Technical Assistance for Improvement of Capacity for Planning of Road Tunnels

Japan Sri Lanka

Guideline

for

Rock Mass Classification System

February 2018

Road Development Authority (RDA)

Japan International Cooperation Agency (JICA)

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1 Introduction

Tunneling work includes constructing a tunnel and associated temporary works of setting up access roads and working yards. The design of a tunnel is not similar to that of a plant or structure and it is difficult to assume accurate geological conditions, properties and variability of rock mass along the tunnel. Tunnel design is generally carried out on the basis of less reliable geotechnical assumptions in comparison with other designs in the preliminary stage. To minimise the risks in tunneling work, the following tasks are recommended to be performed:

- To review existing information and conduct site investigations to assume accurate geological and geotechnical conditions along the tunnel alignment.
- To implement an inspection of excavation surface in the construction stage to compare the actual geological and geotechnical conditions with the assumed conditions in the design stage.
- To monitor deformation of the excavated tunnel cavern and stress and/or strain of installed measurement devices around the tunnel to detect any abnormalities in order to conduct countermeasures if necessary.
- To review and modify the specification of the rock mass classification system and selection of tunnel support based on the actual geological and geotechnical conditions confirmed in the construction stage, if necessary.

Implementing tasks to assess and manage changes in conditions and the adequacy of the tunnel design and ground support are required in order not to create a risk to safety. This may include suspension of tunneling works to reassess and review the changed conditions, the cost performance and safety control measures. The flow chart of tunneling work is shown in Figure 1-1.

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

Both empirical methods and numerical modelling are recommended to conduct a stability analysis of the tunnel in each stage including the preliminary stage of the project. The results obtained from both methods can be compared, and select the most suitable support system. The empirical method can be used when the results obtained through the both methods indicate resemble results. It is considered that the empirical methods is be useful, in the absence of sufficient information for numerical analysis, mainly in the preliminary stage of the project.

The empirical method of rock mass classification system is proposed to be employed in Sri Lanka in this guideline. This system is proposed considering geological condition of Sri Lanka on the basis of various and numerous data obtained in civil engineering projects including tunnel projects.

Some empirical methods have been utilised for classification of rock mass to propose the tunnel support system. The empirical method of Rock Mass Rating (RMR), Q system is often utilised for the rock mass classification of tunnel projects. The proposed system includes visual judgement of the rock material and rock mass with a relatively wide range of definition contrary to the RMR or Q system. One of the empirical methods of Geological Strength Index (GSI) also includes similar visual judgement in the system.

The Relationship or correlation of these systems with the proposed rock mass classification have been studied and some correlations have been proposed in some papers, however these studies haven’t met with thee conclusion.

The rock mass classification system for a road tunnel project of Sri Lanka appears to be finalized on the basis of accumulated field data of the coming road tunnel projects. The Concept of the proposed system might be a part of the finalised system of Sri Lanka.

In-situ stress of the rock mass around the tunnel is an important factor when thickness of overburden above the tunnel level exceeds some amount. This factor is not included in the proposed system, however phenomena caused by this factor can be roughly assumed by using a simple calculation. The degree of acting force and durability of rock mass against the in-situ stress can be roughly estimated. The procedure is described in this guideline. A sequential excavation method which can cope with the in-situ stress by immediate installation of support is commonly applied in these days. However, the in-situ stress is closely measured to check the suitability of the tunnel supporting system in general.

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

“Rock Mass Classification and Tunneling works” is shown in Figure 1-2.

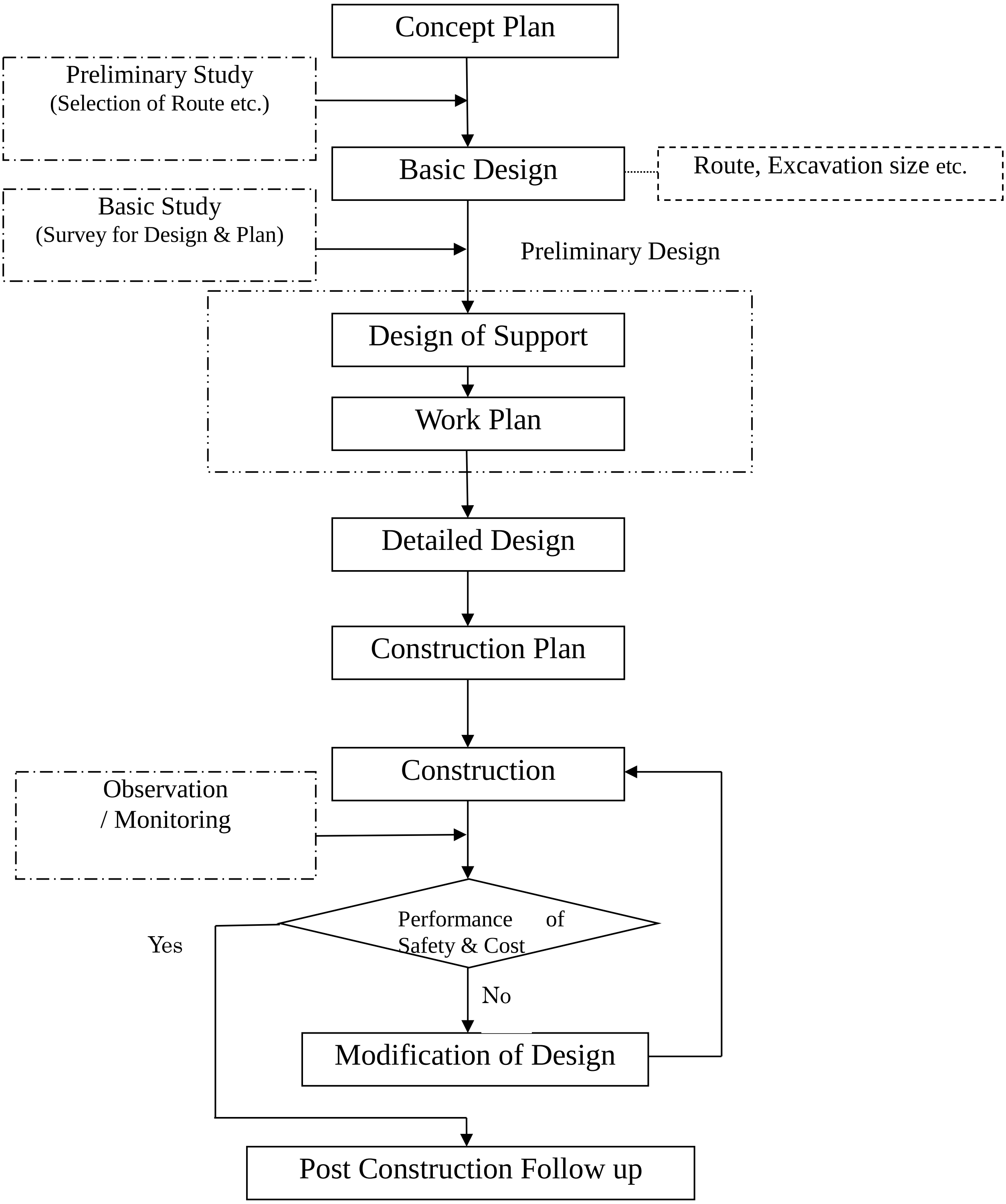


Figure 1-1 Flow Chart of Tunneling Work

(Source: JICA project)

