63
05/02/2018	182.61

Well No.	T2-BT-03
Date	GWL(m a.s.l.)
28/09/2017	178.90
16/11/2017	179.85
01/12/2017	180.02
20/12/2017	180.26
30/01/2018	180.21
05/02/2018	180.15

Well No.	T2-W3
Date	GWL(m a.s.l.)
15/09/2017	180.43
19/09/2017	180.27
22/09/2017	180.16
16/11/2017	181.46
01/12/2017	181.49
20/12/2017	181.46
30/01/2018	181.28
05/02/2018	181.28

Well No.	T2-W4
Date	GWL(m a.s.l.)
25/07/2017	177.09
10/08/2017	177.06
14/08/2017	177.11
19/08/2017	177.21
22/08/2017	176.69
29/08/2017	176.98
15/09/2017	177.34
19/09/2017	177.26
22/09/2017	177.24
16/11/2017	177.76
01/12/2017	177.81
20/12/2017	177.79
30/01/2018	177.77
05/02/2018	177.71

Well No.	T2-W5
Date	GWL(m a.s.l.)
25/07/2017	175.60
10/08/2017	175.50
14/08/2017	175.52
19/08/2017	175.62
22/08/2017	175.50
29/08/2017	175.58
19/09/2017	175.74
22/09/2017	175.72
26/09/2017	175.74
16/11/2017	175.96
01/12/2017	175.93
20/12/2017	175.82
30/01/2018	175.78
05/02/2018	175.70

Well No.	T2-W6
Date	GWL(m a.s.l.)
25/07/2017	174.45
12/08/2017	174.25
14/08/2017	174.44
19/08/2017	174.55
22/08/2017	174.34
23/08/2017	174.55
29/08/2017	174.49
15/09/2017	175.10
19/09/2017	175.19
22/09/2017	175.25
16/11/2017	175.53
01/12/2017	175.53
20/12/2017	175.47
30/01/2018	175.28
05/02/2018	175.27

Well No.	T2-W7
Date	GWL(m a.s.l.)
25/07/2017	172.64
10/08/2017	173.26
14/08/2017	173.46
19/08/2017	173.56
23/08/2017	173.66
29/08/2017	173.71
16/11/2017	173.92
20/12/2017	173.78
30/01/2018	173.57
05/02/2018	173.62

Well No.	T2-S8
Date	GWL(m a.s.l.)
25/07/2017	172.55
10/08/2017	171.88
14/08/2017	172.21
19/08/2017	172.88
23/08/2017	172.83
29/08/2017	172.83
15/09/2017	173.13
19/09/2017	173.18
22/09/2017	173.10
16/11/2017	173.08
01/12/2017	173.11
20/12/2017	173.12
30/01/2018	173.07
05/02/2018	173.08

2.2 Observed Water Levels of Wells and Springs in Tunnel No.2 Area (unit: m.a.s.l.) (continued)

Well No.	T2-W9
Date	GWL(m a.s.l.)
25/07/2017	172.65
10/08/2017	172.69
14/08/2017	172.81
19/08/2017	172.63
23/08/2017	172.80
29/08/2017	172.87
15/09/2017	173.43
19/09/2017	173.51
22/09/2017	173.50
16/11/2017	173.34
01/12/2017	173.53
20/12/2017	173.57
30/01/2018	173.36
05/02/2018	173.60

Well No.	T2-W10
Date	GWL(m a.s.l.)
25/07/2017	175.50
10/08/2017	175.38
14/08/2017	175.52
19/08/2017	175.67
23/08/2017	175.71
29/08/2017	175.39
15/09/2017	176.21
19/09/2017	176.12
22/09/2017	176.17
16/11/2017	176.61
01/12/2017	176.47
20/12/2017	176.35
30/01/2018	176.26
05/02/2018	176.24

Well No.	T2-S11
Date	GWL(m a.s.l.)
25/07/2017	175.45
10/08/2017	175.34
14/08/2017	175.47
19/08/2017	175.77
23/08/2017	175.80
29/08/2017	175.54
15/09/2017	176.02
19/09/2017	176.04
22/09/2017	175.99
16/11/2017	176.25
01/12/2017	176.24
20/12/2017	176.11
30/01/2018	175.90
05/02/2018	175.83

Well No.	T2-W12
Date	GWL(m a.s.l.)
25/07/2017	176.31
10/08/2017	176.17
14/08/2017	176.17
19/08/2017	176.22
23/08/2017	176.19
29/08/2017	176.19
15/09/2017	176.88
19/09/2017	176.87
22/09/2017	176.80
16/11/2017	177.56
02/12/2017	177.37
20/12/2017	177.18
30/01/2018	176.90
05/02/2018	176.88

Well No.	T2-W13
Date	GWL(m a.s.l.)
25/07/2017	175.39
10/08/2017	174.96
14/08/2017	174.96
19/08/2017	174.82
23/08/2017	175.37
29/08/2017	175.27
15/09/2017	175.97
19/09/2017	175.88
22/09/2017	175.88
16/11/2017	176.20
01/12/2017	176.11
20/12/2017	176.15
30/01/2018	175.96
05/02/2018	175.95

Well No.	T2-W14
Date	GWL(m a.s.l.)
25/07/2017	175.90
10/08/2017	175.87
14/08/2017	175.85
19/08/2017	176.02
23/08/2017	176.05
29/08/2017	175.98
15/09/2017	176.52
19/09/2017	176.45
22/09/2017	176.44
16/11/2017	176.80
01/12/2017	176.60
20/12/2017	176.63
30/01/2018	176.46
05/02/2018	176.25

Well No.	T2-W14
Date	GWL(m a.s.l.)
25/07/2017	175.90
10/08/2017	175.87
14/08/2017	175.85
19/08/2017	176.02
23/08/2017	176.05
29/08/2017	175.98
15/09/2017	176.52
19/09/2017	176.45
22/09/2017	176.44
16/11/2017	176.80
01/12/2017	176.60
20/12/2017	176.63
30/01/2018	176.46
05/02/2018	176.25

Well No.	T2-W15
Date	GWL(m a.s.l.)
15/09/2017	176.74
19/09/2017	176.77
22/09/2017	176.71
16/11/2017	177.28
01/12/2017	177.09
20/12/2017	177.00
30/01/2018	176.75
05/02/2018	176.74

