Road Development Authority (RDA),
Ministry of Higher Education and Highways
The Democratic Socialist Republic of Sri Lanka

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Technical Assistance for Improvement of Capacity for Planning of Road Tunnels

Final Report

February 2018

Japan International Cooperation Agency (JICA)

Earth System Science Co., Ltd.
Nippon Koei Co., Ltd.

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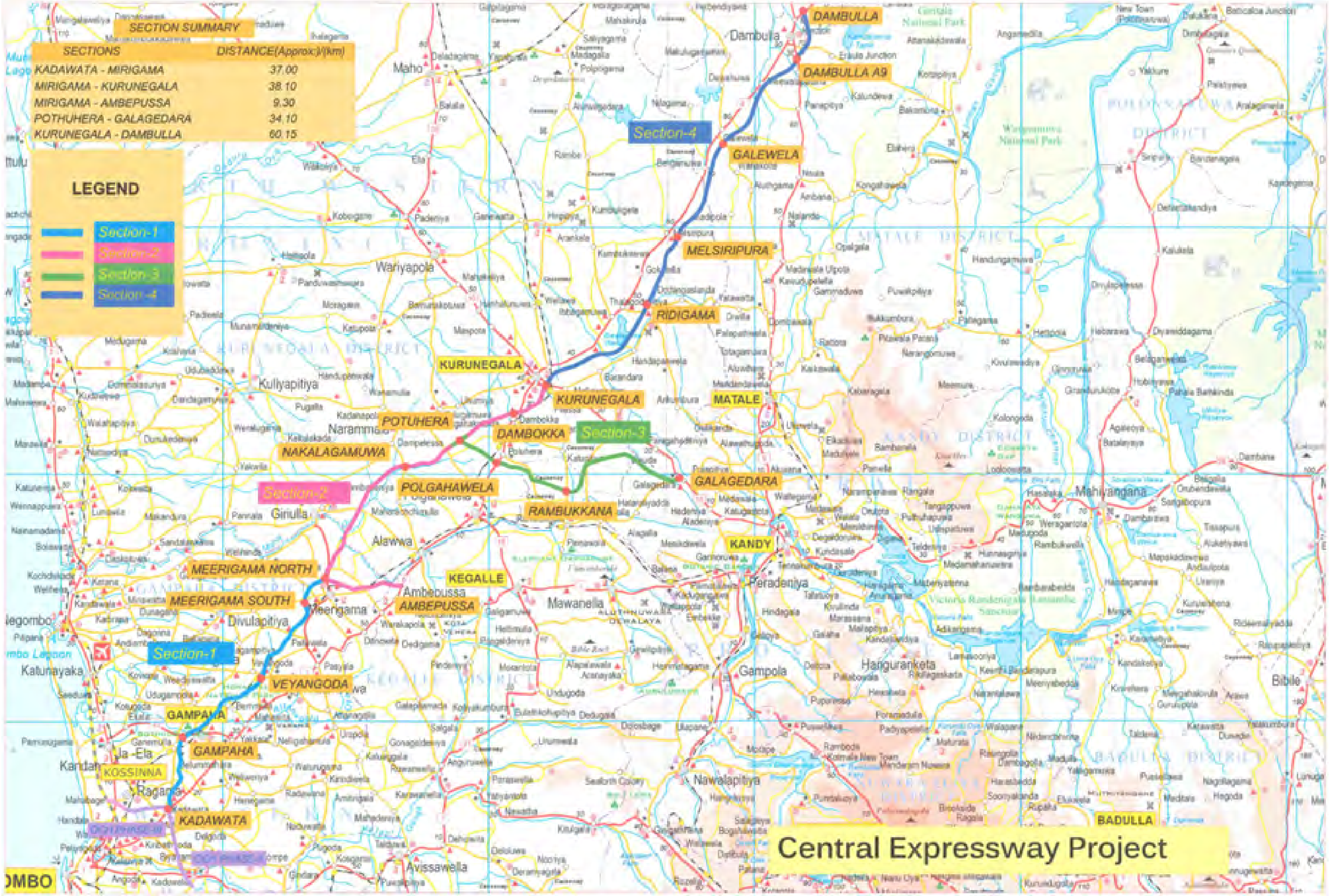

Map of the Central Expressway

(Source: RDA)

Map of the Pilot Sites

(Source: RDA)

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Abbreviations

Term	Definition
ADB	Asian Development Bank
CEP-III	Central Expressway Project-III
C/P	Counterpart
ELS	Engineering & Laboratory Services (Pvt) Ltd
FIM	First Inter-monsoon season
F/S	Feasibility Study
GNSS	Global Navigation Satellite System
GPS	Global Positioning System
GSMB	Geological Survey and Mines Bureau
ITCZ	Inter-Tropical Convergence Zone
JICA	Japan International Cooperation Agency
NATM	New Austrian Tunneling Method
NBRO	National Building Research Organization
NEM	Northeast Monsoon season
NEXCO	Nippon Expressway Company Limited
OJT	On the Job Training
RDA	Road Development Authority
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SIM	Second Inter-monsoon season
SIRT	Simultaneous Iterative Reconstruction Technique
SPT	Standard Penetration Test
SWM	Southwest Monsoon season
UCS	Uniaxial Compressive Strength

Chapter 1: Description of the Project

1.1 Background of the Project

Roads are the major means of transportation in Sri Lanka. Therefore, development of an efficient and safe road network is essential for Sri Lanka's social and economic development.

Almost 90% of passengers as well as cargo transportation depend on roads. Since the conflict ended in 2009, traffic demand has increased: the number of registered vehicles has increased by about 1.8 times from 2007 to 2014 while the number of newly registered vehicles per year has increased by about 1.4 times. The total extension of the roads of Sri Lanka is approximately 115,906 km (as of 2014). Roads are classified into highways and community roads. The Highways, including national expressways, are operated and maintained by the Road Development Authority (RDA) while the community roads are managed by the Ministry of Local Government and Provincial Councils and other organizations/institutions. Although the road network is improved from year to year in Sri Lanka, roads constructed in the colonial period are narrow and old considering the recent traffic condition. In addition, the road condition has become worse recently. The ratio of expressways out of the total length of the roads is still 0.13% (as of 2014). Therefore, promotion of efficiency of domestic transport is required by development of the traffic network between major cities by expressways.

Under this situation, the Government of Sri Lanka has been constructing highways connecting major cities in recent years. As a part of the road development works, JICA conducted the "Road Network Improvement Project (Loan agreement in 1999)", which includes extension and improvement of main roads (40km), repairing and widening of bridges and constructing a 2-lane road tunnel (220 m) of national highway A2.

The Government of Sri Lanka is now conducting projects to expand expressway inland, especially from Colombo to Kandy (Central Expressway), as a part of further highway and expressway development in Sri Lanka. However, construction of tunnels must be required for the road development across central Sri Lanka which is a highland and mountainous region (altitude more than 2,000m). Nevertheless capacities and experiences of RDA on actual road planning including tunnel construction are limited since the number of previous road tunnel projects, including the "Road Network Improvement Project", is few. The experiences of the previous tunnel projects are utilized as reference information for the coming tunnel projects, but that is not enough. Guidelines and standards for investigation, design, construction and management are still not developed. As a result, there are issues on necessary information collection for construction bidding.

Therefore, support for capacity development of RDA in tunnel investigation and design through development of guidelines and standards regarding rock mass classification is required for future road tunnel planning including road tunnels.

1.2 Characteristics of Tunnel Planning and Necessity of Guidelines and Standards

1.2.1 Tunnel Planning and Geology

Tunneling work includes constructing a tunnel and associated temporary works of setting up access roads and working yards. Designing a tunnel is not similar to that of a plant or structures as it is difficult to assume accurate geological and geotechnical conditions such as properties and variability of rock mass along the tunnel. In comparison with other designs, tunnel design is generally carried out using less reliable geological and geotechnical assumptions.

Existing information and results of site investigation to assume accurate geological and geotechnical conditions along the tunnel alignment shall be reviewed to minimize the risks in the tunneling work.

Inspection of excavation surface in the construction stage to compare the actual geological and geotechnical conditions with the assumed conditions of the design stage is carried out to review the accuracy of rock mass classification system and tunnel design. Monitoring deformation of excavated tunnel cavern and stress and/or strain by installed measurement devices around the tunnel is also required to detect any abnormalities and cope with any changes in geological conditions.

1.2.2 Tunnel Design and Rock Mass Classification System

The design of tunnels is performed based on classification of rock mass. Even in the tunnel construction stage, assessment and management of changes in the tunnel design based on rock mass classification are necessary to avoid safety risk. This may include suspension of tunneling works to reassess the changed conditions, cost performance, and safety control measures. Review and modification of the rock mass classification system and selection of tunnel support by using rock mass classification are often required based on the actual geological and geotechnical conditions confirmed in the construction stage. Therefore, the rock mass classification system is essential to the design of tunnels.

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 during the research works. Numerical modelling using various modelling techniques and computing power can be a way of solving very complex problems and selecting the most suitable tunnel 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.

Therefore, preparation of a rock mass classification system for tunneling works is important for RDA gaining the knowledge and enhancing the ability to conduct road tunnel projects such as the Central Expressway Project. However, no proper rock mass classification system is utilized in Sri Lanka at the present stage. Thus, the guidelines for rock mass classification system, site investigations and tunnel design are necessary for tunneling works in Sri Lanka.

1.3 Project Purpose

(1) Objective of Project

Draft of standards and procedure manuals for ground assessment for road tunnels in Sri Lanka are prepared so that ground assessment necessary for preparation of a road tunnels is appropriately performed. Moreover, draft guidelines of road tunnel design and evaluation and environmental impact study on groundwater are prepared.

(2) Expected Outcomes

Outcome i) Draft of criteria for rock mass classification for road tunnel construction is prepared.

Outcome ii) Draft of procedure manuals for geological survey for rock mass classification system, road tunnel design and evaluation and environmental impact study on groundwater is prepared.

Outcome iii) RDA acquires, through the pilot survey, the skills to utilize the guidelines prepared by the outcomes i) and ii) and knowledge to prepare the design document necessary for actual tunnel construction.

1.4 Relevant Agencies

- Counterpart organization: RDA, the Ministry of Higher Education & Highways
- Indirect beneficiary: the public (road users) and other relevant sectors

1.5 Details of Operation and Project Implementation Schedule

The project activities and implementation schedule (plan and actual) as well as detail of operation is shown in Table 1.1. The results of the project activities such as geological investigation, advice for tunnel design and preparation of guidelines/manuals are described in Chapters 3 to 6; outcomes of technical transfer are described in Chapter 7.

1.6 Input

1.6.1 Japanese Side Input

The dispatch record of JICA Experts is shown in Table 1.2.

1.6.2 Sri Lankan Side Input

(1) Assignment of Counterpart Personnel for Overall Coordination

Two RDA engineers from Central Expressway Project-III (CEP-III), RDA are assigned for overall coordination of the project activities.

(2) Assignment of Counterpart Personnel for Technical Transfer

RDA has nominated RDA engineers for technical transfer from CEP-III, Bridge Design Division, Highway Design Division, Research and Design Division, Ruwanpura Expressway Project and Kandy Tunnel Project. They joined in the technical seminars and workshops held by the project team, and made discussions about site investigation and tunnel design with the project team. Furthermore, seven counterparts of the RDA engineers made presentations and gave the presentations on the final seminars of this project in December 2017 and January 2018.

Table 1.1 Project Activities and Implementation Schedule

Work in Sri Lanka (Plan:■, Actual:■) Work in Japan (Plan:□, Actual:□)	2017												2018		
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3
1. Preliminary Study (work in Japan)															
【1.1】 Discuss with JICA about contents and purpose of work															
【1.2】 Collect and review previous studies															
【1.3】 Prepare draft standards and manuals for ground assessment on tunnel construction															
【1.4】 Discuss with JICA about work schedule															
【1.5】 Prepare and submit draft Work Plan															
2. Work in Sri Lanka (1st batch)															
【2.1】 Discuss with RDA and finalize draft Work Plan															
【2.2】 Introduce knowledge and experience of road and tunnel construction in Japan and draft standards and manuals for ground assessment															
【2.3】 Collect existing data from RDA and relevant agencies/companies															
【2.4】 Investigate candidate pilot sites															
a. Collect overall topographic information for pilot survey															
b. Collect overall geological condition at pilot sites															
【2.5】 Select position and section of tunnel for the pilot site															
【2.6】 Examine the accuracy and scale of topographic survey and assist on topographical survey planning															
【2.7】 Identify necessary geological information and consider any additional investigating methods															
【2.8】 Discuss with RDA and finalize Progress Report															
3. Work in Sri Lanka (2nd batch)															
【Topographical and Geological Survey】															
【3.1】 Carry out topographical survey based on the plan prepared by 【2.6】															
【3.2】 Develop a new topographic map reflecting the results from the survey															
【3.3】 Technical assistance on development of presumed geological map from existing geological profiles															
【3.4】 Carry out boring geological survey															
a. Develop soil boring logs															
b. Develop a geological map															
【3.5】 Carry out seismic exploration to determine geological profile and physical property															
【3.6】 Develop the integrated geological map applying the results from the previously carried out investigations															
【3.7】 Examine planned route of the road and tunnel location															
【Analysis and utilization of the survey result for design and construction】															
【4.1】 Set an outline of design and construction using the integrated geological map															
【4.2】 Provide advice for design of tunnel (up to selection of tunnel support structure)															
【4.3】 Provide advice for construction method of tunnel															
【4.4】 Provide advice for tunnel cross-section															
【4.5】 Provide advice for plan for tunnel construction															
【Finalization of the standards and manuals for ground assessment on tunnel construction】															
【5.1】 Provide advice for finalization of the standards and manuals for ground assessment															
【5.2】 Hold a seminar to share the standards and manuals for ground assessment among RDA members															
【5.3】 Prepare the Final Report															
4. Work in Japan (2nd batch) and Work in Sri Lanka (3rd batch)															
【Developing guildeline for road tunnel design and providing advices for tunnel design】															
【6.1】 Collect and review existing information and examples of tunnel design															
【6.2】 Prepare a guideline (draft) for tunnel design															
【6.3】 Analyze the result of the site investigation at No.2 tunnel site, and discuss principle(s) in tunnel design															
【6.4】 Provide avdice to the plan and design of tunnel No.2															
【6.5】 Finalize the guideline for tunnel design by discussing with RDA															
【Developing guildeline for environmental impact study in groundwater assessment and monitoring】															
【7.1】 Collect and review existing environmental impact studies and incident cases of groundwater depletion															
【7.2】 Prepare a guideline (draft) for environmental impact study in groundwater assessment and monitoring															
【7.3】 Analyze the result of the site investigation at No.2 tunnel site, and discuss impacts on groundwater															
【7.4】 Provide avdice to hydrological investigation at tunnel No.1-3															
【7.5】 Finalize the guideline for environmental impact study in groundwater assessment and monitoring															
【Seminar to share the guidelines for tunnel design and environmental impact study in groundwater assessment】															
【8.1】 Hold a seminar to share the guidelines for tunnel design and environmental impact study in groundwater ass															
Reports															
	△W/P ▲P/R F/R▲ W/P: Work Plan P/R:Progress Report F/R:Final Report														

(Source: Project team)

Table 1.2 Expert Assignment Schedule

1. Work in Sri Lanka

Name (Assignment)		2017											2018		M/M Total
		2	3	4	5	6	7	8	9	10	11	12	1	2	
Kimihiko KOTOO (Team Leader / Geological Survey)	Plan			45(1.5)			30(1.0)			69(2.3)				15(0.5)	5.30
	Actual			5/1 - 6/8 39(1.3)			7/3 - 8/10 39(1.3)			10/18 - 12/22 66(2.2)			1/22 - 2/5 15(0.5)		5.30
Pucal YANG (Tunnel Entrance Geotechnical Planning)	Plan						30(1.0)			60(2.0)					3.00
	Actual						7/7 - 9/5 61(2.03)				11/24 - 12/22 29(0.97)				3.00
Toshimasa KOBAYASHI (Geophysical Investigation)	Plan						75(2.5)				45(1.5)				4.00
	Actual						7/3 - 9/14 74(2.47)				11/7 - 12/22 46(1.53)				4.00
Yuichi NISHIZONO (Tunnel Design and Evaluation 1)	Plan									15(0.5)	30(1.0)		30(1.0)		2.50
	Actual									10/30-11/9 11(0.37)		12/7-12/30 24(0.8)	1/7-1/28 22(0.73)		1.90
Kyoichi KAWAKAMI (Tunnel Design and Evaluation 2)	Plan												30(1.0)		1.00
	Actual											12/18-12/23 6(0.2)	1/18 - 2/10 24(0.8)		1.00
Shigeo SUIZU (Environmental Impact Study - Groundwater)	Plan										45(1.5)			15(0.5)	2.00
	Actual										11/23 - 12/22 30(1.0)		1/11 - 2/9 30(1.0)		2.00
Tomoyuki WADA (Topographical Survey/ Project Coordinator)	Plan			45(1.5)			78(2.6)				45(1.5)				5.60
	Actual			5/1 - 6/8 39(1.3)			7/3 - 9/24 84(2.8)				11/8 - 12/22 45(1.5)				5.60
										sub-total		Plan			23.40
												Actual			22.80

2. Work in Japan

Name (Assignment)		2017											2018		M/M Total
		2	3	4	5	6	7	8	9	10	11	12	1	2	
Kimihiko KOTOO (Team Leader / Geological Survey)	Plan			2(0.1)									2(0.1)		0.20
	Actual			4/3-4 2(0.1)										2/19-20 2(0.1)	0.20
Yuichi NISHIZONO (Tunnel Design and Evaluation 1)	Plan										20(1.0)				1.00
	Actual									10/16-20,23-27 10(0.5)	11/10,13-17,20-22,24,27-30 18(0.9)	12/1,4-6 2(0.1)		2/19-20 2(0.1)	1.60
Kyoichi KAWAKAMI (Tunnel Design and Evaluation 2)	Plan										20(1.0)				1.00
	Actual									10/23-27,30-31 11/1,2,6,7 11(0.55)	11/28-12/1 4(0.2)	12/25-27 3(0.15)		2/19-20 2(0.1)	1.00
Tomoyuki WADA (Topographical Survey/ Project Coordinator)	Plan			3(0.15)											0.15
	Actual			4/3-5 3(0.15)											0.15
										sub-total		Plan			2.35
												Actual			2.95

Legend: Plan Actual

Total	25.75
	25.75

Reports	3	4	5	6	7	8	9	10	11	12	1	2
		△ W/P			△ P/R							△ F/R

* W/P:Work Plan P/R:Progress Report F/R:Final report

(Source: Project team)

1.7 Activities

1.7.1 Existing Information Collection and Discussion with RDA

At the beginning of the project, information about previous tunneling works, standards and existing data were collected to prepare the project working plan and first draft of rock mass classification system (Activities 1.1-1.5, 2.3). A meeting about the working plan and rock mass classification system was held on 30th May 2017. Participants of the meeting were RDA Chairman and other directors/engineers involved. Based on the result of the meeting, progress report was prepared in June 2017 (Activities 2.1, 2.2, 2.8).

1.7.2 Selection of Pilot Site

The study area has three tunnel sites (Figure 1.1). The project team has inspected the outline of the topography and geology of these sites on 18th to 21st May 2017. The three members from RDA Kurunegala office joined the site inspection and discussed the appropriate tunnel site. As a result, Tunnel No.2 was selected on the basis of topographic, social conditions as well as geomorphologic and geological constraints (Activities 2.4, 2.5). The results of the site inspection are shown in Table 1.3.

Table 1.3 List of Tunnels for Survey

Tunnel Section	Topographic condition	Social Impact	**Location**	Consideration of Selection and Remarks	Selection
Tunnel 1	Good	Medium	**Good**	・Largest Thickness of Overburden is 25m (1.82D) ・Preferable location for on-the-job training ・Large numbers of houses distributed	―
Tunnel 2	Excellent	Small	**Excellent**	・Largest Thickness of Overburden is 29m (2.12D) ・Preferable location for on-the-job training ・A few numbers of houses distributed	Selected
Tunnel 3	Good	Small	**Bad**	・Largest Thickness of Overburden is 25m (1.82D) ・Less preferable location for on-the-job training ・Less number of houses distributed	―

(Source: Project team)

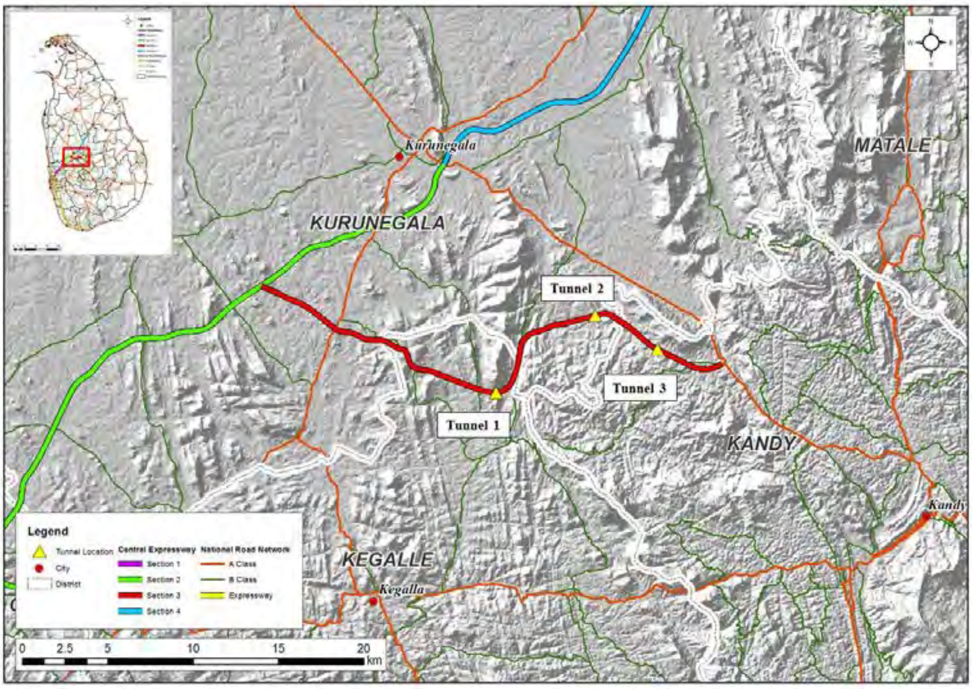


Figure 1.1 Location of Tunnels for Survey

(Source: Project team)

1.7.3 Geological and Hydrological Investigations

Topographic survey, seismic refraction prospecting and drilling survey were conducted in the selected pilot site (Tunnel No.2) by sub-contract work (Activities 3.1-3.7). The specification of the survey was prepared based on the collected information and result of site reconnaissance and was decided through discussion with RDA (Activities 2.6, 2.7). Engineering and Laboratory Services (Pvt) Ltd. (ELS) was selected as a sub-contractor on 5th June 2017 by the collected multiple quotes. The site investigation was conducted from 12th July to 28th November; 140 days. In addition, hydrological investigation was conducted by the project team from July 2017 to February 2018 cooperating with RDA. Detail of the investigations is described in Chapter 3 to 5.

1.7.4 Evaluation of Geological and Hydrological Investigation Results and Advice for Tunnel Design

Evaluation, discussion and advice about the results of the investigations, rock mass classification and tunnel design including consideration of support and cross-section of the tunnel were conducted from November 2017 to February 2018 (Activities 4.1-4.5, 6.3, 6.4, 7.3, 7.4). Details are described in Chapter 3 to 6.

1.7.5 Preparation of Guidelines

During the project implementation, guidelines on "Rock Mass Classification System", "Site Investigation for Rock Mass Classification System", "Design of Tunnels" and "Environmental Impact Study on Groundwater" are also prepared regarding project outcome i) and ii) (Activities 1.3, 5.1, 6.1, 6.2, 6.5, 7.1, 7.2, 7.5). Some parts of the guidelines are mentioned and quoted in this final report.

Technical seminars and meetings had been held in 12 occasions for sharing and improving the technical information on the guidelines and tunneling works during the project period (Activities 5.2, 8.1). Details of the seminars are described in the section 1.7.6, Chapter 7 and Appendix 1.

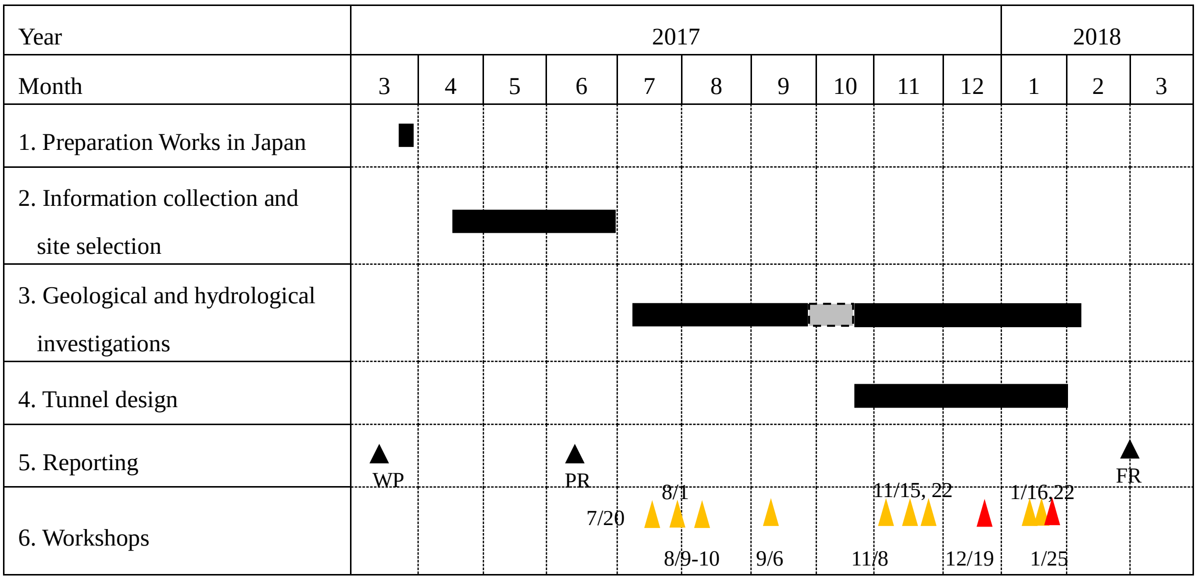
1.7.6 Technical Seminars and Technical Transfers

Regarding the project outcome iii), the project team held 12 technical workshops to share knowledges with RDA engineers regarding investigations and design of road tunnels. Table 1.4 shows the schedule, contents and number of participants of the seminars.

15 to 25 RDA engineers generally joined in the technical seminars. The RDA engineers made presentations on the two Final Seminars based on the technical knowledges transferred by this project.

Furthermore, technical transfer was conducted through the discussions with the C/P in RDA Colombo office about the result of investigations and the tunnel design. At the pilot site, the technical transfer for RDA Kurunegala office engineers was conducted through the site investigation. Details are described in Chapter 7.

Table 1.4 Record of Technical Seminars and Workshops



Note: "Technical Seminars", "Final Seminar"

Seminar	Date	Subjects of Workshop	Partici-pant
1	July 20	1) Geotechnical Survey, 2) Seismic Refraction Survey	13
2	Aug 01	1) Hydrogeology, 2) Mitigation of Tunnel Portal Slope	14
3	Aug 09	1) Introduction to Site Investigation to be performed on Aug 10, 2) Hydrogeology, 3) Mitigation of Tunnel Portal Slope	25
4	Aug 10	Site Workshop 1) Seismic Refraction Survey, 2) Core Drilling	34
5	Sep 06	Progress of Geotechnical Surveys 1) Groundwater Survey, 2) Seismic Refraction Survey, 3) Core Drilling	25
6	Nov 08	1) Introduction of Tunneling Methods (NATM), 2) Rock Mass Classification System, 3) Topographic Interpretation, 4) Rock Testing	25
7	Nov 15	Laboratory Testing	13
8	Nov 22	Laboratory Testing	8
9	Dec 19	**"Final Seminar Stage1"** 1) Sub-surface Exploration for Road Tunnels with Specific Reference to Seismic Refraction Prospecting, 2) An Introduction to Laboratory Tests in Respect of Road Tunnels, 3) Rock Mass Classification System for Road Tunneling, 4) Hydro-geological Aspects of Road Tunnels, 5) Road Tunnels Construction Methods, 6) Proposed Kandy Tunnel	56
10	Jan 16	1) Groundwater Environment around road Tunnels 2) Road Tunnel design (Geometric, structural design aspects and Introduction to Tunnel design guideline.)	24
11	Jan 22	1) Groundwater Environment around road Tunnels 2) Road Tunnel design - Alignment, Geometric (Horizontal, Vertical and CSS)	7
12	Jan 25	**"Final Seminar Stage2"** 1) Road Tunnel Design -Alignment Geometric (horizontal , vertical and CSS), 2) Groundwater Environment around Road Tunnels	31

(Source: Project team)

Chapter 2: Natural Conditions

Basic natural conditions in Sri Lanka are described in this Chapter. The information was collected in the activities 1.2, 2.3 and 7.1 for preparing guidelines and site investigations.

2.1 Topography

2.1.1 Sri Lanka

Sri Lanka has an area of 65,000 km^2, 1,340 km of shorelines with topographic features of the mountain massif. The highest peak of the massif is 2,524 m above sea level (a.s.l.), in the south-central part of the island. (See Figure 2.1)

The topography of the mountain massif appears to reflect the combined effects of geologic and climatic processes. Geologic processes including partly tectonic processes and vertical and horizontal displacements of rock and climatic processes influence the topography in general. The mountain massif mainly consists of compact and massive rocks in comparison with the other parts of the island. Rocks forming the mountain massif are mostly resistant to weathering. The remainder of the island is practically flat except for several small hills that rise abruptly in the lowlands.

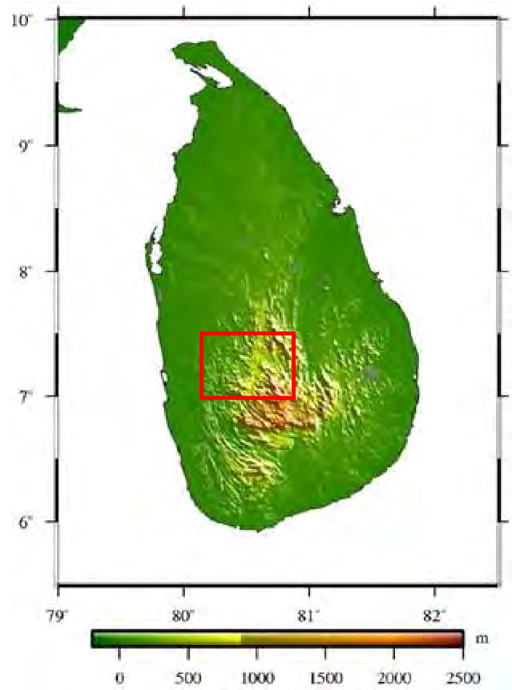


Figure 2.1 Topography of Sri Lanka

(Source: Project team)

2.1.2 Project Area

The project area is 100 to 500 m a.s.l. and located around the north-western edge of the mountain massif neighboring to a flat land area. The topography of the project area consists of many complex topographical features such as ridges, peaks, plateaus, basins, valleys and escarpments. (See Figure 2.2 and Figure 2.3)

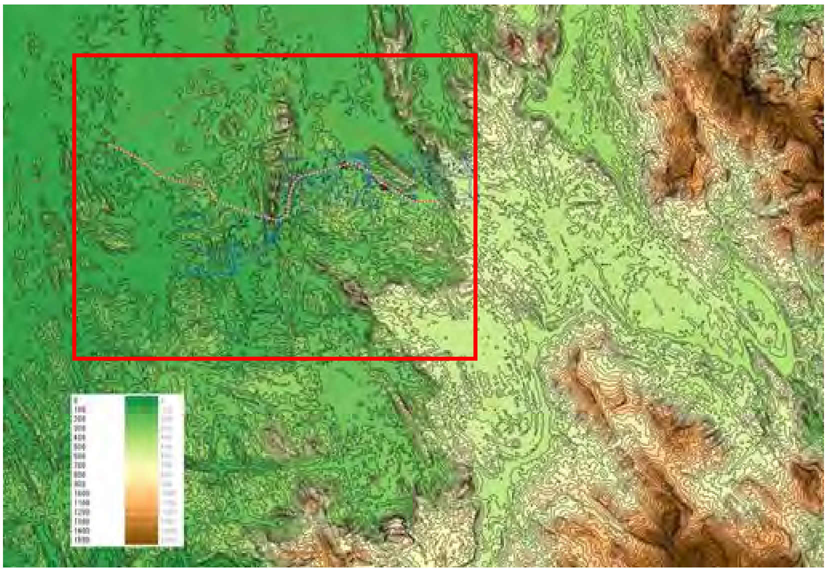


Figure 2.2 Topography of Surrounding Area of the Tunnel Sites

(Source: Project team)

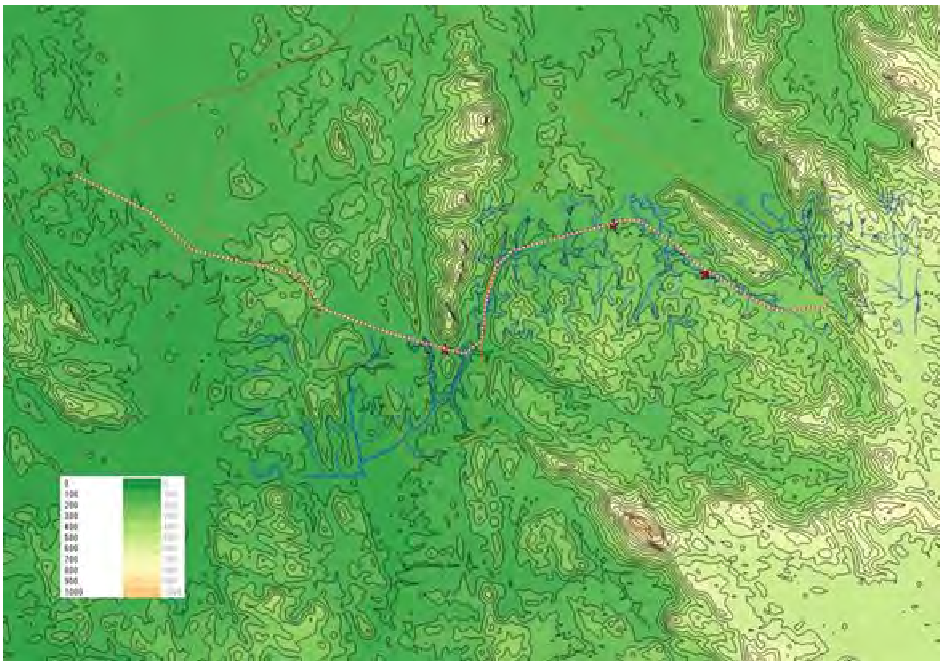


Figure 2.3 Topography around Tunnel Sites

(Source: Project team)

2.2 Geology

2.2.1 Sri Lanka

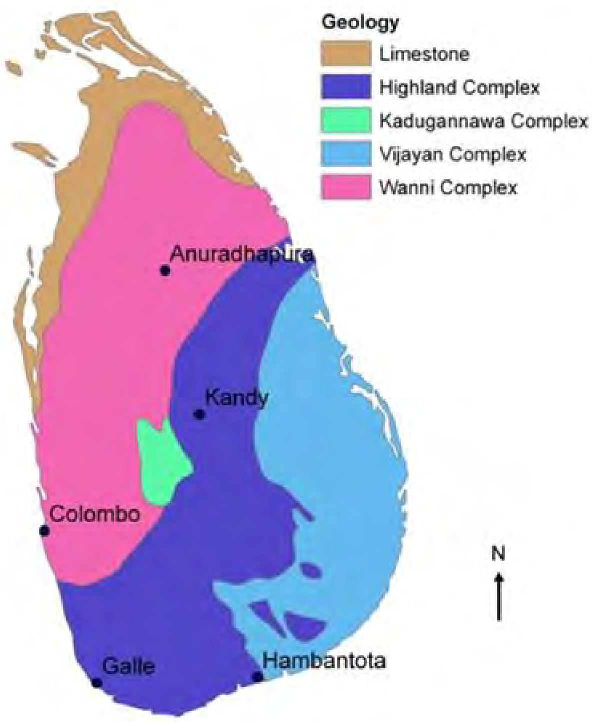


Figure 2.4 Geological Map of Sri Lanka

(ESDAC - European Soil Data Centre)

The geology of Sri Lanka consists mainly of crystalline Precambrian metamorphic rocks subdivided as Highland Complex, Wanni Complex and Vijayan Complex and other small divisions. Major rock types of the Highland Complex are meta-igneous rocks (charnockites, hornblende-biotite gneiss, migmatitic and quartzofeldspathic rocks) and metasedimentary rocks (quartzites, marble, dolomite and garnet-sillimanite-graphite schist). The Vijayan Complex that lies east of the Highland Complex consists of metamorphosed granitoids, charnockitic gneisses, migmatites, microcline-bearing quartzofeldspathic rocks, amphibolite and biotite gneiss. The Wanni Complex is located in the west side of the Highland Complex, and its rock types are similar to those of the Vijayan Complex. In addition, a small unit named "Kadugannawa Complex", composed of amphibolite or hornblendebiotite gneisses, and migmatites, is located in the central part of the country.