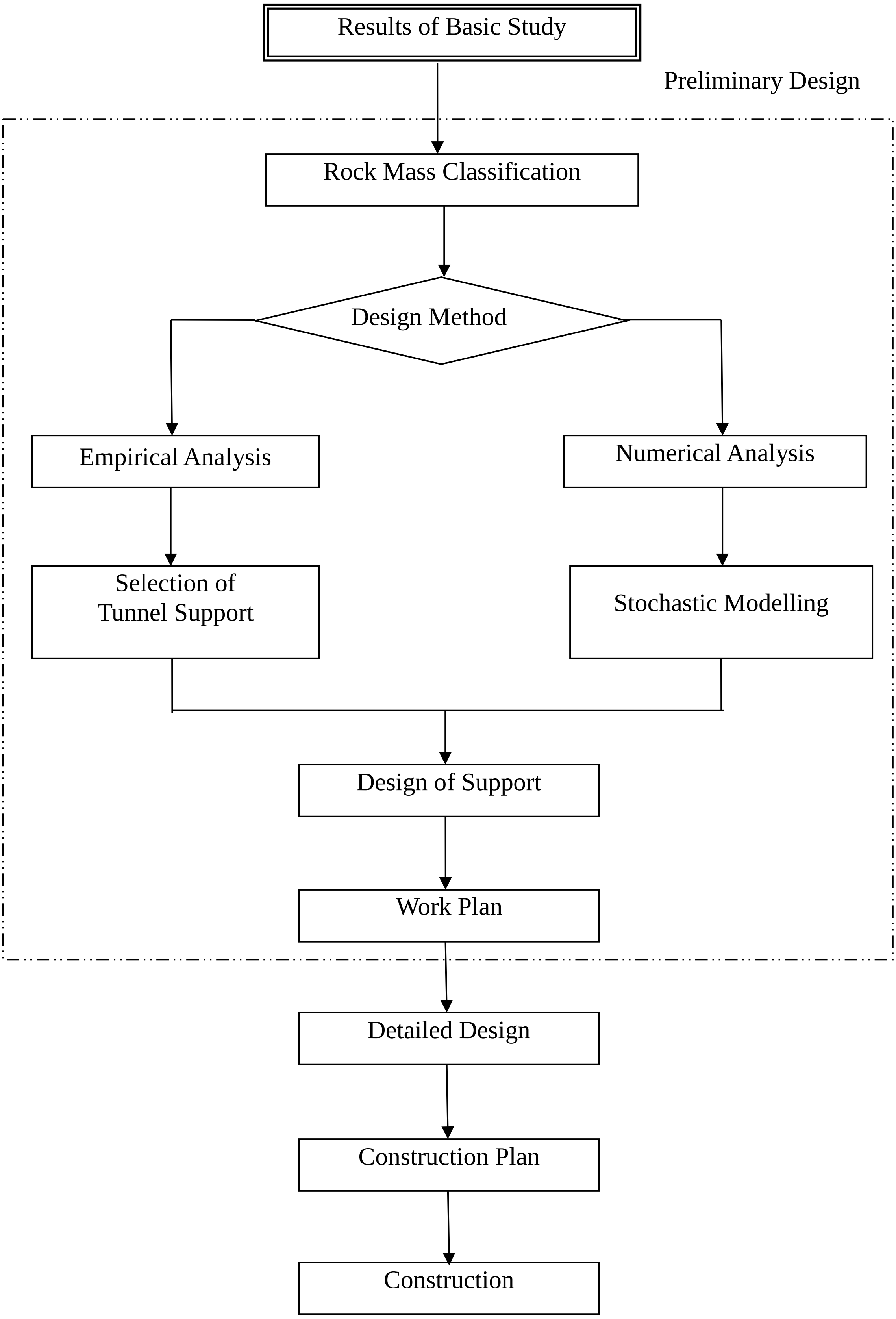


Figure 1-2 Rock Mass Classification and Tunneling Works

(Source: JICA project)

2 Geological and Geotechnical Condition of Sri Lanka

2.1 Sri Lanka

2.1.1 Geology

The Geology of Sri Lanka consists mainly of crystalline Precambrian metamorphic rocks and they are subdivided as Highland Complex, Wanni Complex,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 lies east of the Highland Complex and consists of metamorphosed granitoids, charnockitic gneisses, migmatites, microcline-bearing quartzofeldspathic rocks, amphibolite, 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 the “Kadugannawa Complex” composed of amphibolite or hornblende biotite gneisses and migmatites is located in the central part of the country.

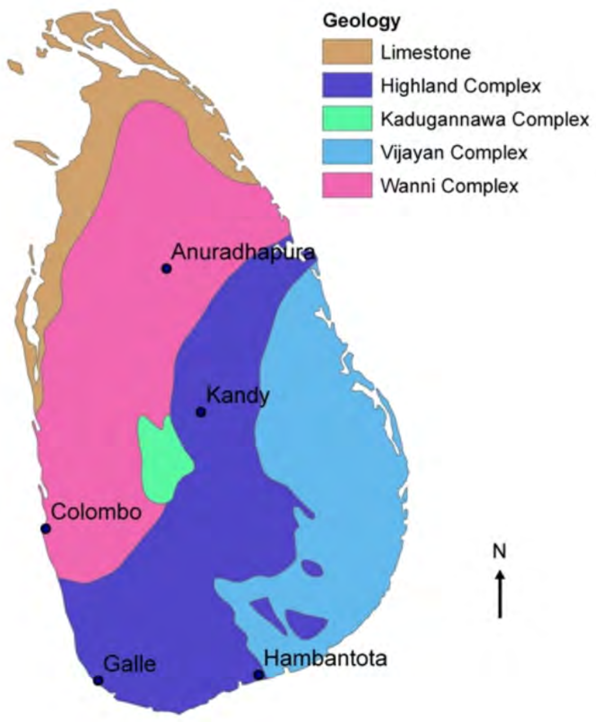


Figure 2-1 Geological Map of Sri Lanka
(Source: ESDAC - European Soil Data Centre)

2.1.2 Geotechnical Conditions

The geology of Sri Lanka consists mainly of crystalline Precambrian metamorphic rocks and they are hard and compact in a fresh condition, however the top portion of the rock is cracky and relatively soft due to weathering especially above the groundwater table. Overburden of talus deposits, residual soils, sand and gravel with clayey materials are distributed covering the bedrock of the Precambrian age.

The bedrock is generally hard and compact in mountainous area, however deep weathering is often observed in hilly areas. Geological investigation including core drilling is highly recommended to be performed along the tunnel alignment. A hydrogeological survey is also recommended to be conducted along the tunnel alignment especially in mountainous areas where pressurized groundwater is assumed to exist. Sufficient hydrogeological data shall be collected and analyzed prior to any construction works.

2.2 Project Area

2.2.1 Geology

The geology of the project area is mainly composed of the Kadugannawa Complex Wanni Complex is distributed in limited area of north-western part of the study area. The Kadugannawa Complex is composed of amphibolite or hornblende biotite gneisses and migmatitesis, and The Wanni Complex consists of metamorphosed granitoids, charnockitic gneisses, migmatites, microcline-bearing quartzofeldspathic rocks, amphibolite and biotite gneiss which more or less similar to those of the Vijayan Complex. Distribution of rock types is shown in Figure 2-2.

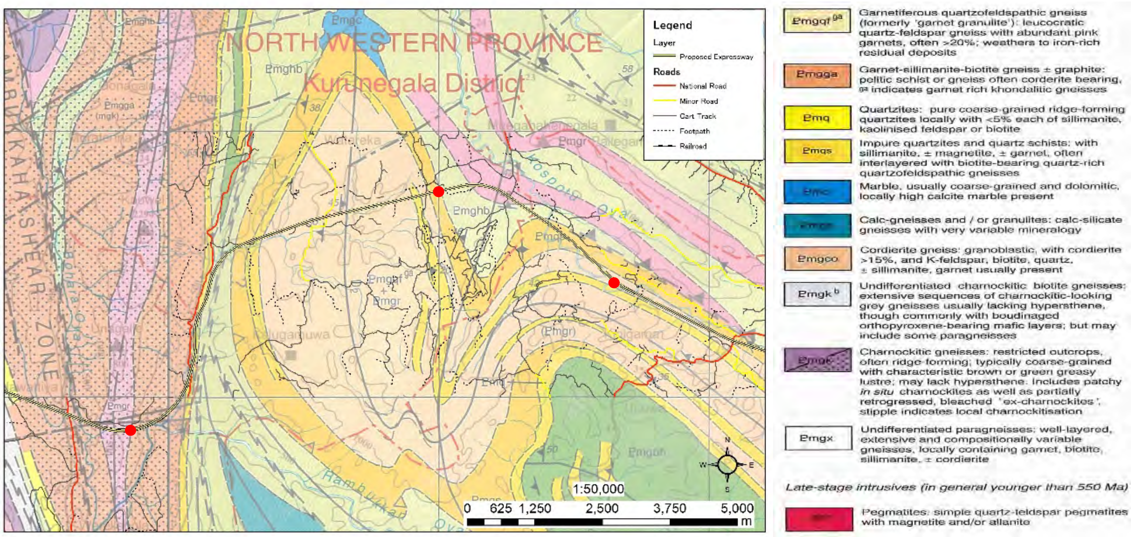


Figure 2-2 Geological Map around Tunnel sites
(Source: Geological Survey of Sri Lanka)