Well No.	T2-W16
Date	GWL(m a.s.l.)
15/09/2017	177.31
19/09/2017	177.29
22/09/2017	177.24
26/09/2017	177.26
01/10/2017	177.23
16/11/2017	178.52
01/12/2017	178.08
20/12/2017	177.92
30/01/2018	177.47
05/02/2018	177.40

2.2 Observed Water Levels of Wells and Springs in Tunnel No.2 Area (unit: m.a.s.l.) (continued)

Well No.	T2-W17
Date	GWL(m a.s.l.)
25/07/2017	174.67
10/08/2017	174.53
14/08/2017	174.58
19/08/2017	174.33
23/08/2017	174.53
29/08/2017	174.53
16/09/2017	175.48
19/09/2017	175.61
22/09/2017	175.46
16/11/2017	177.38
01/12/2017	177.34
20/12/2017	177.10
30/01/2018	176.70
05/02/2018	176.63

Well No.	T2-W18
Date	GWL(m a.s.l.)
25/07/2017	174.01
10/08/2017	174.12
14/08/2017	173.94
19/08/2017	174.18
23/08/2017	173.73
29/08/2017	174.00
16/09/2017	175.45
19/09/2017	175.14
22/09/2017	175.19
16/11/2017	176.99
01/12/2017	176.83
20/12/2017	176.68
30/01/2018	176.30
05/02/2018	176.25

Well No.	T2-W19
Date	GWL(m a.s.l.)
25/07/2017	174.73
10/08/2017	174.79
14/08/2017	174.69
19/08/2017	174.82
23/08/2017	174.74
29/08/2017	174.69
16/09/2017	175.08
19/09/2017	175.32
22/09/2017	175.27
16/11/2017	176.90
01/12/2017	176.76
20/12/2017	176.63
30/01/2018	176.28
05/02/2018	176.19

Well No.	T2-W20
Date	GWL(m a.s.l.)
25/07/2017	174.65
10/08/2017	174.82
14/08/2017	174.84
19/08/2017	174.87
23/08/2017	174.89
29/08/2017	174.67
16/09/2017	174.87
19/09/2017	175.05
22/09/2017	175.13
16/11/2017	176.87
01/12/2017	176.83
20/12/2017	176.65
30/01/2018	176.28
05/02/2018	176.17

Well No.	T2-W21
Date	GWL(m a.s.l.)
25/07/2017	174.08
10/08/2017	173.45
14/08/2017	173.35
19/08/2017	173.45
23/08/2017	173.55
29/08/2017	173.45
16/09/2017	174.35
19/09/2017	174.18
22/09/2017	174.25
16/11/2017	175.29
01/12/2017	175.19
20/12/2017	174.98
30/01/2018	174.40
05/02/2018	174.23

Well No.	T2-W22
Date	GWL(m a.s.l.)
25/07/2017	175.60
10/08/2017	175.67
14/08/2017	175.55
19/08/2017	175.55
23/08/2017	175.55
29/08/2017	174.75
16/09/2017	175.85
19/09/2017	175.83
22/09/2017	175.83
16/11/2017	176.97
01/12/2017	176.95
20/12/2017	176.94
30/01/2018	176.52
05/02/2018	176.53

Well No.	T2-W23
Date	GWL(m a.s.l.)
25/07/2017	177.03
10/08/2017	176.27
14/08/2017	176.25
19/08/2017	176.21
23/08/2017	176.27
29/08/2017	176.47
16/09/2017	176.65
19/09/2017	176.91
22/09/2017	176.89
16/11/2017	178.97
01/12/2017	178.58
20/12/2017	178.86
30/01/2018	178.41
05/02/2018	178.09

Well No.	T2-W24
Date	GWL(m a.s.l.)
25/07/2017	174.91
10/08/2017	174.88
14/08/2017	174.98
19/08/2017	174.89
23/08/2017	174.76
29/08/2017	174.89
16/09/2017	175.80
19/09/2017	175.74
22/09/2017	175.64
16/11/2017	176.91
01/12/2017	176.78
20/12/2017	176.71
30/01/2018	176.32
05/02/2018	176.30

Well No.	T2-W25
Date	GWL(m a.s.l.)
25/07/2017	174.36
10/08/2017	174.59
14/08/2017	174.44
19/08/2017	174.46
23/08/2017	174.51
29/08/2017	174.51
16/09/2017	175.21
19/09/2017	175.06
22/09/2017	175.02
16/11/2017	177.01
01/12/2017	176.78
20/12/2017	176.72
30/01/2018	176.04
05/02/2018	175.81

2.2 Observed Water Levels of Wells and Springs in Tunnel No.2 Area (unit: m.a.s.l.) (continued)

Well No.	T2-W26
Date	GWL(m a.s.l.)
16/09/2017	175.05
19/09/2017	175.00
22/09/2017	174.98
16/11/2017	176.92
01/12/2017	176.69
20/12/2017	176.42
30/01/2018	175.92
05/02/2018	175.88

Well No.	T2-W27
Date	GWL(m a.s.l.)
16/11/2017	180.72
01/12/2017	180.80
20/12/2017	180.90
30/01/2018	180.30
05/02/2018	180.32

Well No.	T2-W28
Date	GWL(m a.s.l.)
25/07/2017	177.01
10/08/2017	177.10
14/08/2017	177.09
19/08/2017	177.25
22/08/2017	177.30
29/08/2017	177.15
16/09/2017	177.45
19/09/2017	177.43
22/09/2017	177.47
16/11/2017	177.55
01/12/2017	177.53
20/12/2017	177.55
30/01/2018	177.45
05/02/2018	177.42

Well No.	T2-W29
Date	GWL(m a.s.l.)
29/08/2017	174.80
16/09/2017	175.59
19/09/2017	175.36
22/09/2017	175.57
16/11/2017	176.02
01/12/2017	176.03
20/12/2017	175.86
30/01/2018	174.99
05/02/2018	175.43

Well No.	T2-W31
Date	GWL(m a.s.l.)
19/09/2017	176.95
22/09/2017	177.07
16/11/2017	178.80
01/12/2017	178.45
20/12/2017	178.03
30/01/2018	177.84
05/02/2018	177.72

Well No.	T2-S51
Date	GWL(m a.s.l.)
10/08/2017	175.47
14/08/2017	175.57
19/08/2017	175.72
23/08/2017	175.77
29/08/2017	175.70
15/09/2017	176.07
19/09/2017	176.04
22/09/2017	176.06
19/09/2017	176.04
16/11/2017	176.09
01/12/2017	176.10
20/12/2017	176.08
30/01/2018	176.01
05/02/2018	176.01

