Technical Assistance for Improvement of Capacity for Planning of Road Tunnels

Japan Sri Lanka

Guideline on Site Investigation for Rock Mass Classification System in Sri Lanka

February 2018

Road Development Authority (RDA)
Japan International Cooperation Agency (JICA)

Introduction

Tunnels can be defined as underground routes or passages driven through the ground without disturbing the overlying rock or soil, and constructed through mountains or under cities and rivers for transportation. When considering the tunnel alignment, there might be many available alternate alignments. However, an alignment with the least geologically negative factors should be the first choice. Therefore, geological and geotechnical investigations are essential for selection of routes and design of tunnels.

Site investigations for tunneling work include core drilling, geophysical exploration and laboratory tests described in “Structure of the Guideline” (Table 1), and they are mainly carried out in two stages of a preliminary and basic study as shown in the work flow of tunneling work in Figure 1.

Preliminary study for tunneling work is carried out to select the tunnel route for the basic design of the tunnel in general. Alignment of a tunnel and the excavation size is chiefly defined in the course of the basic designing. Primary items of site investigations are generally core drilling, geophysical explorations and laboratory test of materials collected at site in the preliminary study stage. The other items of rock tests using drilled core samples and others as well as in-situ tests; they are often carried out in the basic study stage.

This guideline explains the survey methods of “A. Study on exiting data”, “B. Topographic interpretation”, “C. Geological Mapping”, “D. Core Drilling”, “E. Seismic Exploration”, “F. Electrical Resistivity Survey”, “G. Permeability Test”, “H. Laboratory Test” and “I. Hydrogeological Survey”.

The methods are optimized on the basis of main subjects of investigation items of “Geology & Geological Structure”, “Stability of Portal”, “Hydrogeology”, “In-situ Stress”, “Geothermal & Gas” and summarized in Table 1. The main subjects of investigation items are selected on the basis of the factors to influence the safety and efficiency of tunneling works. It is known that design of a tunnel is not similar to that of plant or structure, and it is difficult to presume accurate geological conditions along the tunnel alignment, therefore it is highly recommended that properties and variability of the rock mass along the tunnel shall be presumed on the basis of results of various investigation methods. Besides, it is important to review existing geological information and conduct a site investigation to determine whether existing information is accurate and sufficient.

This guideline is developed taking into account geological conditions and technical situations in Sri Lanka. The contents of this guideline are selected considering importance of each method and purpose of geological investigations in Sri Lanka (e.g. relatively high priority for Geological Structure, and low priority for Geothermal and Gas). This guideline is the first edition. It is desired to update this guideline continuously by reflecting experiences of the future tunnel projects.