Jurassic sandstones, shales and mudstones are distributed in a limited area of faulted basins at Tabbowa and Andigama. Miocene limestone lies in a small marginal area unconformably on the Precambrian basement of the north, northwestern and southeastern coast.

By the Upper Jurassic Period, Sri Lanka was detached from the southern supercontinent Gondwanaland and the Indian Ocean began to open-up. During this drifting period, Sri Lanka was subjected to at least four major uplift through Jurassic, Miocene, Pliocene and Pleistocene times.

Prior to the Jurassic period, the Earth's surface was characterized by a relatively simple landmass configuration with three main continents (Gondwanaland, Laurasia and Angaraland) coalescing to form the Pangaean supercontinent. The oceanic domain was defined in its major part by Panthalassa and partially by the Tethys.

The interior of Gondwanaland was very arid and hot during the Lower and Middle Jurassic period (200-160 Ma). During the Upper Jurassic period (160-150 Ma), the global climate began to change due to the break-up of Pangaea. The interior of Pangaea became less dry, and seasonal snow and ice frosted the Polar Regions. By Lower to Middle Jurassic (200-160 Ma), it is assumed that Sri Lanka is emerged as a separated landmass. Sri Lanka reached its present position by the Lower to Middle Pleistocene period(1.806-0.781 Ma).

2.2.2 Project Area

Geology of the project area is mainly composed of Kadugannawa Complex. Wanni Complex is distributed in limited area of the north-western part of the project area. Kadugannawa Complex is composed of amphibolite or hornblende biotite gneisses and migmatites, whilethe Wanni Complex consists of metamorphosed granitoids, charnockitic gneisses, migmatites,microcline-bearing

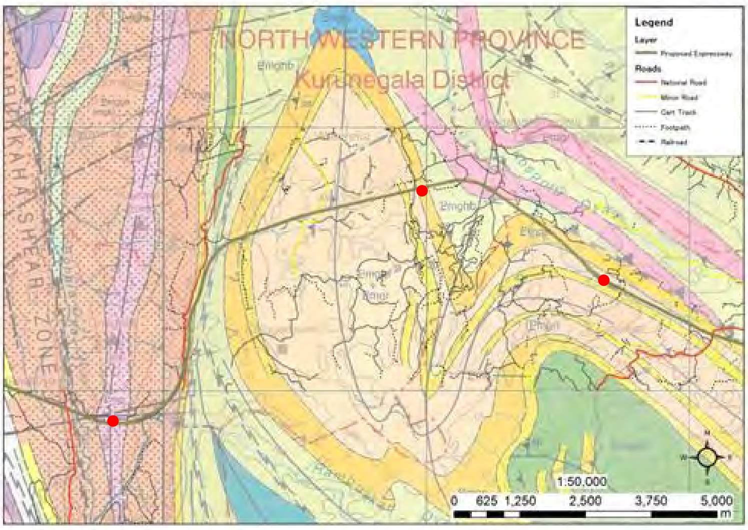


Figure 2.5 Geological Map around Tunnel sites

(Source: Geological Survey of Sri Lanka)

quartzofeldspathic rocks, amphibolite and biotite gneiss. Rocks of Wanni Complex are similar to those of the Vijayan Complex as mentioned above. Distribution of rock types are shown in Figure 2.5.

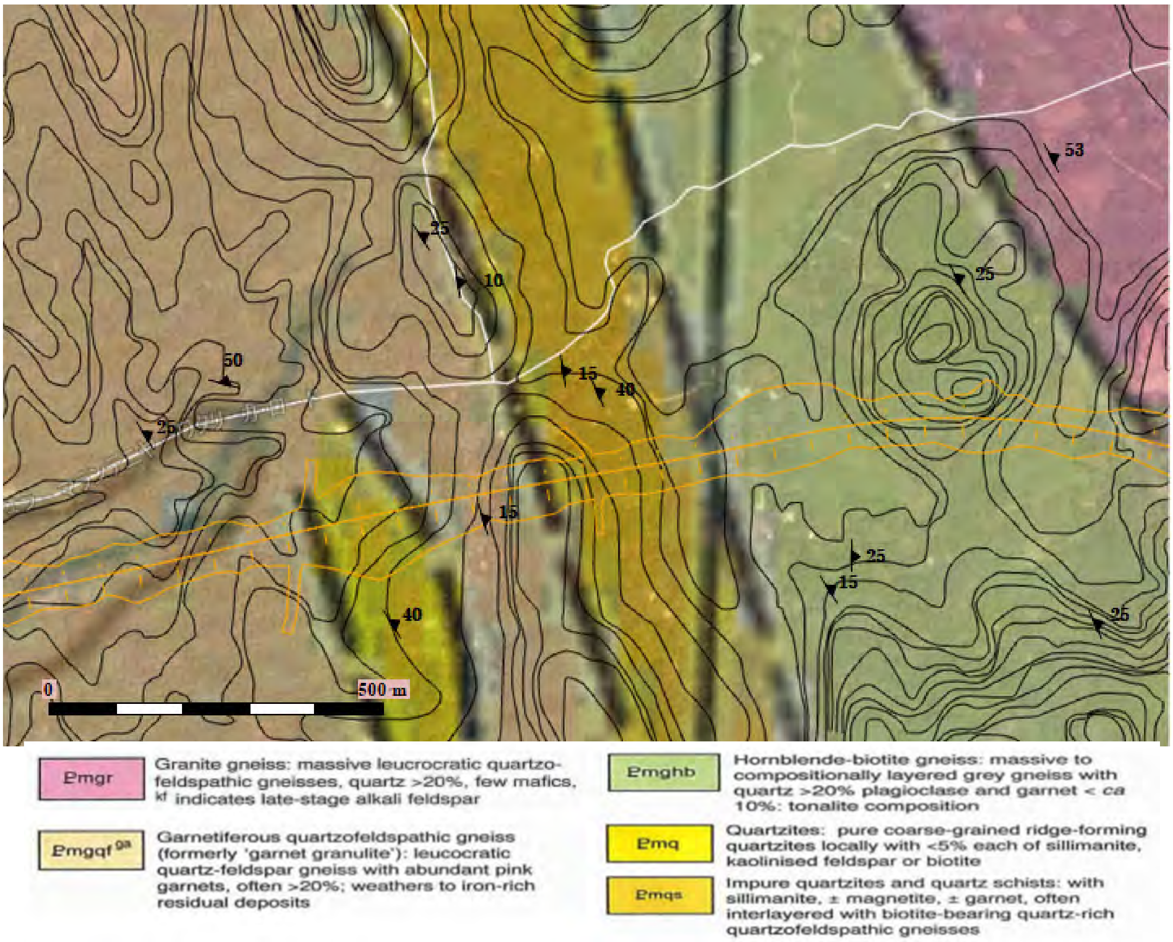


Figure 2.6 Geological Map around Tunnel No.2 Site

(Source: Modified after Geological Survey of Sri Lanka)

The project area is located in a small unit of the Kadugannawa Complex neighboring to the Highland Complex and Wanni Complex. Formations of granitic gneiss, garnetiferousquartzofeldspathic gneiss, hornblende biotite gneiss, quartzite and impure quartzite are distributed according to the geological map published by the geological survey of Sri Lanka. The distribution of these formations is confirmed by the site investigation. The foliation of these formations is dipping eastward in general.

Impure quartzites of biotite and garnet-bearing quartz-rich quartzofeldspathic gneiss are found mainly in the tunnel section. Intercalation of pure quartzites is confirmed by the surface geological mapping as well as by the drilling works in the impure quartzite. Fresh and compact bedrocks were hardly outcropped; weathered bedrocks, partly covered by overburden of talus and fluvial deposits, were observed.

2.3 Hydrology and Hydrogeology

2.3.1 Sri Lanka

(1) Meteorology

Sri Lanka is located between 5° 55' to 9° 51' North latitude and between 79° 42' to 81° 53' East longitude with climate characterized as tropical. The ITCZ (inter-tropical convergence zone) dominates the climate in Sri Lanka. The ITCZ is formed near the equator with a high solar radiation as a low pressure zone in the global circulation and ITCZ generates an ascending air current. The ascending air current divides into southward and northward current in upper atmosphere. Both currents form descending currents and then flow into ITCZ near the earth surface. Air mass from ITCZ completes the circulation. ITCZ moves to north and south according to the change of solar radiation accompanied with earth's revolution. In Sri Lanka, the year is divided into the following four seasons:

a. First Inter-monsoon season (FIM): March to April

b. Southwest Monsoon season (SWM): May to September

c. Second Inter-monsoon season (SIM): October to November

d. Northeast Monsoon season (NEM): December to February

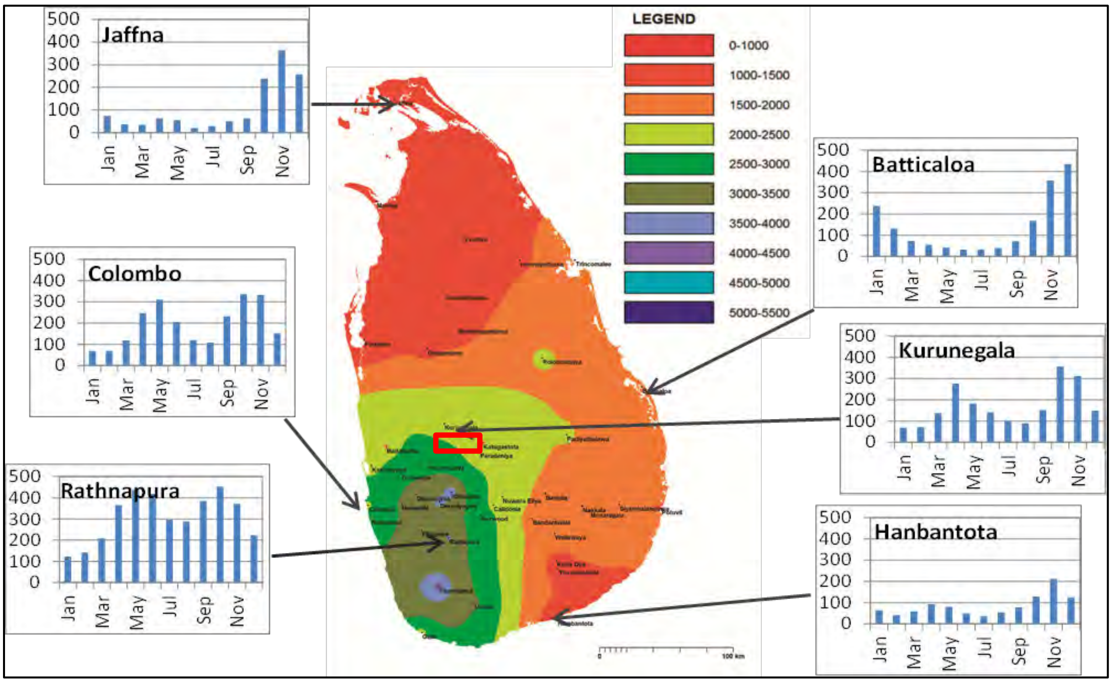


Figure 2.7 Mean Annual Distribution and Monthly Change of Rainfall in Sri Lanka (Oct. 1970-Sep.2015), (Red frame shows study area)

(Source: Hydrological Annual 2015/16, Irrigation Department)

These seasons are formed depending on the location of ITCZ. ITCZ is located across or near Sri Lanka during FIM then move north. ITCZ is located north of Sri Lanka during SWM then move south. ITCZ is located across or near Sri Lanka again during SIM then move south. ITCZ is located south of Sri Lanka during NEM then move north. The wind blowing toward ITCZ is monsoon and the name of monsoon period shows its prevailing wind direction.

The annual rainfall distribution and the seasonal change of rainfall at the representative sites in Sri Lanka are shown in Figure 2.7. The average annual rainfall of Sri Lanka is 1,861 mm. The rainfall amount is very varied depending on regions; the mountainous region in the southwestern part receives the maximum rainfall of more than 5,000 mm but the northwest coastal part receives less than 1,000 mm. Rainfall in the inter-monsoon season occurs due to convection by ascending air current in ITCZ and often accompanied by thunder. As rainfall in the monsoon season is caused by lifting the wet air of the Indian Ocean with the prevailing wind on the slope, and rainfall on the wind oblique slope is much. Therefore, the rainfall is significantly influenced by the terrain and the area; the seasonal change of the rainfall also varies depending on the terrain and the area.

The average annual air temperature in lowlands of Sri Lanka is 27 °C and it drops to 16 °C in Nuwara Eliya with 1,800 m of elevation. The coldest month is January and the hot month is March and April. The annual range of the air temperature is 1 to 5 °C, and the temperature

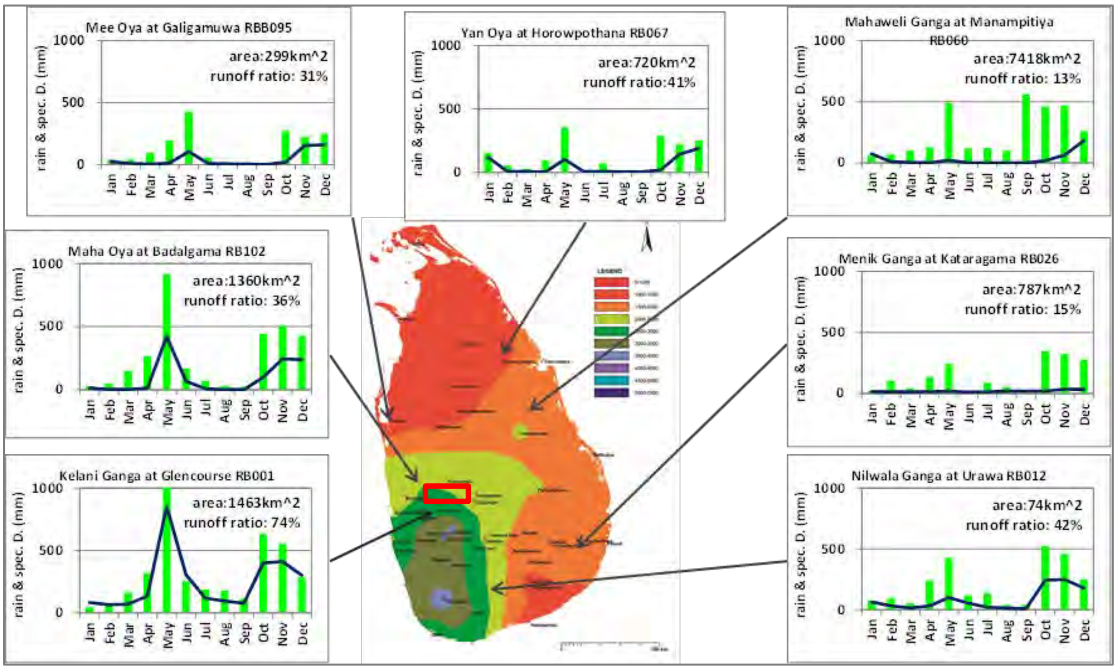


Figure 2.8 Monthly Changes of River Discharge and Rainfall (2015/16) with Annual Distribution of Rainfall (Oct. 1970-Sep.2015)
(Red frame shows study area)

(Source: Hydrological Annual 2015/16, Irrigation Department) change through the year is small. The daily range of air temperature is 5 to 10 ° C, and the daily range is larger than the annual range.

(2) Hydrology

There are 103 rivers in Sri Lanka. Most rivers flow radially from the central mountains. The largest basin is Mahaweli Ganga with a catchment area of 1,946 km^2. In Sri Lanka, more than 90% of the land is underlain by Pre-Cambrian crystalline rocks with low permeability that reduce the infiltration. As a result, a relatively high percentage of rainfall converts to surface water. The runoff ratio (equals runoff / rainfall) exceeds 70% in the wet zone while it is 20 to 30% in the dry zone. The runoff ratio from entire Sri Lanka is estimated to be about 45%.

The monthly discharge and rainfall graph, together with the annual rainfall distribution, are shown in Figure 2.8 to show the seasonal change of discharge. Discharge is indicated by specific discharge (equals discharge / catchment area) to avoid the influence of the size of the basin. It also shows the catchment area and runoff ratio (equals flow rate / rainfall amount). As with rainfall, the discharge is also varied. The figure shows that the discharge changes seasonally according to rainfall and the runoff ratio tends to increase closer to the mountain.

(3) Hydrogeology

Seven distinctive types of aquifers have been identified in Sri Lanka (see Figure 2.9). The seven types are:

a) Shallow karstic limestone aquifers,

b) Coastal sand aquifers,

c) Deep confined aquifers,

d) Laterite aquifers,

e) Alluvial aquifers,

f) Shallow regolith aquifers, and

g) Deep fractured zone aquifers.

The shallow aquifers, excluding shallow karst aquifers, are pore water among unconsolidated particles such as gravel, sand, silt and weathered rock, and generally spread horizontally. The deep groundwater is mainly fissure water. Fissure water exists in fissures or joints or cracks opened in the hard rock. This groundwater flows in the water vein and behaves differently from pore water. In general, the flow of deep groundwater is localized in the fractured rock. The flow velocity and the hydraulic gradient of deep groundwater irregularly change depending on the location and size of the fissures and an intermittent groundwater surface is formed.

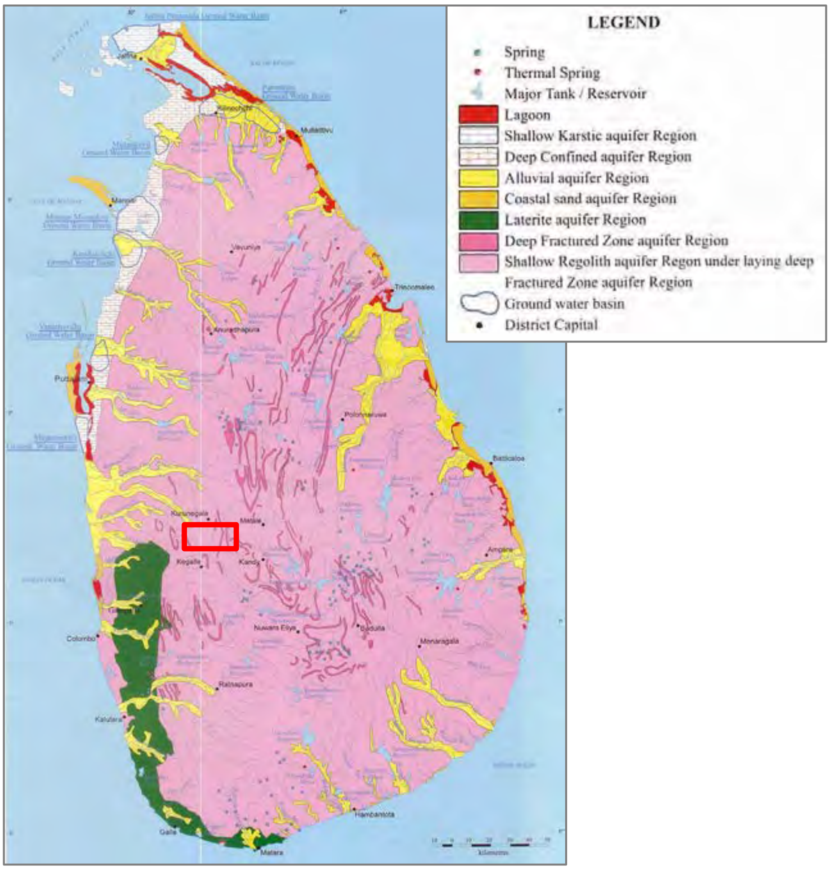


Figure 2.9 Groundwater Aquifer in Sri Lanka

(Red frame shows study area.)

(Source: National Atlas of Sri Lanka second edition, 2007, Survey Department)

Investigation of deep groundwater is more difficult than that of shallow groundwater. The reasons are that the deep groundwater exists deep from the earth's surface; one-dimensional form and irregular presence of water vein.

Drilling and blasting are the main methods of tunneling in hard rock, in general. A gush of groundwater sometimes occurs during drilling and blasting. Geological investigations are performed to locate joints, faults and fractured zones to evaluate water-tightness and potentiality of a gush of groundwater as well as the stability of the rock mass.

2.3.2 Study Area

(1) Meteorology

The annual rainfall of Kurunegala is shown in Figure 2.10 while the average monthly rainfall is shown in Figure 2.11. The Figure also shows the average monthly rainfall plus its standard deviation and monthly rainfall minus its standard deviation. Kurunegala is located 10 km northeast of the study area and is the closest observation station that can provide long-term meteorological data. The average annual rainfall is 2,111 mm; there is no significant change year to year. On the other hand, the monthly rainfall shows a large standard deviation and it shows that the monthly rainfall fluctuates largely from year to year. Regarding the standard deviation, it can be said roughly that there is a variation of more than the standard deviation every three years. Figure 2.11 indicates two peaks of rainfall in April and October to December and two rainy seasons. It also indicates the two dry seasons in February and June each year. Therefore, rainfall is large in the inter-monsoon period, and it is small in the monsoon period.

In the southwest monsoon season especially after rainfall in the mountainous area in the southwestern part of the study area, dry air mass flows. During the northeast monsoon season, dry air mass flows after the rainfall in the northeastern part of Sri Lanka, and thus it is derived that rainfall is small during the monsoon seasons.

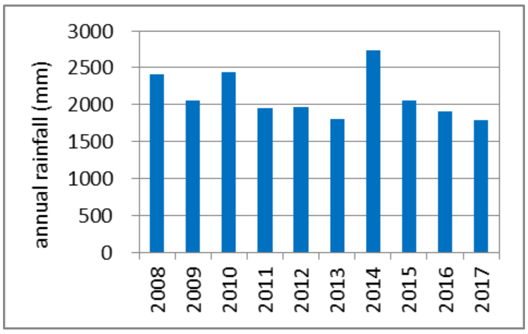


Figure 2.10 Annual Rainfall at Kurunegala

(Source: Meteorological Department)

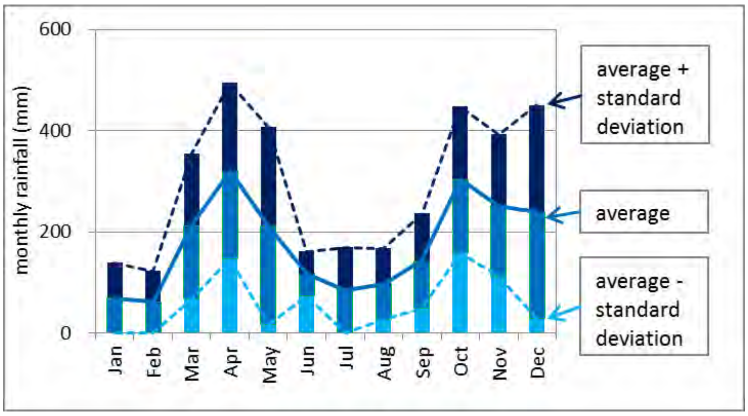


Figure 2.11 Monthly Average Rainfall, with its Standard Deviations at Kurunegala (2008-2017)

(Source: Project team)

(2) Hydrology

The study area straddles the basins of Deduru Oya and Maha Oya, and is near the upper basin of Mahaweli Ganga. Average monthly rainfall and average monthly specific discharge at Holombuwa from 2008 to 2017 are shown Figure 2.12. Holombuwa is located 25 km southeast of the study area and is a hydrological station near the study area. It is considered to represent the hydrological environment similar to the study area because it is located close to the mountain with small catchment area with 155 km^2. The seasonal change in rainfall is similar to Kurunegala, with some slight deviation. Reflecting the seasonal change in rainfall, the discharge also has two high water periods and two low water periods. The runoff ratio at Holombuwa is 50%, and half of the rain is received in the catchment area flows as surface water. The average of monthly specific discharge at Holombuwa is shown in Figure 2.13. It also shows the monthly specific discharge plus its standard deviation and monthly specific discharge minus its standard deviation. It is shown that monthly specific discharge fluctuates largely from year to year, as with rainfall.

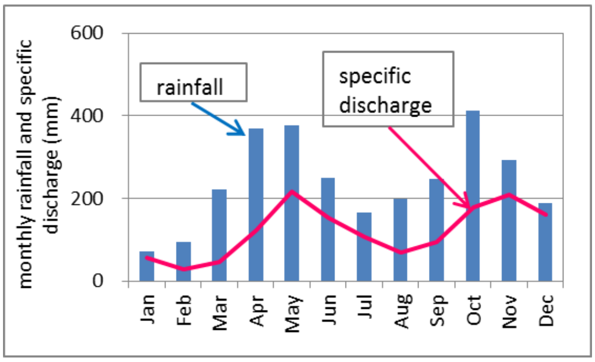


Figure 2.12 Monthly Rainfall and Specific Discharge at Holombuwa

(Source: Project team)

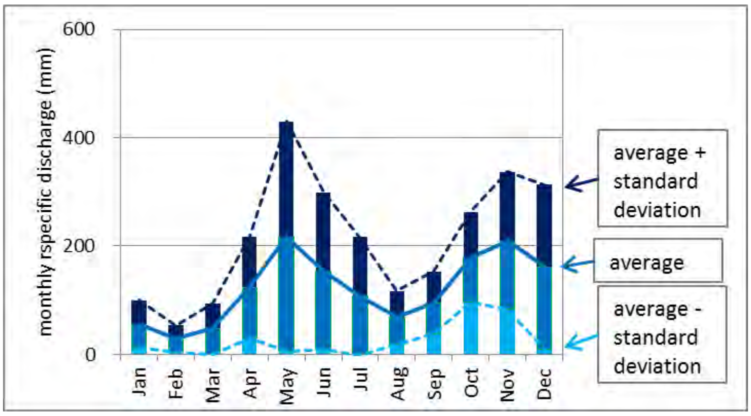


Figure 2.13 Monthly Average Specific Discharge with its Standard Deviations at Holombuwa (2008-2017)

(Source: Project team)

(3) Hydrogeology

The study area belongs to "shallow regolith aquifers region underlying deep fractured zone aquifers" of the seven sub-divided aquifer categories. This category includes shallow regolith aquifer without deep fracture zone aquifer.

The basement rocks at the tunnel points are quartzite, quartz gneiss, granite gneiss and others. It is considered that weathered rocks and weathered unconsolidated topsoil cover the basement rocks at tunnel point. If the tunnel is shallow, it is only necessary to consider the pore water in the unconsolidated regolith of the surface layer and in the weathered rock as groundwater, but if the tunnel is deeper, it is necessary to consider the fractured zone aquifer in the hard rock.

Chapter 3: Site Investigation

The geological investigation was conducted in the selected pilot site of Tunnel No.2, CEP-III (Activities 3.1-3.7). Processes of the actual investigation and its results were shared with C/P to promote technical transfer regarding the project outcome iii). Following sections are detail of the geological investigation conducted by the project.

3.1 Topographic Survey

3.1.1 Methodology

To obtain fundamental topographic data in the pilot site, control point survey, cross-section/plan survey and point survey to identify borehole/well/spring locations were conducted from 13th to 30th July 2017. Specifications of the topographic survey and arrangement of the survey lines are shown in Table 3.1and Figure 3.1, respectively.

(1) Control Point Survey

Two GPS control points were established near the tunnel site by static GPS survey. The procedure of the GPS control point survey is as follows:

- Two GNSS instruments (Leica GPS System 1200) were set up on two Survey Department control points (32B200005 and 32B20017) as base point.
- Another two GNSS instruments (Trimble 5700 GPS) were set up on new control points near the tunnel site as rover points (GPS-01 and GPS-02).
- GNSS raw data was logged simultaneously and post-processed by using Trimble Business Center software.

The accuracy of the GPS observation and post data processing were checked and approved by the Survey Department. In addition, leveling survey was conducted by using automatic level (NIKON AP-8) for vertical control. RDA control point (TBM No.1021) was utilized as a base control point of the leveling survey.

After the two GPS control points were established, control traverse survey was started from GPS-02 and closed to GPS-01 by using total station (NIKON NPL-632). 20 traverse points were established.

All control points were marked by steel pins fixed by concrete.

(2) Topographic (Plan) Survey

Topographic survey was done for 300meters x 100meters area around the planned tunnel by using total station. More than 1,100 points were surveyed for the topographic survey.

(3) Section Survey

Two longitudinal section surveys along the inbound/outbound lanes of the tunnels and six cross-section surveys were done by using the total station. The longitudinal survey lines don’t completely match the tunnel alignment because the tunnel alignment was changed after the section survey.

(4) Point Survey

Location of test drillings, geophones for seismic prospecting and water points (wells/springs/rivers) were surveyed by using the total station.

Table 3.1 Specification of Topographic Survey

Location	Mawathagama Division, Kurunegala District, North Western Province 7°23′49″N, 80°27′22″E
Section Survey	300m x 2 lines, 100m x 5 lines, 200m x 1 lines Total: 1300m Survey point: every 10m + Landform change points
Plan Survey	300m x 100m = $30,000m^2$ Scale = 1:1,000, Contour interval = 2m
Set Temporary Bench Mark	2 points (benchmark pile, identify elevation and coordinate)
Establishment of GPS Points	2 points
Identify Drilling Positions	3 boreholes (identify elevation and coordinate)
Identify Position of Water Points (wells/springs) near the site	30 points (identify elevation and coordinate)

(Source: Project team)

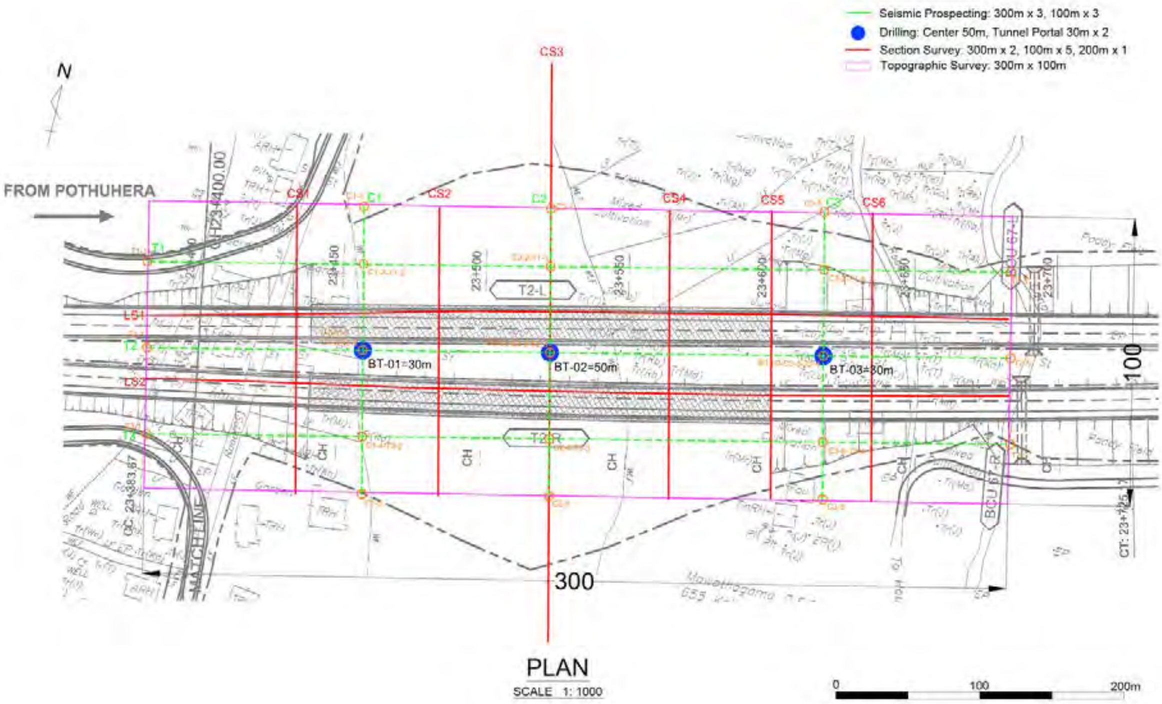


Figure 3.1 Arrangement of Survey Lines

(Source: Project team)

3.1.2 Result

(1) Topographic (Plan) Map and Control Points

Figure 3.1 and Appendix 2 show the results of the topographic survey and control point survey. A list of control points is attached in Appendix 2. Closing error of the traverse survey was 1/68,290.

The site is located in the north-west edge of the mountainous area, North Western Province, The tunnel is aligned from west to east direction through a hill which is a part of low ridge extending from south to north. This low ridge is a basin boundary between Maha Oya basin and Deduru Oya basin. The east side of the hill, Deduru Oya side, is covered by forest; some houses are located near the foot of the hill. The east end of the hill is a steep slope. An alluvial flatland and streams are situated beyond the east side of the steep slope. The flat land is utilized as paddy fields. The west side of the hill, Maha Oya side, is rubber plantation. The west end of the hill is gentle slope; some houses are located near the foot of the hill.

Northern and southern areas of the hill are saddleback of the low ridge, and the tunnel goes through directly under the hilltop.