Well No.	T2-S52
Date	GWL(m a.s.l.)
10/08/2017	175.40
14/08/2017	175.62
19/08/2017	175.66
23/08/2017	175.74
29/08/2017	175.69
22/09/2017	175.99
16/11/2017	175.92
01/12/2017	175.85
20/12/2017	175.83
05/02/2018	175.84

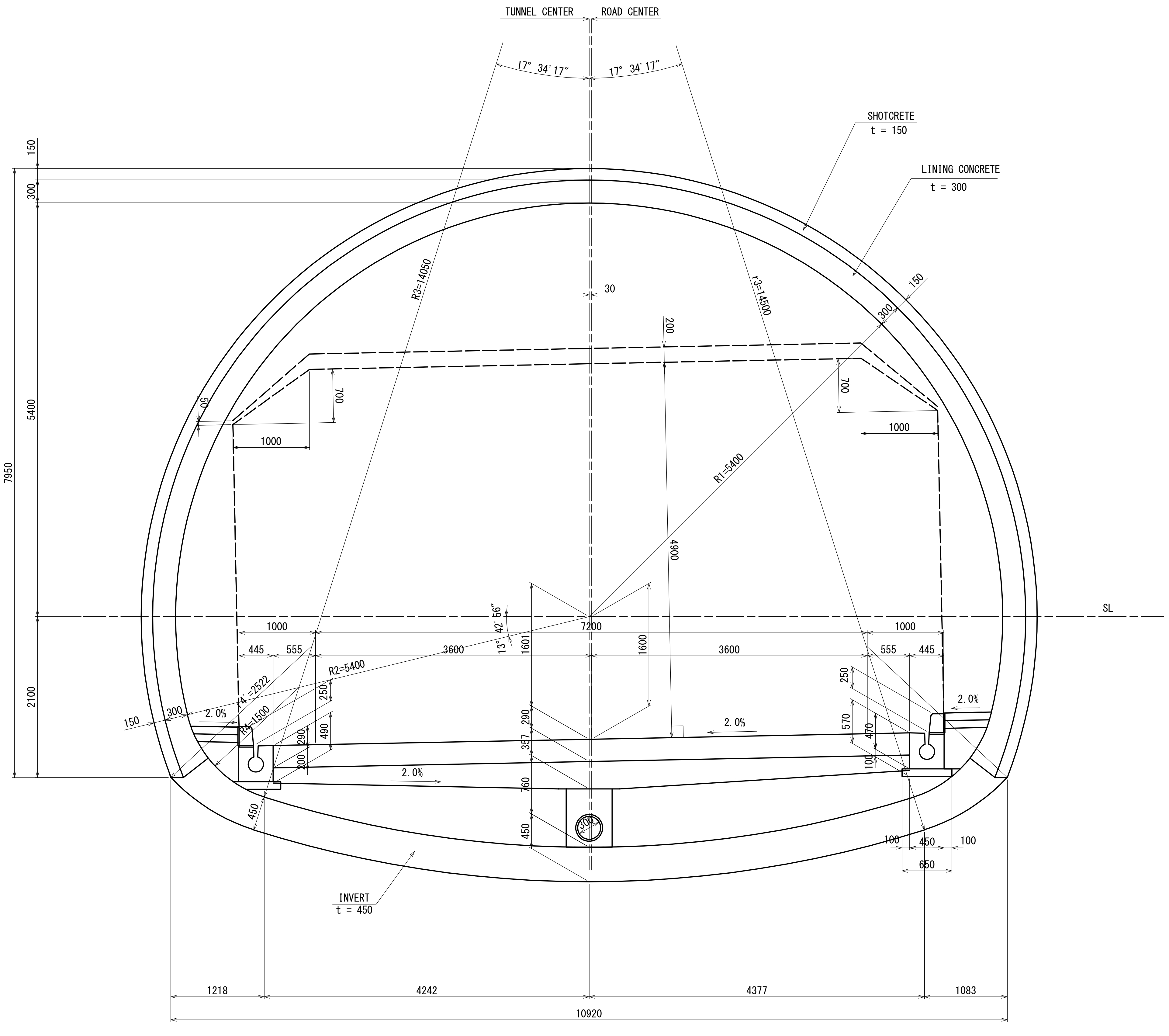
Well No.	T2-W53
Date	WL(m, depth from the ground.)
26/07/2017	-3.70
10/08/2017	-3.98
14/08/2017	-4.00
19/08/2017	-3.81
23/08/2017	-3.70
29/08/2017	-3.90
16/09/2017	-2.82
19/09/2017	-3.00
22/09/2017	-3.09
16/11/2017	-2.17
02/12/2017	-2.25
20/12/2017	-2.49
30/01/2018	-2.84
05/02/2018	-2.89

添付資料　6

トンネル設計断面

STANDARD CROSS SECTION SCALE=1:30

(TYPE D I -b)