2.2.2 Geotechnical Condition

The geology of Sri Lanka consists mainly of crystalline Precambrian metamorphic rocks and they are subdivided into several divisions. Geology of the project area consists mainly of crystalline Precambrian metamorphic rocks of the Kadugannawa Complex and Wanni Complex, and they are hard and compact in fresh condition, however the top portion of the rock is cracky and relatively soft due to weathering as confirmed during the geological investigation especially above the groundwater table. Overburden of talus deposits, residual soils are distributed covering the bedrock of Precambrian age the same as other areas of the island.

The bedrock is moderately hard to relatively soft along the tunnel alignment, and intercalation of highly weathered and/or fractured zones is locally distributed in the area.

The groundwater table is confirmed near the topographical boundary between the lowland area and the hilly area, and highly weathered zones are distributed above the groundwater table in general. Occurrence of groundwater appears to be limited in the hilly area and relatively abundant in the lowland area, however quality of groundwater seems to be better in the hilly area. A hydrogeological survey has been executing along the tunnel alignment due to existence of groundwater wells in hilly area. Severe constraints for the excavation of tunnels appear not to be expected in hydrogeological aspects for this project.

However, any unexpected phenomena may occur at any locations in case of insufficient data collection. Sufficient hydrogeological data shall be collected and analyzed so that any unexpected phenomena will not arise prior to any construction works.

3 Proposed Rock Mass Classification System in Sri Lanka

3.1 Concept

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

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

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

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

The classification systems are not suitable for use in elaborated and final design, particularly for complex underground openings in poor rock condition. The rock mass classification systems were designed to act as an engineering design aid, and were not intended to substitute engineering judgment as mentioned above.

Mathematical solutions of empirical methods and numerical modelling can be applied to classify the rock mass, and both empirical methods and numerical modelling are recommended to conduct a stability analysis of the tunnel for selection of the most suitable support system. However, the empirical method can be applied when the results obtained through the both methods indicate similar results, and in the case of insufficient information for numerical analysis, mainly in the preliminary stage as mentioned above. Therefore, one of the empirical methods is proposed for rock mass classification system considering characteristics of the rock mass in Sri Lanka in this guideline.

3.2 Parameters

Various important parameters have to be considered in order to describe a rock mass accurately 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) geological structures (folds and faults).

(5) in-situ stress

(6) groundwater pressure and the flow

3.2.1 Rock Grade

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

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

Rock grade is chiefly determined on the basis of the strength of the cylindrical core sample obtained from the rock mass, and degree of weathering. Rock quality designation (RQD) can be applied to indicate spacing of the discontinuities due to weathering. Rock grade is classified into six states from A to F, and described in detail in Table 3-1.

3.2.2 Condition of Joints

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

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

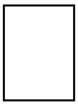
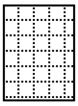
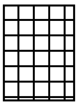
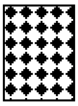
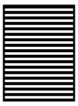
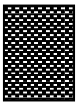
The strength of intact rock mass without discontinuities is supposed to be equivalent to that of the rock constituting the rock mass. However, joints and cracks are generally distributed in the rock mass, and properties of the rock mass highly depend on the discontinuities like joints and cracks.

Condition of joints and cracks are classified into four categories of “a” to “d” from close contact to open, and five stages of their spacing as shown in Table 3-2 and Table 3-3.

3.2.3 Proposed Rock Mass Classification System

Rock mass classification can be assumed on the basis of the rock grade and condition of joints and cracks described above. Rank S1 to S7 of rock mass classification is proposed on the basis of said parameters in this guideline as shown in Table 3-4 and Table 3-5. This rock mass classification system is studied in further chapters considering other rock mass classification systems.

Table 3-1 Classification of Rock Grade

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

(Source: JICA project)

Table 3-2 Spacing of Joints / Cracks

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

Table 3-3 Condition of Joints / Cracks

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

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

Table 3-4 Classification of Rock Mass based on Combination of Parameters

	A					**B**					**C**					**D**					**E**					**F**				
	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV	V
a	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: JICA project)

Table 3-5 Rock Mass Classification System

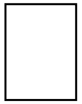
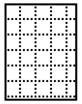
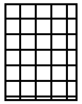
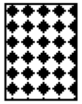
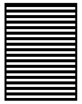
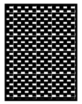
Rock Mass Classification	Condition of Rock Mass
S1	Fresh and hard, no deterioration in the rock-forming minerals. Crack spacing lager than 50 cm. Rocks are cohesive, no deterioration nor discoloration. Rock is hard and compact in light colour. Crack spacing about 15-50cm. Limonite stains along cracks.
S2	Relatively hard. Biotite and plagioclase are somewhat deteriorated. Crack spacing about 5-30 cm. Very thin clay is intercalated along the opening.
S3	Broken when struck by hammer blows. Deterioration of plagioclase developed. Crack spacing smaller than 15 cm. Clay is rarely intercalated along the opening face.
S4	Biotite turns golden color, but quartz particles are hard. Plagioclase is deteriorated. Broken into pieces when struck by hammer. Crack spacing smaller than 5 cm.
S5	Can be broken by a hammer blow and by hand. Biotite turning to golden color, and brown in the periphery. Particles are hard, forming small, sand-like pieces. Apparent spacing of cracks becomes wider.
S6	Broken by hand, it becomes sand-like remaining crystal of quartz and potassium feldspar. Mica loses its crystal form and plagioclase is mostly deteriorated. Apparent spacing of cracks becomes even wider.
S7	Broken by hand, mostly becomes powder, expect for partly sand form. Most feldspar is deteriorated and becomes clayish soil. Original joint planes become indistinguishable.

(Source: JICA project)

3.2.4 Properties of Classified Rock Mass

Properties of rock grade, one of the parameters of rock mass classification system, is estimated on the basis of data accumulated in a large number of projects. Properties of classified rock mass are also proposed considering accumulated data and other information. Classification of rock grade and properties and properties of each classified rock mass are shown in Table 3-6 and Table 3-7. Photos of each classified rock mass are shown in Table 3-8.