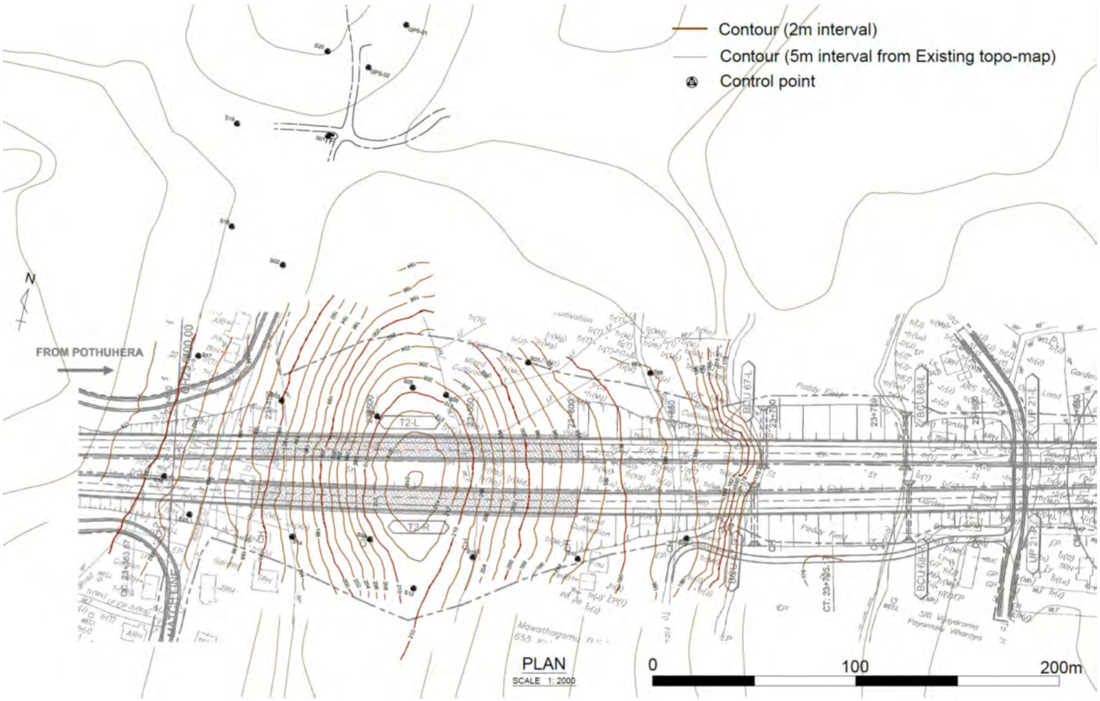


Figure 3.2 Topographic Map and Control Points in Pilot Site

(Source: Project team)

(2) Section Survey

Results of the section survey are shown in Figure 3.3 and Appendix 2. The elevation of the hilltop is ca. 216m, and the maximum overburden of the tunnel is ca. 26m. Hill slope angles range from 12° near the tunnel portal to 30° at the hillside. Mean slope angles of the west side and east side slopes are ca. 24° and 20°, respectively.

(3) Point Survey

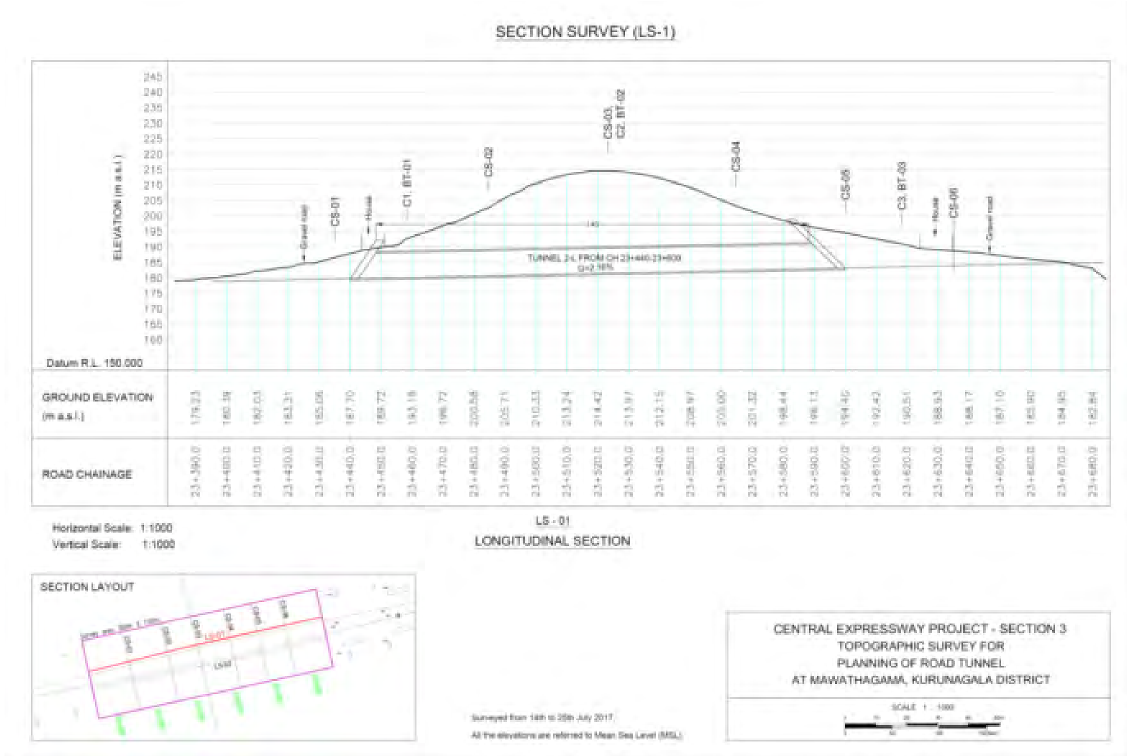
Most of wells and springs are situated around the foot of the hill. The point survey results of the water points are utilized for hydrogeological analysis in Chapter 5.

The point survey results of the drillings and geophones for seismic prospecting are also utilized for the geophysical and geological analysis in the sections 3.2 and 4.3.

3.1.3 Evaluation and Discussion

The pilot site is a gentle hill (mean slope angle: 20°-24° at hill side and 12°-15°at tunnel portals, max. overburden: ca. 26m). The direction of tunnel portals and hill slope cross at right angle. Therefore, unsymmetrical pressure is not likely to occur.

Landslide and slope failure were not observed in the pilot site. Loose rocks and source of rock fall (outcrop of weathered bedrock) were also few in the hill slope above the tunnel portal.



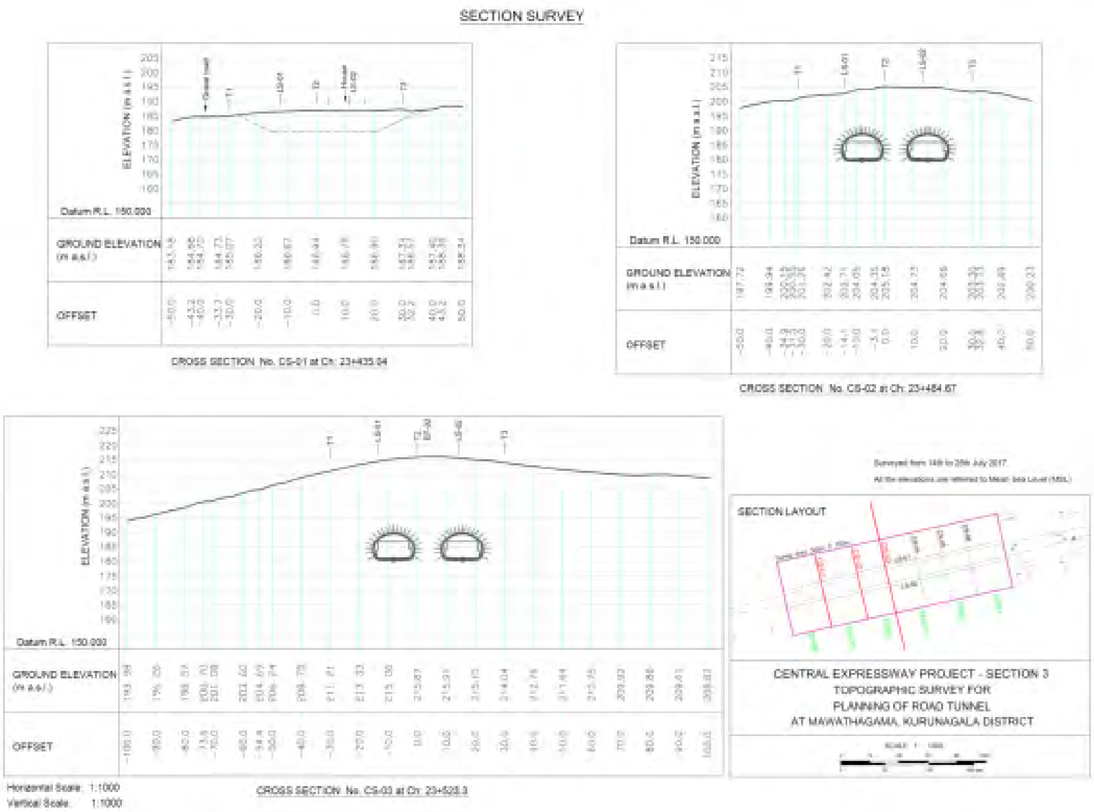


Figure 3.3 Cross-section of Tunnel No.2

(Refer to Appendix 2 for other cross-section drawings)

(Source: Project team)

3.2 Seismic Refraction Prospecting

The seismic refraction prospecting produces artificial seismic wave by blasting or by using a large hammer. The seismic wave is refracted at the underground geological boundary and comes back to the surface. This refracted wave (P-wave) is measured by the surface instrument, and the underground velocity structure is interpreted.

It is widely used to obtain important indicators for evaluation of natural ground and evaluation for basic rock and engineering judgment of unstable soil lump.

3.2.1 Methodology

(1) Survey Setting

As shown in Table 3.2, the survey quantity was 1,200 meters in six survey lines.

For each survey line, stationary piles were set at 5-meter horizontal intervals, and leveling surveys were carried out.

Figure 3.4 shows the line layout.

Table 3.2 Seismic Exploration Quantity Table

Survey line name	Survey line length (m)	Survey line direction	Remarks
T1	300	Tunnel direction	
T2	300	Tunnel direction	
T3	300	Tunnel direction	
C1	100	Tunnel traversal direction	Tunnel portal (Inlet side)
C2	100	Tunnel traversal direction	Ridge
C3	100	Tunnel traversal direction	Tunnel portal (Exit side)
Total	1,200		

(Source: Project team)

(2) Measurement

In this survey, a digital measuring instrument, which can arrange 24 geophones at a time (1 span), was utilized.

Since the measurement interval is 5m, 1 measurement span is 115m. In the longitudinal line (300m), the first span was 0 to 115m, the second span was 100 to 215m, and the third span was

200 to 300m. The vibrating point of 1 span is 6 to 10 points, and the vibrating point interval is 30 to 40m.

Dynamite was planned to be used as the seismic source. However, it was unlikely that the use of explosives would be permitted during the investigation period; thus, the stacking method using hammering and falling down of “MONKEN” (Heavy weight-63.5kg) was used.

The stacking method is a method of compensating for a small excitation signal and adding signals by hitting the same part several times (Ground noise is averaged and decreased). Dropping a MONKEN can generate a signal larger than a hammer hit.

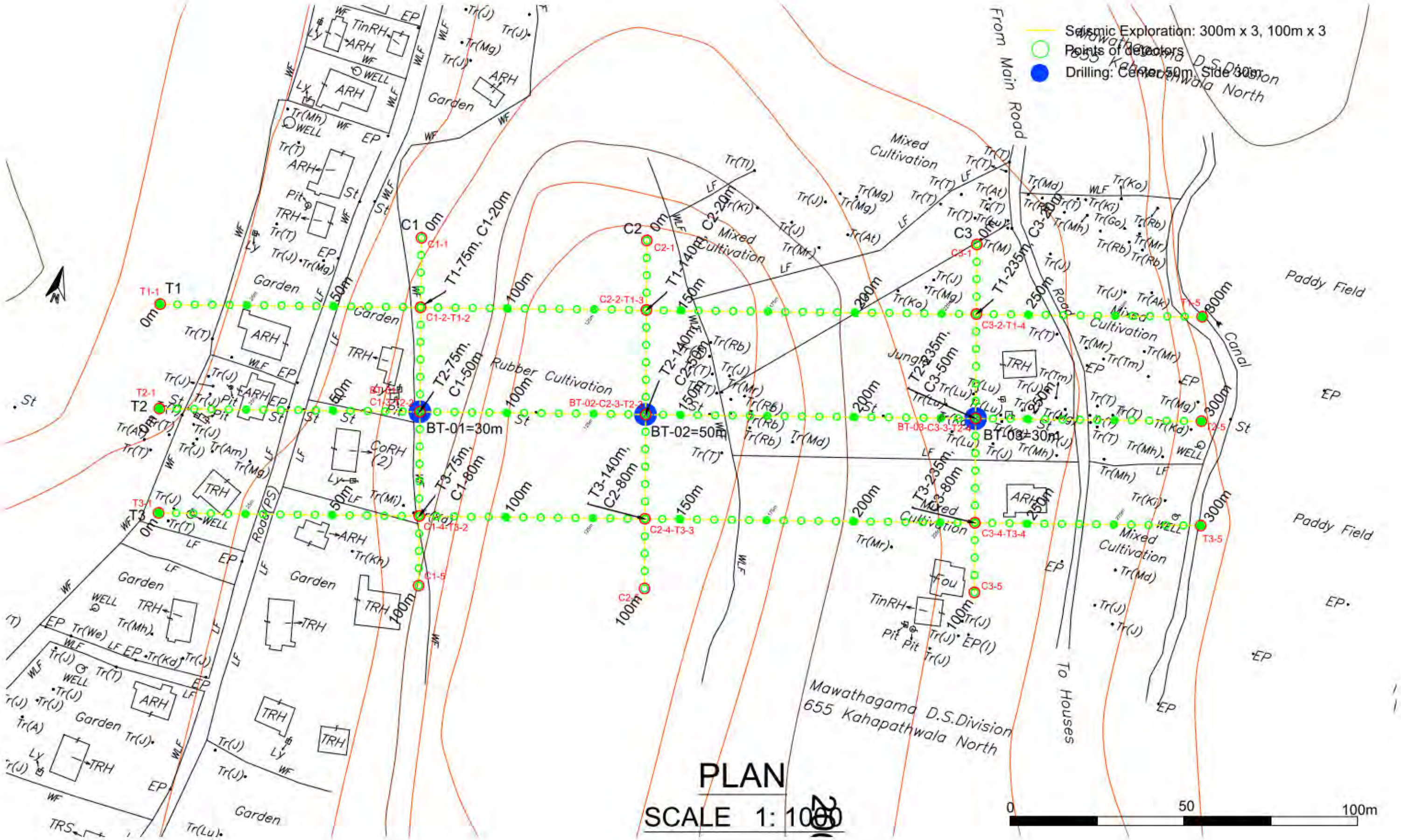


Figure 3.4 Survey Line Arrangement

(Source: Project team)

Figure 3.5 shows a schematic diagram of seismic refraction prospecting, and Figure 3.6 shows the procedure of seismic refraction prospecting.

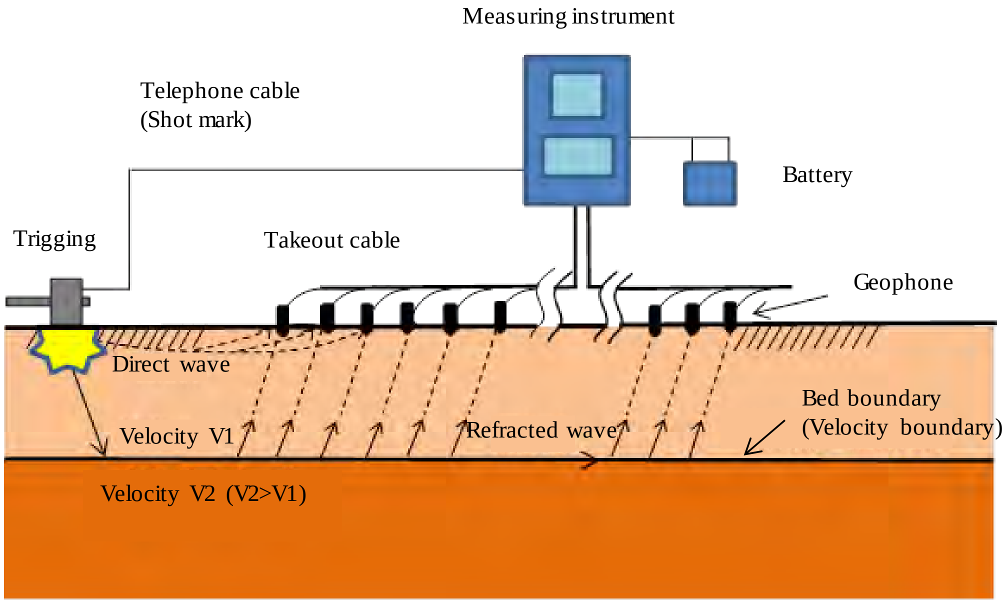


Figure 3.5 Seismic Refraction Prospecting Pattern Diagrams

(Source: Modified after SeisImager/SW: basis of surface wave exploration, OYO)

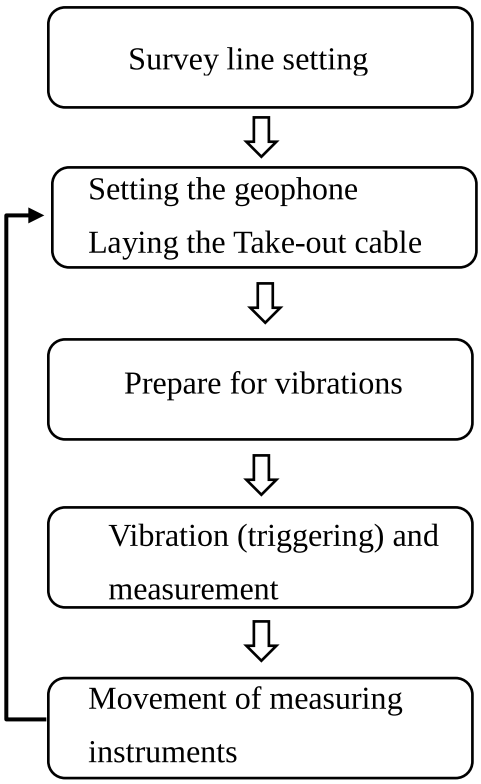


Figure 3.6 Procedure of Seismic Refraction Prospecting

(Source: Project team)

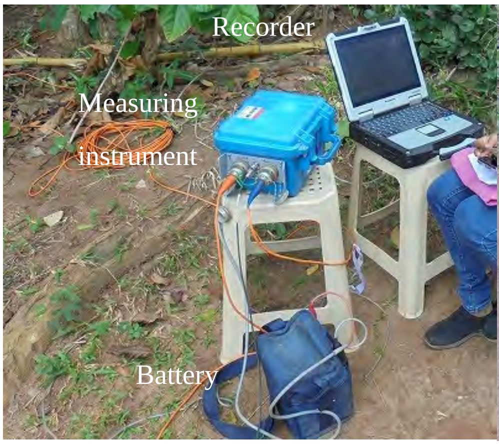
(3) Used Equipment

The measuring instruments used in this survey are shown in the table below.

Table 3.3 List of Equipment Used

Equipment		Specification	Quantity
Measuring instrument	McSEIS-SW MODEL 1109	Numbers of Channels:24ch, Digital recording method, Gain: 36dB	1set
Recorder	Laptop		1set
Receiver	Geophones	Natural frequency:14Hz	24 sets
Observation line	Take-out cables	5m Interval and 12 ingredient	2 sets
Other	Battery(12V,24A) Wooden hammer Trigger Trigger-cord Iron plate Drop hammer (63.5kg) Tripod		

(Source: Project team)



Measuring equipment-1

Measuring equipment-2

Measuring equipment-3

Measuring equipment-4

(4) Analysis (Hagiwara's method)

To obtain accurate analytical results by seismic refraction prospecting, the following is necessary.

1. Obtain good measurement record.
2. Prepare a triggering plan to obtain travel-time curve propagating through the basement rock.
3. Obtain accurate basement rock velocity.

1) Data and Travel-time Curve

The time taken for the wave to travel between the triggering source and the receiver is called travel-time. A graph plotting travel-time is a travel-time curve. In the travel-time curve diagram, the horizontal axis is the distance from seismic vibration source to the receiver geophone while the vertical axis is the measured travel-time. The initial travel-time is the time which takes for the seismic wave to arrive at the geophone for the first time from the vibration source.

The travel-time curve plots the relationship between the vibration generating point and the distance between the geophones.

The measurement record is illustrated with a clock line every 1/100 second (4mm interval), and read it up to 1/1000 second by eye measurement. Since the propagation velocity of the wave can be calculated through the relationship velocity equals distance divided by time, the reciprocal of the slope of the travel-time curve represents the wave propagation velocity.

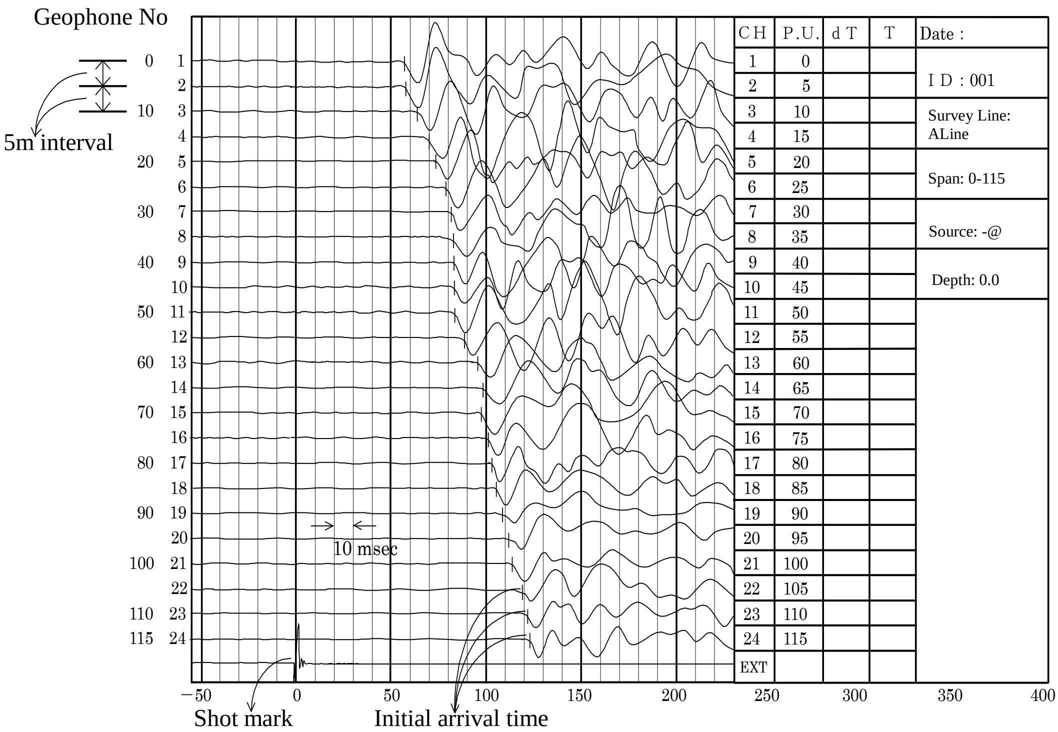


Figure 3.7 Measurement Data

(Source: Project team)

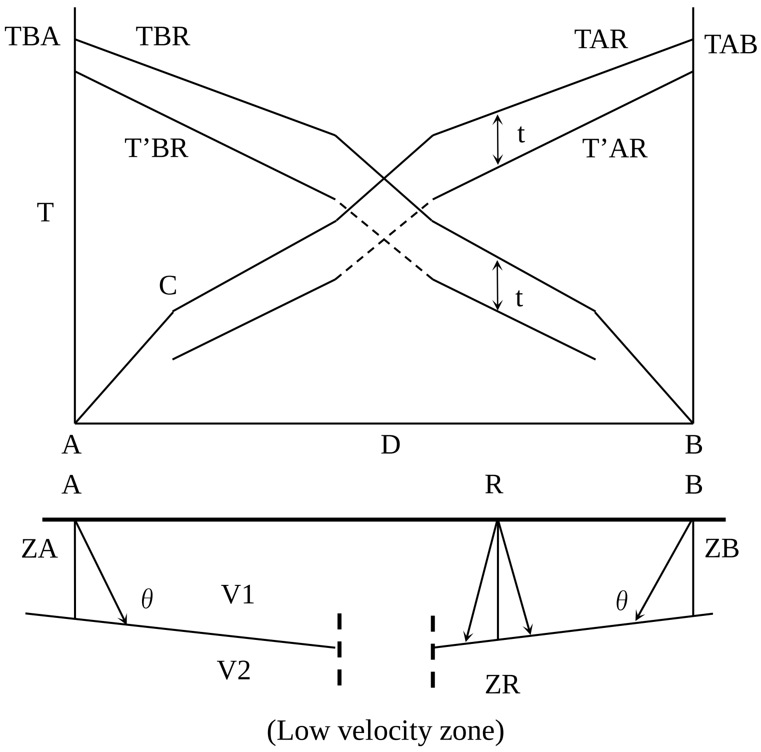
2) Analysis of Travel-time Curve

The travel-time curve shows a shape corresponding to the distribution condition of the propagation speed of the wave traveling through the ground. Consequently, by analyzing the shape of the travel-time curve, it is possible to estimate the underground velocity distribution.

In this survey, "Reciprocal method - Hagiwara's method" is deemed suitable for analysis of seismic wave exploration in sloping lands such as tunnel survey, and was employed.

3) Principle of the "Hagiwara's Method"

For the analysis of the travel-time curve, Hagiwara's method, so-called "Hagitori method" (a.k.a: surface layer removal method), was employed. The principle of the method is briefly described as follows.



A, B: Vibration point — t: Delay time

R: Measurement point — Z: Depth

T: Travel-time — V_1, V_2: Velocity

T': Principle line — θ : Critical angle of incidence

Figure 3.8 Principle of "Hagiwara's Method (Hagitori Method)"

(Source: Modified after Geophysical exploration handbook: Society of Exploration Geophysicists of Japan, 1998)

In the two-layer structure as shown in Figure 3.8, TAR, TBR, TAB, are given by the following equations.

$$TAR = ZACOS\theta / V_1 + ZRCOS\theta / V_1 + AR / V_2$$

$$TBR = ZBCOS\theta / V_1 + ZRCOS\theta / V_1 + BR / V_2$$

$$TAB = ZACOS\theta / V_1 + ZBCOS\theta / V_1 + AB / V_2$$

Consider the amount of T\`AR given by the following equation.

$$T`AR = TAR - (TAR + TBR - TAB) / 2 = ZACOS\theta / V_1 + AR / V_2$$

Since AR represents the distance from the triggering point to each measurement point, it can be considered as variable X, it becomes a straight dotted line as shown in the figure (T'AR, T'BR). This is a "Hagitori-line" which slope is 1 / V_2 and represents the velocity of the lower layer. When the velocity of the lower layer changes, the slope of the T 'straight line changes. For example, in a low velocity part such as a fractured zone, the T 'straight line becomes a staggered shape and a fractured zone can be detected.

Next,

$$t = TAR - T'AR = ZRCOS\theta / V_1$$

$$ZR = tV_1 / COS\theta$$

In the above equation, the depth ZR at each measurement point R can be obtained by multiplying t by a constant V_1 / COS θ.

"t" is called depth travel-time. When "Hagitori" method is applied correctly between travel-times that passed through the lower layer, depth travel-time can be obtained continuously. As a consequence, a cross section showing the depth in time is obtained on the "Hagitori line".

The velocity of the lower layer and its distribution are able to be described by performing "Hagitori", and can continuously obtain depth travel-time t = TAR-T\`AR. These two outcomes are excellent features of the "Hagitori method".

On the other hand, , in the case of multi-layer (more than two layers) structures, it is only necessary to geometrically expand the concept of the two-layer structure, and the same methodology can be utilized regardless of the number of layers.

4) Policy of this Analysis

Points considered when conducting analysis are given below.

In general, error of reading always occurs in travel-time curve. However, the travel-time curve theoretically must satisfy the following conditions:

i) Travel-time difference does not increase.

ii) Reciprocal travel-time should be same.

iii) Intercept time from the both sides of travel-time curves should be same.

(5) Analysis (seismic refraction prospecting)

1) Creation of travel-time curve diagram

The plotted initial travel-time curve contains some inconsistency because of error in reading the record and difference on conditions of vibration source and geophone setting. In preparing the travel-time curve, correct the travel-time curve to satisfy the conditions described below. If large correction is necessary, it is required to read the measurement data again.

The required policy of this analysis is described in detail as follows:

i) Travel-time parallelism : The travel-time propagating in the same direction and velocity layer keep parallelism.

ii) Match of reciprocal travel-time : The time required for the waves between the excitation points facing each other are identical.

iii) Match of Intercept time : The T' curve extending from the excitation point to both points coincide when there is no abrupt change in the vicinity of the excitation point (origin).

The above policies expressed by the type of travel-time curve are shown in Figure 3.9.

It is often difficult directly to determine the velocity value from the gradient of the travel-time curve due to the inclination of the velocity layer or the roughness of the travel-time curve which reflects uneven distribution of the low velocity layer near the surface of the ground.

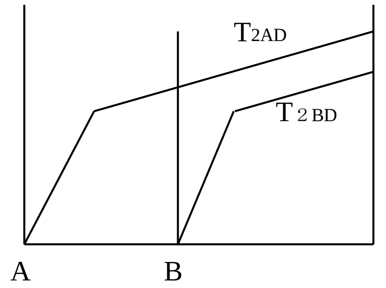
Therefore, to remove irregularities and apparent slopes of these travel-time curves, a velocity travel-time curve (T' curve) is created from the opposite reciprocating travel-time curve and the velocity value is determined from the gradient of the T' curve. Furthermore, the time corresponding to each velocity layer is obtained by combining the travel-time curve and the T' curve.

When the velocity and time are known, the depth is calculated, and it can be expressed as a velocity layer section by calculating the depth for each velocity layer at each measurement point.

On the survey and the analysis, velocity values in the range of 0.3 to 4.7 km / sec were obtained for each of the survey lines, so analysis was conducted within that range.

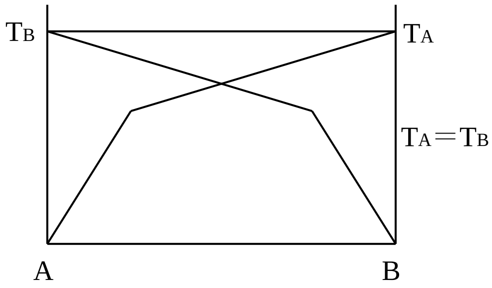
i) Travel-time parallelism

Travel-time difference does not increase



ii) Reciprocal travel-time

Reciprocal travel-time should be same



iii) Intercept time

Intercept time from both sides of travel-time curves should be same

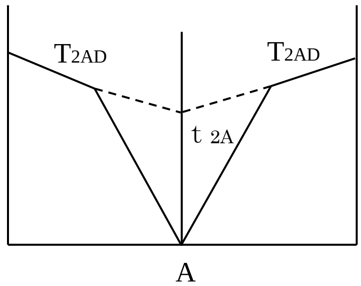


Figure 3.9 Nature of Travel-time Curve

(Source: Modified after Geophysical exploration handbook: Society of Exploration Geophysicists of Japan, 1998)

2) Correction of Cell Speed Using Tomography Method

The tomography method is described as follows:

i) Create an initial model that velocity increases with depth.

ii) For the initial model, calculate the theoretical travel-time.

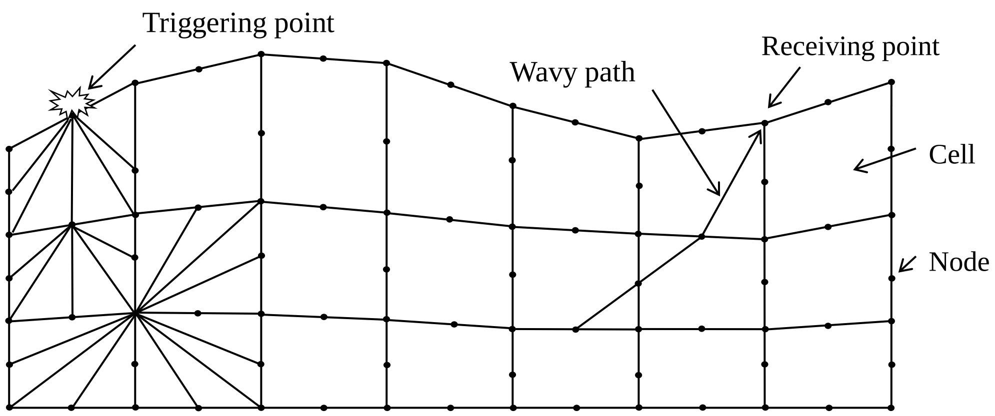
For calculation of theoretical travel-time, a calculation method based on the principle of Huygens shown in Figure 3.10 is used.

iii) Calculate the difference (residual) between the theoretical travel-time and observed travel-time and correct the model to reduce the residual.

Modification of the model corrects the speed of each cell by SIRT method.

In the SIRT method, the residuals are distributed to all the cells through which each wavy line (wave propagation path from the excitation point to the receiving point) passes, and the speed of each cell is corrected to eliminate the residual.

iv) The calculation of this theoretical travel-time and the modification of the model are repeated, and the iteration is completed when the residual becomes sufficiently small.



Mimic the velocity layer cross section to multiple quadrilaterals (cells).The elastic wave velocity in each cell shall be constant.

- The plurality of nodes is provided around each cell (on the speed boundary).
- The wavy line from the excitation point passes between these nodes.
- The initial travel-time between the triggering point and the receiving point is the running time of the shortest traveling route among all the paths following these nodes.

Figure 3.10 Calculation of Theoretical Travel-time (initial travel-time) by Huygens principle

(Source: Modified after SeisImager/SW: basis of tomography analysis, OYO)

Figure 3.11 shows the procedure of seismic prospecting tomography analysis.

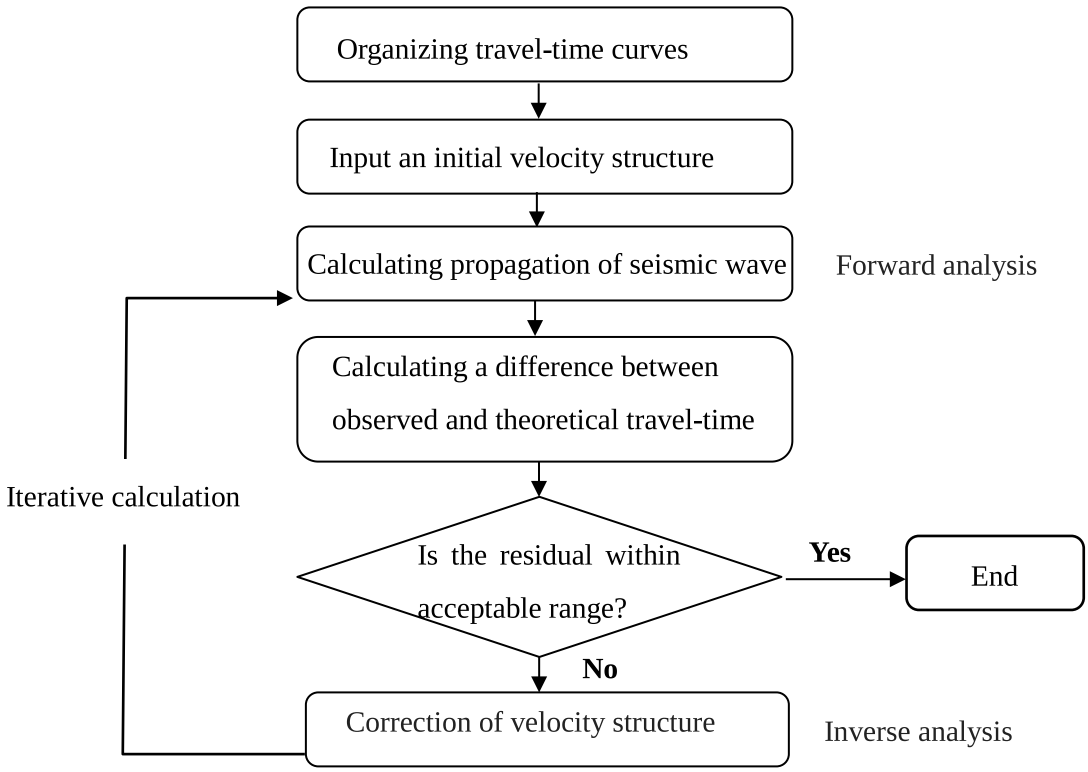


Figure 3.11 Procedure of Seismic Prospecting Tomography Analysis

(Source: Project team)

3.2.2 Analysis Result

The analysis results are shown in the analytical cross section along with the travel-time curve diagram.