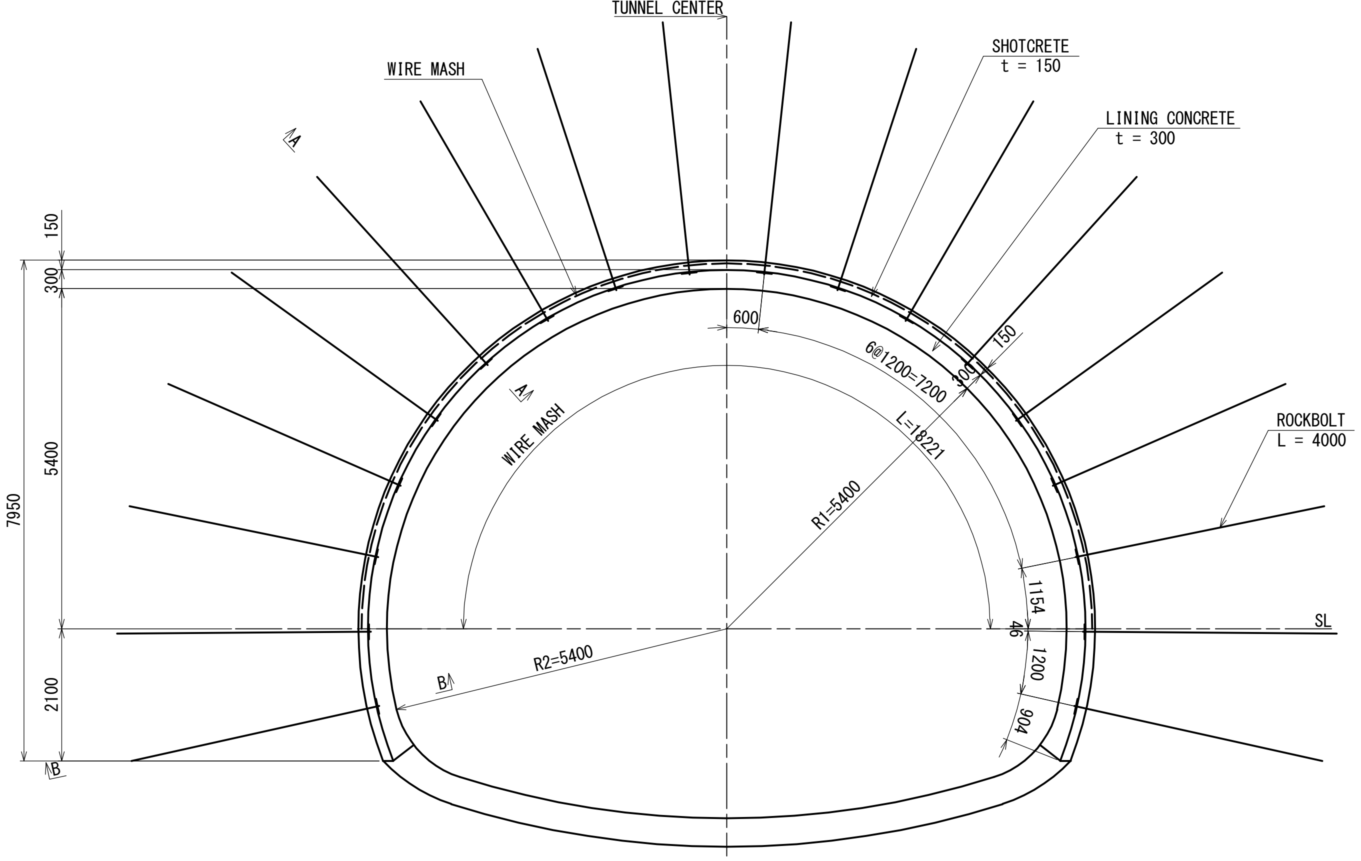


SHEET CONTENTS	SCALE	DRAWING NO.
STANDARD CROSS SECTION (TYPE D I -b)	1:30	

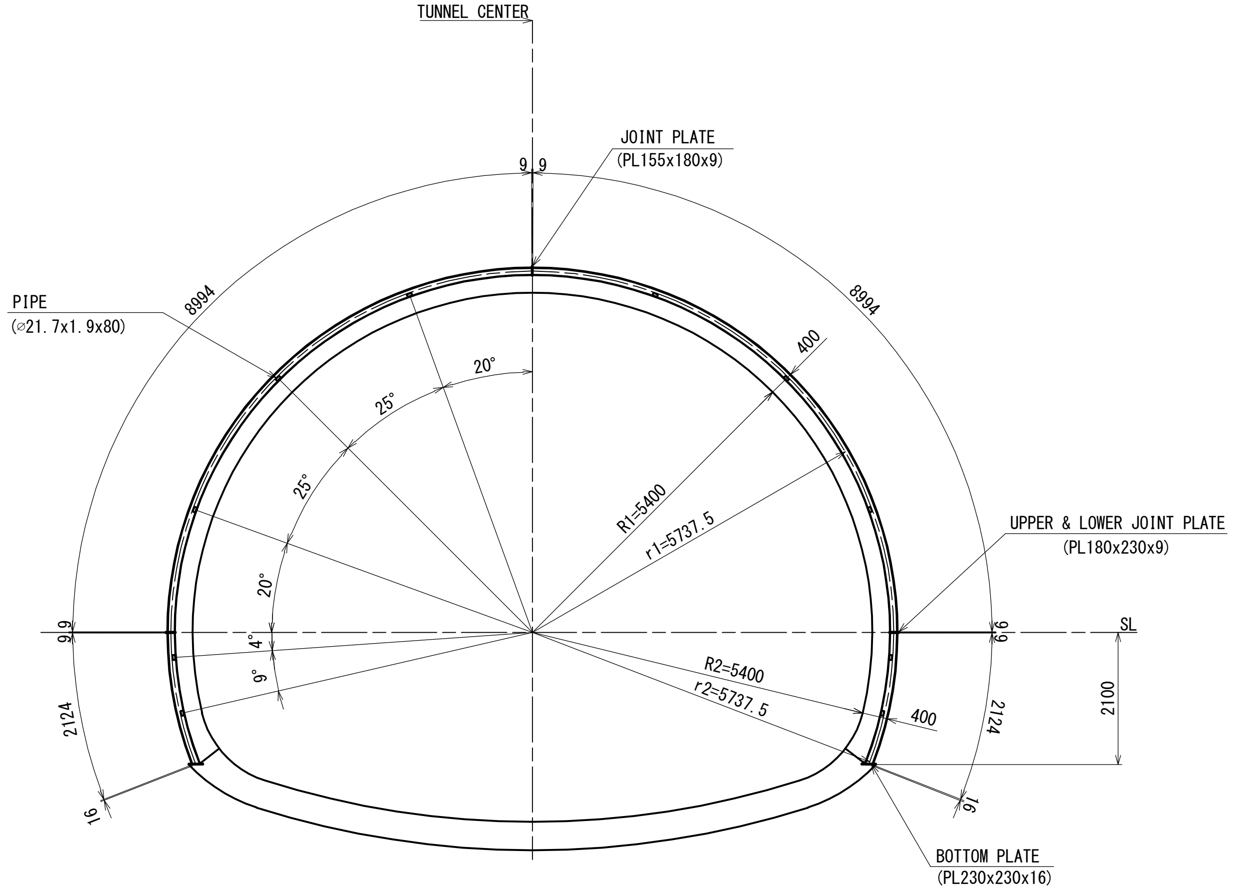
SUPPORTING PATTERN

(TYPE D I -b)