Table 3-6 Classification of Rock Grade and Properties

Class		Criteria for Judgment	Drilled Core sample	Weathering (Alteration)	Seismic Velocity (m/sec)	Unified Compsv. Strength(MPa)
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.)	More than 4000	More than 100
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.)	3000 to 6000	50 to 100
C		Rock often becomes broken when struck by hammer. Rock pieces keep almost intact when broken. Joints are slightly to moderately weathered in general.	Cylindrical core recovery and/or are 40% to-70%. Some fragmental pieces of rock is recovered. Cranky in general.	Moderately (Ratio of discoloration is less than half)	2000 to 4000	25 to 50
D		Broken by hand and slightly penetrated by hammer blow. Joints are generally not clear mainly due to highly weathered condition.	Cylindrical core recovery and/or RQD are less than 40%. Fragmental core with sand & clayey materials is recovered.	Highly (Ratio of discoloration is more than half)	1000 to 3000	10 to 25
E		Broken and/or squeezed by finger, remaining particles of quartz and feldspar. Proportion of broken pieces is generally 30-50% and 20-50% in powder form.	Pebble, sand and clay samples recovered in general. Cylindrical core recovered mainly in clayey layer.	Completely (All discolored, Texture of rock can be confirmed)	1000 to 2000	Less than 10
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)	Less than 1000	—

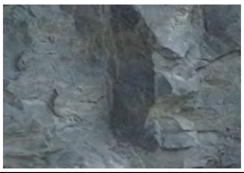
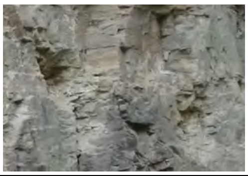
(Source: JICA project)

Table 3-7 Properties of each Classified Rock Mass

Rock Mass Classification	Modulus of Deformation (MPa)	Shear Strength τ (MPa)	Friction Degree Φ (°)	Seismic Velocity 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: JICA project)

Table 3-8 Photos of Classified Rock Mass (for reference)

Rock Mass Classification	Condition of Rock Mass		
S1			
S2			
S3			
S4			
S5			
S6			
S7			

(Source: JICA project)

3.3 Design of Tunnel Support

3.3.1 General

Design of tunnel support is performed generally on the basis of classification of rock mass. Tunnel support of classified rock mass of S1 to S7 is proposed in Table 3-9. The table is subject to change when the classification of rock mass is modified due to changes of actual condition of the site.

Table 3-9 Rock Mass Classification System and Tunnel Support

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

(Source: JICA project)

3.3.2 Project Area

Design of tunnel support is performed generally on the basis of classification of rock mass as mentioned above. Tunnel support of each classified rock mass of S1 to S7 of the project area can be applied to Table 3-9.

4 Discussion

4.1 Empirical Rock Mass Classification Systems

All these systems have a quantitative estimation of the rock mass quality linked with empirical design rules to estimate adequate rock support measures.

4.1.1 Rock Mass Rating

The RMR, published by Bieniawski in 1973, is one of the most commonly applied and performed ratings 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 4-1.

Table 4-1 The 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		
		$p_w / \sigma 1$	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; $\sigma 1$ = 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 4-2.

Table 4-2 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 appears to be unclear since no input parameter for rocks stresses in the RMR system.

4.1.2 Q 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,

andSRF = ratings for the rockmass stress situation. See Table 4-3 and Table 4-4.

Table 4-3 The 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
Partly or no wall contact	**FILLING OF:**	**Type**		*Wall contact before 10 cm shear*	*No wall contact when sheared*
	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 4-4 The 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 for the tunnel or cavern. The Excavation Support Ratio (ESR) of the Q value defines the rock support in a support chart as shown in Table 4-5.

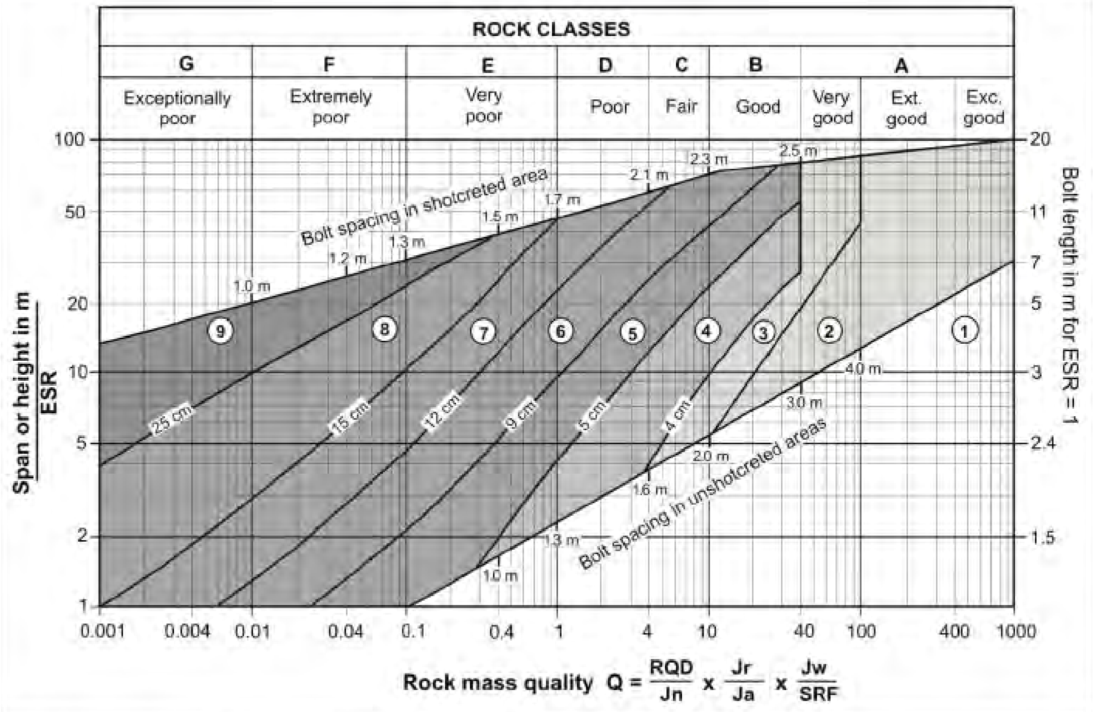
The Q system has several limitations, and it is said to work between Q = 0.1 and Q = 40 for tunnels with spans between 2.5m and 30m in general.

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

Table 4-5 The Support Chart Used in the Q 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)

4.1.3 Geological Strength Index

The Geological Strength Index (GSI) is proposed by Hoek et al. 1995, and it is one of the rock mass classification systems that has been developed in engineering rock mechanics to meet required reliable input data related to rock mass properties as input for numerical analysis or closed form solutions for designing tunnels, slopes or foundations in rocks. The geological character of the rock material, together with the visual assessment of the mass is used as direct input parameters for the prediction of rock mass strength and deformability. This approach is considered as a mechanical continuum without losing the characteristics of mechanical properties of the rock mass.

The GSI classification system is based on the assumption that the rock mass contains a sufficient number of “randomly” oriented discontinuities that behave as a homogeneous isotropic mass. Therefore, the GSI system should not be applied to those rock masses in which there is a clearly defined dominant structural orientation or structurally dependent gravitational instability.