The geology distributed in the surveyed area can be identified comprehensively from the geological map, the core drilling result and the seismic refraction prospecting results. It is judged that the geology distributed in the surveyed area is composed of the quartzite layer (Q) and the quartzite/feldspar gneiss (Q-Fg).

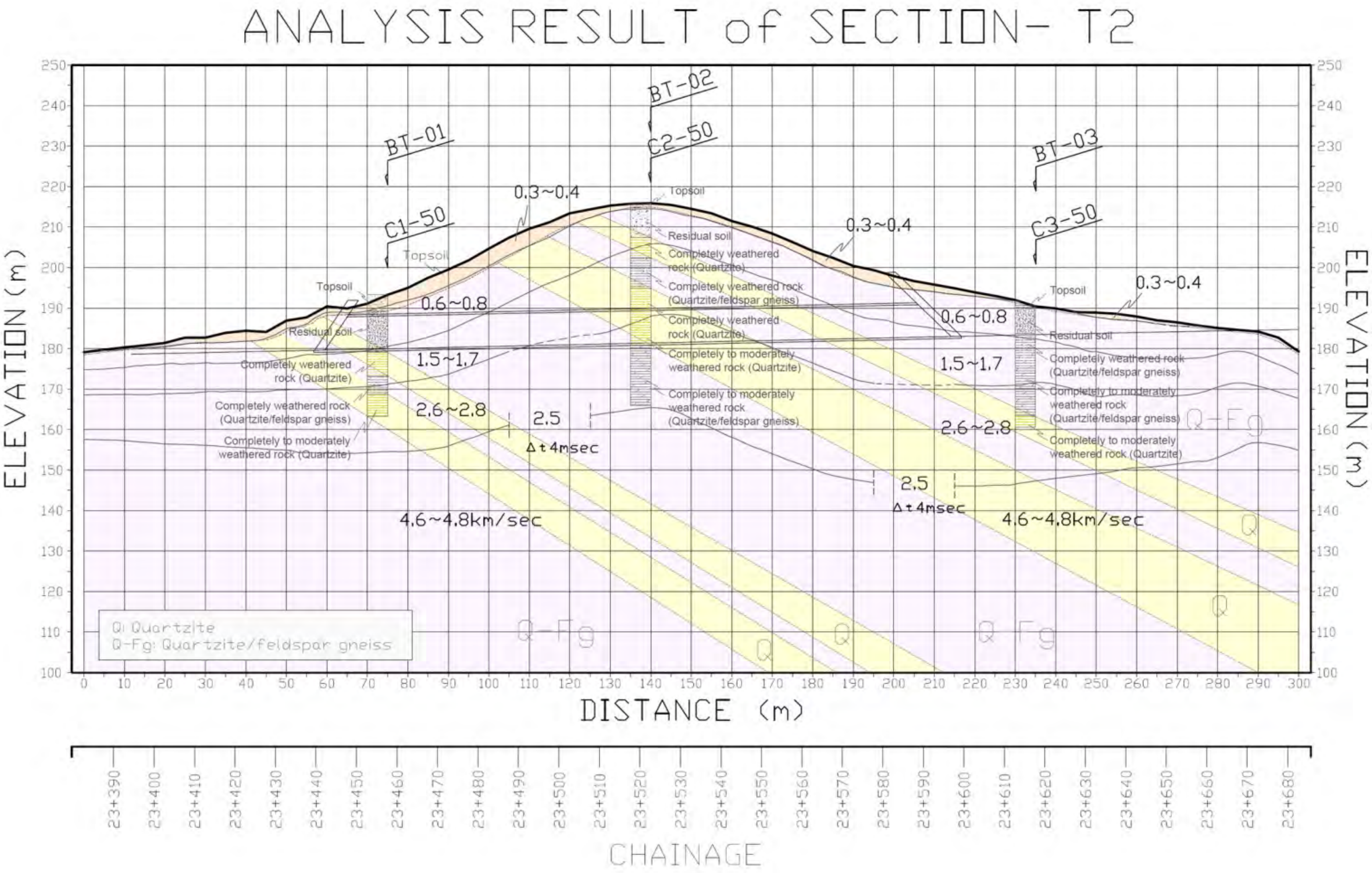


Figure 3.12 Estimated Stratum Slope

In this tunnel planning area, the quartzite/feldspar gneiss layer (Q - Fg) and the quartzite layer (Q) form mutual layers with a structure of gentle dip angle to east in the north - south strike.
The quartzite layer (Q) is distributed in the central part of the tunnel. Estimated from geological maps, seismic refraction exploration results and BT - 02 core drilling (Figure 3.4) results, the distribution area of quartzite layer (Q) is strongly resistant to weathering as compared with other rocks because the ridges are almost continuous and of the hard rock mass.
For other strata, the quartzite layer (Q) in the west side is composed by weathered quartzite. Looking at the geological map, there are many terrains such as valley and stream in the distribution area of this layer. Moreover, it is concerned that the bedrock in the area seems composing fragile rock state from the analysis result that the distribution direction coincides with the recessed region of the basement rock in tomographic analysis.

The velocity layer obtained by the “Hagitori method” can be classified into the five-layer structure. The geological situation of each velocity layer is estimated from the elastic wave velocity value and summarized in Table 3.4.

Table 3.4 Geological Condition Estimated as Elastic Wave Velocity Value

Velocity layer	Elastic wave velocity (Vp=Km/sec)	Estimated geology
1st velocity layer	0.3〜0.4	Surface soil and talus accumulation
2nd velocity layer	0.6〜0.8	Heavily weathered soil of the basement rock is the main body. There are also some heavily weathered rocks.
3rd velocity layer	1.5〜1.7	Weathering rock of basement rock which is deteriorated from the crack face and softened.
4th velocity layer	2.6〜2.8	Weak weathering～weathered rock of basement rock
5th velocity layer	4.6〜4.8	Basement rock of gneiss and quartzite
	Δt	Low velocity zone (The numerical value is the section speed.) Δt is the time difference of the section.

(Source: Project team)

(1) Geological Circumstances of each Velocity Layer

i) 1st velocity layer (0.3 - 0.4km/sec)

It seems to be a surface soil and a talus layer. It seems to be very soft unstable sediment.

ii) 2nd velocity layer (0.6 - 0.8km/sec)

It is mainly composed of heavily weathered soil of the basement rock including some heavily weathered rock. It is brown soil (clay to sand) and contains gravel of weathered rock. The N value is approximately 20 to 50 or more.

iii) 3rd velocity layer (1.5 - 1.7km/sec)

It is originally the weathered rock area where there is deterioration from the crack face; softening is also recognized. In the drilling survey, there were many places that can be taken with gravel-like cores.

iv) 4th velocity layer (2.6 - 2.8km/sec)

Although it is difficult to make an assumption from the collected core, it is originally a distribution area of weak weathered or weathered rocks. The most parts of the rocks are considered to be in a good condition regardless of observed some weathered rocks. In the survey, some rod-shaped core samples were collected at several points in this layer..

v) 5th velocity layer (4.6 - 4.8km/sec)

It corresponds to basement rock velocity layer. The geological condition is expected to be in a fresh and hard state. In the drilling investigation, the excavation depth didn’t reach this speed layer and it hasn’t been identified.

vi) Low velocity zone (⋮ ⋮)

In the survey, five low velocity zones were observed, and the results are shown in the analysis sectional view and the low velocity zone list is shown in Table 3.5.

Table 3.5 Low Velocity Zone List

Survey line name	Low velocity section (m)	Time lag Δt (msec)	Velocity value (km/sec)	Remarks
T1	115-120	2	1.5	Weathered rock
T2	105-125	4	2.5	Weathered rock
T2	195-215	4	2.5	Weathered rock
T3	110-125	3	3.0	Weathered rock
T3	215-235	6	2.3	Weathered rock

(Source: Project team)

There are low velocity zones of Vp = 1.5, 2.3, 2.5, 3.0 km / sec for the T1 to T3 survey lines of the "hagitori' analysis". Both of these are consistent with the geological boundary line and the vulnerability is a concern. The rock formations of quartzite are kept up to the depth of 35 m of BT-2 core (refer to Geological Log: Appendix 4). The place deeper than that is almost in the state of earth and sand.

The distribution area of the low velocity zone is summarized below.

i) T1, T2, T3-Line, L = low velocity zone of 105-125 m

It corresponds to the geological boundary line between the quartzite layer (Q) and the quartzite/feldspar gneiss layer (Q - Fg), and there is a possibility that the periphery of the boundary portion is weakened geologically and forms a weak line. Specifically, it is the sedimentation of the rock by progress of weathering and alteration.

ii) T2,T3-Line, L = low velocity zone of 195-235 m

It corresponds to the geological boundary line between the quartzite/feldspar gneiss layer (Q - Fg) and quartzite layer (Q), and there is a possibility that the periphery of the boundary is weakened geologically and forms a weak line. There is also a possibility that decomposed rock progresses due to weathering and alteration. However, as mentioned above, considering that the quartzite layer (Q) is relatively hard and highly resistant to weathering and others., there is a risk that the weakening of the quartzite/feldspar gneiss layer (Q - Fg) distribution area side is proceeding high.

(2) Specificity and Problems

i) The phenomenon of the dent of the basement rock layer in tomographic analysis

The dent of the basement rock layer in the tomographic analysis is a common phenomenon appearing in any of the T1 to T3 survey lines. These are almost overlapped with the distribution areas of the quartzite layer (Q) in the west side and the quartzite/feldspar gneiss layer (Q - Fg). Investigation infers that at least the distribution of rocks in the fresh and hard state may remain in the deep part, and it is further inferred that a fragile rock is widely distributed, although there is uncertainty in the observation of the boring core.

ii) The difference in velocity value between crossing line (T line × C line)

The seismic refraction prospecting is a method to analyse the ground using method to measure the fastest P-wave arrival time from the vibration point through the ground . Therefore, when there are a substance with a high P-velocity and a substance with a low

P-velocity, the velocity value passing through the substance with the high P-velocity is firstly detected.

In the surveyed area, in the tunnel direction (T survey direction), the velocity values that passed through the whole geological structures were observed by the seismic prospection.

On the other hand, in the tunnel crossing direction (C survey line), the velocity values that passed through the geological structures with high-speed values were preferentially observed since the geographical distribution direction of this area and the survey line are nearly parallel.

The survey line where this phenomenon is obvious is the C2 survey line. On the C2 survey line, the quartzite layer (Q) forming the ridges is widely distributed. The quartzite layer (Q) is more resistant to weathering than the surrounding rocks, and seems to be in a more rigid state. Therefore, the P wave that passed through this quartzite layer (Q) was detected in the C2 survey line. For this reason, it is considered that a significant difference occurs in the elastic wave velocity boundary between the tunnel direction line and the tunnel transverse line.

iii) Civil engineering issues in tunnel planning

Based on the result of the seismic refraction prospecting, identified civil engineering issues in tunnel planning are shown below.

- In the pilot site, most of the tunnel section passes through the rock area of VP = 1.5 ~ 1.7 km / sec or below, and geological condition is quite weak.
- Because of the gentle hilly terrain and the thin soil coverage as well as above-mentioned geological conditions, there is a high risk that the tunnel construction will face difficulties.
- The geology of the quartzite layer and quartzite/feldspar gneiss layer distributed in the western half are likely to be weak, so it is necessary to select tunnel construction method carefully.
- Both tunnel portals have thin soil coverage. Moreover, weathering of rock mass is strongly observed. Therefore, there is a high possibility that tunnel drilling with auxiliary method will be forced at a considerable depth.
- The possibility of the occurrence of sudden spring water seems to be extremely small from the topographic observation.

3.3 Core Drilling

3.3.1 Methodology

The core drilling, together with standard penetration test (SPT) and water pressure test (Lugeon tests), were performed to obtain subsurface conditions of the proposed tunnel alignment and to provide geotechnical parameters for the design and construction plan of the tunnel structure.

The core drilling of 110 meters in total length was performed at three locations along the proposed tunnel alignment. Two boreholes (BT-01, 03) were at the portal area and the other borehole (BT-02) at the top point of the tunnel alignment. The locations of the core drilling and number of the in-situ tests performed within each borehole are shown in Table 3.6 and Figure 3.13, respectively.

Table 3.6 Summary of Core Drilling Quantity and Relevant In-Situ Tests

No.	Borehole No.	Drilling Depth (m)	SPT (nos.)	Lugeon Test (nos.)
1	BT-01	30.0	14	2
2	BT-02	50.0	6	5
3	BT-03	30.0	7	2
Total		110.0	27	9

(Source: Project team)

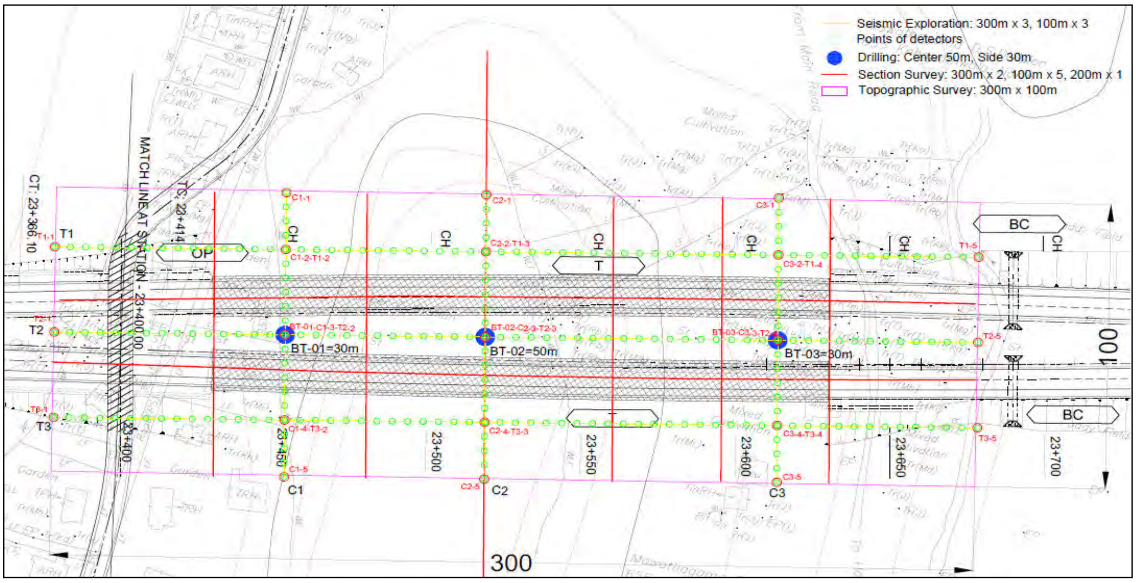


Figure 3.13 Location of the Core Drillings

(Source: Project team)

The core drilling work was performed in accordance with ASTM D2113 standard procedure and the specifications for this project, using a conventional rotary drilling machine equipped with single tube core barrel. The diameter of borehole was 90 mm and 76 mm for the overburden soil and weathered rock respectively, with a core diameter of 54.7 mm.

The SPTs were conducted in accordance with BS1377-9 every 1.0 meter depth. The tests were carried out mainly in the overburden soils and its underlying completely weathered rocks. The purpose is to roughly estimate their index and engineering properties through empirical correlations and to take disturbed soil sample for laboratory tests.

Water pressure tests (packer or Lugeon tests) were conducted in accordance with BS1377-9. The tests were performed in bedrocks or weathered rocks to measure the permeability of the rocks surrounding the tunnel. The test results were presented in section 3.4.

3.3.2 Results

In addition to the above-mentioned core drillings, a review of local geological map and site reconnaissance were performed. These results of the investigation together with core drillings were presented below.

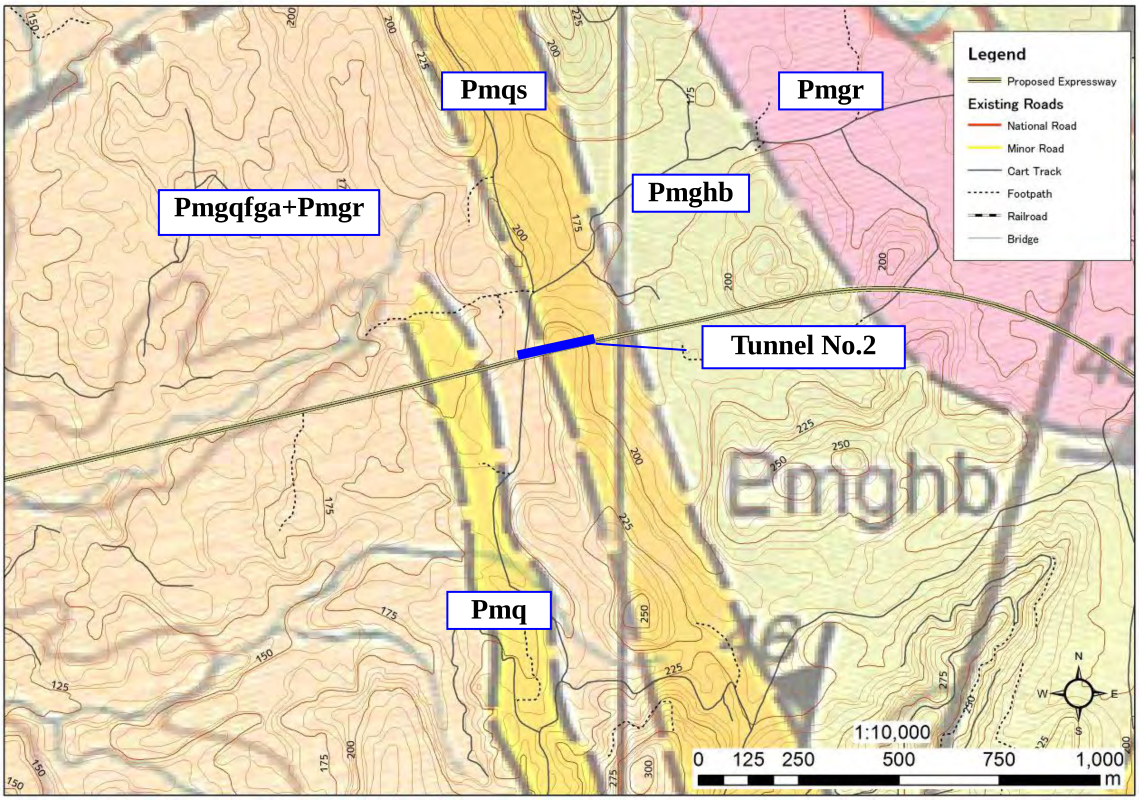
(1) Local Geology and Site Conditions

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 3.14 and summarized in Table 3.7. According to GSMB (1996), the selected Tunnel No. 2 is crossing quartzite and quartz-feldspar gneiss of the Proterozoic metamorphic rocks.

Table 3.7 Stratigraphic Units of the Project Site

Geological Unit		Geological Description
Wanni Complex	Pmgr	Granite gneiss: massive leucocratic quartzofeldspathic gneiss, quartz>20%, few mafics.
	Pmghb	Hornblende-biotite gneiss: massive to compositionally layered grey gneiss with quartz>20%, plagioclase and garnet<10%.
Highland Complex	Pmq	Quartzites: pure coarse-grained ridge-forming quartzites locally with <50% each of sillimanite, kaolinized feldspar or biotite.
	Pmqs	Impure quartzites and quartz schists: with sillimanite, garnet, often interlayered with biotite-bearing quartz-rich quartzofeldspathic gneiss
	Pmgqfga	Garnetiferousquartzofeldsparthic gneiss (formerly garnet granulite): leucocratic quartz-feldspar gneiss with abundant pink garnets, often>20%, weathers to iron-rich residual deposits.

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



Note: Geological description and symbols are the same as those presented in Table 3.7 above

Figure 3.14 Site Geology Together with Tunnel Alignment

(Source: Modified after Geological Survey of Sri Lanka)

Some outcrops of quartzite are observed around the project site. These exposures show a closely well-developed foliation within the rock mass that coincides with the plane of joint (Figure 3.15). The foliation joints generally dip toward east with a dip angle of 10 to 30 degrees. In addition, the foliation joints show some opening (Figure 3.15(b)) at exposures presumably due to stress relief.

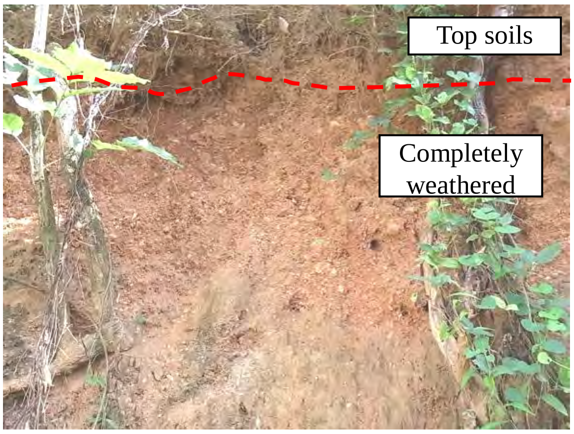
(a) Closely foliated quartzite

(b) Quartzite with foliation open joints

Figure 3.15 Photographs Showing Fracturing of the Quartzite

(Source: Project team)

The bedrocks are completely weathered in outcrops in general. Field observation on cut exposures indicates that the bedrocks at the present ground surface are completely decomposed into sandy gravels and residual soils. The soils are very dense and can vertically stand at natural condition (Figure 3.16). On the other hand, the soils are generally susceptible to erosion by surface water because of its composition of sand and gravel.



(a) Weathering Zone Observed in Cut Slope

(b) Close View of Completely Weathered Rock

Figure 3.16 Photographs Showing Surface Weathering of the Bedrocks

(Source: Project team)

A thrust is inferred to pass through the planned tunnel alignment, which means that the underlying bedrocks along the tunnel alignment have a high potential for fracturing, further indicating a possibility of over break 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 and suspected on the portal slopes and along the tunnel alignment from site reconnaissance. No spring water was observed around the tunnel alignment.

(2) Core Conditions

Geological logs and core photographs are given in Appendix 4, respectively. Log description was related mainly to lithological features, rock mass weathering (refer to Table 3.8) and discontinuity data, and summarized for each boreholes below:

BT-01:

1) Topsoil, Clayey Sand to Clayey Gravel, 0.0 to 4.0m thick, dark to brownish grey, medium dense, dry to moist, fine to medium coarse-grained gravels of quartz.
2) Residual soil of quartzite and quartz-feldspar gneiss, 4.0 to 14.2m thick, the bedrocks were completely decomposed into sandy soils, brownish grey to grey, generally medium dense.
3) Completely weathered rocks, 14.2 to 24.5m thick, locally containing moderately weathered rocks, foliated joints dominate the moderately weathered rocks.
4) Completely weathered rocks, 24.5 to 30.0m thick.

BT-02:

1) Topsoil, Clayey Sand/Gravel, 0.0 to 1.0m thick, brown to brownish grey, loose to medium dense, wet, fine to medium coarse grained gravels of quartz.
2) Residual soil, 1.0 to 8.5m thick, same as those observed in BT-01.
3) Completely weathered rocks, 8.5 to 26.5 meters thick, locally containing moderately weathered rocks. Core recovery was poor, and RQD cannot be measured.
4) Completely weathered rocks, 26.5 to 50.0m thick, brownish grey to brown. Core recovery was poor, and RQD cannot be measured.

BT-03:

1) Topsoil, 0.0 to 0.5m thick, geological characteristics were the same as those observed at BT-01.
2) Residual soil, 0.5 to 7.0m thick, same as those observed at BT-01.

3) Completely weathered rocks, 7.0 to 15.5m thick.
4) Completely weathered rocks, 15.5 to 30.0m thick, from moderately weathered rock joints along foliation were observable (Figure 3.17). The RQD is mainly between 0-10% and 70-100%, and mostly concentrated 0-10% (Figure 3.18).

Table 3.8 Representative Rock Mass Weathering Zone Identified

Weathering Zone	Core Observation	Core Photo
Residual soils	a) All rock material is decomposed to soils and deposited without movement. b) The mass structure is completely destroyed.	
Completely/highly weathered	a) More than half of the rock material is decomposed and/or disintegrated to a soil. b) Discoloured rock is present as a corestone.	
Moderately weathered	a) Discolouration indicates weathering of rock material and discontinuity surfaces. b) Rock material is partially discoloured by weathering.	
Completely/highly weathered	a) More than half of the rock material is decomposed and/or softened to a soil.	

(Source: Project team)

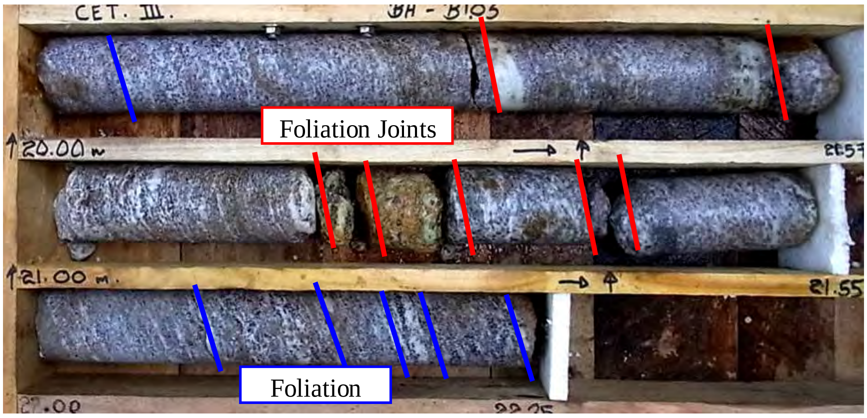


Figure 3.17 Photographs Showing Foliation Joints Formed within Quartz-Feldspar Gneiss

(Source: Project team)

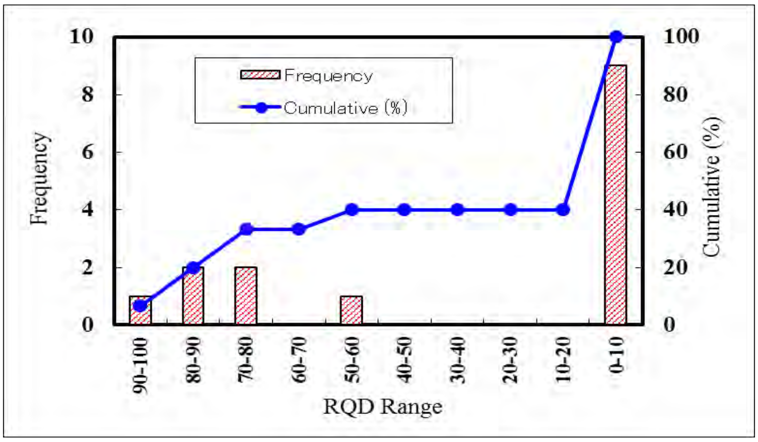


Figure 3.18 Histogram Distribution of the RQD Values in BT-03

(Source: Project team)

(3) SPT Results

SPTs were conducted in the overlying residual soils. The relationship between depth and N value for each borehole location is illustrated (Figure 3.19), and described as follows:

1) All the N values of the residual soil at BT-02 were more than 50 blows/30cm, indicating the residual soil was dense to very dense.
2) The N values of the residual soils at western portal (BT-01) and eastern portal (BT-03) generally increased with depth. The N values varied mostly from 12 to 30 blows/30cm and averaged approximately 27 blows/30cm above the ground level (7.0 m), and increased to more than 30 blows/30cm below the ground level (7.0 m).

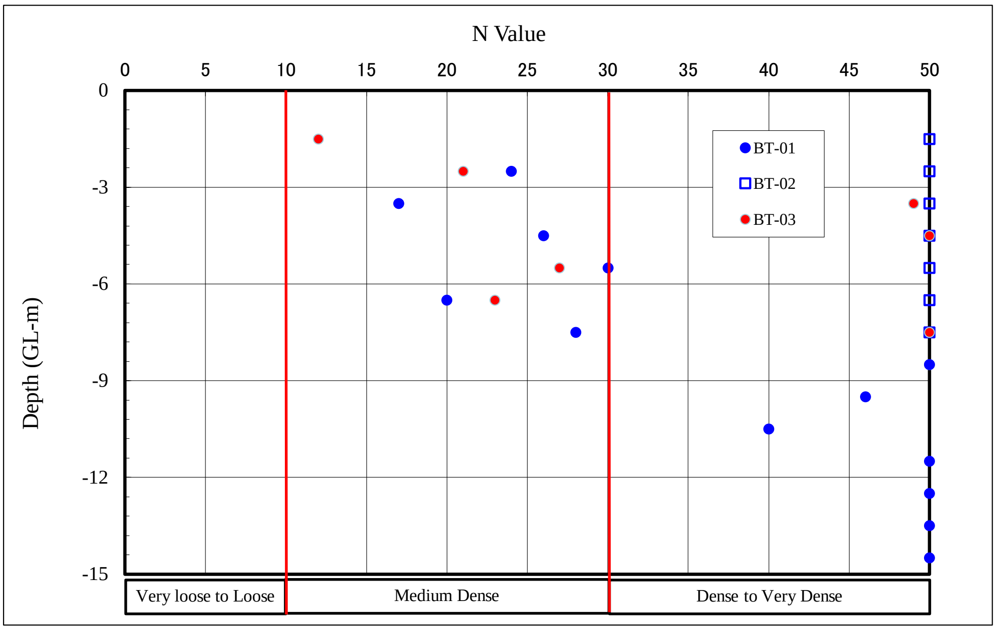


Figure 3.19 Photographs Showing Surface Weathering of the Bedrocks

(Source: Project team)

3.3.3 Evaluation and Discussion

(1) Geological Sections

The geological section along the tunnel alignment was prepared based mainly on rock type, rock mass weathering conditions, RQD and joint spacings. The prepared geological section is given in Appendix 4 and summarized below:

1) The bedrocks of the tunnel alignment is composed of quartz-feldspar gneiss and inter-bedded with quartzite. The major foliation joints are developed in both bedrocks; they generally dip toward the east with a dip angle of 10 to 30 degrees.
2) The quartzite is generally less weathered than the quartz-feldspar gneiss. The bedrocks below the topsoil are generally classified into three zones for engineering purpose, including a) Residual soil, b) Completely weathered rocks locally containing moderately weathered rocks, and c) Completely to moderately weathered rocks, as summarized in Table 3.9 and Table 3.10.
3) No fault was identified across the tunnel alignment from site investigation.

Table 3.9 Geotechnical Zonation at Drilled Borehole Locations

Geological/Geotechnical zone		Corresponding Depth (m) at Borehole		
		BT-01	BT-02	BT-03
1	Topsoil (TS)	0.0 -4.0	0.0 –1.0	0.0 – 0.5
2	Residual Soil (RS)	4.0 - 14.2	1.0 - 8.5	0.5- 7.0
3	Completely weathered, locally with moderately weathered rocks (CW)	14.2 -24.5	8.5 -26.5	7.0-15.5
4	Completely to moderately weathered rock (CM)	24.5 - 30.0	26.5 -50.0	15.5 - 30.0

(Source: Project team)

Table 3.10 Simplified Geological Logs

Zone	Geology/Rock Type and Its Depth at Each Borehole		
	BT-01	BT-02	BT-03
TS	Topsoil (0.0–4.0m)	Topsoil (0.0–1.0m)	Topsoil (0.0–0.5m)
RS	Residual soil (4.0–14.2m)	Residual soil (1.0–8.5m)	Residual soil (0.5–7.0m)
CW	Quartzite (14.2–20.0 m) Quartz/feldspar gneiss (20.0-24.5m)	Quartzite (8.5-13.5m) Quartz/feldspar gneiss (13.5-21.0m) Quartzite (21.0-26.5m)	Quartz/feldspar gneiss (7.0-15.5m)
CM	Quartzite (24.5-30.0m)	Quartzite (26.5-35.0m) Quartz/feldspar gneiss (35.0-50.0m)	Quartz/feldspar gneiss (15.5-27.3m) Quartzite (27.3-30.0m)

(Source: Project team)

(2) Rock Mass Classification

According to the recommended rock mass classification system described in Chapter 4: The geological and geotechnical conditions along the tunnel alignment can be classified into 3 classes (Table 3.11), as follows:

1) Class DII – It consists mainly of the residual soils. The topsoil is also included in the class.
2) Class DI – It includes the completely weathered rocks, locally with some moderately weathered rocks. The RQD is less than 20 blows/30cm.
3) Class CII – It includes completely to moderately weathered rocks. The RQD was mostly between 10 and 70 blows/30cm.

Table 3.11 Geotechnical Zonation and Corresponding Rock Class

Geotechnical Zonation			Rock Mass Classification
1	Topsoil (TS)	TS	DII Class
2	Residual Soil (RS)	RS	DII Class
3	Completely weathered, locally with moderately weathered rocks	CW	DI Class
4	Completely to moderately weathered rock (CM)	CM	CII Class

(Source: Project team)

(3) Stability of Tunnel Portal Slope

Both portals are located in the foot of small gentle slopes, about 15 to 20 degrees. No landslide and existing slope failure were identified around the portal areas in the topographic interpretation and at the site investigation.

The geology in the tunnel portal areas is expected to be residual soils – medium dense to very dense silty sands and gravels, which can keep stable even in sub-vertical cut slope at natural condition. However, the strength of the residual soils sharply changed at about 6 to 7 m below the present ground level (refer to Figure 3.19 above), the boundary of the strength variation is a geologically weak plane – a potential sliding surface. The foliation joint surface dips toward the east at degrees of 10 to 30, also contributing geologically to instability after excavation of the portal area, especially in the eastern portal area. In addition, the residual soils, consisting of sand and gravel, were susceptible to erosion by surface flow water.

Accordingly, cut slope protection and stabilization measures should be designed for permanent stability of portal areas after excavation. The standard slope gradients for the portal cut slope is recommended in Table 3.12 according to the Manual for Highway Earthworks (1990), as shown in Table 3.13 below.

Table 3.12 Recommended Geometric Standards for Cut Slopes of Portal Areas

Geological division	Residual Soil (RS)	Completely weathered (CW)	Completely/Moderately weathered (CM)
Cut gradient (V:H)	1:.1.2	1:1.0	1:0.8
Cut slope height (m)	5 to 7	5 to 10	5 to 10

Note: V:H = vertical (height): horizontal (length).