SHOTCRETE AND ROCKBOLT SCALE=1:60

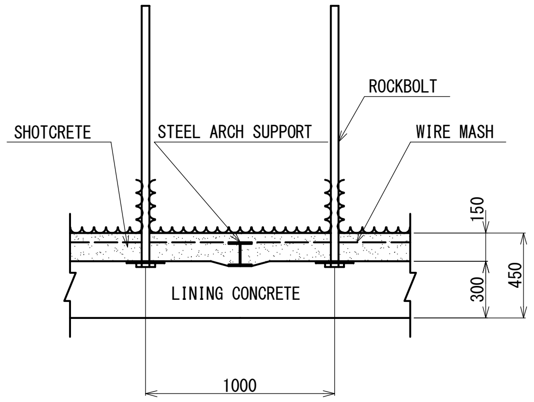


STEEL ARCH SUPPORT SCALE=1:60

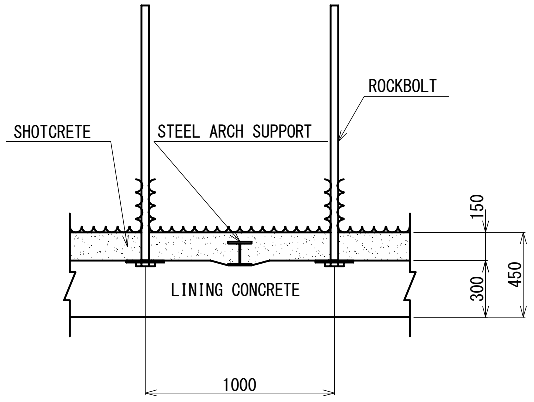


CROSS SECTIONAL DETAIL SCALE=1:20

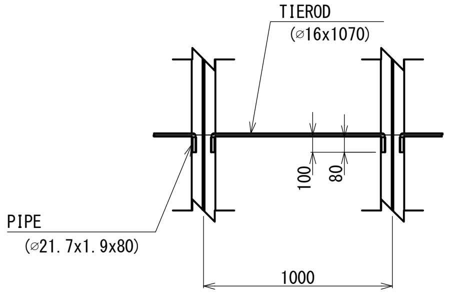
SECTION A-A



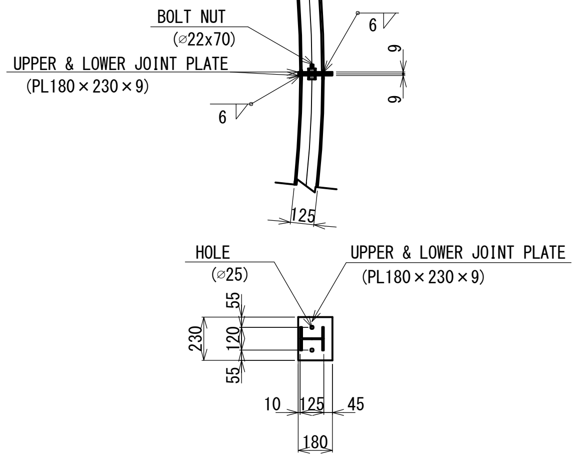
SECTION B-B



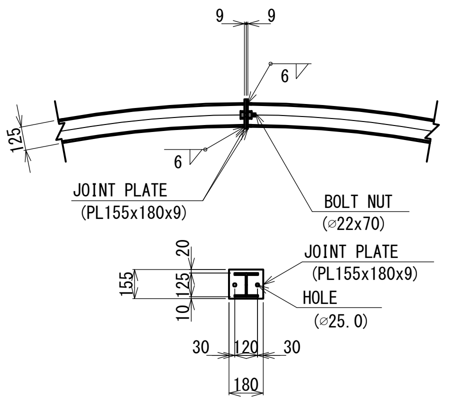
TIERROD DETAIL SCALE=1:20



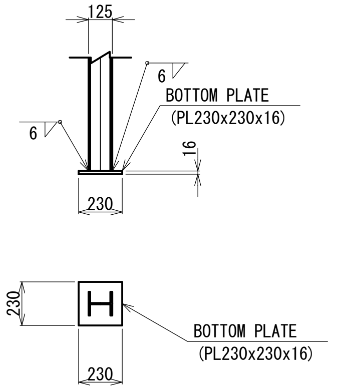
UPPER & LOWER JOINT PLATE DETAIL SCALE=1:20



JOINT PLATE DETAIL SCALE=1:20



BOTTOM PLATE DETAIL SCALE=1:20



SUPPORT PATTERN

ROCKBOLT			STELL SUPPORT		SHOTCRETE THICKNESS (cm)	LINING THICKNESS (cm)		WIRE MESH	THE AMOUNT OF ALLOWABLE DEFORMATION (cm)		
LENGTH (m)	CIRCUM-FERENTIAL DIRECTION (m)	LONGI-TUDINAL DIRECTION (m)	TOP HEADING	BENCH		ARCH, SIDE WALL	INVERT		TOP HEADING	BENCH	INVERT
4.0	1.2	1.0	H-125	H-125	15	30	45	**TOP HEADING**	—	—	—

SHOTCRETE AND ROCKBOLT MATERIAL LIST

(P=1.000mPER)

ITEM	DIMENSION	STANDARD	UNIT	QUANTITY	REMARK
ROCKBOLT	L = 4000	YIELD LOAD170KN (18t) OVER	NUMBER	18	MORTAR CONPLETE ANCHORAGE SYSTEM
PLATE	150 × 150 × 9	SS400	SHEETS	18	
NUT		M24	PIECE	18	
SHOTCRETE	t = 150		m^2	22.209	
WIRE MESH	∅5x150x150	JIS G 3551	m^2	18.221	WIRE MESH