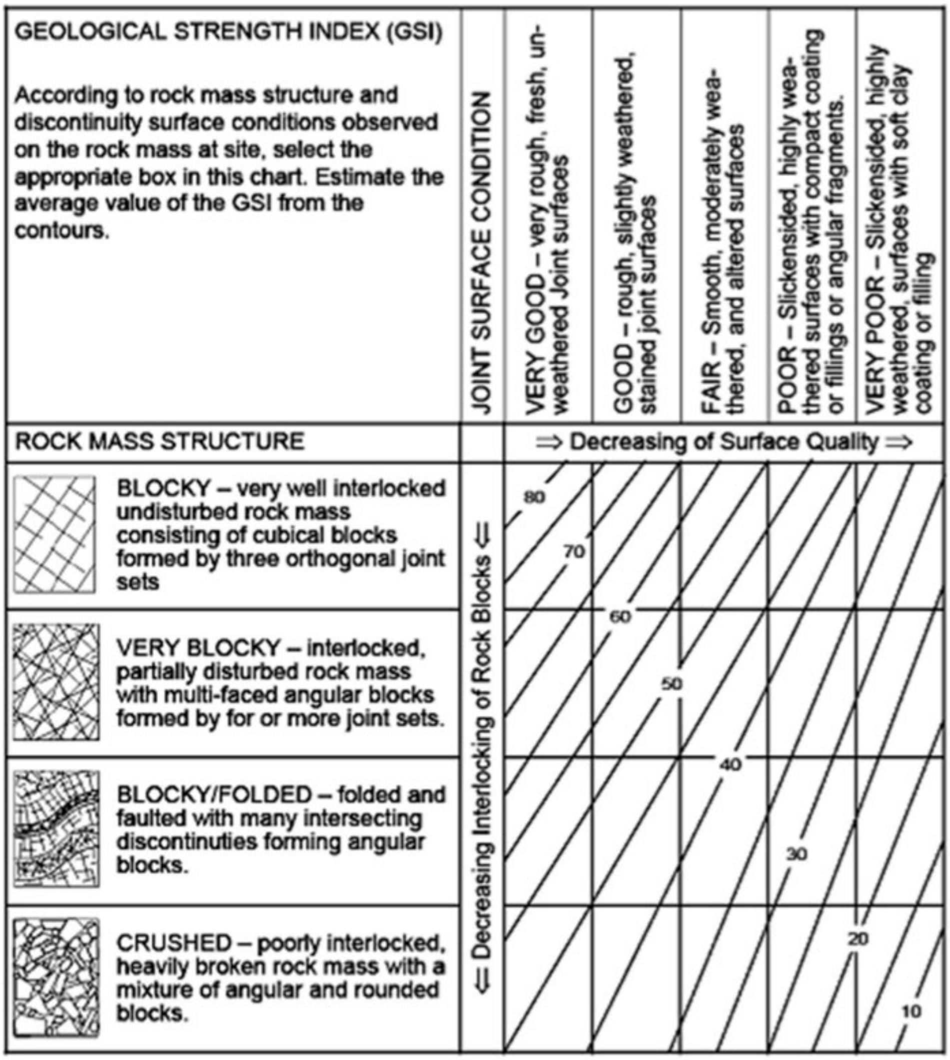


Figure 4-1 Estimate of GSI Based on Visual Inspection of Geological Conditions

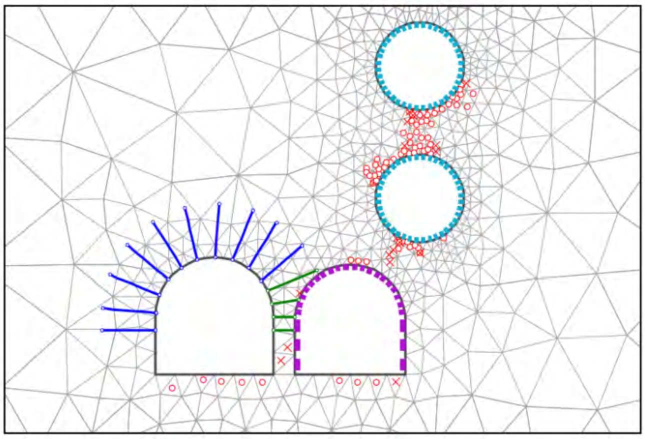
(Source: Hoek and Brown, 1997)

4.2 Numerical Analysis Method

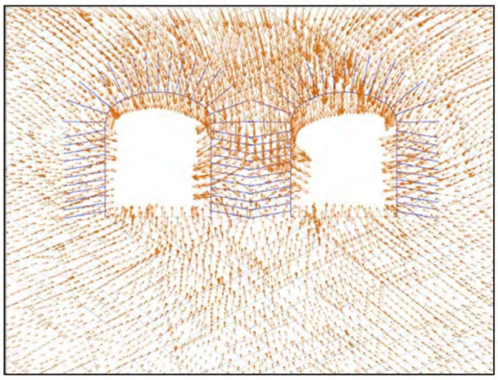
Numerical analysis is often applied in these days, and the easy operation of the software products has found broad acceptance, and they are used in geotechnical engineering practice.

4.2.1 Finite Element Method

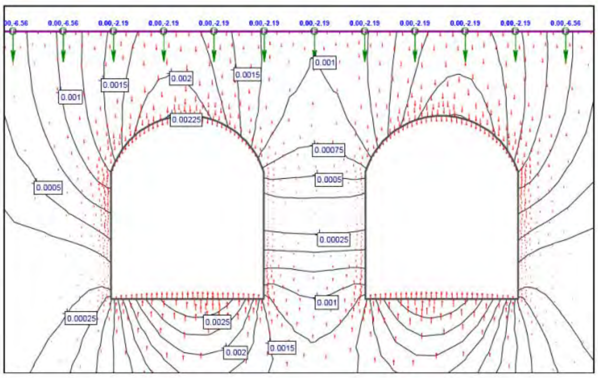
The finite element method (FEM) is a numerical method commonly applied for solving problems of engineering and mathematical physics. The aim of the finite element method analysis is to verify the empirically evaluated tunnel support design. The FEM software module is often applied to evaluate induced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



Design of Support System



Graphical Result of Discrete



Ground Displacement Contours calculated by FEM

Figure 4-2 Numerical Analysis and Tunnel (Finite Element Analysis method)

(Source: Analysis of tunnels by Design and Programming)

4.2.2 Others

Other analysis methods are the Boundary Element Method (BEM), Finite Difference Method (FDM) and the others. They are sometime applied for analysis of induced stresses and maximum deformation for the excavated portion of tunnels, although they are mainly developed and/or related to the finite element method. The finite element method (FEM) is to be chiefly applied for analysis of induced stresses and maximum deformation for the excavated portion of the tunnels.

4.2.3 Consideration

Careful choice and utilization of these powerful tools are necessary to avoid wrong calculation and simulation results based on careful inspection of model set-up and in-depth analysis. Therefore, comparison with results of other methods such as the empirical methods is strongly recommended.

4.3 Proposed System and Others

4.3.1 Correlation

Several systems are used for that rock mass classification to perform design of tunnels, and RMR and Q systems are frequently used among the systems developed on the basis of these two systems. The main rock mass classification systems of RMR and Q systems use similar rock mass parameters.

The quality of the ground is calculated differently in the two systems and several correlations 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)

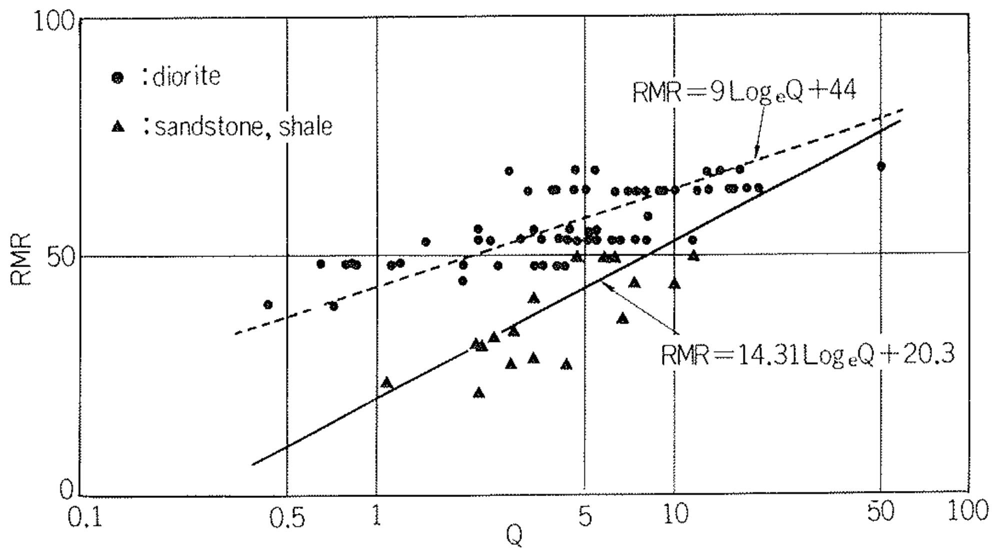


Figure 4-3 Correlation between RMR and Q-system (Diorite)

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

The proposed rock classification system is somehow similar to the Geological Strength Index (GSI). The geological character of the rock material, together with the visual assessment of the mass is used as direct input parameters for the prediction of rock mass strength.

4.3.2 Design and Construction Stages

Mathematical solutions of the empirical method and numerical modelling are applied to evaluate the stability of a tunnel. Empirical method is based on the experience obtained in the course of research works. Numerical modelling is using various modelling techniques and solving very complex problems, and selecting the most suitable ground support system.

The empirical method including the proposed system is used, in the absence of sufficient information for numerical analysis, mainly in the preliminary stage of the project.