(Source: Modified from Highway Earthwork Series, Manual for Highway Earthworks, Published by Japan Road Association, March 1990)

Table 3.13 Geometric Standards for Cut Slopes

Character of soil or bedrock		Cut slope height(m)	Gradient (i=V:H)
Hard rock			1:0.3 ~ 1:0.8
Soft rock			1:0.5 ~ 1:1.2
Sand	Not dense and poorly graded	(1)	1:1.5 ~
Sandy soil	Dense and solid	Less than 5 m	1:0.8 ~ 1:1.0
		5~10 m	1:1.0 ~ 1:1.2
	Not dense	Less than 5 m	1:1.0 ~ 1:1.2
		5~10 m	1:1.2 ~ 1:1.5
Sandy soil mixed with gravels or rock masses	Dense and well graded	(2) Less than 10 m	1:0.8 ~ 1:1.0
		10~15 m	1:1.0 ~ 1:1.2
	Note dense or poorly graded	(3) Less than 10 m	1:1.0 ~ 1:1.2
		10~15 m	1:1.2 ~ 1:1.5
Clayey soil		Less than 10 m	1:0.8 ~ 1:1.2
Clayey soil mixed with rock or cobble stones		Less than 5 m	1:1.0 ~ 1:1.2
		5~10 m	1:1.2 ~ 1:1.5

(Source: Modified from Highway Earthwork Series, Manual for Highway Earthworks, Published by Japan Road Association, March 1990)

3.4 Water Pressure Testing

3.4.1 Methodology

The water pressure tests were carried out by the following five-stage method (refer to Figure 3.20 below) which is called as packer test or Lugeon test. Test was conducted for every 5m section, in the parts of boreholes through bedrock in order to evaluate the seepage potential of the bedrocks surrounding the tunnel. The test was conducted with a single packer as follows:

1) Wash the borehole with clean water. When drilling reaches the bottom of a tested section in the bedrock, the borehole shall be washed by flushing clean water through the inserted drill rod to the bottom of the borehole until the water emerging from the top of the borehole becomes clear.
2) Install a packer at the top of the test section and expand the packer rubber with air pressure. Next, inflates 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) Inject water into the borehole under a given pressure. Water is injected into the borehole below the packer under a given pressure (refer to Table 3.14 below) – the volume of injected water at each pressure level should be measured till at least 15 minutes passed or the flows get stabilized.
4) Record elapsed time and volume of water injected. The injected volume of water at each pressure is measured at least 3 times of five-minute intervals. 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.
5) Test at five pressures. The water injection pressure is increased in stages. After maximum injection pressure stage is completed, the water pressure is decreased stepwise. Each test consists of measurements of water inflow at five pressures (refer to Table 3.14 below) measured at the surface pressure and flow gauge.

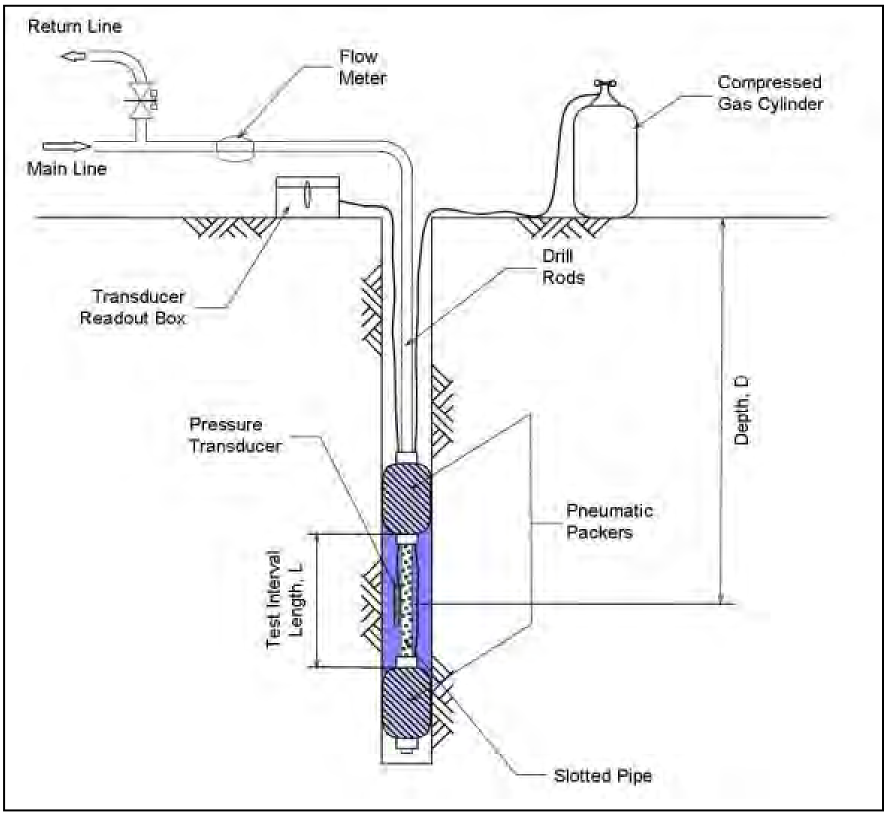


Figure 3.20 Lugeon Test Schematic

(Source: Quiñones-Rozo, 2010)

Table 3.14 Pressures Used for Each Test Stage

Stage	Description	Pressure	
1st	Low	0.50 Pmax	1.50 bar
2nd	Medium	0.75 Pmax	2.25 bar
3rd	Maximum (Peak)	1.00 Pmax	3.00 bar
4th	Medium	0.75 Pmax	2.25 bar
5th	Low	0.50 Pmax	1.50 bar

Note: 1 bar = 100 kPa.

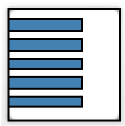
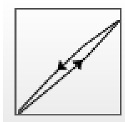
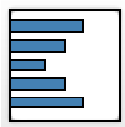
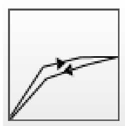
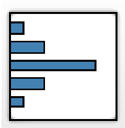
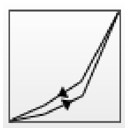
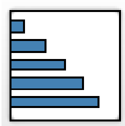
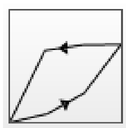
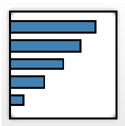
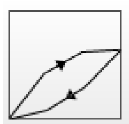
(Source: Project team)

The maximum test pressure (Pmax), shown in table above was calculated to 300 kPa assuming the minimum ground coverage was 15 m.

3.4.2 Results

The Lugeon values were obtained through the interpretation of injected water volume and pressure pattern (refer to Table 3.15). All of the measured Lugeon values were less than 5 (summarized in Table 3.16), and corresponding to 5 x 10^{-7} m/sec, water coefficient of permeability.

Table 3.15 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, relating 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)

Table 3.16 Summary of Lugeon Test results

Borehole No.	Test Depth (m)	Lugeon Value	Geology
BT-01	20.0 – 25.0	0.0	Completely weathered rock
	25.0 – 30.0	1.1	Completely/moderately weathered rocks
BT-02	20.0 – 25.0	0.0	Completely weathered rocks
	30.0 – 35.0	0.1	Completely/moderately weathered rock
	35.0 – 40.0	0.0	Completely/moderately weathered rock
	40.0 – 45.0	0.0	Completely/moderately weathered rock
	45.0 – 50.0	0.0	Completely/moderately weathered rock
BT-03	20.0 – 25.0	2.5	Completely/moderately weathered rock
	25.0 – 30.0	1.0	Completely/moderately weathered rock

(Source: Project team)

3.4.3 Evaluation and Discussion

The coefficient of permeability of rock mass is generally controlled by rock discontinuities such as joints. Therefore, the Lugeon value represents not only the conductivity but also the rock jointing condition. Typical range of Lugeon values and the corresponding rock condition is indicated in Table 3.17 below.

Table 3.17 Indicative Rock Mass Permeability from Lugeon Value

Lugeon Range	Classification	Coefficient of permeability (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/voids

(Source: Quiñones-Rozo, 2010)

The all obtained Lugeon values are less than 5, indicating the major foliation joints developed in the rocks surrounding the tunnel are very tight, and the permeability is low. Because of the low permeability and the low groundwater level reaching the tunnel bottom, the water inflow into the tunnel during construction is likely limited.

3.5 Laboratory Testing (Soil and Rock)

3.5.1 Methodology

(1) Test Method

All laboratory tests were conducted in accordance with international standard test methods, mainly including the American Society for Test Materials (ASTM) standard methods and British Standards (BS) test methods. Table 3.18 summarizes the tests performed and methods used.

Table 3.18 Summary of the Item and Quantity of Laboratory Tests Performed

Laboratory Test Item		Standard/Description	Unit	Quantity
Soil sample	Specific gravity	BS1377-2	test	13
	Unit weight	BS1377-2	test	13
	Grain size analysis	BS1377-2	test	13
	Atterberg Limit	BS1377-2	test	5
	Unconfined compressive strength	BS1377-7	test	0
	Direct shear strength	BS1377-7	test	0
Rock core sample	Unit weight (dry)	ASTM D2216	Test	10
	Water absorption/Specific gravity	ASTM C97	Test	5
	Point load test	ASTM D5731	Test	25
	Uniaxial compression strength	ASTM D2938	Test	10
	Petrographic analysis	BS812-2 [149]	test	2

(Source: Project team)

The laboratory testing on soil and rock samples was conducted in the Laboratory of ELS (Engineering & Laboratory Services (Pvt) Ltd) except for thin section petrographic analysis which was carried out by the Department of Earth Resources Engineering, University of Moratuwa.

In addition, the planned mechanical properties tests of soil samples were cancelled due to unavailability of undisturbed soil samples.

(2) Sampling Method

Samplings of disturbed soils were done with SPT sampler, mainly from the residual soils and completely weathered rocks.

Rock materials for laboratory testing were taken from rock cores with a core dimeter of 54.7 mm. In addition, the unconfined compressive strength was basically determined on a specimen with a length-to-diameter ratio (L/D) of more than 2. These rock cores retrieved for laboratory tests were either slightly weathered or moderately weathered.

3.5.2 Results

(1) Specific Gravity of Soils

The specific gravity of the residual soils varied from 2.61 to 2.74 averaging 2.67. This index property indicates normal soil.

Table 3.19 Summary of Specific Gravity Data for Soil Samples

Specific Gravity Test Result	
Test count	13
Maximum value	2.74
Minimum value	2.61
Mean value	2.67
Median value	2.68

(Source: Project team)

(2) Grain Size Analysis

Figure 3.21 shows the representative grading curves for the residual soil at Boreholes BT-01 to BT-03. The residual soils, in its natural form, are classified mostly as Silty Sands and Gravels by the British Standard BS 1377 Part II. The fine fraction (smaller than 0.06mm in diameter) was less than 25%. The gravels were fine- to medium coarse-grained in size and accounted for 10 - 50%.

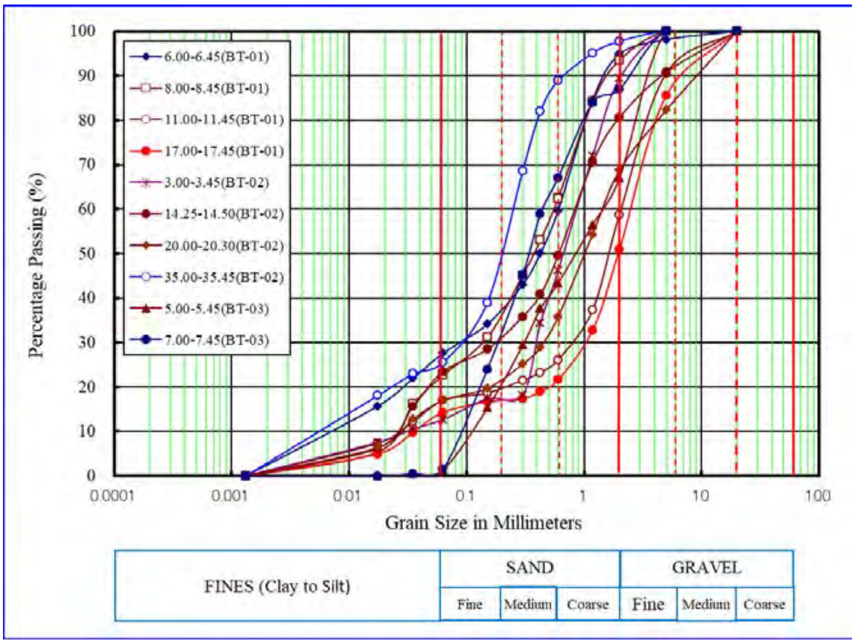


Figure 3.21 Grain Size Distribution Curve for Representative Soil Samples

(Source: Project team)

(3) Specific Gravity of Intact Rock

The specific gravity of different rocks depends on the minerals present and their relative percentage of mineralogical composition. In general, common rocks have an average value of specific gravity about 2.70.

The specific gravity data are summarized in the following table for the bedrocks within the project site. The obtained specific gravity varied from 2.64 to 2.90, and averaged 2.74, closely corresponding to the general average of 2.70 as mentioned above.

Table 3.20 Summary of Specific Gravity Data for Intact Rocks

Specific Gravity Data	
Test count	4
Maximum value	2.90
Minimum value	2.64
Mean value	2.74
Median value	2.74

(Source: Project team)

(4) Intact Rock Density (or Unit Weight)

Rock densities are necessary to estimate overburden stress for the design of the tunnels. The intact rock density obtained from the test was between 2.7 and 3.0 g/cm^3, as shown in the following table.

Table 3.21 Summary of Density Data for Intact Rocks

Intact Rock Density (g/cm^3)	
Test count	10
Maximum value	3.04
Minimum value	2.72
Mean value	2.91
Median value	2.95

Note: 1 g/cm^3 = 10 kN/m^3. (Source: Project team)

(5) Water Absorption of Intact Rock

The values of water absorption show some scatter and deviations, as shown in Table 3.22. The test procedure is generally not sensitive to mistakes and failure, and the scatter and deviations are probably due to the weathering and alteration of collected core samples.

Table 3.22 Summary of Water Absorption Data for Intact Rocks

Intact Rock Water Absorption (%)	
Test count	4
Maximum value	0.86
Minimum value	0.25
Mean value	0.44
Median value	0.38

(Source: Project team)

(6) Unconfined Compressive Strength of Intact Rock

The test results from the Uniaxial Compressive Strength (UCS) compressive tests were highly variable, generally between 23 and 66 MPa, presumably due to varying weathering degrees of the rock specimen and show no consistent trend between strength and depth. The UCS of the intact rock cores are summarized in Table 3.23.

Table 3.23 Summary of Unconfined Compressive Strength for Intact Rocks

Intact Rock UCS (MPa)	
Test count	10
Maximum value	66.1
Minimum value	23.0
Mean value	46.5
Median value	48.8

(Source: Project team)

(7) Point Load Strength Index ($I_{s(50)}$)

The proportional coefficient of the point load strength index ($I_{s(50)}$) and the UCS is usually expressed in terms of an equivalent factor: ISRM standard and literatures usually quote the value of 24. The data obtained gives a factor of 19 that is considerably lower for intact rock cores (Figure 3.22). Further, correlation factors are undefined. The poor correlation is presumably due to different shapes and size of point load test samples.

Table 3.24 Summary of Point Load Strength Index Obtained for Intact Rocks

Intact Rock $I_{s(50)}$ (MPa)		Estimated UCS (MPa)
Test count	12	None
Maximum value	3.06	73.4
Minimum value	0.31	7.4
Mean value	1.69	40.6
Median value	1.65	39.6

Note: Estimated UCS = 24 x $I_{s(50)}$. (Source: Project team)

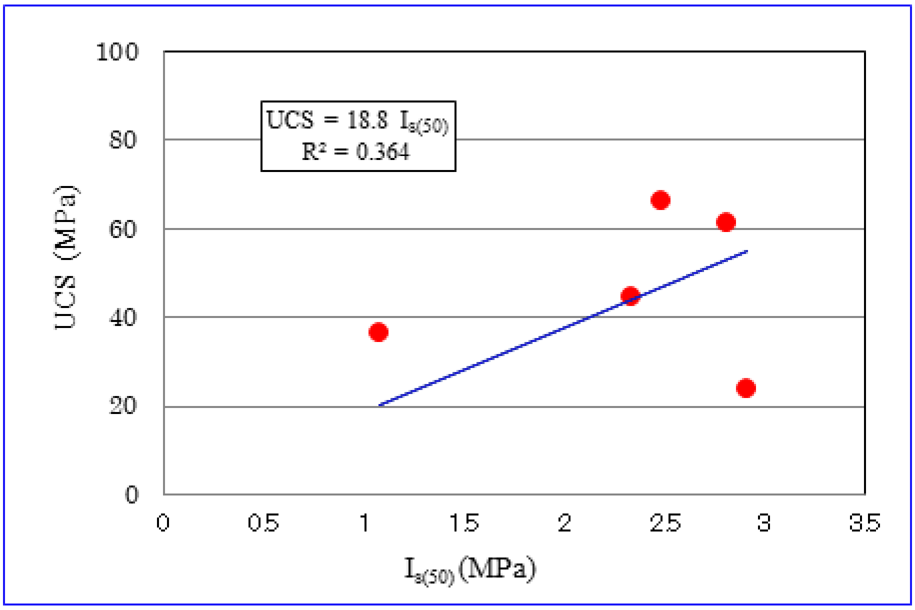


Figure 3.22 Plot of $I_{s(50)}$ vs. UCS for Intact Rocks

(Source: Project team)

(8) Thin Section Petrographic Analysis

Two thin section samples from rock cores were prepared for petrographic analysis. The results of the thin section petrographic analysis identifies that the bedrocks within the project site are coarse-grained high-grade metamorphic rocks – quartzite or pure quartzite with more than 95% quartz, as shown in Table 3.25.

Table 3.25 Mineralogical Composition of Project Site Bedrocks

Sample No.	Mineral	Composition (%)	Remarks
BT-01 (15.58-15.65)	Quartz	94 to 96	Major mineral, felsic
	Feldspar+Clay	4 to 6	Minor mineral, felsic
	Magnetite	< 1	
BT-02 (28.00-28.08)	Quartz	97 to 99	Major mineral, felsic
	Feldspar	1	Minor mineral
	Magnetite	None	Accessory mineral

(Source: Project team)

3.5.3 Evaluation and Discussion

Geotechnical design parameters of soils and rocks were evaluated based on the laboratory test results, core drillings and in-situ test results and summarized in this section.

(1) Geotechnical Properties of Residual Soils for Portal Slope Design

As stated before, the portal area is underlain by the residual soils - medium dense to dense clayey sands with an average N value of 27 blows/30cm.

Because no laboratory mechanical property tests were conducted, the shear strength of the residual soil can be estimated from the following empirical relationship with N value and/or from the conventionally adopted empirical table of design parameter (Table 3.26 and Table 3.27)

For CLAYEY soil   cu = (6 to 10) N   (kN/m^2)

For SANDY soil $\phi = \sqrt{12N} + 15 \leq 45$ (degree)

Coefficient of static soil pressure, $Ko = 1 - \sin \phi$

Table 3.26 Recommended Internal Friction Angle and Cohesion

Soil Type	Soil Conditions	Unit Weight (kN/m^3)	Internal friction angle (Degrees)	Cohesion (kN/m^2)	Group Symbol
Gravel	Dense and well graded	20	40	-	GW, GP
	Not dense, poorly graded	18	35	-	
Gravelly sand	Dense	21	40	-	GW, GP
	Not dense	19	35	-	
Sand	Dense or well graded	20	35	-	SW, SP
	Not dense or poorly graded	18	30	-	
Sandy soil	Dense	19	30	-	SM, SC
	Not dense	17	25	-	
Clayey soil	Firm	18	25	Less than 50	ML, CL
	Slightly soft	17	20	Less than 30	
	Soft	16	15	Less than 15	
Clay and silt	Firm	17	20	Less than 50	CH, MH, ML
	Slightly soft	16	15	Less than 30	
	Soft	14	10	Less than 15	

Note: Group symbols are the same as those used in the Unified Soil Classification System.

(Source: Modification from Design Guide－Earthworks, Published by Japan Highway Public Corporation, May 1998)

Table 3.27 Empirical Estimation of Allowable Bearing Capacity

Ground Type		Allowable bearing capacity(kN/m^2)	N-value
Gravelly Soil	Very dense	600	—
	Dense	300	—
Sandy Soil	Very dense	300	30 - 50
	Dense	200	20 - 30
	Medium dense	100	10 - 20
	Loose	50	5 - 10
	Very loose	0	< 5
Clayey Soil	Very firm or stiff	200	15 - 30
	Firm	100	8 - 15
	Slightly firm	50	4 - 8
	Soft	20	2 - 4
	Very soft	0	0 - 2

(Source: Manual for Highway Earthworks (1990), Published by Japan Road Association)

Table 3.28 gives the recommended geotechnical properties for the residual soil for the design of portal structures based on laboratory and field test results, empirical estimation as well as engineering judgments.

Table 3.28 Recommended Geotechnical Parameters for the Residual Soil

Geotechnical Properties		Recommended value	Remarks
Physical	Specific gravity, Gs	2.68	Table 3.19
	Unit weight, γ_t (kN/m^3)	20	Table 3.25
Mechanical	Cohesion, c (kN/m^2)	0	Table 3.25
	Internal friction angle, φ	35	Table 3.25
Allowable bearing capacity, (kN/m^2)		300	Table 3.26
Coefficient of static soil pressure,*Ko*		0.4	Equation in the previous page (section 3.5.3)

(Source: Project team)

(2) Index Properties of Completely Weathered Rock

Intact rock bulk density was obtained through laboratory tests and given in Table 3.21 above. The following design value is recommended for the design of the proposed tunnel:

Table 3.29 Recommended Index Properties for Bedrocks

Geotechnical Properties		Test results	Proposed value
Slightly weathered	Specific gravity, Gs	2.74	2.70
	Unit weight, γ_t (kN/m^3)	2.91	2.90
Completely weathered	Specific gravity, Gs	-	-
	Unit weight, γ_t (kN/m^3)	-	21

(Source: Project team)

(3) Unconfined Compressive Strength of Completely Weathered Rock

The intact rock strength parameter is determined for each core run section or each rock type by using the mean unconfined compressive strength (UCS) data from different sources, such as unconfined compression and point load on intact rock (fresh and slightly weathered).

The strength of the completely weathered rocks of which test data was not available, and it was estimated according to the following table and relation:

Table 3.30 Classification of Rock Weathering and Suggested Alternation Factor (fw)

Grade	Term	Description	Rating (f_w)
I	Un-weathered	Fresh: No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.	1
II	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.	1.75
III	Moderately weathered	Less than half 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.	2.5
IV	Highly weathered	More than half the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present as a discontinuous framework or as corestones.	10
V	Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	-
VI	Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	-

(Source: ISRM, 1978)

For completely weathered rocks, the adopted values (shown in Table 3.31), are derived from the following equation:

$$\sigma_c(UCS) = \frac{\sigma_{cFresh}}{f_w} = k_{50} \times \frac{I_{S50Fresh}}{f_w} = 50/10 = 5 \text{ MPa}$$

Table 3.31 Recommended Unconfined Compression Strength for Bedrocks

Rock condition	Test results (MPa)	Proposed value (MPa)
Slightly weathered rocks (Intact core)	48.8	50
Completely weathered rocks	-	5

(Source: Project team)

(4) Mechanical Properties of Completely Weathered Rock Mass

No in-situ mechanical property tests were done. The mechanical parameters of the weathered rock masses were roughly estimated based on relationship between rock mass classification and existing in-situ test results in Japan as reference, as shown in Table 3.32.

The completely weathered rock mass around the proposed tunnel alignment corresponded to rock classes III and IV in Table 3.32, and had seismic velocity of 1.5 to 1.7 km/sec. Accordingly, the mechanical parameters of rock masses were roughly estimated as follows:

1) Deformation modulus: 0.5 GPa
2) Internal friction angle: 40 degrees
3) Cohesion: 1.0 Mpa

Table 3.32 Rock Mass Classification and Rock Mass Parameters

Rock classification	Modulus of deformation (GPa)	Modulus of elasticity (GPa)	Shear strength		Seismic velocity(km/sec)
			Cohesion(MPa)	Internal friction angle (°)	
I	5.0or more	8.0 or more	4.0 or more	55-65	3.7 or more
II	2.0-5.0	4.0-8.0	2.0-4.0	40-55	3.0-3.7
III	0.5-2.0	1.5-4.0	1.0-2.0	30-45	1.5-3.0
IV, V	0.5 or less	1.5 or less	1.0 or less	15-38	1.5 or less

(Source: Modified after Kikuchi, et al., 1982)

Chapter 4: Rock Mass Classification System

Regarding the project outcome i), the draft rock mass classification system was developed (Activities 1.3, 5.1). Following sections are the contents of the rock mass classification system.

4.1 Procedure for Setting Parameters

The behaviour of intact rock material or blocks is continuous while that of highly fractured rock mass is discontinuous in nature. The engineering characteristics of rock material and discontinuities are taken into consideration for any engineering design in the rock mass.

Various important parameters have to be considered in order to describe a rock mass satisfactorily for assuring stability of the rock mass. The various important parameters used for description and classification of 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) groundwater pressure and the flow
(5) *in-situ* stress
(6) major geological structures (folds and faults)

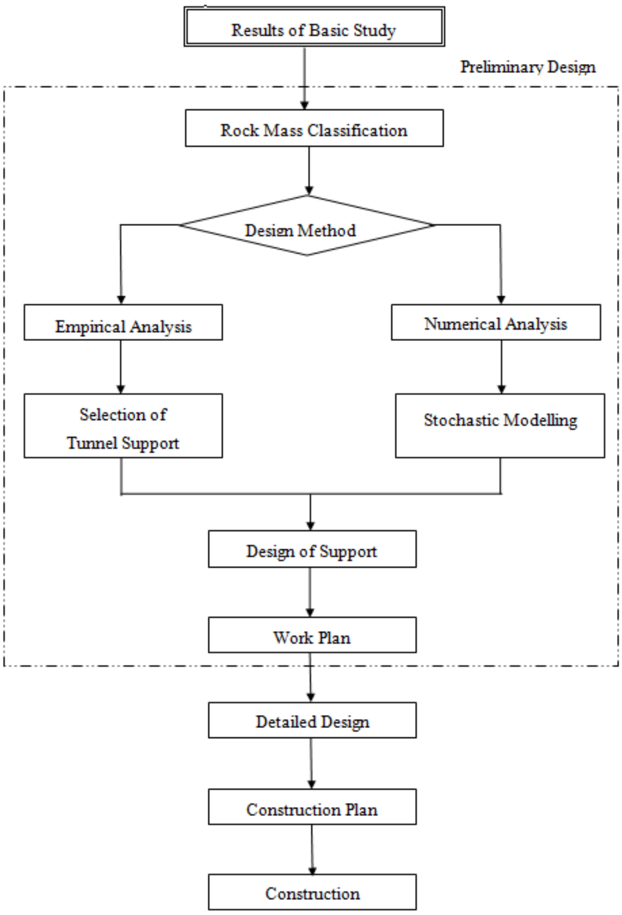


Figure 4.1 Analysis Method of Rock Mass Classification

(Source: Project team)

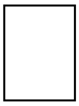
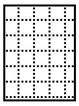
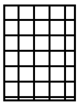
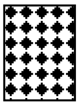
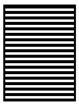
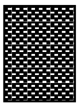
The parameters of rock mass classification system are obtained from geological and geotechnical investigations mentioned above and other studies during the tunneling work as shown in Table 4.1 and Table 4.2.

Parameters obtained as the result of investigations are utilized for rock mass classification. Design of tunnel support is made on the basis of rock mass classification in two major design approaches of empirical and numerical methods (See Figure 4.1).

4.1.1 Rock Grade

Rock grade indicates "the strength (or hardness) of rock" in this case. Rock mass classification is generally made on the basis of parameters of strength and condition of joints and others of the rock mass. The strength or hardness of the rock mass can be classified into categories as described in Table 4.1.

Table 4.1 Rock Grade of Rock Mass

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 remain 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. Cracky in general.	Moderately (Ratio of discolouration 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 discolouration 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 discoloured, 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)

(Source: Project team)

4.1.2 Condition of Joints and Cracks

Rock mass classification is generally made on the basis of parameters of condition of joints together with other parameters like strength of the rock mass. Condition of joints can be classified into categories as described in Table 4.2.

4.1.3 Rock Mass Classification

Rock mass is classified on the basis of parameters of rock grade (Table 4.1), condition of joints (Table 4.2). Table 4.3 shows the classification based on Table 4.1 and Table 4.2.

Table 4.2 Condition of Joints

Class	Judgement Criteria
I	> 50cm
II	30 –50cm
III	15–30cm
IV	5 –15cm
V	< 5cm

Class	Judgement Criteria
a	Closely adhered, no deterioration or dis-colouring
b	Filled with adhesion of limonite along cracks or very thin clay (brown in color)
c	Cracks are deteriorated and filled with about 1-2 cm clay (white-grayish white)
d	Open

(Source: Rock mass classification, Japan civil engineering society 1985)

Table 4.3 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	S1	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.

(Source: Project team)

4.1.4 Properties of Classified Rock Mass

Parameters of rock mass classification system are obtained as a result of geological and geotechnical investigations and other studies. Parameters of the rock mass are defined on the basis of properties of the rock mass. Properties of the classified rock mass withproposed values are shown in Table 4.4. These values can be applied for numerical analysis of the rock mass. Properties of the classified rock mass are subject to change when data accumulate in further stages of the project after discussion with RDA.

Table 4.4 Properties of each Classified Rock Mass

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

(Source: Project team)

4.2 Other Rock Mass Classification Systems

Several rock mass classification systems are utilized to build up characteristics of a rock mass and to provide properties of the rock mass and initial estimates of support requirements. RMR (Rock Mass Rating) and Q-system are most commonly utilized. These two systems are explained in the following sub-section.

4.2.1 RMR (Rock Mass Rating) Method

The RMR, published by Bieniawski in 1973, is one of the most commonly applied and performed method for classification of rock mass. Some changes have been made over the years since then and the 1976 and the 1989 versions of the classification system are mostly used. The equation of RMR is described as follows:

$$RMR = A1 + A2 + A3 + A4 + A5 + B$$

where

A1 = ratings for the uniaxial compressive strength of the rock material,

A2 = ratings for the RQD,

A3 = ratings for the spacing of joints,

A4 = ratings for the condition of joints,

A5 = ratings for the ground water conditions,and

B = ratings for the orientation of joints. (See Table 4.5.)

Table 4.5 Input Parameters Used in the RMR1989 Classification System

A. Classification parameters and their ratings in the RMR system

	PARAMETER		Range of values // RATINGS						
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range, uniaxial compr. strength is preferred		
		Uniaxial compressive strength	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
		RATING	**15**	**12**	**7**	**4**	**2**	**1**	**0**
2	Drill core quality RQD		90 - 100%	75 - 90%	50 - 75%	25 - 50%	< 25%		
		RATING	**20**	**17**	**13**	**8**	**5**		
3	Spacing of discontinuities		> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm		
		RATING	**20**	**15**	**10**	**8**	**5**		
4	Condition of discon-tinuities	a. Length, persistence	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m		
		Rating	**6**	**4**	**2**	**1**	**0**		
		b. Separation	none	< 0.1 mm	0.1 - 1 mm	1 - 5 mm	> 5 mm		
		Rating	**6**	**5**	**4**	**1**	**0**		
		c. Roughness	very rough	rough	slightly rough	smooth	slickensided		
		Rating	**6**	**5**	**3**	**1**	**0**		
		d. Infilling (gouge)	none	Hard filling		Soft filling			
			-	< 5 mm	> 5 mm	< 5 mm	> 5 mm		
		Rating	**6**	**4**	**2**	**2**	**0**		
		e. Weathering	unweathered	slightly w.	moderately w.	highly w.	decomposed		
		Rating	**6**	**5**	**3**	**1**	**0**		
5	Ground water	Inflow per 10 m tunnel length	none	< 10 litres/min	10 - 25 litres/min	25 - 125 litres/min	> 125 litres /min		
		p_w / σ1	0	0 - 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General conditions	completely dry	damp	wet	dripping	flowing		
		RATING	**15**	**10**	**7**	**4**	**0**		

p_w = joint water pressure; σ1 = major principal stress

B. RMR rating adjustment for discontinuity orientations

		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
RATINGS	Tunnels	**0**	**-2**	**-5**	**-10**	**-12**
	Foundations	**0**	**-2**	**-7**	**-15**	**-25**
	Slopes	**0**	**-5**	**-25**	**-50**	**-60**

C. Rock mass classes determined from total RMR ratings

Rating	100 - 81	80 - 61	60 - 41	40 - 21	< 20
Class No.	I	II	III	IV	V
Description	VERY GOOD	GOOD	FAIR	POOR	VERY POOR

D. Meaning of ground classes

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	< 45°	35 - 45°	25 - 35°	15 - 25°	< 15°

(Source: Guidance for Tunnelling work, Australia 2013)

The rock support can be estimated on the basis of the value of RMR in the actual excavation as shown in Table 4.6.

Table 4.6 The Tunnel Support Proposed on the Basis of the RMR1989 Classification System

Ground mass class	Excavation	Support		
		Rock bolts (20 mm diam., fully bonded)	Shotcrete	Steel sets
1.Very good rock RMR: 81-100	Full face: 3 m advance	Generally no support required except for occasional spot bolting		
2.Good rock RMR: 61-80	Full face: 1.0-1.5 m advance; Complete support 20 m from face	Locally bolts in crown, 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
3.Fair rock RMR: 41-60	Top heading and bench: 1.5-3 m advance in top heading; Commence support after each blast; Commence support 10 m from face	Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown	50-100 mm in crown, and 30 mm in sides	None
4.Poor rock RMR: 21-40	Top heading and bench: 1.0-1.5 m advance in top heading; Install support concurrently with excavation - 10 m from face	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh	100-150 mm in crown and 100 mm in sides	Light ribs spaced 1.5 m where required
5.Very poor rock RMR < 21	Multiple drifts: 0.5-1.5 m advance in top heading; Install support concurrently with excavation; shotcrete as soon as possible after blasting	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert	150-200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert

Note: Applies to tunnels with 10m span width

(Source: Guidance for Tunnelling work, Australia 2013)

Overstressing phenomena like rock bursting as well as squeezing, and weakness zone of faults appear to be unclear since there is no input parameter for rock stresses in the RMR system.

4.2.2 Q Classification System

Barton et al. (1974) of the Norwegian Geotechnical Institute proposed the Q system for estimating rock support in tunnels.

The Q value is defined by six parameters combined in the following equation:

$$Q = RQD/Jn \times Jr/Ja \times Jw/SRF$$

where

RQD = given as the value for this parameter,

Jn = ratings for the number of joint sets,

Jr = ratings for the joint roughness,

Ja = ratings for the joint alteration,

Jw = ratings for the joint or ground water, and

SRF = ratings for the rockmass stress situation. (See Table 4.7 and Table 4.8.)