STEEL ARCH SUPPORT MATERIAL LIST

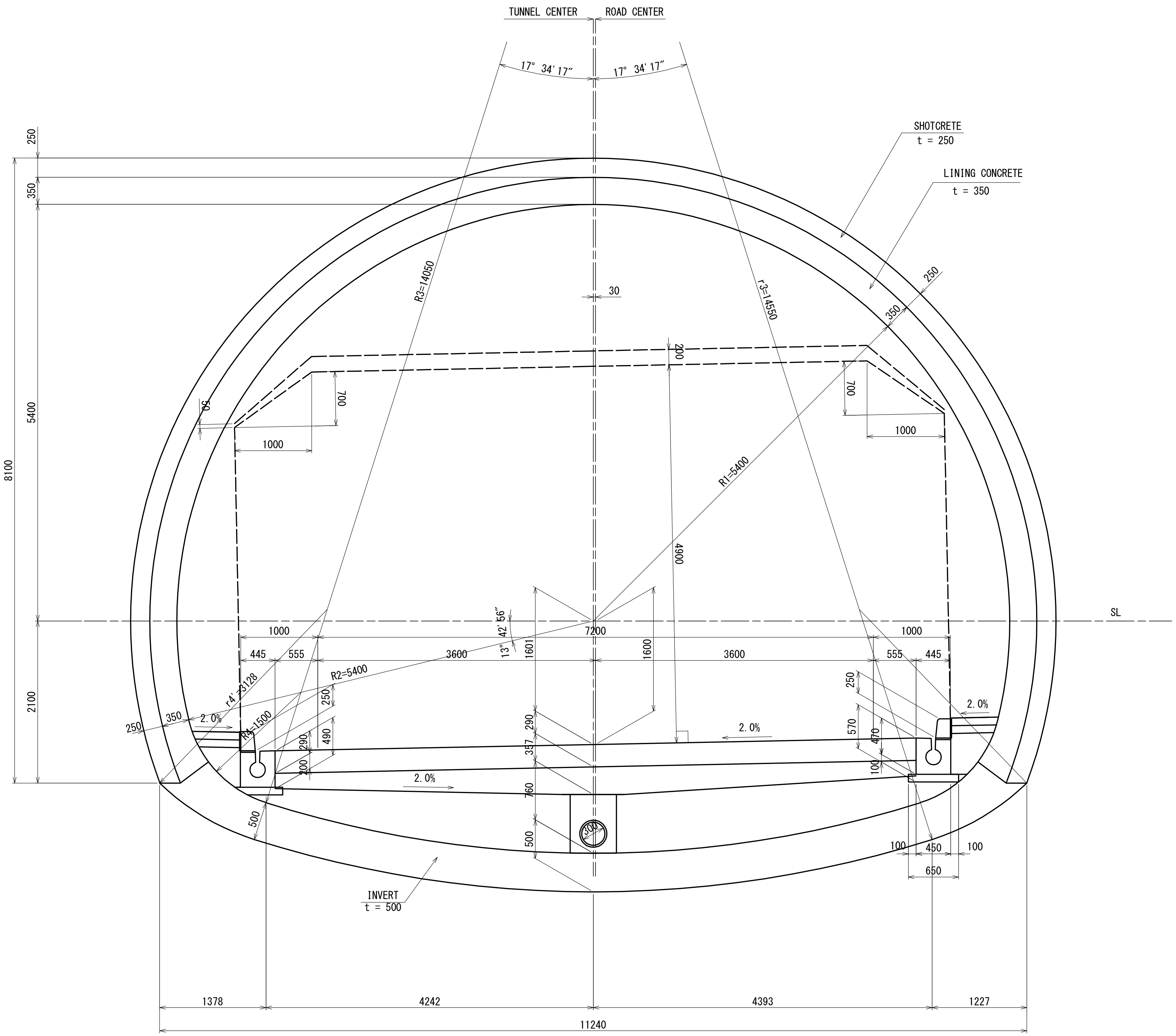
(1mPER UNIT)

ITEM	DIMENSION	UNIT	QTY	UNIT WEIGHT	TOTAL WEIGHT	REMARK
H-BEAM	H-125x125x6.5x9	kg	2	212.258	424.5	23.600 kg/m
H-BEAM	H-125x125x6.5x9	kg	2	50.126	100.3	23.600 kg/m
JOINT PLATE	PL155x180x9	kg	2	1.971	3.9	70.650 kg/m^2
BOLT NUT	∅22 × 70	NUMBER	2			
UPPER & LOWER JOINT PLATE	PL180x230x9	kg	4	2.925	11.7	70.650 kg/m^2
BOLT NUT	∅22x70	NUMBER	4			
BOTTOM PLATE	PL230x230x16	kg	2	6.644	13.3	125.600 kg/m^2
PIPE	∅21.7x1.9x80	kg	20	0.074	1.5	0.928 kg/m
TIEROD	∅16x1070	kg	10	1.691	16.9	1.580 kg/m
TOTAL		572.1 kg				

SHEET CONTENTS	SCALE	DRAWING NO.
SUPPORTING PATTERN (TYPE D I -b)	AS SHOWN	

STANDARD CROSS SECTION SCALE=1:30

(TYPE DⅢa)