4.3.3 Consideration

Both empirical method and numerical modeling are recommended to conduct a stability analysis of the tunnel in each stage of the project. The results obtained from the both methods can be compared for selection of the most suitable support system. The empirical method can be used when the results obtained through the both methods indicating similar results.

4.4 Regional Geotechnical Conditions

4.4.1 In-situ Stress

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

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

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

Considering an element of the rock mass at a certain depth below the surface, the weight of the vertical column of rock resting on this element is the weight of the overlying rock mass, and the weight of the overlying rock mass can be simplified to be in-situ stress as mentioned above.

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

Ta = Po /qu,

Po =γ x H

Where:

Ta = in-situ stress intensity

Po = in-situ stress (simplified)

γ = rock unit weight

H = tunnel depth below ground surface

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

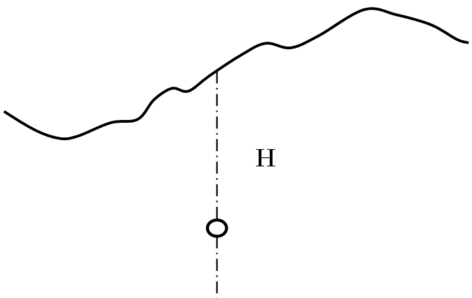


Figure 4-4 Tunnel and In-situ Stress

(Source: JICA project)

As a result of analysis based on accumulated data regarding this value of in-situ stress intensity, the condition of the stress of rock mass is substantially changed from two (2) to four (4) of the value. Deformability of the underground opening due to in-situ stress appears to be less when the value exceeds four (4), however deformation and squeeze of the rock mass are often observed when the value is below two (2) as shown in Figure 4-5.

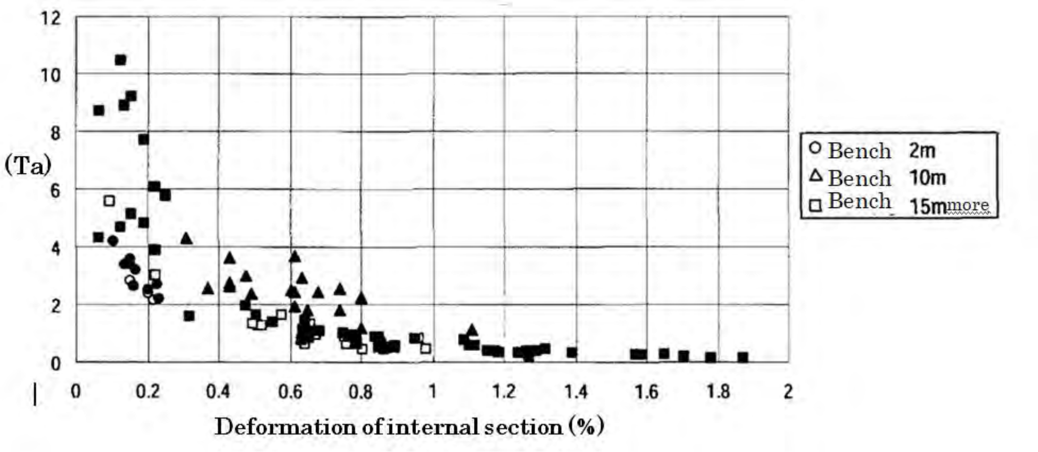


Figure 4-5 Relationship between In-situ Stress and Deformation of Internal Section
(Source: Rock Mass Classification and its Application, Doboku-kogakusya)

4.4.2 Hydrogeology

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

Inflow of groundwater to the tunnel is observed during and even after excavation of the tunnel. The natural groundwater level before the excavation is generally lowered after starting tunnel excavation, and some amount of groundwater flows into the tunnel. A gush of groundwater inflow to the tunnel is also observed in the course of the tuneling work. Steady inflows of groundwater are often recorded after construction of the tunnel.

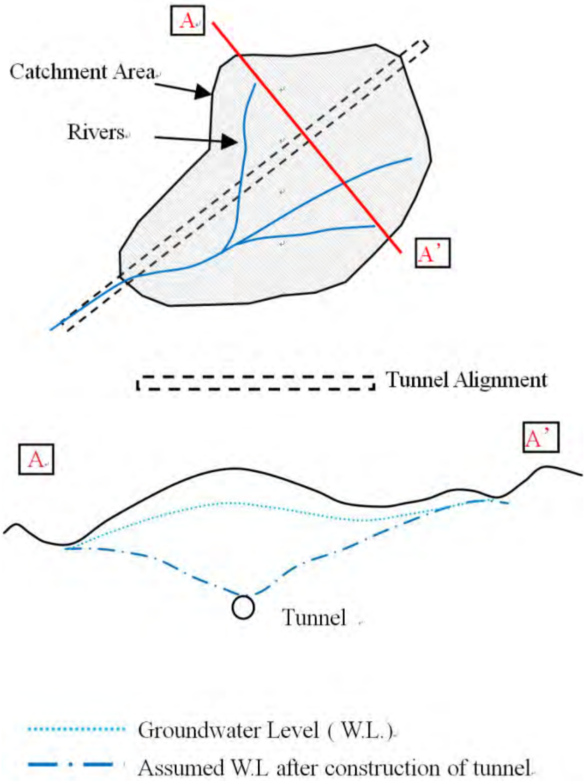


Figure 4-6 Hydrogeology around Tunnel
(Source: JICA project)

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

The hydrogeological condition around the tunnel cavern is changeable from time to time when the excavation has progressed. There are two types of hydraulics conditions of unsteady-state flow and steady-state flow around the tunnel. An unsteady-state flow is generally observed during the excavation stage. The hydraulic condition of steady–state flow is generally observed after completion of the excavation of the tunnel.

4.5 Excavation Method

4.5.1 General

A new excavation method for an underground opening is proposed and applied to improve the progress, and safety of works since excavation of the underground opening started. Significant progress of excavations of underground openings has been made since the blasting method has been applied after dynamite was invented. A conventional method using drilling and blasting is still applied in some projects. A tunnel boring machine (TBM) and the sequential excavation method of the New Austrian Tunneling Method (NATM) are often applied as a result of the latest concept of rock mechanics to improve the progress and safety of the tunneling works in these days.

Supporting measures such as pre-supports including spiles and forepiling are applied through and ahead of the tunnel surface. These recent excavation methods are mainly proposed on the basis of the latest concept of rock mechanics including rock mass classification system. A conventional method, tunnel boring machine and sequential excavation method are described as follows.

Conventional method

Drilling and blasting are applied in a conventional excavation method, and a basic approach is to drill a pattern of small holes, load them with explosives, and then detonate those explosives thereby creating an opening in the rock. The blasted and broken rock (muck) is then removed and the rock surface is supported so that the whole process can be repeated as many times as necessary to advance the desired opening in the rock.

Tunnel Boring Machine (TBM)

This conventional excavation method had been applied up to the 1960’s, however the actual advance rates were still quite low. Some progress was made, but it was slow because the machines attempted to remove the rock by grinding it rather than by excavating it. Tunnel boring machines (TBM) excavate rock mass in a form of rotating and crushing by applying enormous pressure on the surface with large thrust forces while rotating and chipping with a number of disc cutters mounted on the machine face.

Sequential Excavation Method

Tunnel support is proposed on the basis of rock mass classifications and equivalent dimensions of the tunnel. The concept of the sequential excavation method is for the application of sequencing of excavation and quick initial support installation. Disturbance of the rock mass around the tunnel can be limited. The New Austrian Tunneling Method (NATM) is proposed on the basis of this concept; this method has been globally applied in recent years.

4.5.2 Conventional Method

Explosives are used for this tunneling method and drilling rigs are used to bore blast holes on the tunnel surface for blasting. Waste rocks and soil are transported out of the tunnel before further blasting. This excavation method commonly generates a higher level of vibration during construction, and this method is applied to stable rock mass in general.