Table 4.7 Input Parameters Used in the Q Classification System (1/2)

A. Rock quality designation (RQD)

Very poor	RQD = 0 - 25%
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

Notes:
(i) Where RQD is reported or measured as < 10 (including 0), a nominal value of 10 is used to evaluate Q
(ii) RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate

B. Classification with ratings for the Joint set number (Jn)

Massive, no or few joints	Jn = 0.5 - 1
One joint set	2
One joint set plus random	3
Two joint sets	4
Two joint sets plus random	6
Three joint sets	9
Three joint sets plus random	12
Four or more joint sets, heavily jointed, "sugar-cube", etc.	15
Crushed rock, earth-like	20

***Notes:** (i) For tunnel intersections, use (3.0 x Jn); (ii) For portals, use (2.0 x Jn)*

C. Classification with ratings for the Joint roughness number (Jr)

a) Rock-wall contact, b) rock-wall contact before 10 cm shear	
Discontinuous joints	Jr = 4
Rough or irregular, undulating	3
Smooth, undulating	2
Slickensided, undulating	1.5
Rough or irregular, planar	1.5
Smooth, planar	1.0
Slickensided, planar	0.5

***Note:** i) Descriptions refer to small scale features, and intermediate scale features, in that order*

c) No rock-wall contact when sheared	
Zone containing clay minerals thick enough to prevent rock-wall contact	Jr = 1.0
Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0

Notes:
i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
ii) Jr = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are oreintated for minimum strength

D. Classification with ratings for the Joint alteration number (Ja)

	JOINT WALL CHARACTER		Condition	*Wall contact*
Contact between joint walls	CLEAN JOINTS:	Healed or welded joints:	filling of quartz, epidote, etc.	Ja = 0,75
		Fresh joint walls:	no coating or filling, except from staining (rust)	1
	JOINTS WITH COATING OF:	Slightly altered joint walls:	non-softening mineral coatings, clay-free particles, etc.	2
		Friction materials:	sand, silt calcite, etc. (non-softening)	3
		Cohesive materials:	clay, chlorite, talc, etc. (softening)	4

	FILLING OF:	Type	*Wall contact before 10 cm shear*	*No wall contact when sheared*
Partly or no wall contact	Friction materials	sand, silt calcite, etc. (non-softening)	Ja = 4	Ja = 8
	Hard cohesive materials	compacted filling of clay, chlorite, talc, etc.	6	5 - 10
	Soft cohesive materials	medium to low overconsolidated clay, chlorite, talc, etc.	8	12
	Swelling clay materials	filling material exhibits swelling properties	8 - 12	13 - 20

E. Classification with ratings for the Joint water reduction factor (Jw)

Dry excavations or minor inflow, i.e. < 5 l/min locally	p_w < 1 kg/cm^2	Jw = 1
Medium inflow or pressure, occasional outwash of joint fillings	1 - 2.5	0.66
Large inflow or high pressure in competent rock with unfilled joints	2.5 - 10	0.5
Large inflow or high pressure, considerable outwash of joint fillings	2.5 - 10	0.3
Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2 - 0.1
Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1 - 0.05

***Note:** (i) The last four factors are crude estimates. Increase Jw if drainage measures are installed*
(ii) Special problems caused by ice formation are not considered

(Source: Forty years with Q-systems in Norway and abroad, Norway 2014)

Table 4.8 Input Parameters Used in the Q Classification System (2/2)

F. Classification with ratings for the Stress reduction factor (SRF)

Weakness zones intersecting excavation	Multiple weakness zones with clay or chemically disintegrated rock, very loose surrounding rock (any depth)				SRF = 10
	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation < 50 m)				5
	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)				2.5
	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)				7.5
	Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation < 50 m)				5
	Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation > 50 m)				2.5
	Loose, open joints, heavily jointed or "sugar-cube", etc. (any depth)				5
Note: *(i) Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence, but do not intersect the excavation*			σ_c / σ_1	σ_θ / σ_c	SRF
Competent rock, rock stress problems	Low stress, near surface, open joints		> 200	< 0.01	2.5
	Medium stress, favourable stress condition		200 - 10	0.01 - 0.3	1
	High stress, very tight structure. Usually favourable to stability, maybe except for walls		10 - 5	0.3 - 0.4	0.5 - 2
	Moderate slabbing after > 1 hour in massive rock		5 - 3	0.5 - 0.65	5 - 50
	Slabbing and rock burst after a few minutes in massive rock		3 - 2	0.65 - 1	50 - 200
	Heavy rock burst (strain burst) and immediate dynamic deformation in massive rock		< 2	> 1	200 - 400
Notes: *(ii) For strongly anisotropic stress field (if measured): when $5 < \sigma_1/\sigma_3 < 10$, reduce σ_c to $0.75\ \sigma_c$. When $\sigma 1/\sigma 3 > 10$, reduce σ_c to $0.5\sigma_c$* *(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for low stress cases*				σ_θ / σ_c	SRF
Squeezing rock	Plastic flow of incompetent rock under the influence of high pressure	Mild squeezing rock pressure		1 - 5	5 - 10
		Heavy squeezing rock pressure		> 5	10 - 20
Swelling rock	Chemical swelling activity depending on presence of water	Mild swelling rock pressure			5 - 10
		Heavy swelling rock pressure			10 - 15

(Source: Forty years with Q-systems in Norway and abroad, Norway 2014)

The Q-system is developed as an empirical design method for estimating rock support and the stability requirements to the tunnel or cavern. Excavation Support Ratio (ESR) of the Q value defines the rock support in a support chart as shown in Table 4.9.

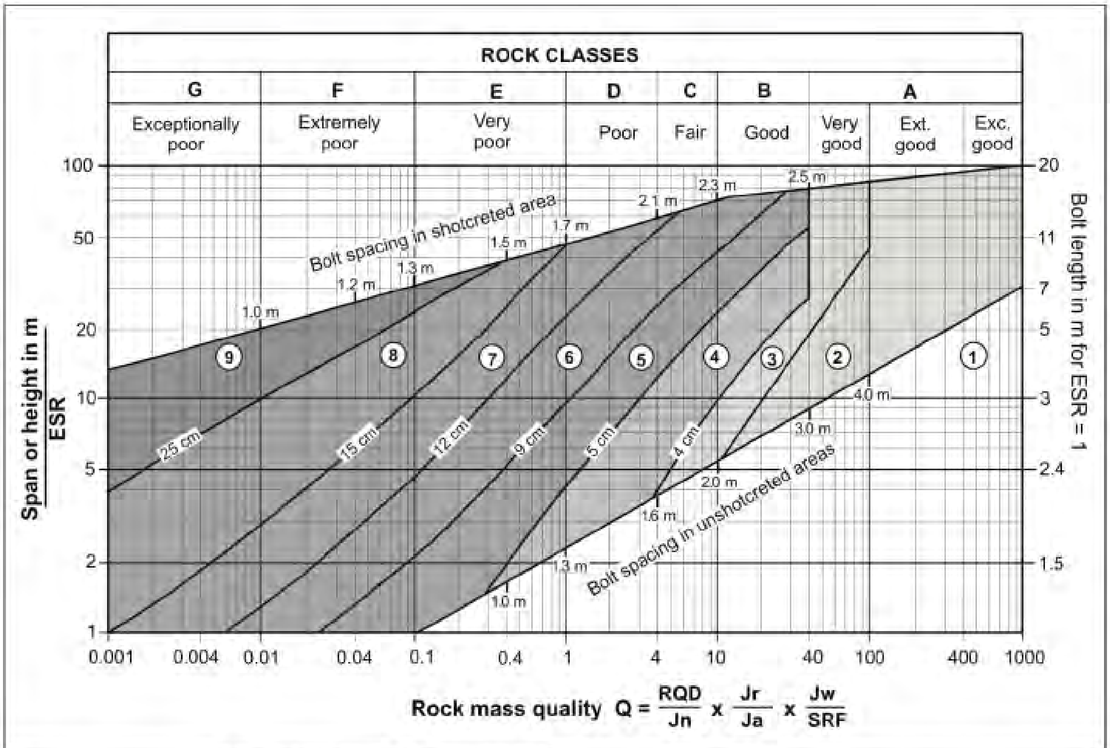
The Q system has several limitations, working best between Q = 0.1 and Q = 40 for tunnels with spans between 2.5m and 30m.

The Q system appears to be unclear in overstressing condition of rock mass. Though there are input parameters for overstressing, the system should be used with care in rock bursting, squeezing and swelling condition of the ground.

Table 4.9 Support Chart Used in the Q Classification System (1993)

The Excavation Support Ratio, ESR

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



REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting, (and unreinforced shotcrete, 4 - 10 cm)
5) Fibre reinforced shotcrete and bolting, 5 - 9 cm
6) Fibre reinforced shotcrete and bolting, 9- 12 cm
7) Fibre reinforced shotcrete and bolting, 12 - 15 cm
8) Fibre reinforced shotcrete, > 15 cm, reinforced ribs of shotcrete and bolting
9) Cast concrete lining

(Source: Forty years with Q-systems in Norway and abroad, Norway 2014)

4.2.3 Evaluation and Discussion

Rock mass classification systems often applied for tunnel project are RMR method and Q classification system; other systems are mainly formed on the basis of these two systems.

Comparisons between the systems of RMR and Q-system have been made on numerous occasions. These two systems show no discrepancies in good rock conditions. However, small discrepancies are indicated in poor rock conditions. A similar tendency of showing discrepancies between each system in poor rock conditions are commonly recognized in other countries.

The two systems have several common parameters. However, there are some differences as described below.

(1) Parameters of Rock Mass Classification System

- RMR system uses strength of intact rock
- Q system applies no direct rock property

(2) Design of Tunnel Support

- RMR system indicates support with 10m span from the table
- Q system indicates support with span or wall height on the basis of Q value (Quality of rock mass)

Both the rock mass classification system of RMR and Q-system cover almost any classified rock mass except weak and fragile rock mass zones. There is no consideration on rock bursting and squeezing due to over-stress caused by thick overburdened rock above the tunnel.

Quality of the same ground is calculated in the two different systems, and several correlations between the two different systems are confirmed in different geology of rock mass. Granitic gneiss is predominantly distributed on the island; therefore, the correlation of Diorite can be applied to Sri Lanka including the project area. (Refer to Figure 4.2; RMR = 9 ln Q+ 44)

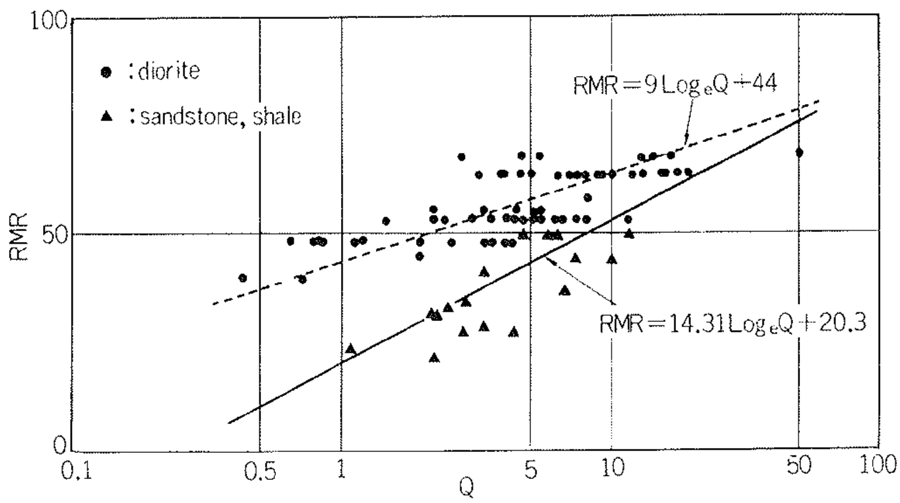


Figure 4.2 Correlation between RMR and Q-system (Diorite)

(Source: Correlation between RMR and Q system, Yoshinaka 1988)

4.3 Design of Tunnel Support

Design of the tunnel support is performed generally on the basis of classification of rock mass. Suitable tunnel support can be selected on the basis of geological and geotechnical conditions of the project area. The bedrock of the project area consists mainly of Precambrian metamorphic rocks of compact and hard intact condition. Deep weathering is often observed in the bedrocks of hilly and terrain areas. A detailed geological face mapping is carried out in the construction stage, and the geotechnical condition of the rock mass along the tunnel alignment can be identified with more details as the project progresses.

The tunnel support type is changed on the basis of classification of the rock mass; structural arrangement of the tunnel support is also modified as a result of the actual work performance of tunnel support. Structural models for computer analysis have been developed and can be used to design the structure of tunnel support considering the applied loads, the calculated loads from the outer surface of the structures and others.

4.3.1 Rock Mass Classification System and Tunnel Support

Correlation diagram of tunnel support and the classified rock mass of S1 to S7 is proposed in Table 4.10. This table is subject to change when the classification of rock mass is modified due to changes of actual condition of the site.

Table 4.10 Tunnel Support Type Proposed for Classified Rock Mass

Rock Mass Classifi -cation	Shot Creting		Rock Bolting			Steel Support	
	Thickness (cm)	Area	Length (m)	Lateral	Longi -tudinal	Material	Pitch (m)
S1	5	Arch	3	1.5	At random	---	---
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

(Source: Project team)

4.3.2 Evaluation and Discussion on Rock Mass Classification System and Tunnel Support

Suitable rock mass classification system can be modified on the basis of geological and geotechnical conditions of the project area. The geotechnical condition of the rock mass along the tunnel alignment will be confirmed when a detailed mapping of geological face is performed as the excavation work proceeds.

The rock mass classification system shall also be modified when detailed geological mapping of excavation face and other site investigations are carried out on the construction stage.

Suitable tunnel support can be selected on the basis of geological and geotechnical conditions of the project area. A detailed geological face mapping is carried out in the construction stage, and the geotechnical condition of the rock mass along the tunnel alignment can be identified with more detail as the project progresses.

The tunnel support type is changed on the basis of classification of the rock mass, and structural models for computer analysis are developed, which can be used to design the structure of tunnel support.

Suitable rock mass classification system is applied for every tunnel project, and it is modified on the basis of actual geological and geotechnical conditions of the project area.

Suitable tunnel support is also selected on the basis of geological and geotechnical conditions of the project area. The tunnel support is chiefly designed on the basis of the rock mass classification system which is proposed based on the geological and geotechnical conditions of the project area.

The tunnel support type is changed on the basis of classification of the rock mass; structural arrangement of the tunnel support is also modified as a result of the actual work performance of support.

These project-related data including rock mass classification and tunnel support shall be built up for future use.

The tunnel support system of “Tunnel No.2” was proposed considering the results of site investigations including the rock mass classification at drilling sites of BT-1, BT-2 and BT-3, and they are described in the Chapter 7. Table 4.11 shows the rock mass classification of the sampled rocks at three points of BT-1, BT-2 and BT-3 evaluated on the basis of this proposed rock mass classification system. However, some parameters regarding joints and cracks of the core sample appear to be difficult evaluation in some cases. The rock mass classification of these core samples was determined on the basis of near-by well recovered core samples.

Table 4.11 Rock Mass Classification of Each Borehole

Borehole No. BT-1		
Depth (m)	Rock Mass Classification	Combination of Parameters
4.00 ～ 14.20	S7	F- V - d
14.20 ～ 30.00	S6	D- IV - d

Borehole No.BT-2		
Depth (m)	Rock Mass Classification	Combination of Parameters
1.00 ～ 14.35	S7	F- V - d
14.35 ～ 26.50	S6	D- V - d
26.50 ～ 46.00	S5	D- IV - c
46.00 ～ 50.00	S4	C- IV - c

Borehole No.BT-3		
Depth (m)	Rock Mass Classification	Combination of Parameters
0.50 ～ 12.00	S7	F- V - d
12.00 ～ 15.50	S6	D- IV - d
15.50 ～ 19.00	S5	D- IV - c
19.00 ～ 22.40	S4	C- III - c
22.40 ～ 28.00	S6	D- V - d
28.00 ～ 30.00	S5	D- IV - c

(Source: Project team)

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"Rock Mass Classification System", Syed Huntazir Abbas, University of Peshawar

Chapter 5: Hydrological and Hydrogeological Investigation

The hydrological and hydrogeological investigations was conducted in the pilot site (Activities 7.3, 7.4). The processes of the actual investigation and its results were shared with C/P to promote the technical transfer. Details of the investigations are described in this Chapter.

5.1 Tunnel No.2 area

5.1.1 Hydrogeological Investigation

Tunnel No.2 penetrates the hill at the height of 215 m in east-west direction as shown in Figure 5.1. A stream flows in the north-south direction on the east side of the hill, and a rice field spreads in the east of the stream. There are north-south direction roads in both side of the hill; houses are scattered along the roads and north side of the hill. Residents of these houses use shallow hand dug wells for domestic use. The depths of the wells are 6 to 10 m in the west side of the hill and shallower than 5 m in the north and east side.

A seismic survey and its analysis were conducted in the Tunnel No.2 area. In addition, geological logs were obtained by drilling three boreholes. An aquifer test was also conducted using a borehole after the drilling. Besides, periodic measurements of the water level of the borehole dug by this project, household wells, the discharge of the streams and the springs were carried out. The locations of these measurements are shown in Figure 5.2.

Upon the measurements, an inventory listing of parameters such as coordinates, well elevation and the other indicators (see Figure 5.3) was created to share information and to continue the measurement easily and efficiently.

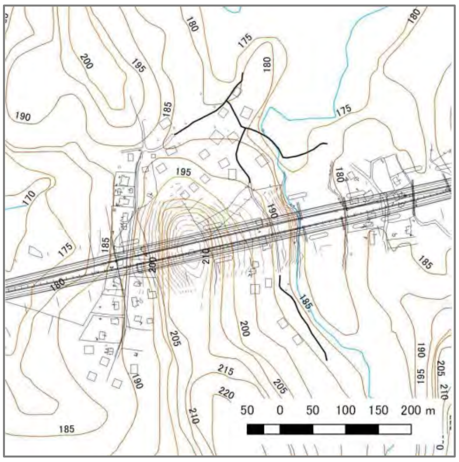


Figure 5.1 Topographic Map of Tunnel No.2 Area

(Source: Project team)

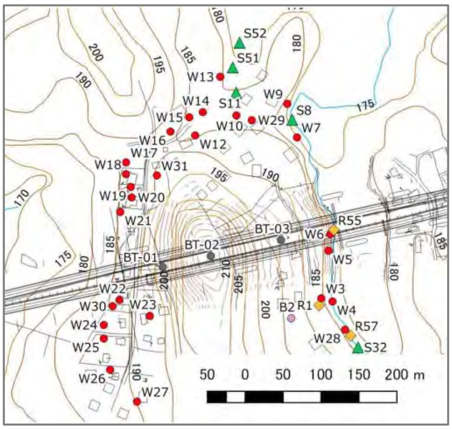


Figure 5.2 Location of Hydrogeological Observation Sites (Tunnel No.2, BT: borehole, R: river, S: spring, W:well)

(Source: Project team)

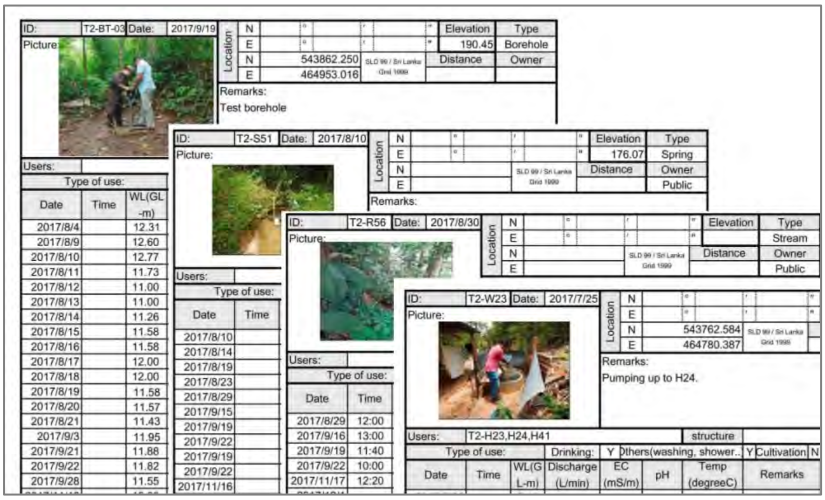


Figure 5.3 Example of Inventories of Observation Sites

(Source: Project team)

5.1.2 Present Groundwater Environment

(1) Results of Hydrogeological Measurement

The time series of the data measurement on the wells, streams and springs are shown in Figure 5.4. The discharge in streams and springs, which had little discharge in August, sharply increased from thereon, reaching the maximum in November and decreasing thereafter. The water levels of the wells show similar tendency in change.

In the well graph, the five points with high water levels are distinct. The solid lines of green and orange are the data of the wells in the valley in southwest side and the south east side of the hill respectively. Because the valley gathers water and outflows, the groundwater level in the valley is high. Three boreholes are indicated by the dotted lines. The water level in the three boreholes continue rising while the water levels of other wells start declining in November. The depths from the ground surface to the groundwater surface of these boreholes are deeper than other wells. It takes time for rain water to infiltrate and reach the groundwater surface, so it is considered that the influence of rainfall will be delayed in the boreholes.

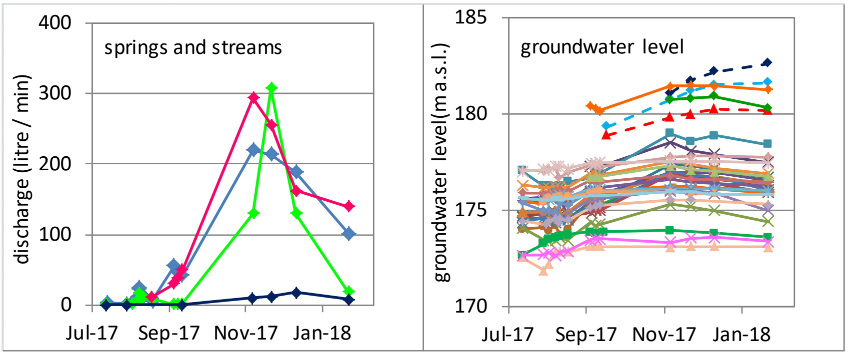


Figure 5.4　Time Series of Observation Data of Discharges of Springs and Streams and Water Level of Wells (Tunnel No.2)

(Source: Project team)

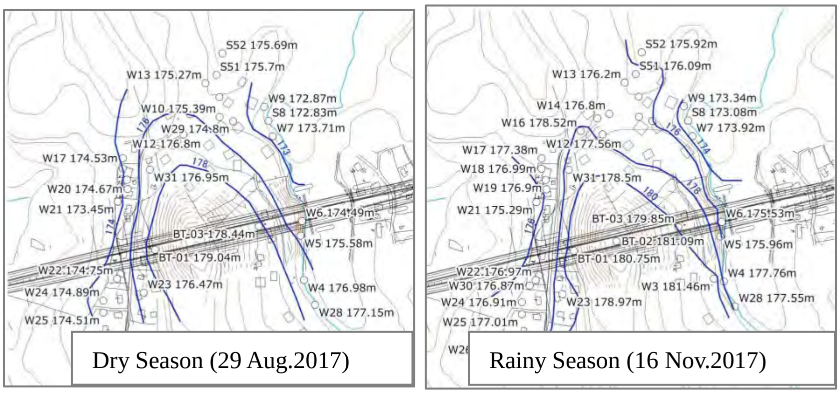


Figure 5.5　Measured Groundwater Levels and Their Contour Map in Dry and Rainy Season (Tunnel No.2)

(Source: Project team)

Groundwater levels in the dry and rainy seasons and groundwater contour lines with every 2 meters drawn based on the groundwater level are shown in Figure 5.5. The data on 19th November 2017 when the water levels of the wells other than the borehole are the highest is shown as the rainy season. The data on 29th August 2017 which recorded the lowest water level after observation started is shown as dry season. The groundwater contour line is almost along the ground elevation in both seasons of rainy and dry season. However, there is no contour line near the top of the hill and the groundwater level is almost constant there.

A longitudinal sectional view of the ground surface, tunnel and groundwater level in the tunnel direction is shown in Figure 5.6. The data on 29th August 2017 are used for the dry season. The data on 30th January 2018, when the water levels of boreholes recorded the highest, is used for rainy season. Although the position and height of the road have been fixed, the cross section of the tunnel has not been decided. Therefore, the bottom of the tunnel is assumed to be 2 m lower than the road surface. The height of the groundwater level on the tunnel bottom is 1.0 m in the dry season and 3.7 m in the rainy season at the highest position.

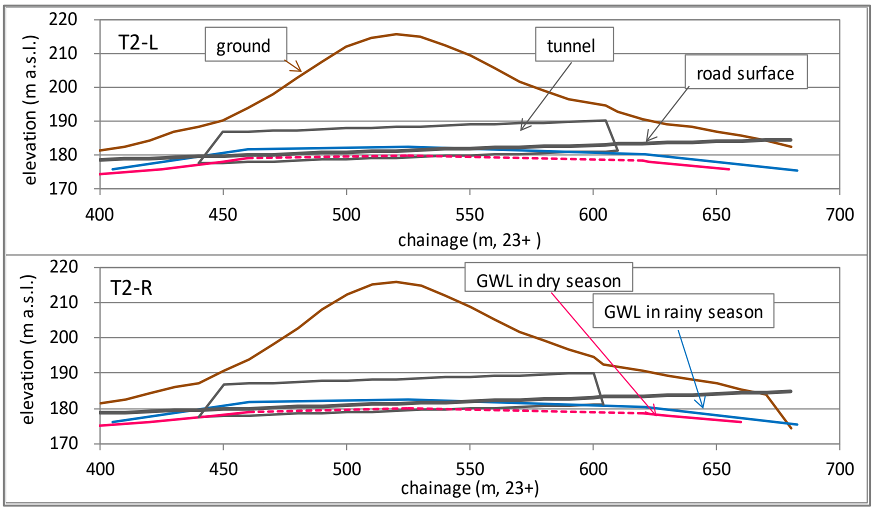


Figure 5.6 Longitudinal Section of Ground, Tunnel and Groundwater Levels of Dry and Rainy Season (Tunnel No.2)

(Source: Project team)

(2) Evaluation of the results of hydrogeological measurement

The observation period is short, from August 2017 to January 2018. However, using long-term rainfall and discharge data, it shows that the observation period was sufficient as it covers the dry season and the rainy season, not in the wet dry season or the dry rainy season. The rainfall data at Kurunegala and the hydrologic data at Holombuwa were used. Kurunegala is the closest meteorological station that can provide long-term data. Holombuwa is a hydrological station near the study area. It is considered to represent resemble hydrological environment of the study area because it is located close to the mountain with a small catchment area.

First, it is indicated that the measurement covered both the rainy and dry season. The average monthly rainfall from 2008 to 2017 and the monthly rainfall of 2017 at Kurunegala are shown in Figure 5.7. The average monthly specific discharge from 2008 to 2017 and the monthly specific discharge of 2017 at Holombuwa are shown in Figure 5.8.

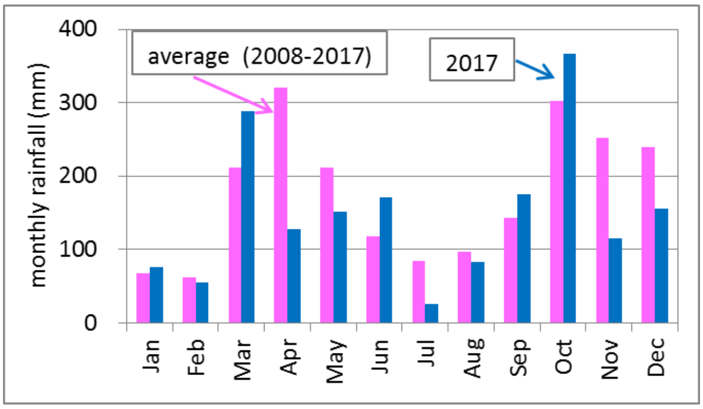


Figure 5.7 Monthly Average Rainfall (2008-2017) and Monthly Rainfall of 2017 at Kurunegala

(Source: Project team)

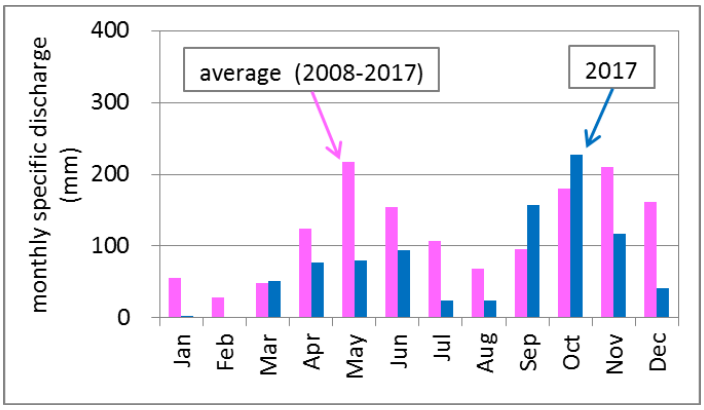


Figure 5.8 Monthly Average Specific Discharge (2008-2017) and Monthly Specific Discharge of 2017 at Holombuwa

(Source: Project team)

The graphs of rainfall and specific discharge show that the measurement period of August to January covers one of the dry and rainy season cycle which occurs two times in a year. It covers the rainy season, centered in October-November, which has more rain with higher water level than the rainy season centered in March and May. Regarding the dry season, there is a possibility that the water level in January to February may be lower than in July to August, and the measurement may not cover the lowest water level in a year. However, the difference in rainfall and specific discharge between July to August and January to February is small and is almost equivalent. Therefore, it is considered that the measurement period covers the rainy and the dry season cycle.

Next, it is shown that the measurement was conducted not in a dry rainy period or a wet dry period. The rainfall in June and August 2017 and the rainfall in September to December 2017 are compared with average rainfall in the same months of 2008 to 2017. The specific discharge at

Holombuwa is also compared in the same manner. The results are shown in Table 5.1. The value of % shows the percentage of the amount in 2017 to the average of 2008 to 2017. Excluding the specific discharge at Holombuwa in June to August 84 to 94%, which is slightly lower than the average value, is almost an average value. The specific discharge at Holombuwa in dry season from June to August 2017 is as small as 43% compared with the average, the measurement was conducted in drier period. Therefore, it was shown that the measurement was conducted in the appropriate period.

Table 5.1 Percentage of Rainfall and Specific Discharge in the Dry Season and Rainy Season during the Measurement Period to the Long-term Average

	June to August			September to December		
	Average (2008-2017)	2017	%	Average (2008-2017)	2017	%
Rainfall at Kurunegala	299 mm	281 mm	94 %	938 mm	814 mm	87 %
Specific discharge at Holombuwa	329 mm	142 mm	43 %	646 mm	543 mm	84 %

(The value of % shows the percentage of the amount in 2017 to the average of 2008 to 2017)

(Source: Project team)

5.1.3 Forecast of Groundwater Environment after Tunneling Work

The water level drawdown range is calculated by simple calculation method based on the hydraulic formula to forecast the groundwater environment after tunneling work. The target point is Borehole No.1 where the water level was the highest above the tunnel bottom. Bear's hydraulic formula was used for the calculation. The calculation method is shown in Section 5.4.2 of "Groundwater Guideline". Evaluation of the impact on the groundwater environment should be conducted for the dry season when the groundwater level is at its lowest, but the calculation was also made on the data of 30th January 2018 when the water level was the highest. Table 5.2 summarizes the data obtained by boring and seismic survey and the parameters necessary for the simplified hydraulic formula calculation.

Table 5.2 Hydrogeological Parameters of Borehole No.1 of Tunnel No.2

Name of parameter	Value or contents of parameter
Description of aquifer	Shallow regolith aquifer region underlying deep fractured zone aquifer
Component material of aquifer	weathered rock, highly weathered rock and sand
Groundwater level (Borehole BT-01 site)	179.043 m a.s.l. (dry season) 181.625 m a.s.l (rainy season, 30 January 2018)
Elevation of upper surface of impermeable layer	155 m a,s,l,
Hydraulic coefficient	4.78×10^{-4}cm/min
Rain infiltration into ground	664mm/year (1,794mm: annual rainfall at Kurunegala (2017), 37%: runoff ratio of observed at Holombuwa (2017)

(Source: Project team)

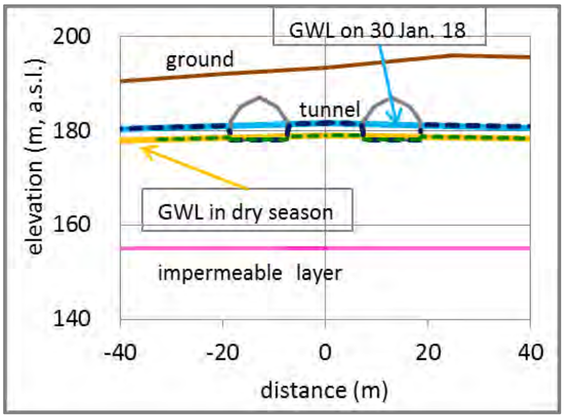


Figure 5.9 Present and Forecaster Groundwater Level after Tunneling Work (Tunnel No.2)

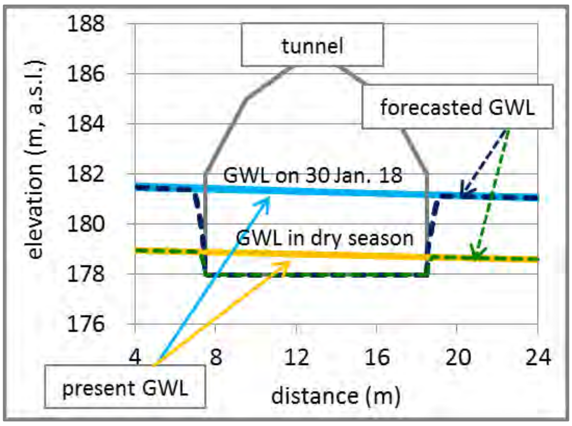


Figure 5.10 Enlarged View of Right Tunnel. (Tunnel No.2)

The calculation result is shown in Figure 5.9. Although the drawdown of water level after tunneling work is shown in the figure, it is too small to be identified. Therefore, the right side of tunnels is enlarged and is shown in Figure 5.10. A lowering of the water level at the tunnel bottom and a water level drawdown in small parts along the tunnel walls are observed in Figure 5.10. The drawdown range is 0.29 m in the dry season and 0.5 m on the 30th of January 2018. The ranges are limited to extremely close to the tunnel. The groundwater flowing into the two tunnels is a small amount of 1.8 m^3, 4.0 m^3 per meter of length of tunnel per day in the dry and rainy season respectively. The larger the coefficient of permeability becomes, the bigger influenced range becomes. The range becomes 0.91 m and 1.80 m respectively by the use of 10 times of the coefficient of permeability; but it is still limited to the places close to the tunnel walls.

Although it is the forecast based on the short period measurement, the tunneling work is considered to have little impact on the groundwater environment in the Tunnel No.2 area for the following reasons:

- the forecasted impact is extremely minor

- the measurement used for the forecast covers the dry and rainy cycle in a year
- the measurements were made not in dry rainy period or a wet dry period

5.1.4 Future Tasks

In the Tunnel No. 2 area, it is considered that there is almost no impact of tunnel drilling on groundwater; but it is necessary to continue hydrogeological observations until the completion of the tunneling work, and to confirm that the forecast is correct. In the case of groundwater lowering, the residents are likely to accuse that the tunnel is the cause. In that case, it is possible to judge whether the decrease in groundwater level is due to natural change or due to external factors based on the observation data. It is easier to convince the residents by showing them the data and explaining that it is a natural change. If the tunnel is the cause, it will be useful to analyze the process that brought the results and it will also be resourceful for planning counter measures to prevent the groundwater level lowering.

5.2 Tunnel No.1 Area

5.2.1 Hydrogeological Investigation

Tunnel No.1 penetrates in the east-west direction under the saddle portion of the mountains continuing in the north-south direction as shown in Figure 5.11. There are some springs and streams flowing from the springs on the ground in the planned tunnel location. There are also houses, and the residents use shallow hand dug wells for domestic use.

Periodic measurements of the water level of household wells, the discharge of the streams and the spring were carried out. The locations of these measurements are shown in Figure 5.12.