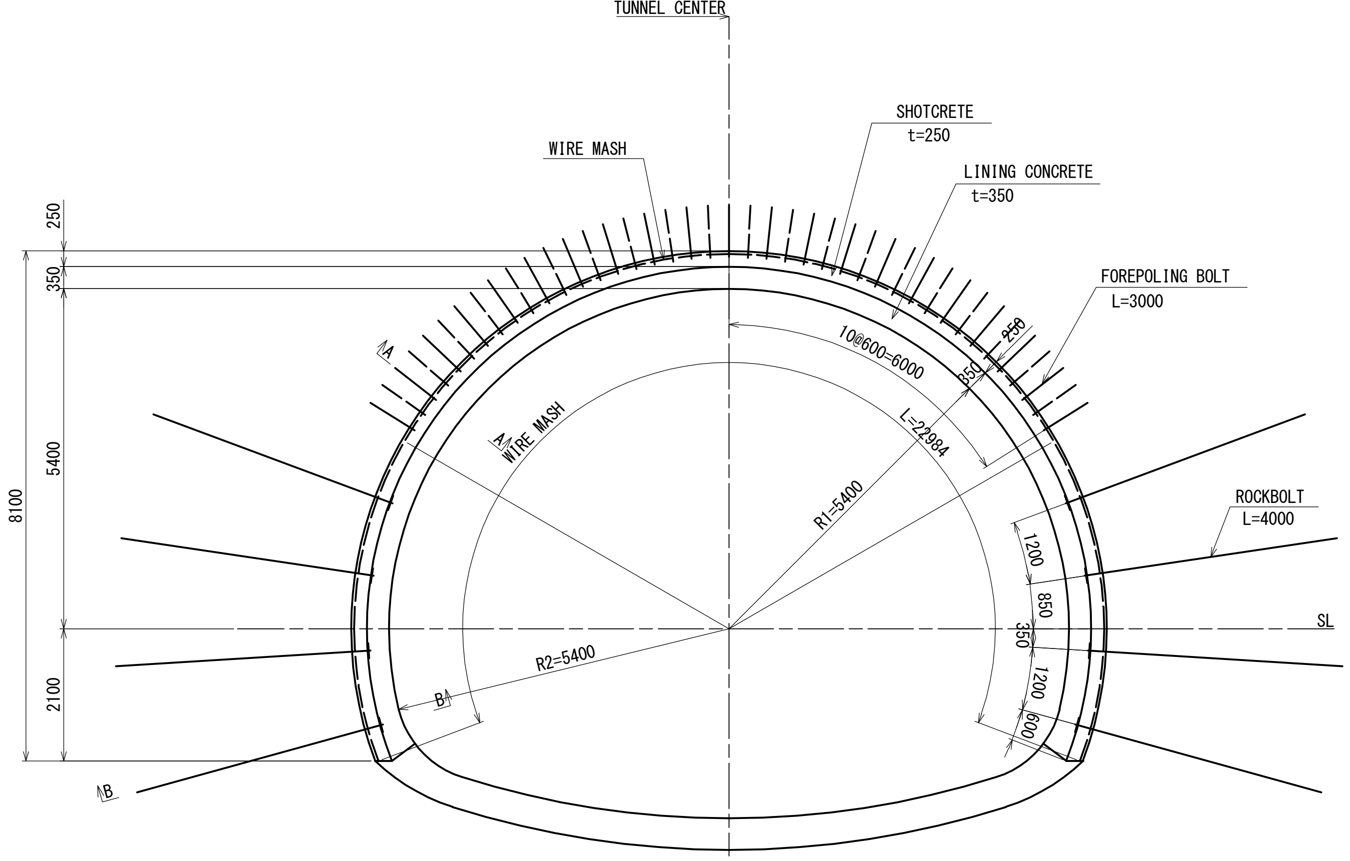


SHEET CONTENTS	SCALE	DRAWING NO.
STANDARD CROSS SECTION (TYPE DⅢa)	1:30	

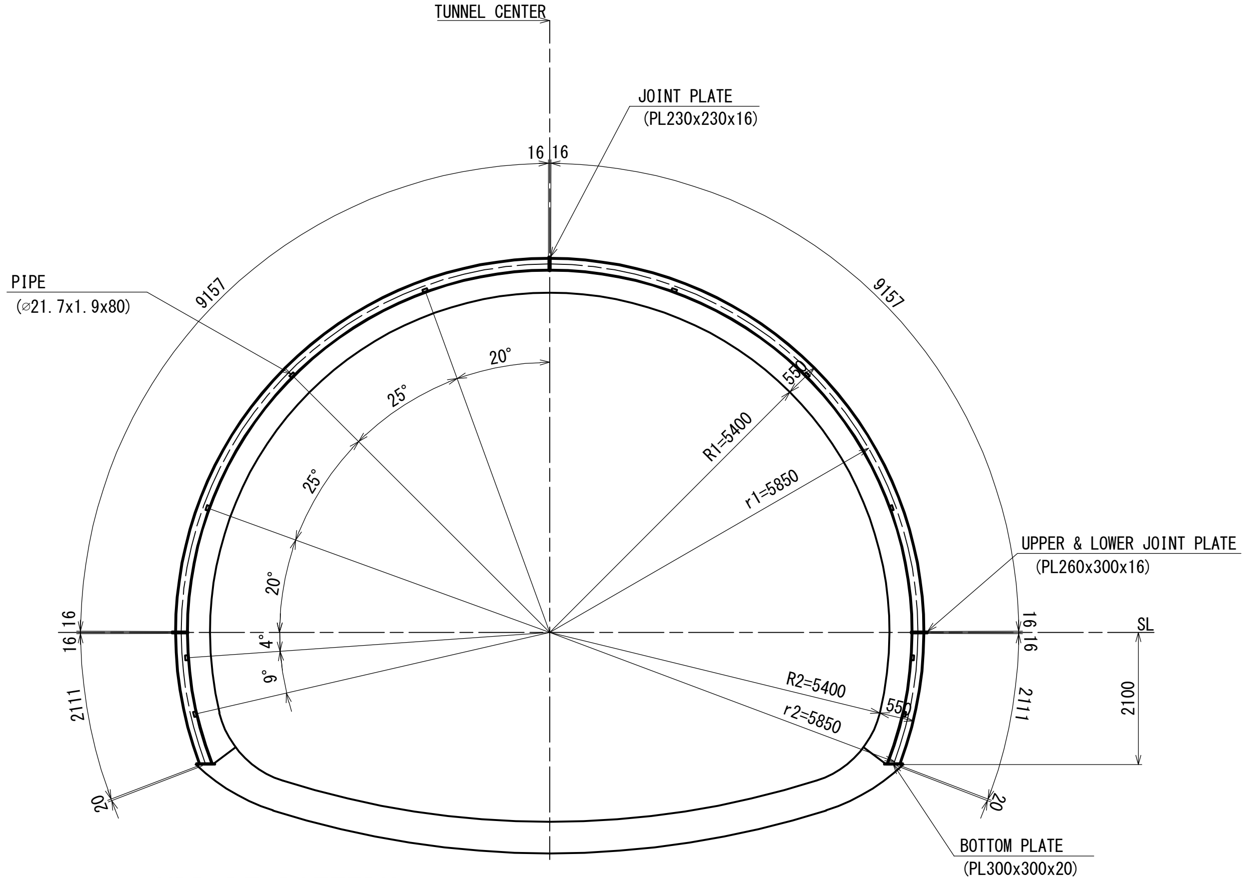
SUPPORTING PATTERN

(TYPE DⅢa)

SHOTCRETE AND ROCKBOLT SCALE=1:60



STEEL ARCH SUPPORT SCALE=1:60



SUPPORT PATTERN

ROCKBOLT			STELL SUPPORT		SHOTCRETE THICKNESS (cm)	LINING THICKNESS (cm)		WIRE MESH	THE AMOUNT OF ALLOWABLE DEFORMATION (cm)		
LENGTH (m)	CIRCUM-FERENTIAL DIRECTION (m)	LONGI-TUDINAL DIRECTION (m)	TOP HEADING	BENCH		ARCH, SIDE WALL	INVERT		TOP HEADING	BENCH	INVERT
4.0 3.0	1.2 0.6 (120°)	1.0 1.0	H-200	H-200	25	35	50	UPPER & LOWER	—	—	—

SHOTCRETE AND ROCKBOLT MATERIAL LIST

(P=1.000mPER)