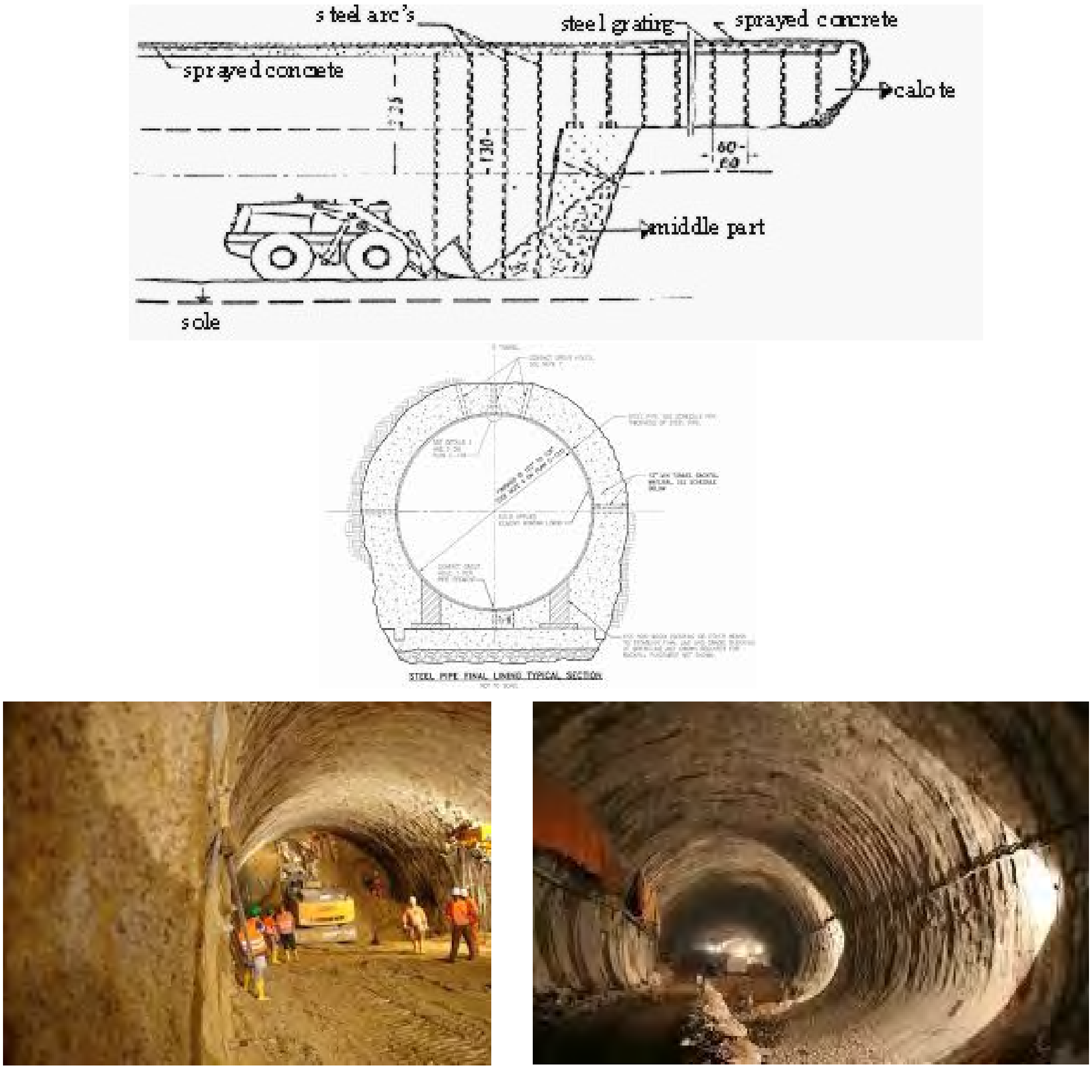


Figure 4-7 General Conditions of Conventional Excavation Method

(Source: International Federation of Syrvey, Database and Galley)

4.5.3 Tunnel Boring Machine (TBM)

Bored tunneling by using a Tunnel Boring Machine (TBM) is often used for excavating long tunnels. The TMB method requires the selection of appropriate equipment for different rock mass and geological conditions. The TBM is generally suitable for excavating tunnels in competent rocks that can provide adequate geological stability for boring a long section tunnel without structural support.

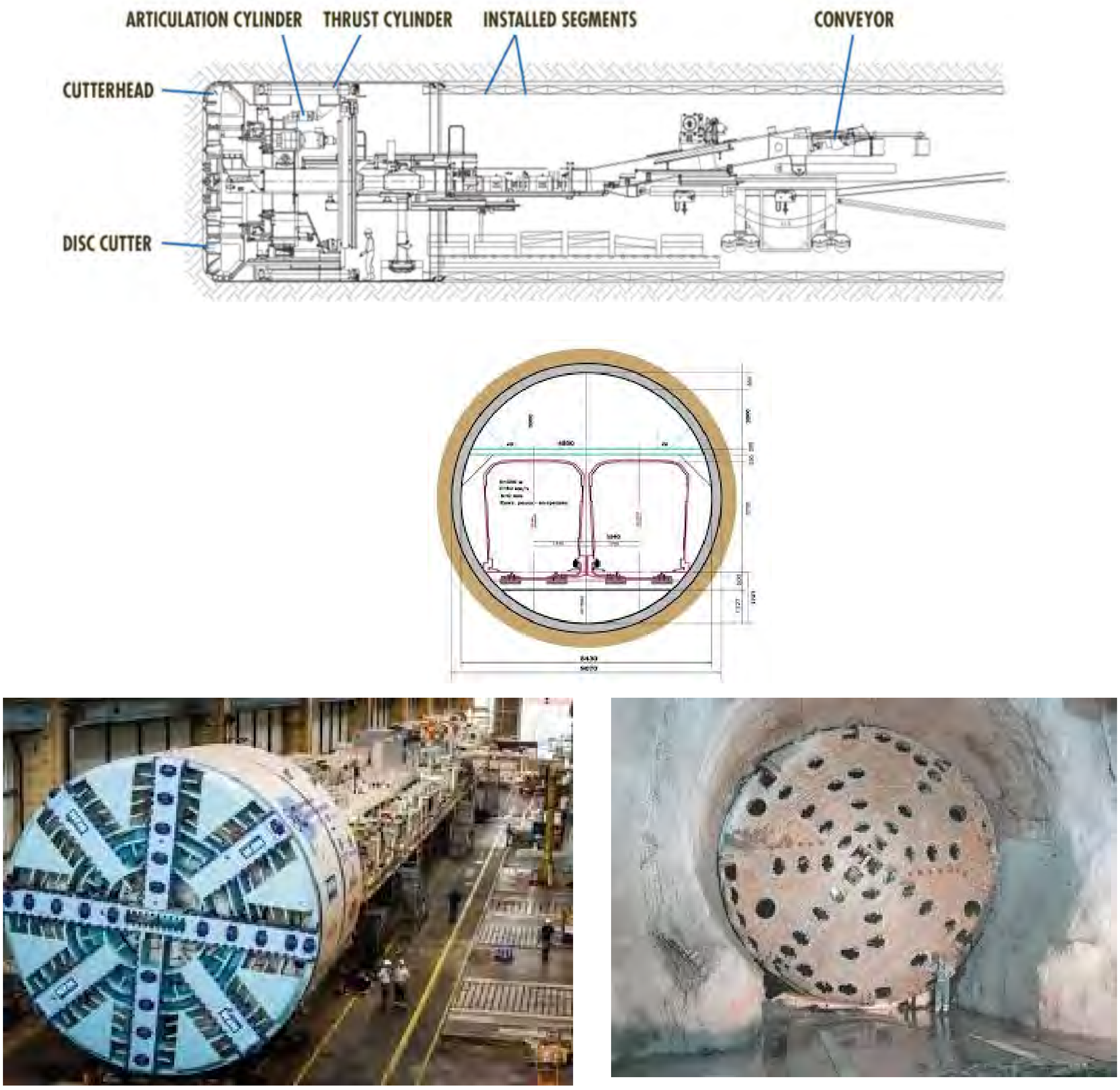
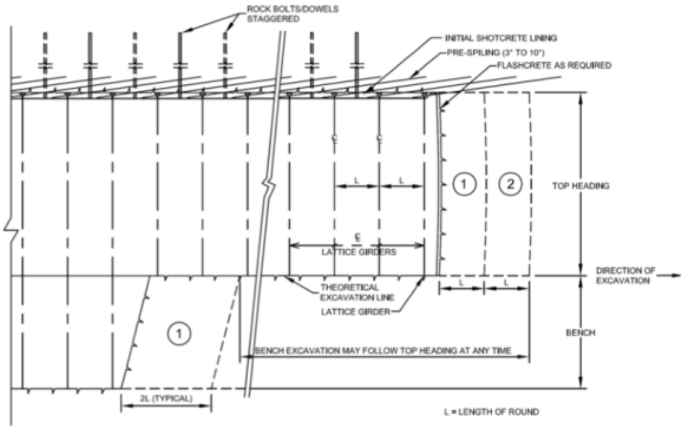