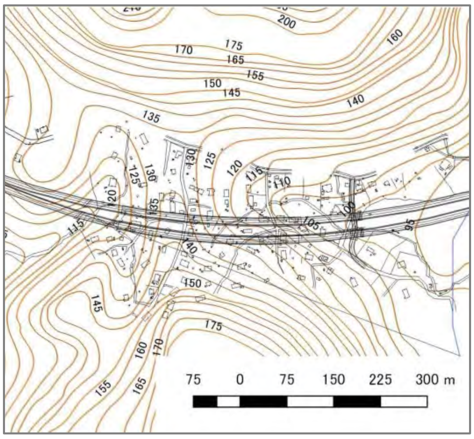


Figure 5.11 Topographic Map of Tunnel No.1 Area

(Source: Project team)

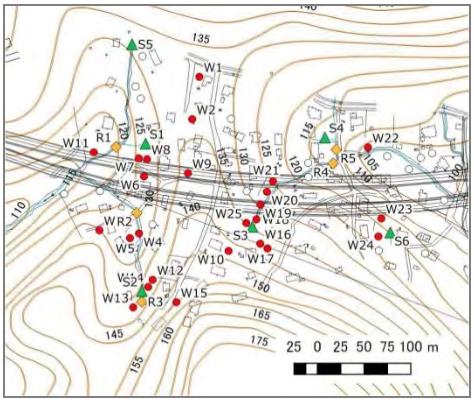


Figure 5.12 Location of Hydrogeological Observation Sites

(Tunnel No.1,R: river, S: spring, W:well)

(Source: Project team)

5.2.2 Present condition of Groundwater Environment

The time series of the measurement data on the wells, streams and springs are shown in Figure 5.13. In August, the measured discharge in streams and springs is small, which increases sharply after August, reaching the maximum in November and decreasing thereafter. The water levels of the wells show a similar trend.

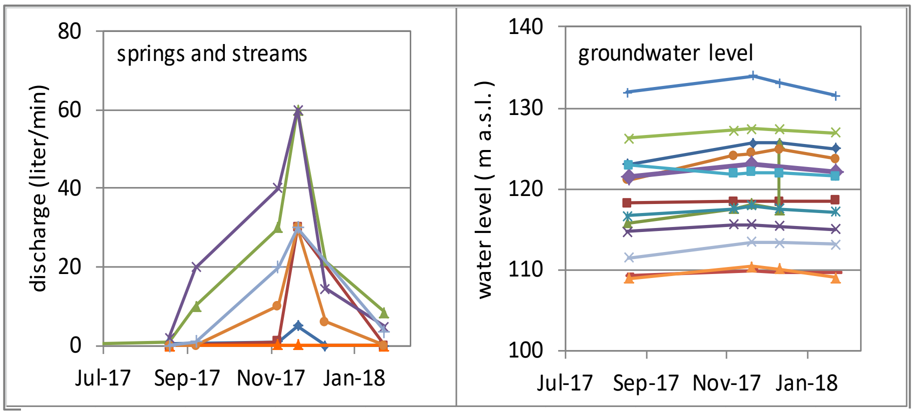


Figure 5.13 Time Series of Observation Data of Discharges of Springs and Streams and Water Level of Wells (Tunnel No.1)

(Source: Project team)

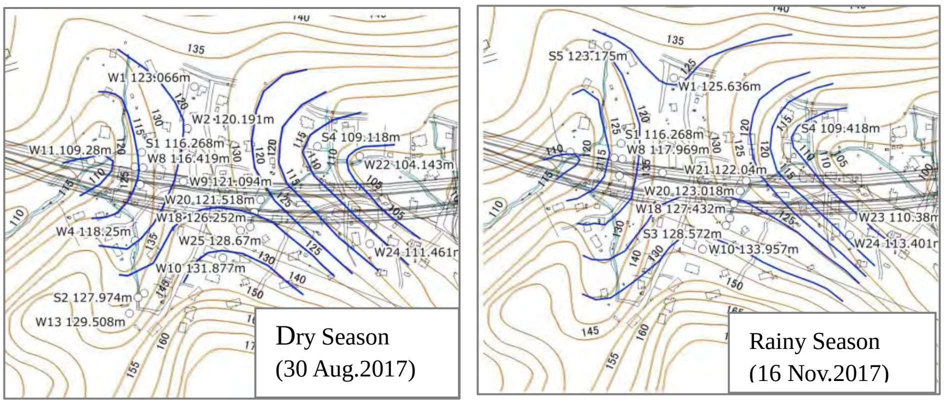


Figure 5.14 Measured Groundwater Levels and Their Contour Map in Dry and Rainy Season (Tunnel No.1)

(Source: Project team)

Groundwater levels in the dry and rainy seasons and groundwater contour lines with every 5 meters drawn based on the groundwater level are shown in Figure 5.14.

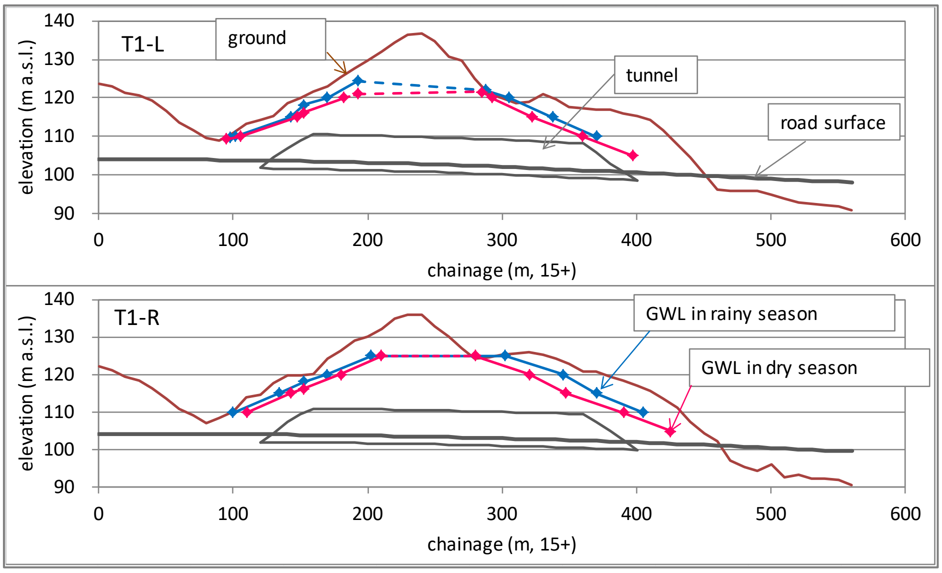


Figure 5.15　Longitudinal Section of Ground, Tunnel and Groundwater Levels of Dry and Rainy Season (Tunnel No.1)

(Source: Project team)

The groundwater contour line is almost along the ground elevation in both rainy and dry seasons. A longitudinal sectional view of the ground surface, tunnel and groundwater level in the tunnel direction is shown in Figure 5.15. Although the position and height of the road have been fixed, the cross-section of the tunnel has not been decided. Therefore the bottom of the tunnel is assumed to be 2 m lower than the road surface. The groundwater level is high at the convex part of the ground in the tunnel center part; the water level becomes lower as the ground elevation becomes lower. Where a stream crosses, the groundwater level is almost the same as the ground elevation. The height of the groundwater level is 24 m above the tunnel bottom at the highest position in the dry and rainy season, and the entire tunnel becomes below the groundwater level.

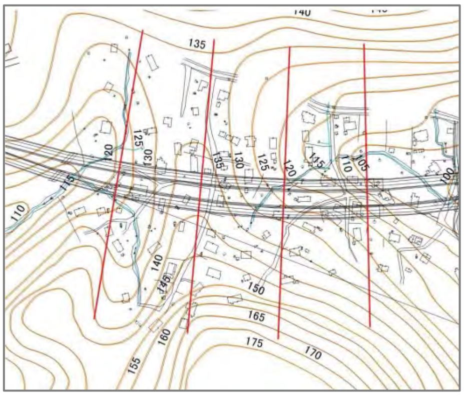


Figure 5.16　Location of Four Cross-sections (Tunnel No.1)

(Source: Project team)

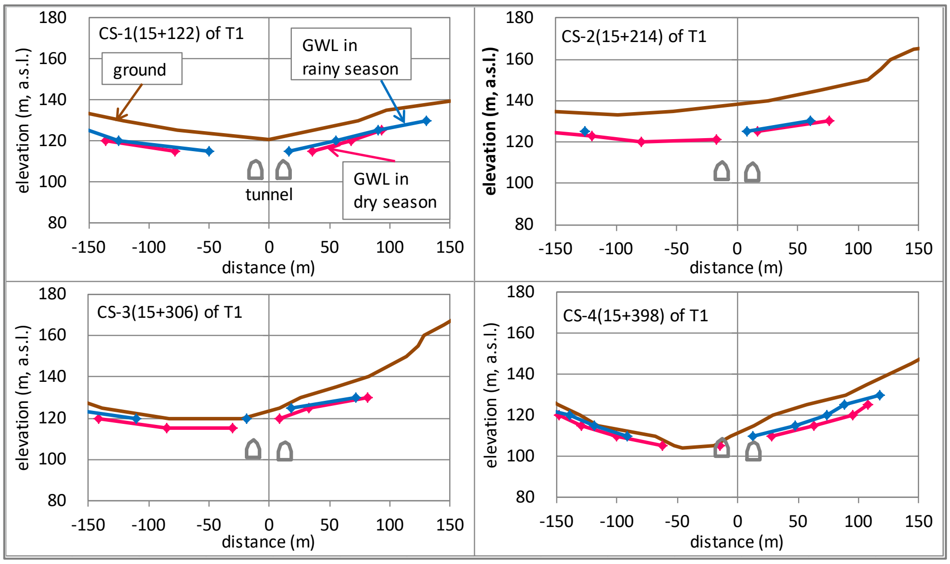


Figure 5.17 Four Cross-sections of Ground, Tunnel and Groundwater Levels of Dry and Rainy Season (Tunnel No.1)

(Source: Project team)

The location of four cross-sections is shown in Figure 5.16, while the ground and groundwater levels are shown in Figure 5.17 to show the relationship between the ground and groundwater. The cross-section at the east and west ends corresponds to the tunnel portal. The topography of the cross-sections is concave and the tunnel passes under the bottom of the valley. The groundwater level is low at the tunnel part and tends to be higher at some distance from the tunnel. Thus, the groundwater flows toward the tunnel.

5.2.3 Forecast of Groundwater Environment after Tunneling Work

Since the investigation such as seismic survey or boring has not been conducted in Tunnel No.1, the information to be used for forecasting groundwater environment after the tunnel work is limited. Takahashi's method was used for the forecast because the method does not require hydrogeological information but only topographic information. Takahashi's method is shown in Section 5.4.3 of "Groundwater Guideline". The method was developed based on the similarity between the groundwater level around the river in the dry season and the groundwater level around the tunnel. It is widely used in Japan, and is evaluated as simple, practical and highly useful. The impact range of tunnel on groundwater is estimated by analyzing the topography of the catchment area of the river around the tunnel.

The estimated results are shown in Figure 5.18. The green dotted line is the Takahashi’s parabola obtained from the analysis and the intersection between the parabola and the ground surface is the impacted range.

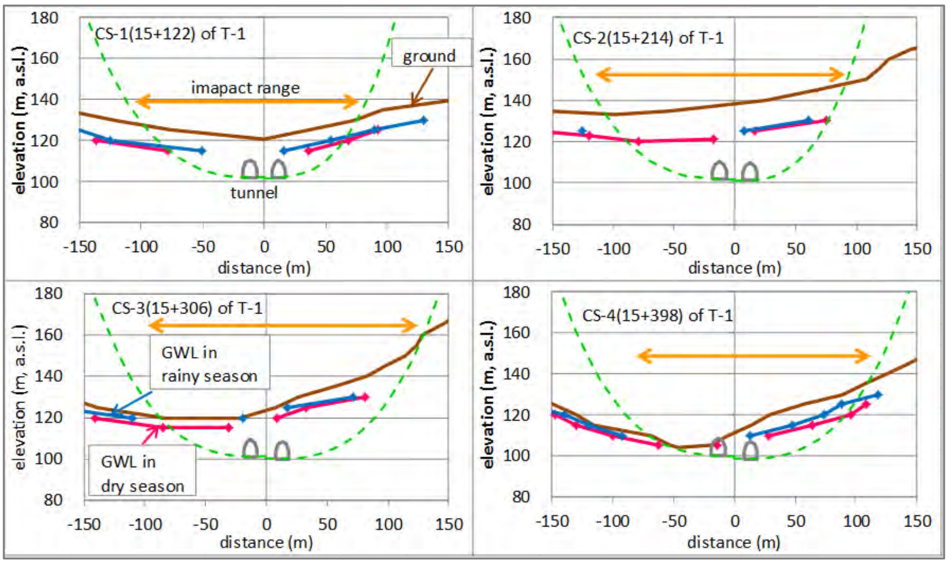


Figure 5.18 Groundwater Impact Ranges of Four Cross-sections Estimated by Takahashi's Method (Tunnel No.1)

(Source: Project team)

The impacted range is shown on the topographic map in Figure 5.19. The impacted range is approximately 100 m on both sides of the tunnel. Approximately 80% of the wells located on both sides of Tunnel No.1 are within the impact range and drawdowns in the water level of these wells are forecasted.

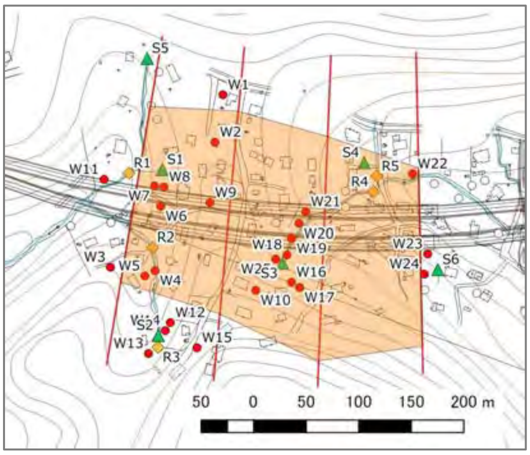


Figure 5.19 Forecasted Impact Area on Groundwater by Tunneling Work (Tunnel No.1)

(Source: Project team)

5.2.4 Current Issues and Future Tasks

Although it is not possible to evaluate the results quantitatively because of limited available hydrogeological information, the results indicate that many wells will be impacted in Tunnel

No.1 area. It is considered necessary to implement the following items to enable quantitative forecasting and to avoid or reduce the impact,

a. Continue to observe the water level of wells, the discharge of the spring and the streams, and accumulate the data to know the fluctuations in a year as well as year to year fluctuation. The accumulation of data can be useful not only for explanation to the residents but also for the groundwater environment forecast in future tunnel work.
b. Start observations and measurements leading to a quantitative forecast by boring, seismic survey and others, and carry out environmental impact assessment.
c. Consider environmental conservation measures such as water supply facilities as part of environmental impact assessment. As an example of water supply facility, it is conceivable to collect groundwater flowing into the tunnel and to supply it to the residents. If the amount of water is insufficient, drilling new well may also be as additional water sources.

5.2.5 Tunnel No.3 Area

Tunnel No.3 penetrates in the north-south direction of the hill surrounded by meandering river shown in Figure 5.20. There are residences on the northeastern side of the tunnel. There are no wells in this area, and the residents use water taps for domestic use and the water source is located upstream of the river. Hydrological or hydrogeological investigation was not conducted in this area. Even if the tunneling work impacts the groundwater environment, there is no effect on the water use of the residents because the residents use the tap water. It is considered that there is no need to conduct immediate hydrogeological investigation. Boring and geological survey will be conducted before the tunneling work and the hydrogeological investigation will be carried out together with the survey. Subsequently, the groundwater environment after tunneling work will be forecasted and no or little impact of tunneling work will be confirmed.

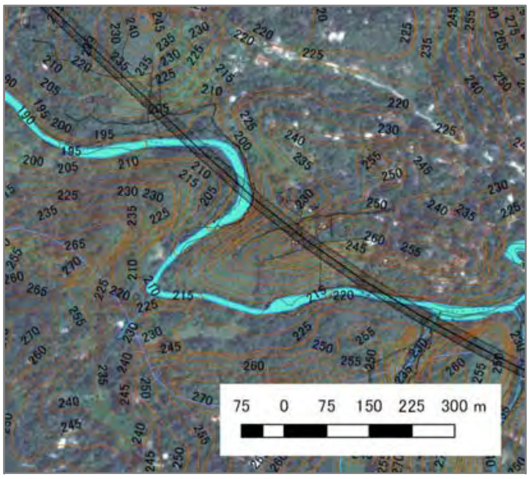


Figure 5.20 Satellite Image with Topographic Map of Tunnel No.3

(Source: Project team)

Chapter 6: Design of Tunnel

Regarding the project outcome iii), the technical transfers were conducted through the preliminary tunnel design in the pilot site (Activities 4.1-4.5, 6.3, 6.4) and preparing tunnel design guideline (Activities 6.2, 6.5). Details of the tunnel design are described in this Chapter.

6.1 Guideline for Design of Road Tunnel

6.1.1 Purpose of the Guideline

The guideline that enables planning of preliminary design levels of mountain tunnels is developed. The shape of the tunnel cross-section, tunnel extension, tunnel lining shape, tunnel portal position, support and shape of portal are determined at the preliminary design stage.

This guideline is aimed at improving the tunnel design skills of engineers of RDA, and is based on "2006 established, tunnel standard specification document [mountain construction method], same commentary, July 2006, The Japan Society of Civil Engineers".

Specific knowledge and technique aimed to gain are following:

- Understand the theory of NATM (New Austrian Tunneling Method)
- Understand tunnel design method
- Judge and validate the cross-section examination, examination of the support, and proposal of an appropriate tunnel portal position
- Prepare a preliminary tunnel plan

6.1.2 Subjects of Guideline

This guideline is prepared for the tunnels with following conditions:

- It is a mountain tunnel
- It is a road tunnel
- Target geology is soft rock, medium hard rock, and hard rock
- The design method is NATM

6.1.3 Contents of Guideline

The guideline consists of the following chapters.

- **Tunnel outline**
  - About tunnel definition and tunnel application
- **Japanese NATM**
  - Difference between NATM and conventional method
  - The concept of Japanese NATM

- Features of Japanese NATM

- **Tunnel design**
  - Features of NATM design
  - Method of design
  - Concept of modified design
  - Specific design method
  - Tunnel's outline construction plan
- **Auxiliary method**
  - Idea of auxiliary method
  - Application of auxiliary method and kinds of auxiliary construction method
- **Review of tunnel equipment**
  - Overview of emergency equipment
  - Consideration of ventilation equipment
  - Review of lighting equipment
  - Review of emergency equipment

6.2 Preliminary Design of CEP-III Tunnel No. 2 (L)

6.2.1 Contents of Preliminary Design

Based on the tunnel design guidelines, a preliminary design was developed.
By the preliminary geological survey, the elastic wave velocity zone, the unit volume weight of the rock, the uniaxial compressive strength of the rock and others are obtained. In addition, the RDA (Deputy Director CEP-III) provided required information to examine the tunnel cross-section. Based on these information, the following items are examined.

- Study on the minimum cross-section
- Consideration of support pattern
- Study of emergency equipment

6.2.2 Study of Minimum Cross-section

(1) Given Conditions

The most fundamental factor in considering the minimum cross-section of the tunnel (most economical cross-sectional area) is the vehicle gauge.

An agreement was made to secure the following items as the minimum requirement to determine the vehicle gauge:

- The road width of one lane shall be 3.6 m
- The vehicle gauge height on the lane is 5.1 m or more (Including the overlay amount of pavement)

(2) Cross Section in F/S

The cross section provided by the RDA at the F/S stage is shown below.

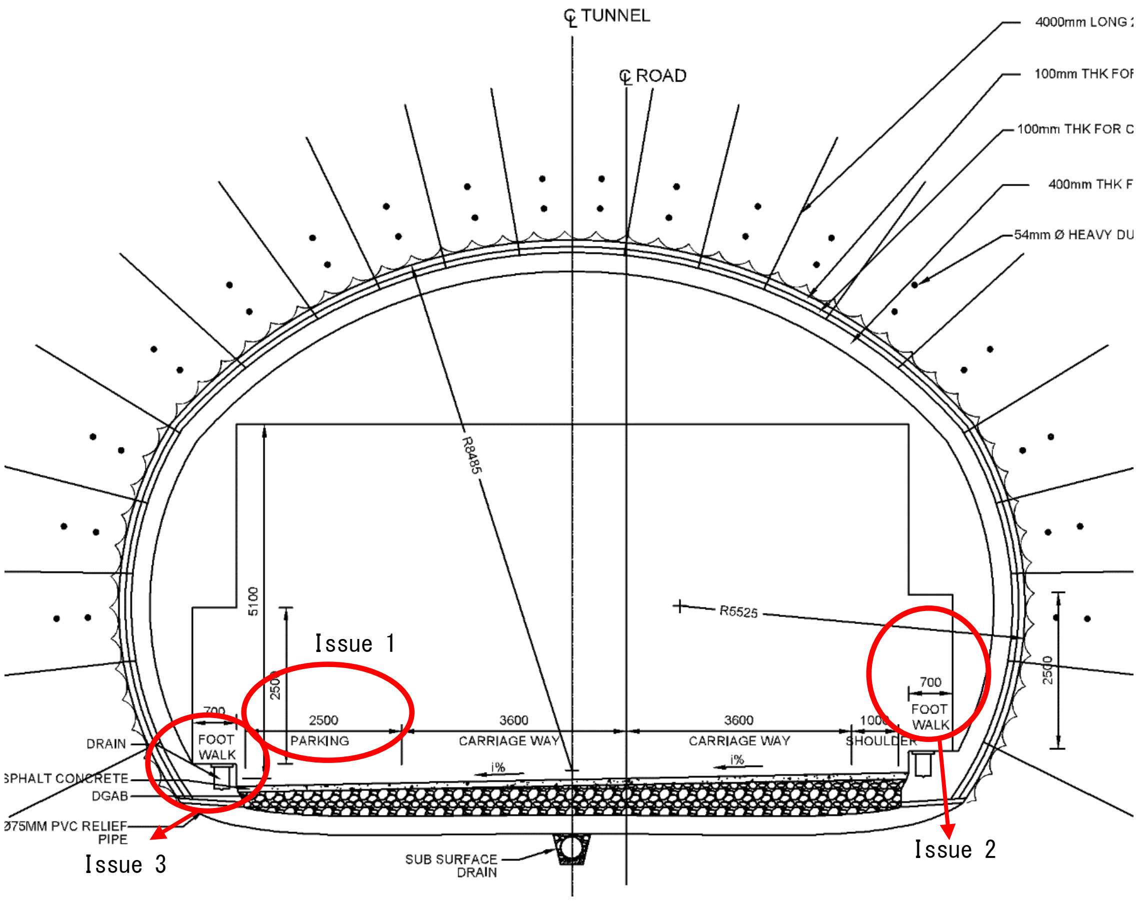


Figure 6.1 Cross Section in F/S

(Source: RDA)

The issues of this cross-section are described below.

a. Although the parking lane is provided, it was agreed that the parking lane is unnecessary since the length of this tunnel is very short, in the order of 160 m.
The presence of the parking lane just increased the cross-section area. Also, when a fire occurs due to a vehicular accident or the like and one lane becomes jammed, another lane can be used as a by-pass lane. Therefore, it is judged that there is no problem in evacuating vehicles. This further justifies removing the parking lane in the design.

b. Audit corridors called "Foot Walk", which is used for maintenance of emergency facilities are set. However, it was concluded by the preliminary examination that it is unnecessary to install emergency facilities, because the tunnel length is short. Therefore, it has been decided that audit corridors are unnecessary. Also, it is not necessary in the regulation of NEXCO (Nippon Expressway Company Limited) of Japan as well.

c. Since the ditches installed under the sidewalk are higher than the road, rainwater remains on the road shoulder. For this reason, breaking distance to stop vehicles becomes long. When this happens, the accident rate increases.

(3) Examination Conditions of Minimum Cross-section

Since there are concerns in the cross-section at the F/S stage, examination of the cross-section using the method generally employed in Japan is conducted. The examination conditions are shown in Table 6.1.

Table 6.1 Examination Conditions of Minimum Cross section

Item	Contents	Conditions
Road specification	1 lane width	3.6m
	Shoulder width	1.0m (both side)
	Parking lane	None (not necessary), tunnel length (L=160m) is short
	Superelevation (slope)	2.0% (temporary setting)
Vehicle gauge	Height	5.1m (including additional pavement layer)
Foot walk		None (not necessary), tunnel length (L=160m) is short
Drainage	Side ditch	Quoted Japanese specifications
	Center drainage pipe	Quoted Japanese specifications
Pavement	Pavement structure	Quoted Japanese specifications

Item	Contents	Conditions
Cross-section	Shape of cross-section	Triangle (top heading semicircle) Japan Expressway Standard
Inertial plate		None (not necessary), quoted Japanese specifications

(Source: Project team)

(4) Study of Ventilation Equipment

In the road tunnel, ventilation is necessary to suppress the pollution of the air in the tunnel due to the exhaust gas from the vehicles running in the tunnel. Natural ventilation may be sufficient for short tunnels; but ventilation facilities are necessary for long tunnels and high traffic volume.

- Examination of necessity of ventilation equipment−

The following empirical formula is used in Japan for outline examination on the necessity of ventilation equipment.

L・N = 600

L: tunnel length [km]

N: traffic volume per hour [vehicles/h]

L・N ＜ 600 Ventilation is unnecessary

L・N ＞ 600 Ventilation is necessary

<u>Outline examination in No 2 tunnel (L)</u>

L: tunnel length [km] = 0.160Km

N: traffic volume per hour [vehicles/h] = 417

*** Design traffic volume is assumed to be 10,000 vehicles/day N=10,000/24=417**

<u>0.160・417 =66.7＜600 Unnecessary</u>

Since there is no information on traffic volume, the planned traffic volume was set at 10,000vehicle/days. It is necessary to know the planned traffic volume on the stage of detailed design. Furthermore, detailed examination is also necessary because this formula is a outline examination method.

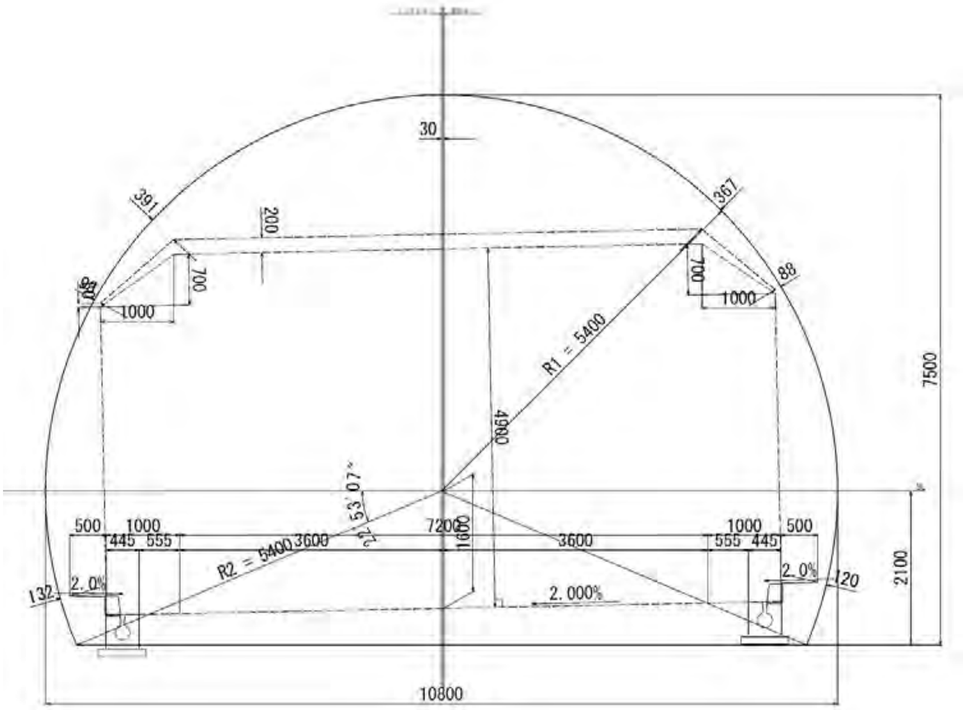
Figure 6.2 Ventilation Equipment (free-suspended Jet Fan)

(Source:jet fan manufacturer)

6.2.3 Results of Calculating Minimum Cross-section

A minimum cross-section was determined by inputting design conditions to an appropriate design program.

The determined cross-section diagram is shown as follows.



Item	number
Cross-section width	10.8m
Cross-section height	7.5m
Cross-section area	67.9m^2

Figure 6.3 Minimum Cross-section

(Source: Project team)

6.2.4 Comparison of Proposed Cross-section and Cross-section in F/S

As shown in Table 6.2, the proposed cross-section width, height and area are lower than the F/S cross-section. It is concluded that the proposed cross-section is economical.

Table 6.2 Comparison between Cross-sections in F/S and Proposed

Item	Cross-section in F/S	Proposed Cross-section
Cross-section width	12.1m	10.8m
Cross-section height	8.5m	7.5m
Cross-section area	86.0m^2	67.9m^2

(Source: Project team)

6.2.5 Cosideration of Portal Zone

The cutting slopes at the tunnel portals start from the position securing 3 m of overburden. Cut slope is basically set to 1: 0.5. At the time of cutting, basically it is necessary to carry out shotcrete to secure the gradient and protect the cut surface. A general portal wall type is assumed since no special limitation.

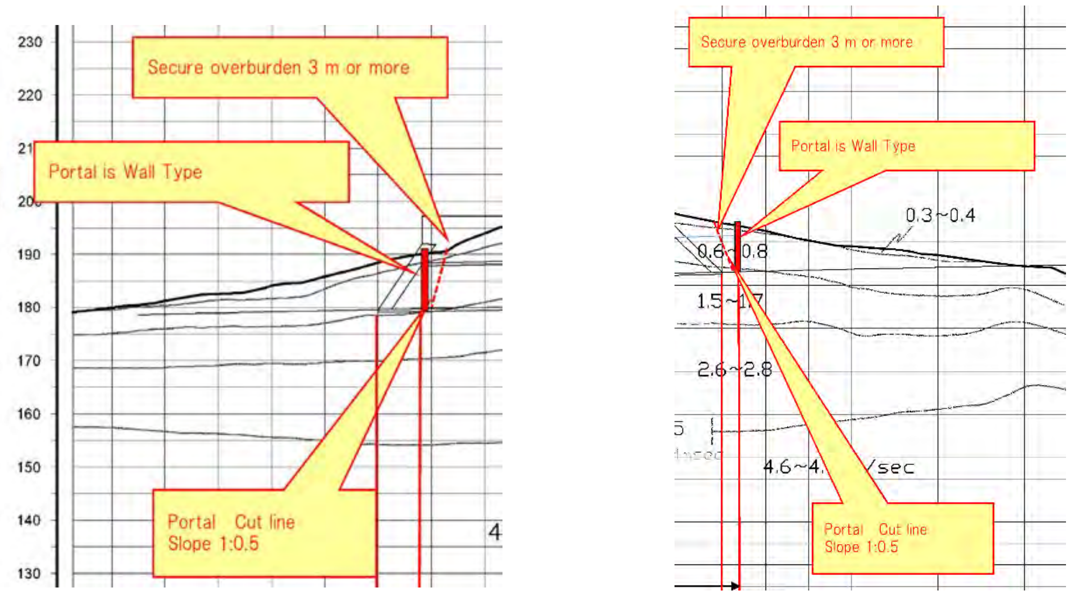


Figure 6.4 Portal Location

(Source: Project team)

In Japan, the portal cut line is basically protected by shotcrete. To adopt this construction method for this tunnel, the cut slope is set to 1: 0.5.