ITEM	DIMENSION	STANDARD	UNIT	QUANTITY	REMARK
FOREPOLING BOLT	L=3000	D25(SD345)	NUMBER	20.5	MORTAR COMPLETE ANCHORAGE SYSTEM
ROCKBOLT	L = 4000	YIELD LOAD170KN (18t)OVER	NUMBER	8	MORTAR CONPLETE ANCHORAGE SYSTEM
PLATE	150×150×9	SS400	SHEETS	8	
NUT		M24	PIECE	8	
SHOTCRETE	t = 250		m^2	22.364	
WIRE MESH	∅5x150x150	JIS G 3551	m^2	22.984	WIRE MESH

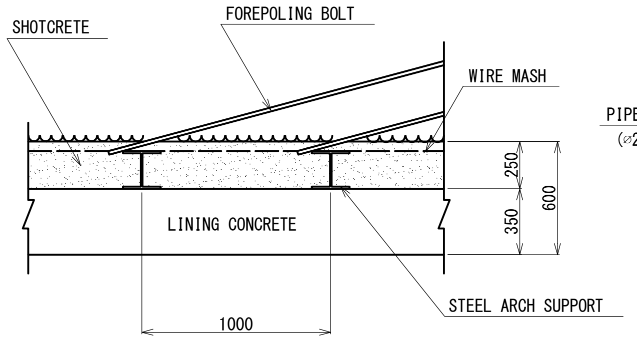
STEEL ARCH SUPPORT MATERIAL LIST

(1mPER UNIT)

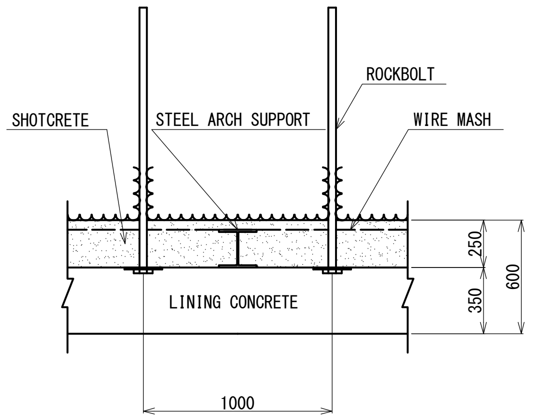
ITEM	DIMENSION	UNIT	QTY	UNIT WEIGHT	TOTAL WEIGHT	REMARK
H-BEAM	H-200x200x8x12	kg	2	456.934	913.9	49.900 kg/m
H-BEAM	H-200x200x8x12	kg	2	105.389	210.8	49.900 kg/m
JOINT PLATE	H-230×230×16	kg	2	6.644	13.3	125.600 kg/m^2
BOLT NUT	∅25x75	NUMBER	2			
UPPER & LOWER JOINT PLATE	PL260×300×16	kg	4	9.797	39.2	125.600 kg/m^2
BOLT NUT	∅25x75	NUMBER	4			
BOTTOM PLATE	PL300x300x20	kg	2	14.130	28.3	157.000 kg/m^2
PIPE	∅21.7x1.9x80	kg	20	0.074	1.5	0.928 kg/m
TIEROD	∅16x990	kg	10	1.564	15.6	1.580 kg/m
TOTAL		1222.6 kg				

CROSS SECTIONAL DETAIL SCALE=1:20

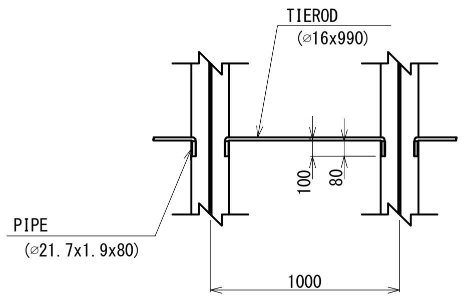
SECTION A-A



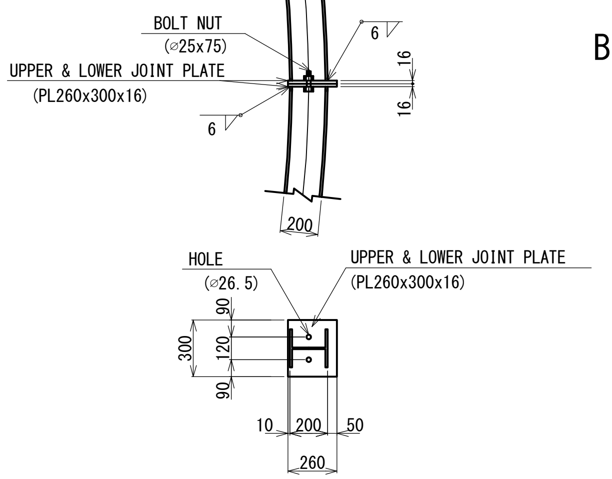
SECTION B-B



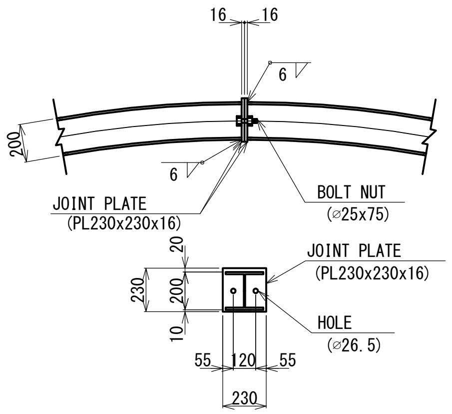
TIEROD DETAIL SCALE=1:20



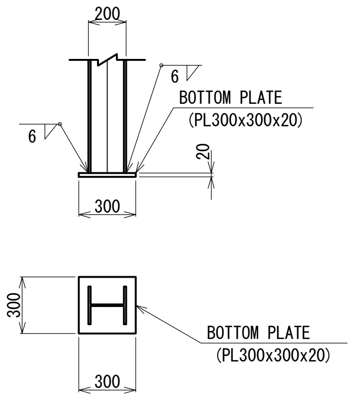
UPPER & LOWER JOINT PLATE DETAIL SCALE=1:20



JOINT PLATE DETAIL SCALE=1:20



BOTTOM PLATE DETAIL SCALE=1:20



SHEET CONTENTS	SCALE	DRAWING NO.
SUPPORTING PATTERN (TYPE DⅢa)	AS SHOWN	