Figure 4-8 General Conditions of Tunnel Boring Machine (TBM)
(Source: International Database and Gallery of Structures)

4.5.4 Sequential Excavation Method

This method includes the New Austrian Tuneling Method (NATM). The tunnel is sequentially excavated and supported and the initial ground support is provided by shotcrete in combination with fabric reinforcement, steel arches of segments, and ground reinforcement, if necessary. Some mining equipment such as loadheaders and backhoes are commonly used for tunnel excavation. NATM is said to be one of the Sequence Excavation Systems (SEM).



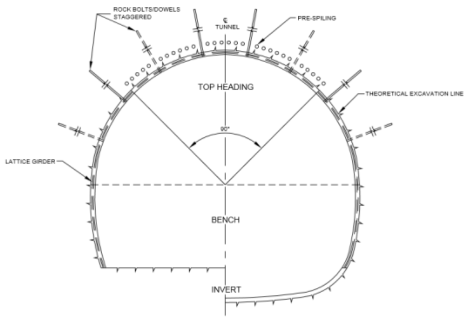


Figure 9-7 Prototypical Excavation Support Class (ESC) Cross Section

Figure 4-9 General Conditions of New Austrian Tunneling Method (NATM)

(Source: International Database and Gallery of Structures)

4.6 Survey and Monitoring

Monitoring of ground deformations in tunneling is a principal means for selecting the appropriate excavation and support methods for ensuring safety during tunnel construction.

Various types of ground deformation measurements are often used in tunneling in obtaining ground measurements and their subsequent evaluation.

Examples of ground deformation monitoring and their application in tunnel design and construction are illustrated in Figure 4-10.

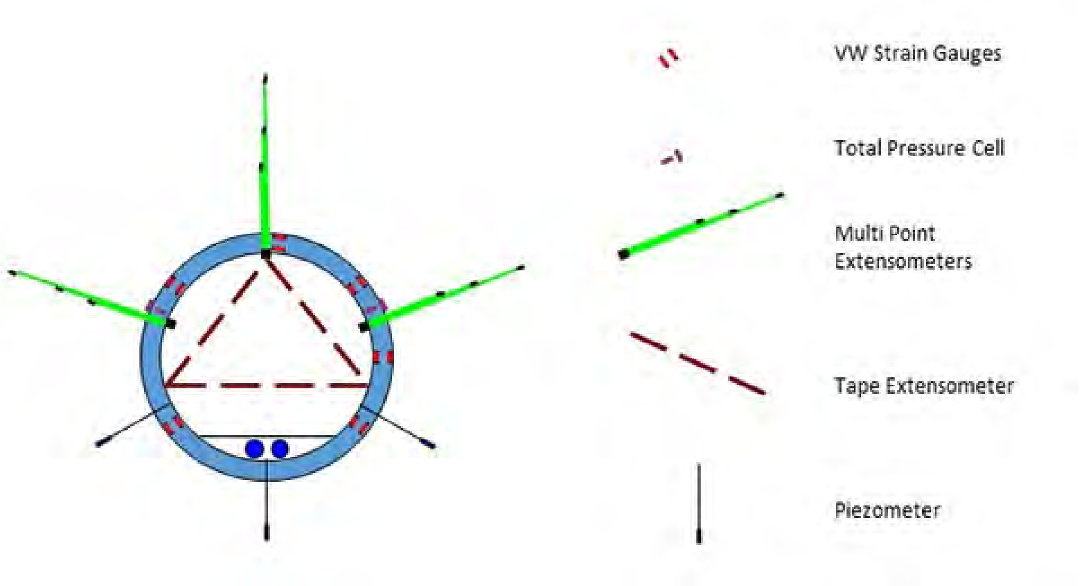


Figure 4-10 Typical Monitoring System for Tunnel
(Source: JICA project)

4.6.1 Modification of rock mass classification system

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

The rock mass classification system shall also be modified as results of detailed geological mapping of the excavation face and site investigations are carried out in the construction stage, and study results of other project sites can be considered.

4.6.2 Design of tunnel support

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 compact and hard in 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 revealed in more detail as the project progresses.

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

4.6.3 Building Data base for Tunneling Work

The 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. The tunnel support is also selected on the basis of geological and geotechnical conditions of the project area as mentioned above.

The tunnel support is chiefly designed on the basis of the rock mass classification system which is proposed considering geological and geotechnical conditions of the project area. The tunnel support type is changed on the basis of classification of the rock mass, and structural arrangement of the tunnel support is also modified as a result of the actual work performance of support.

These projects related data including rock mass classification and tunnel support shall be compiled for future use.

5 Conclusions and Recommendations

5.1 Conclusions

In the course of discussion on the proposed rock mass classification system, existing frequently applied systems were introduced and partly compared with the proposed system in this guideline. The following conclusion can be obtained on the basis of the discussion and research works in various fields. The results of the research on the interaction between tunnel supports and rock mass under special consideration of the geotechnical conditions are considered.

- A methodology is proposed to determine the classification of rock mass according to the rock mass classification system and design of support system for road tunnels in Sri Lanka. The classification system consists of the following steps:
  - Determination of mechanical parameters of the rock mass
  - Transformation of rock mass mechanical parameters into rock mass parameters by using rock mass classification systems
  - Parameters shall be studied and optimized by choosing other different systems
  - Set-up of a numerical model on the basis of the real geological situation, and perform a simulation by using the model and mechanical parameters
- Compared to other methods such as empirical correlations or simple analytical calculations, numerical modelling offers a much more detailed and physically based insight into the interaction between the rock mass and tunnel supports.

Stability of openings of the tunnel and its support system can be confirmed by the applied numerical modelling approach. Realistic parameters can be obtained as a result of numerical modelling analysis for the tunnel and its support system.

Consequently, numerical modelling can be used for detailed tunnel supports dimensioning, design and optimization.

5.2 Recommendations

- The proposed rock mass classification system shall be modified on the basis of actual conditions of the excavation surface and additional data obtained from other tunnel and related projects.
- Effective and comprehensive geological and geotechnical investigations are recommended to be carried out to determine appropriate parameters.
- Numerical simulation of the interaction between rock mass and tunnel supports should be extended towards mathematical based sensitivity analysis and optimization. Further numerical simulations should include the more realistic tunnel supports models.
- In-situ stress of the rock mass around the tunnel section shall be considered when

design of the tunnel is implemented. The rock mass at a certain depth is said to be subjected to stresses resulting from the weight of the overlying strata with additional stress from locked in stresses of tectonic origin. Stability and potential failure mode of tunnels and underground rock caverns are directly related to the magnitude and orientation of the in-situ and induced rock stress.

- Hydrogeological conditions of the rock mass around the tunnel section often affect the progress of the tunneling works in the area of thick overburden from abundant groundwater. Therefore, hydrogeological conditions shall be carefully surveyed when tunneling work is planned in this environment.