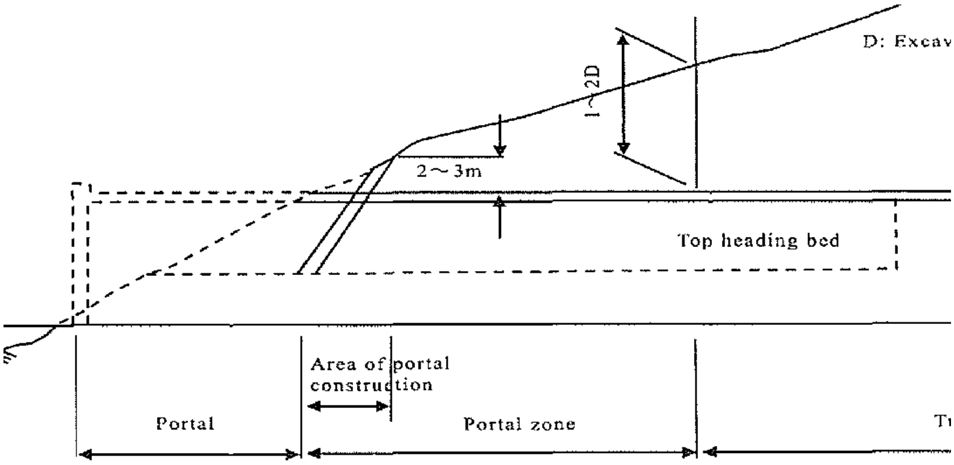


Figure 6.5 Portal Zone

(Source: Standard Specifications for Tunneling-2006 :mountain Tunneis,2007,Japan Society of Civil Enginners,P-113)

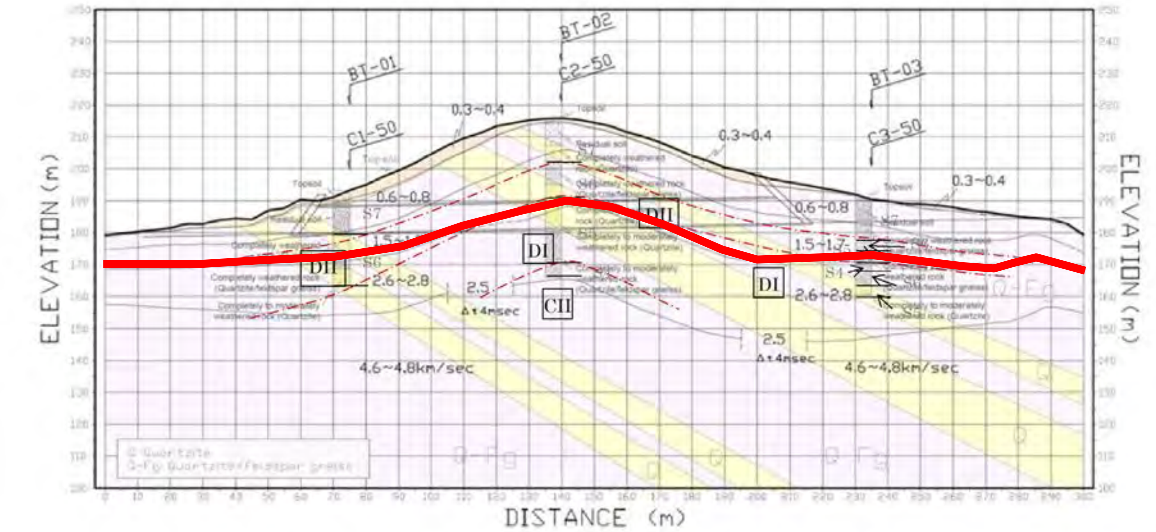
6.2.6 Consideration of Support Pattern

(1) Design of Support Pattern

The proposed support pattern is selected based on the ground classification in Japan. Conditions for selection are as follows:

- **The elastic wave velocity of the bedrock is about 1.5 to 1.7 km / sec**
- **The uniaxial compressive strength of bedrock is almost 50 N / mm ^{2} or more**

Considering the above conditions and a small overburden, **DI-b for the support pattern is proposed.**



* Following correlation can be assumed. CII – S4, DI – S5, DII – S6 (S1～S7: Rock Mass Classification proposed in this project)

Figure 6.6 Seismic Velocity Layer and Rock Mass Classification of Tunnel No. 2

(Source: Project team)

Table 6.3 Setting of Support by Ground Classification of Japan

Class of ground	Rock type	Representative rock name	Elastic wave velocity (Vp km/s) 1.0 2.0 3.0 4.0	Geological condition: Effect by water and lithologic character	Geological condition: Interval of discontinuity	Geological condition: Condition of discontinuity	Boring core condition RQD	Ground competence factor	Situation of the tunneling and standard of the displacement
B	H Massive	Granite, granodiorite, quartz porphyry, hornfels		- Rock is extremely hard and fresh or a slight tendency toward deterioration due to weathering action is to be seen. - No deterioration due to water.	- The interval of the joint is about average 50cm. - Though there is the influence of bedding and schistosity, these influence on the tunneling has been limited.	- It has almost no slickenside and fault clay in discontinuity. - Discontinuity has closed approximately.	- The shape of the core shows large pieces, short cylinders or rods; core length being generally in rang 10-20cm, but with length on order of 5cm also present. - RQD is over 70.	—	- The rock strength is very bigger than the load which is caused by the tunneling. - The condition of discontinuity is good, and the looseness by the tunneling does not occur almost. - Rock falls from the excavation face occur rarely, and the convergence with the excavation is elastic deformation of about 15mm or less. - Cutting face stands up.
		Paleozoic and Mesozoic sandstone,chert							
	M Massive, L Massive	Andesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
		Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							
C I	H Massive	Granite, granodiorite, quartz porphyry, hornfels		- Rock is relatively hard and fresh or a slight tendency toward deterioration due to weathering action is to be seen. - Soft rock which is relatively consolidated. - No deterioration due to water.	- The interval of the joint is about average 30cm. - Bedding and schistosity are remarkable, and these affect the tunneling.	- It has a few slickenside and fault clay in discontinuity. - Though discontinuity is partially opening, the width is small.	- Core length being generally in rang 5-20cm, but with length on 5cm or less also present. - RQD is 40-70.	— Over 4	- The rock strength is bigger than the load which is caused by the tunneling. - The condition of discontinuity is good, and the looseness by the tunneling occur partly. - The partial rock falls along discontinuity which is easy to slide relatively occur rarely, and the convergence with the excavation is elastic deformation of about 15～20mm. - Cutting face stands up.
		Paleozoic and Mesozoic sandstone,chert							
	M Massive, L Massive	Andesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
		Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							
C II	H Massive	Granite, granodiorite, quartz porphyry, hornfels		- Rock is relatively hard and fresh or a slight tendency toward deterioration due to weathering action is to be seen. - Rock becames somewhat soft as result of weathering and alteration. - Soft rock which is relatively consolidated. - It has some deterioration or looseness due to water.	- The interval of the joint is about average 20cm. - Bedding and schistosity are remarkable, and these affect the tunneling.	- It has some slickenside and fault clay in discontinuity. - Opened discontinuity increases, and the width of discontinutiy also expands. - The rock with small fault which is narrow width.	- Core length being generally in 10cm or less, and with numerous pieces of 5cm or less. - RQD is 10-40.	— Over 4	- Though the rock strength is not very bigger than the load which is caused by the tunneling.it is in the range of the plastic deformation. - Sincet the condition of discontinuity is bad, even if the strength of rock is big, rock block intend to fall along the discontinuity which is easy to slide and the looseness by the tunneling increase . - When strength of rock is smaller than the load which affects it, the convergence with the excavation achieved at about 30mm which is the elasto-plasticity boundary,but it is almost settled, until the face separates on the displacement 2 times of the diameter of the tunnel(2D). - Cutting face stands up.
		Paleozoic and Mesozoic sandstone,chert							
	M Massive, L Massive	Andesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
		Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							
D I	H Massive	Granite, granodiorite, quartz porphyry, hornfels		- Though the hard rock remains partly, it received generally strong weathering and alteration. - Bedding and schistosity are remarkable. - The interval of the discontinuity is about average 10cm or less, and most of their are opened. - Width of discontinutiy big and most of thier have been slickenside and fault clay. - The rock within small fault which is narrow width and with clay. - Soil material mixed numerous pumice, talus, etc. - It has remarkable deterioration or looseness due to water.			- Cores take form of small pieces, but sometimes form of clay, or sand with rock fragments mixed in. - RQD is 10 or less.	4~2	- The rock strength is bigger than the load which is caused by the tunneling.it is occur the plastic deformation and elastic deformation partially. - Sincet the condition of discontinuity is very bad, even if it is sufficient in the strength of rock fixing the plastic deformation, the looseness by the tunneling is increased along many discontinuity which is easy to slide. - When strength of rock is smaller than the load which affects it, the convergence with the excavation achieved at about 30-60mm without the case which carried out the invert closing early, and it is almost settled, until the face separates on the displacement 2 times of the diameter of the tunnel(2D). - The face is unstable, and ring cuts and face shotcrete are necessary by the condition of the ground.
		Paleozoic and Mesozoic sandstone,chert							
	M Massive, L Massive	Andesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
		Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							
D II	H Massive	Granite, granodiorite, quartz porphyry, hornfels						2~1	- Though the rock strength is smaller than the load which is caused by the tunneling.it is occur the plastic deformation and large elastic deformation. - Sincet the rock strength is small and condition of discontinuity is very bad, the looseness by the tunneling expands along many discontinuity which is easy to slide, and the displacement also increases.The convergence with the excavation achieved at about 60-200mm without the case which carried out the invert closing early, and it is not settled, even if the face separates on the displacement 2 times of the diameter of the tunnel(2D). - The face is unstable, and ring cuts and face shotcrete are necessary by the condition of the ground.
		Paleozoic and Mesozoic sandstone,chert							
	M Massive, L Massive	Amdesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
		Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							

Note 1) Rock class A is supposed to be good rock which does not correspond to the table, rock class E is supposed to be inferior (convergence; over 200mm).

Note 2) The division of H,M,L; By the strength in the primary and fresh condition of rock, it will be divided at the uniaxial compressive strength as following.

H; qu≧80N/mm², M; 20N/mm²≦qu<80N/mm², L; qu<20N/mm²

Note 3) The division of Massive,Layerd Massive; The rock which the joint plane becomes dominant surface of discontinuity.

Layerd ; The rock which bedding plane or schistosity plane become dominant surface of discontinuity.

Note 4) Convergence means the change of the distance between tunnels wall surface measured under tunneling actually, and it is not contained the displacement before the excavation.

Note 5) Looseness means that the rock block field intends to fall along discontinutiy by the gravity, since discontinuity in rock mass which has closed by earth pressure opened because of the release of in-situ stress by the tunneling.

Note 6) The strength of rock means the strength of the rock which does not receive the influence of the fissure.

(Source: modified after 2016 established, tunnel standard specification document [mountain construction method] 2016, Japan society of civil engineers, P-48)

Suppot Pattern	Length(m)
DⅢa	37
D I	70
DⅢa	48
Total	155

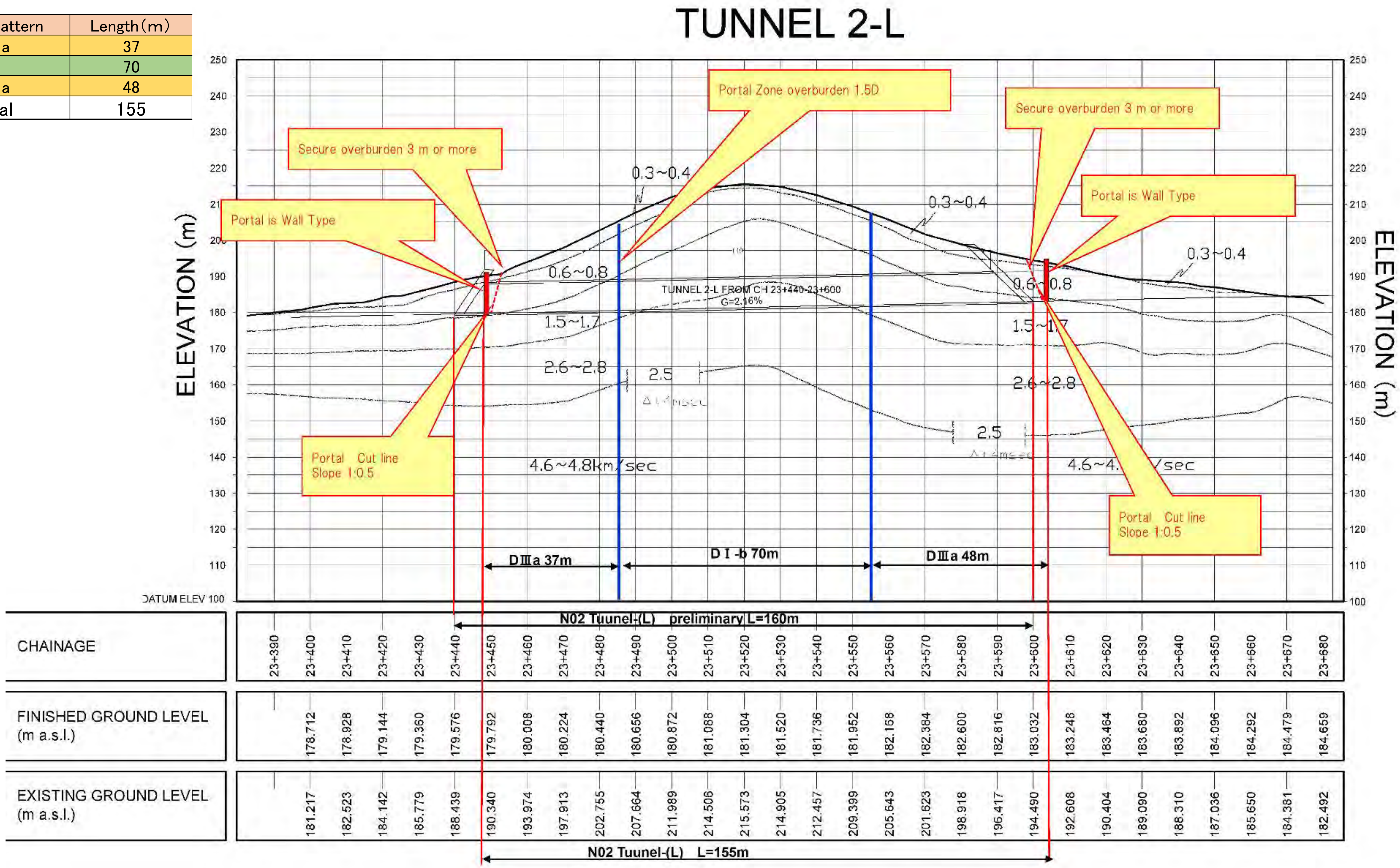


Figure 6.7 Support Pattern of Tunnel No. 2

(Source: Project team)

(2) Auxiliary Method

From the results of the geological survey, it is assumed that fragile stratum is distributed in the upper part of the tunnel. The boring core at 0 to 3 m of BT-01 shows the appearance of viscous soil, and is assumed to be a talus cone. In addition, the core cannot be collected even at 20 to 25 m of BT-02; the RQD is evaluated as zero.

Even after taking into account the underestimation of RQD due to boring operator's poor experience on the core drilling, it is judged that the severe weathered layer distributes around the tunnel.

Therefore, it is considered that the auxiliary construction method is necessary in the portal zone (DIII-a) with the foregoing conditions, where arch action is difficult to be formed.

According to the drilling core condition, the crown of tunnel may collapse without supporting structures. Thus, it is judged that the stabilization methods for crown are necessary, and long steel pipe forepiling is appropriate.

The long steel pipe forepiling is well adopted in Japan and excluding the possibility of extending the construction period (the cycle is earlier than other auxiliary methods).

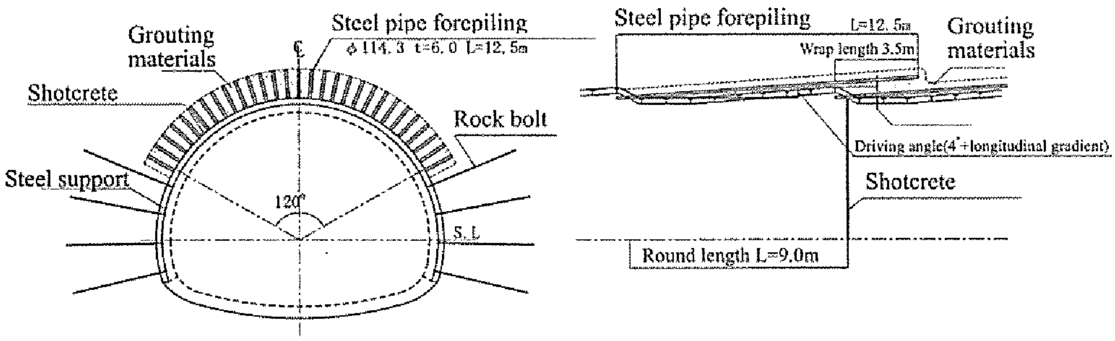


Figure 6.8 Example of Long Steel Pipe Forepiling

(Source: Standard Specifications for Tunneling-2006 :mountain Tunneis,2007,Japan Society of Civil Enginners,P-113)

6.2.7 Standard Cross-section and Support Pattern

(1) DI-b

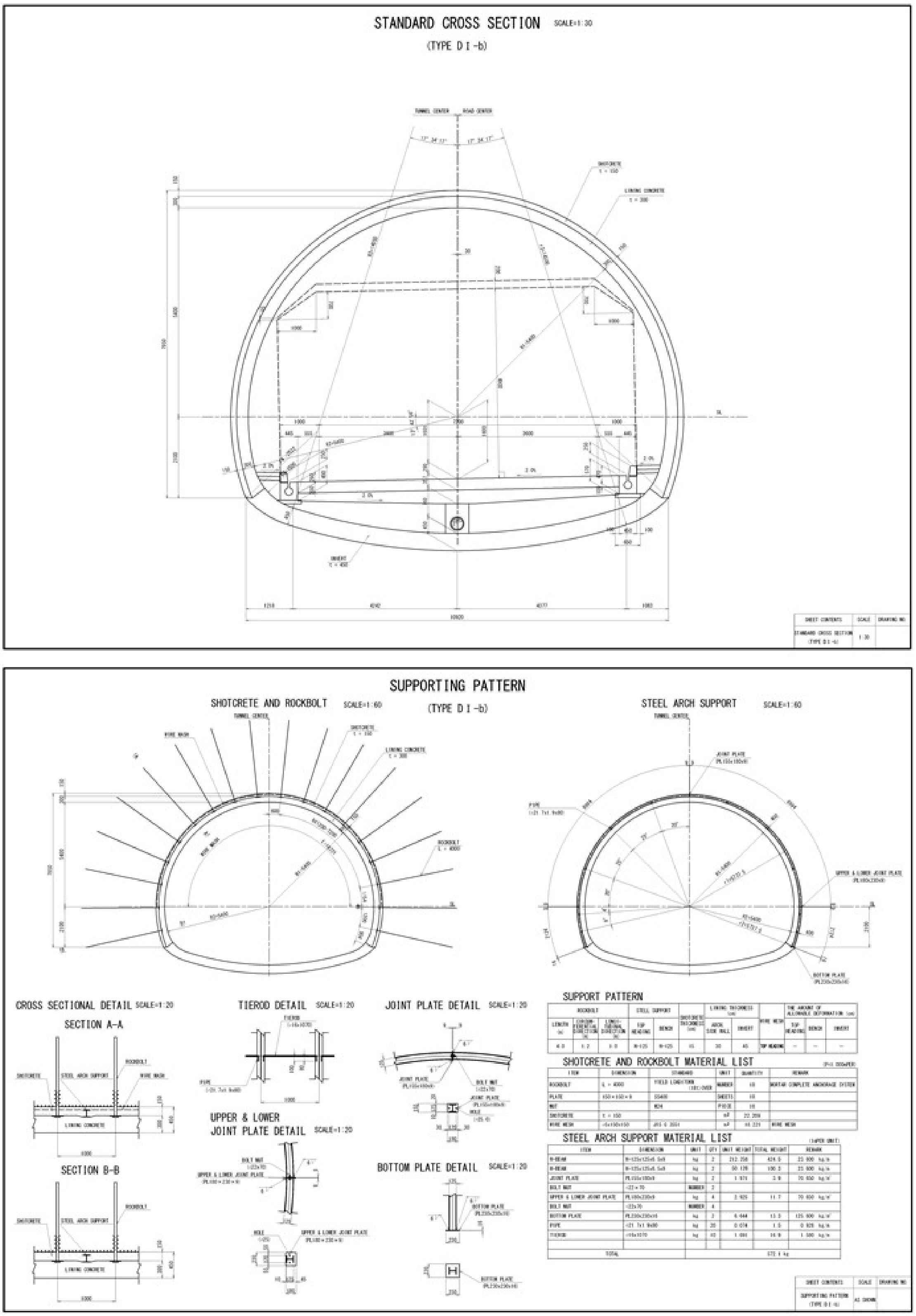
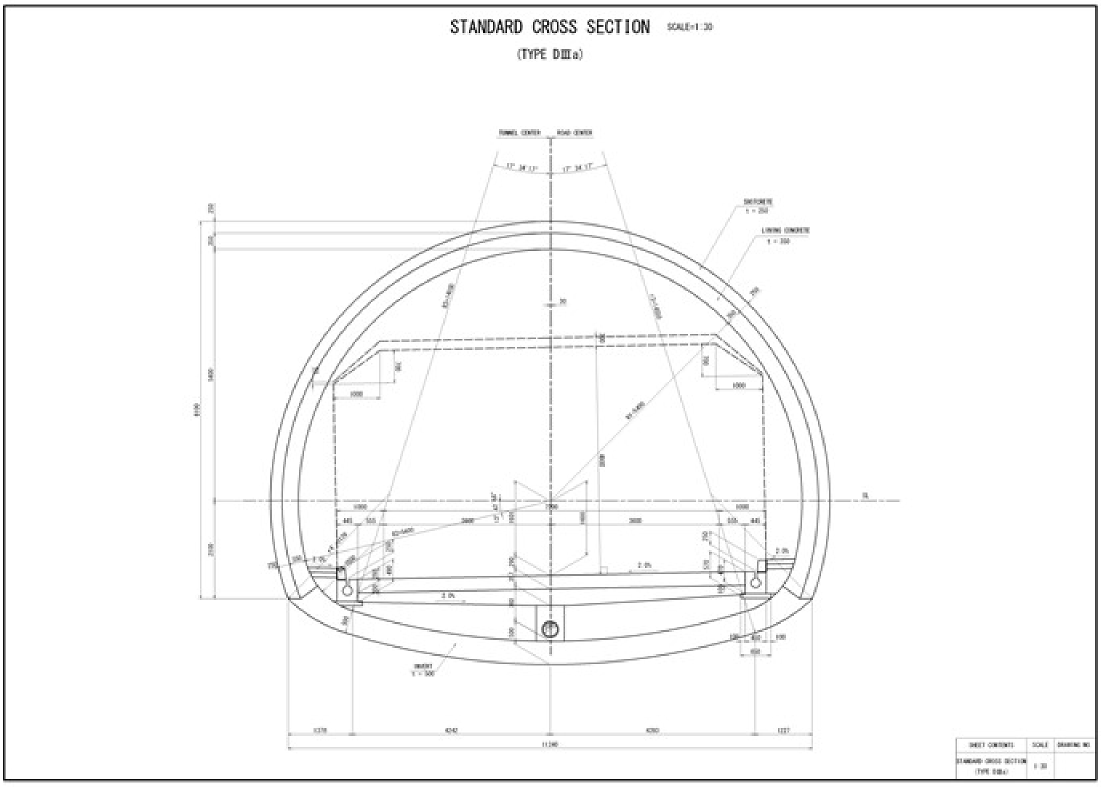


Figure 6.9 Standard Cross-section and Support Pattern (DI-b)

(Source: Project team)

(2) DIII-a



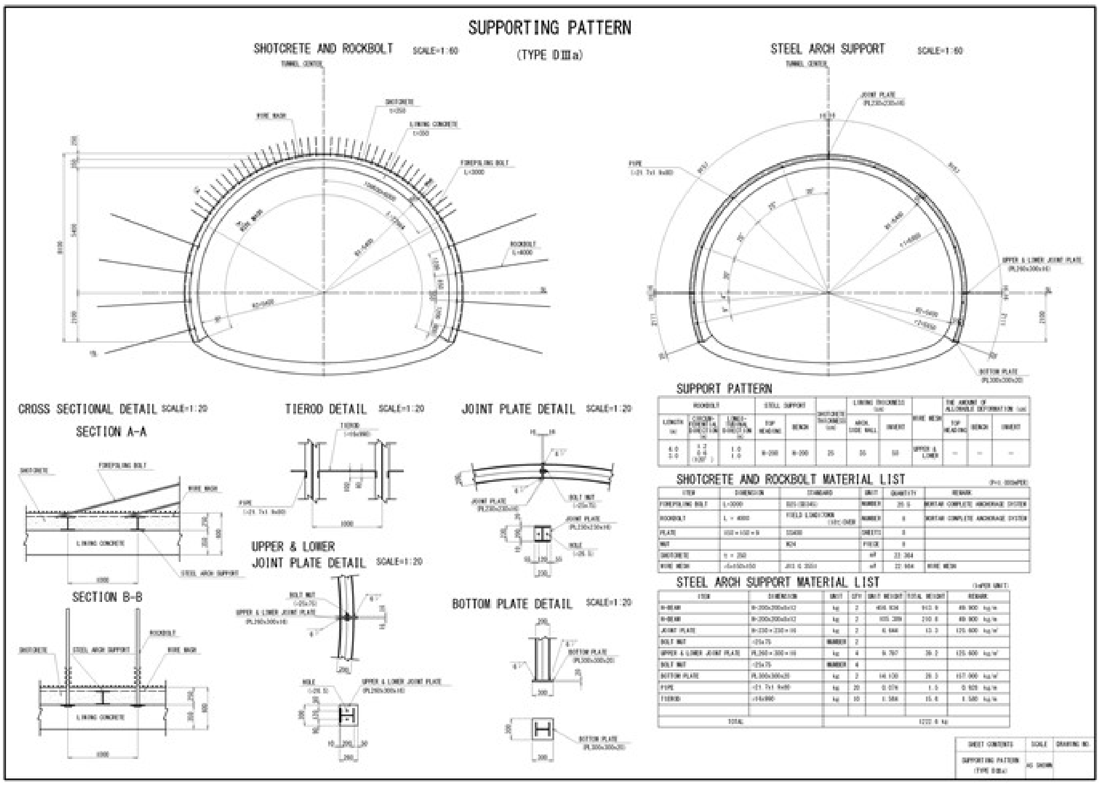


Figure 6.10 Standard Cross-section and Support Pattern (DIII-a)

(Source: Project team)

6.2.8 Consideration of Emergency Equipment

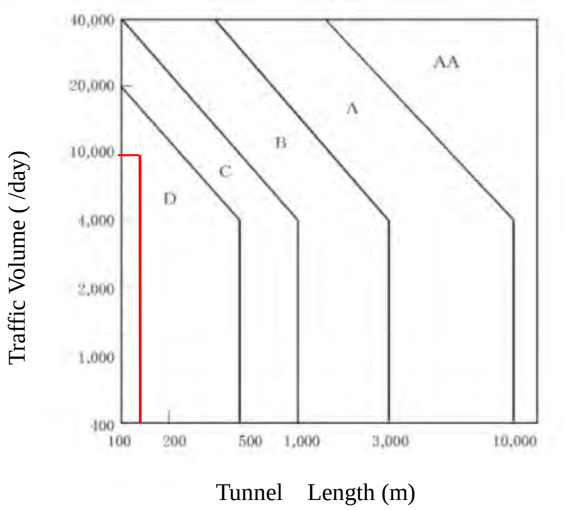
Since the tunnel is a closed space, consideration is necessary for disaster prevention measures against fire and other incidents.

Disaster prevention measures for tunnels include prevention measures through public relations and public awareness on various legal rules for tunnel users. In addition to that, emergency facilities are installed to minimize damage when accidents occur.

In Japan, grades are determined from the relationship between tunnel length and traffic volume; emergency equipment to be installed are identified for each grade.

The length of this tunnel is 155 m, which is provisional; but because it sets the traffic volume to 10,000 units, it is determined to be D grade.

In the case of D grade, emergency equipment in the tunnel is unnecessary.



Emergency equipment	Tunnel grade	AA	A	B	C	D
Notifying and alerting devices	Emergency telephones	○	○	○	○	
	Push button call units	○	○	○	○	
	Fire detectors	○	△			
	Emergency alarms	○	○	○	○	
Fire extinguishing equipment	Fire extinguishers	○	○	○		
	Hydrants	○	○			
Evacuation guidance equipment	Guidance display panels	○	○	○		
	Smoke vents or evacuation passages	○	△			
Other devices	Water plugs	○	△			
	Wireless communication supporting equipment	○	△			
	Radio repeating devices or public-address systems	○	△			
	Water spray apparatus	○	△			
	Monitors	○	△			

○ : Installation required

△ : Decide after consultation with other agencies

Figure 6.11 Rationale for Determining the Need for Emergency Equipment (Japanese standard)

(Source: modified after Road Tunnel Emergency Facility Installation Criteria and Interpretation, 2001, Japan Road Association, P-9.P-14)

6.3 Future Tasks

6.3.1 Outline of Future Tasks

The subject of this survey was the No. 2 tunnel in CEP-III, but overall there are three places where tunnels are applied (including No. 2) with a total of six tunnels.

Overall the overburden is small; at some tunnels, there is little separation between the tunnel portals and bridges. Therefore, it is considered that the required technology level is high in the design and construction of the tunnels.

The anticipated future issues obtained through this preliminary design have been identified. Those concerns are explained in the following sections.

6.3.2 Consideration of Examination of Cross-section

There are many indefinite conditions for examining the cross-section. For instance, it is necessary to examine the gradient (slope) of the pavement, the structure of the pavement, the shape of the drainage, the decision on the need for parking zone and others, to determine the final cross section. Moreover, it is necessary to examine the cross-section based on whether it is economical to make the same cross section for all six tunnels (Since the slope of the pavement that depends on the horizontal alignment is different, the minimum cross-section may be different from each other).

In case that the same cross-section is applied for all tunnels, an inner mold for concrete for lining can be utilized for all tunnels. This is an economically advantageous point.

Table 6.4 Issues of Cross-sections

Item	Subjects
Consideration of Cross-section	Setting the slope of pavement
	Setting structure of pavement (Ensure consistency with the another section)
	Setting of side ditch and center pipe
	Consideration of necessity of parking lane

(Source: Project team)

6.3.3 Review of Support Pattern

After gathering and sorting all the geological information of all six tunnels, it is necessary to ensure consistency of reasons and evidence in selecting the support pattern to secure the validity of the six tunnel designs.

6.3.4 Development of Detailed Construction Plan

Considering market conditions of local materials and the state of procurement of machinery, especially the import of tunnel machinery, it is necessary to formulate a construction plan.

It is also necessary to consider environmental conservation measures such as wastewater treatment by turbid water treatment facilities.

Temporary electric power is always a problem in provisional equipment. In Japan, a 6,600 V lead-in line is installed and converted to 220 V at the substation; but it is necessary to consider whether the Japanese practice is possible in the setting or whether a generator should be installed.

6.3.5 Approximate Cost

Calculation of tunnel construction expenses requires all conditions to be met. For example, methods of excavation (blasting or mechanical), power supply (from temporary source or a generator) and inner mold work (whether one inner mold can be utilized for all tunnels or not) should be decided for the cost estimation.

Furthermore, machinery, which has a great influence on the cost, cannot be finalized until the detailed design is completed. Therefore, it is appropriate to calculate tunnel construction expenses after establishing which machine will be needed.

Chapter 7: Outcome of Technical Transfer and Recommendations for Future

Outline of the status of achievement regarding the expected project outcomes is described in this chapter. Moreover, recommendations on necessary approaches for RDA to utilize the outcomes are also shown.

Expected outcomes of the project:

i) Draft of criteria for rock mass classification for road tunnel construction is prepared.

ii) Draft of procedure manuals for geological survey for rock mass classification system, road tunnel design and evaluation and environmental impact study on groundwater is prepared.

iii) RDA acquires, through the pilot survey, the skills to utilize the guidelines prepared by the outcomes i) and ii) and knowledge to prepare the design document necessary for actual tunnel construction.

7.1 Situations of C/P and Technical Transfer

The draft guidelines and standards for road tunnel planning (geological site investigation, groundwater environment, rock mass classification and tunnel design) were developed by the project activities regarding the outcome i) and ii). Additionally, the technical transfer regarding outcome iii) were conducted through technical seminars and routine discussions with C/P based on the developed guidelines. C/Ps were assigned from the six divisions of RDA (CEP-III, Bridge Design, Highway Design, Research and Design, Ruwanpura Expressway and Kandy Tunnel Project); one to two engineers assigned from the six divisions were the core-C/P. In addition, three to six C/P were joined in the technical seminars continuously. Experience of the C/P on road planning, design and investigation ranges from less than one year to more than 20 years; the experiences of most C/P were less than 10 years.

A lot of C/P has experiences on design and investigation for bridges and other structures, but there few experiences on road tunnels. Furthermore, guidelines and standards for tunnel design and investigation also have not been developed by RDA. Therefore, it seems that basic technical transfer regarding tunneling works is required. A process to develop the draft guidelines and standards and outcome of the technical transfer are described in the following sections.

7.2 Rock Mass Classification System and Guidelines for Tunnel Investigation and Design

7.2.1 Development of Draft Guidelines and Standards

The draft guidelines and standards for road tunnel planning (geological site investigation, groundwater environment, rock mass classification and tunnel design) were developed by the project activities regarding the outcomes i) and ii) (Activity 1.3, 5.1, 6.2, 6.5, 7.2, 7.5). The draft guidelines and standards are attached as separate volumes.

In order to develop the guidelines and standards, previous studies and existing information/standards in Japan and other countries were firstly collected. In addition, information about previous tunneling works and standards in Sri Lanka was collected. Based on the information, the first draft of the guidelines and standards were developed. The first drafts were finalized through discussions with RDA and the technical seminars.

7.2.2 Utilization of Guidelines and Standards

The draft guidelines and standards prepared by the project can be utilized for future tunnel projects. However, there are still some issues on the utilization of the guidelines and standards. Therefore, continuous approaches are required (refer to section 7.4).

7.3 Contents of Technical Transfer and Evaluation

The technical transfers through technical seminars and other activities regarding the outcome iii) were conducted. The contents of the technical transfer and evaluation are mentioned below.

7.3.1 Technical Transfer about Geological Investigation, Hydrological Investigation and Rock Mass Classification System

As mentioned in section 7.1, the experiences of C/P on tunneling works are limited. Thus, 10 technical seminars covering basic techniques on geological investigation, hydrological investigation and rock mass classification system were held to transfer and share the techniques and information (Table 1.4). The C/P asked a lot of questions and had discussions to obtain helpful information for their understanding.

In addition, the two final seminars to share the guidelines and result of the pilot survey were held. The presenters of the final seminars were assigned from the C/P. The presentation materials were prepared by collaborative works of C/P and the project members to ensure effective technical transfer.

In some case, the C/P showed concern on their inadequate understanding due to lack of their experience; nevertheless they finally prepared the presentation materials attached with their own ideas. From this reason, understanding of C/P seems improved.

Technical transfer through the seminar in the pilot site and the laboratories and through routine discussions with C/P was also conducted. Regarding the hydrological investigation, the hydrological survey points and methods were explained to RDA Kurunegala office engineers on site investigation. After the final seminar held in Colombo, the result of the hydrological investigation and forecasted impacts by the tunneling work on groundwater environment were also explained to the RDA Kurunegala to promote understanding toward the importance of hydrological investigation. The hydrological investigation sites and instrument were handed over. Additionally, advices for continuous hydrological investigation were given to RDA.

7.3.2 Technical Transfer about Tunnel Design

Technical transfer about tunnel designs was conducted through four technical seminars and two final seminars; technical transfer about how to reflect the results of geological investigation and rock mass classification to tunnel design was also conducted through discussion with C/P. In addition, Japanese Government Order on Road Design Standards regarding alignment of expressway including standards of longitudinal/cross-sectional gradient and sight distance were introduced to C/P based on a request of C/P.

C/P has much interest in the tunnel design and made inquiries in the technical seminars. The C/P requested to hold continuous technical lecture to clean up their questions after the final seminar. Thus, an additional technical lecture was held with additional materials.

7.3.3 Evaluation of Technical Transfer

C/P seems to obtain basic understanding on investigations and design of tunnel through developing guidelines and making presentations for the final seminars by them. However, there are various approaches for tunnel design and investigation depending on geological and geotechnical conditions in target tunnel sites. Therefore, accumulation of experiences and techniques is required for actual proper investigations and designs.

7.4 Recommendations for Future

Recommendations on necessary approaches for RDA to conduct road tunnel projects in future by utilizing the outcomes of this project are mentioned below.

(1) Accumulation of Experiences and Improvement of Rock Mass Classification System

In the tunnel planning, it is necessary to estimate geological and geotechnical conditions of the target site from limited information obtained by geological investigations and previous tunnel projects and to design the tunnel by using rock mass classification system. Therefore, it is important to accumulate information about previous tunnel constructions under various geological conditions. In fact, it is a weak point of Sri Lanka due to lack of experience on large tunnel design and investigation. Furthermore, it is necessary to continuously update and improve the rock mass classification system by comparing geological conditions estimated by the preliminary investigations with actual geological conditions observed at the tunnel construction stage. It is also required to improve accuracy of impact assessment on groundwater/surface water by accumulation of case examples.

It should be welcomed that the C/P acknowledged the necessity of establishing tunnel group in the RDA design division at the end of this project. To actualize this idea, supports for accumulation of experiences and information, establishment of tunnel group and continuous updating of guidelines are required.

(2) Improvement of Geological Investigation

Default position of RDA is a contractee who manages and supervises the geological investigation. In reality, drilling survey and seismic refraction prospecting are respectively conducted by private company and National Building Research Organization (NBRO) as a sub-contract work in this project. However, there is a concern that the geological conditions may be underestimated because technical level of the geological investigation of sub-contractors is low. Therefore, it is necessary to improve the geological investigation technique of the private sector and NBRO for proper tunnel planning. Furthermore, it is necessary to strengthen linkage between tunnel project teams and road design divisions in RDA by setting up a special section for geological investigations and analysis in RDA.

(3) Capacity Improvement of F/S

The capacity for route planning and structure selection at feasibility study (F/S) considering economic efficiency and environmental impacts should be improved. Continuous technical transfer is important for RDA to improve a capacity for F/S.

(4) Tunnel Design and Construction Plan

As mentioned in section 6.3, necessary information and design conditions are insufficient for detailed design, construction plan and cost estimation. It is necessary to encourage RDA to recognize and collect the required information so that RDA can utilize the information in future tunnel projects.

Therefore, it is important to improve collaboration and information sharing among relevant RDA sections. In addition, activities such as obtaining knowledge from other specialized agencies and collecting previous tunnel project data are required. It is especially recommended to record the results of investigation, design and construction in the future tunnel projects and to share the lesson-learned among the relevant agencies.