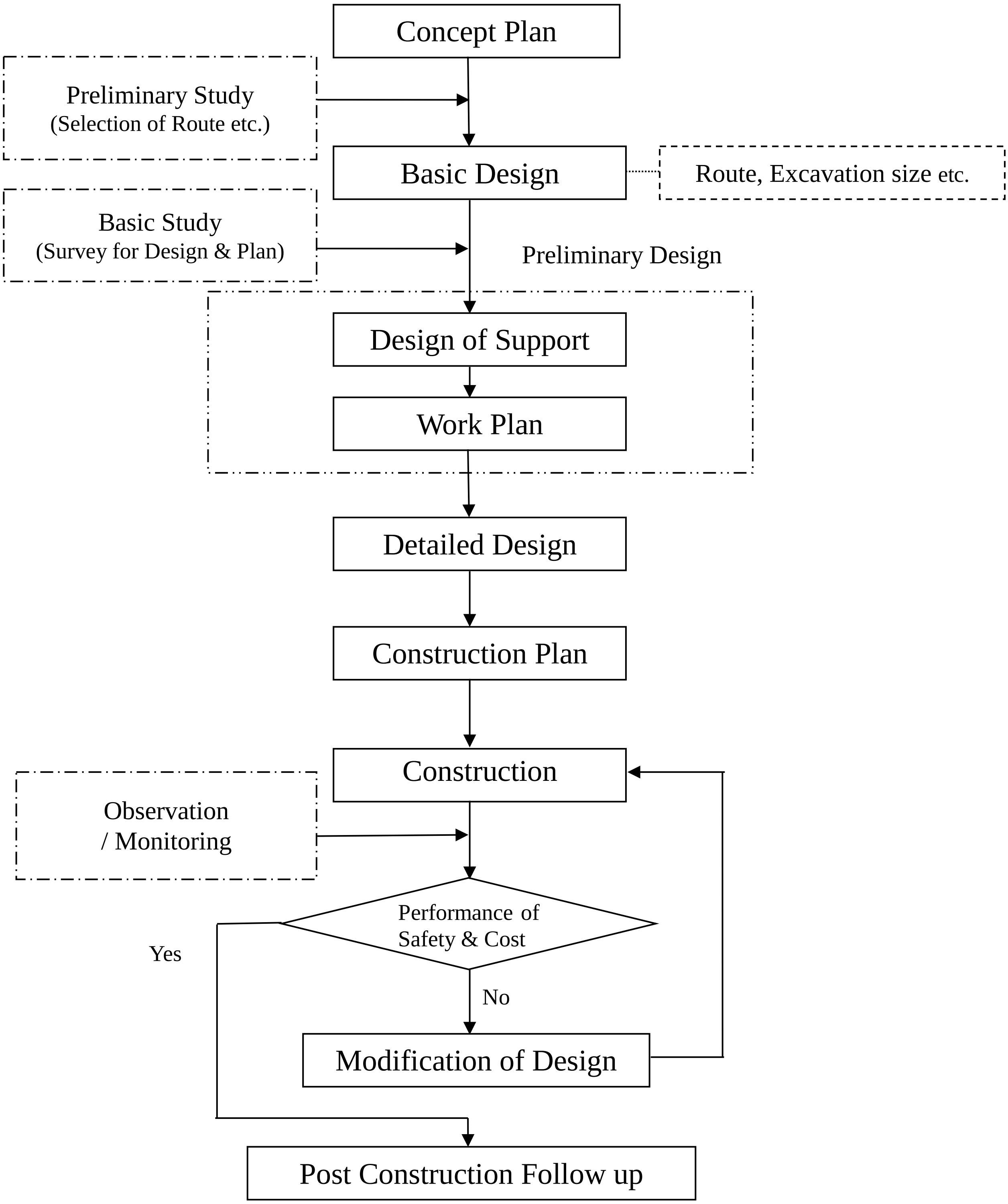

Figure 1 Flow Chart of Tunneling Work

Table 1 Structure of the Guideline and Optimization of Investigation Items

Method \ Purpose	1 Geology & Geological Structure	2 Stability of Portal	3 Hydrogeology	4 In-situ Stress	5 Geothermal & Gas
A. Study on exiting data	1	1	1	1	1
B. Topographic interpretation	1	1	1	1	1
C. Geological Mapping	1	1	1	1	1
D. Core Drilling	1	1	1	1	1
E. Seismic Exploration	1	1	3	2	3
F. Electrical Resistivity Survey	3	3	3	--	--
G. Permeability Test	--	--	1	--	--
H. Laboratory Test	1	2	--	1	2
I. Hydrogeological Survey	--	--	1	--	--
Stage of Study	Preliminary	Preliminary	Preliminary / Basic	Basic	Basic

1 : Highly Recommended 2 : Recommended 3 : When necessary -- : Unnecessary / unrelated

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Tables

A. Study of existing data

1 Introduction

Tunneling works include construction of tunnels and appurtenant structures, and design of the tunnel is complicated in comparison with those of plants or structures due to difficulties in evaluation of accurate geological conditions along the tunnel.

To minimize the risks, study referring existing data is highly recommended to be performed.

2 Topography

2.1 Topographical Maps of the Project Area

An assessment of the existing topographic data in the project area shall be established to secure the adequacy of the selected route and tunnel alignment.

2.2 Collection, Review and Analysis of Available Documents and/or Records

Documents and published reports regarding topography in and around the project area are recommended to be examined.

2.3 Characteristics of Topography of the Project Area

Abnormalities of the topography due to subsurface conditions are examined in the collected topographic maps and analyzed by site reconnaissance. Subsequently a site investigation in derail is planned to carry out in the next stage of the preliminary site investigation. Interpretation of topographic data is mentioned in the next chapter “B Topographic interpretation” in detail.

2.4 Interpretation of the Aerial Photographs of the Project Area

The project area adjoining the tunnel alignment with the reference of collected topographic maps shall also be examined to have a general idea of major structural features of rocks, such as faults, dykes, shear zone, major joints, fractures and others in the vicinity of the tunnel alignment. Aerial photo interpretation is followed by ground truth verification.

2.5 Data Collection

A topographic map and aerial photographs are provided by the Survey Department of Sri Lanka. A list of available topographic data and an index of the topographic map are shown in Table A-1 and Figure A-1, respectively.

Table A-1 List of Topographic Data

Item	Remark	Data Provider
Topographic map	- Hard copy & soft copy are available (Geodatabase/SHP/DXF/jpg) - 89 maps (1:50,000) cover entire country - Each 1:50,000 map is divided into 25 maps of 1:10,000	Survey Department of Sri Lanka No 150, Kirula Road, Narahenpita, Colombo 05, Srilanka (http://www.survey.gov.lk/)
Aerial photograph	- Aerial photographs and ortho photographs are available	

(Source: JICA project)

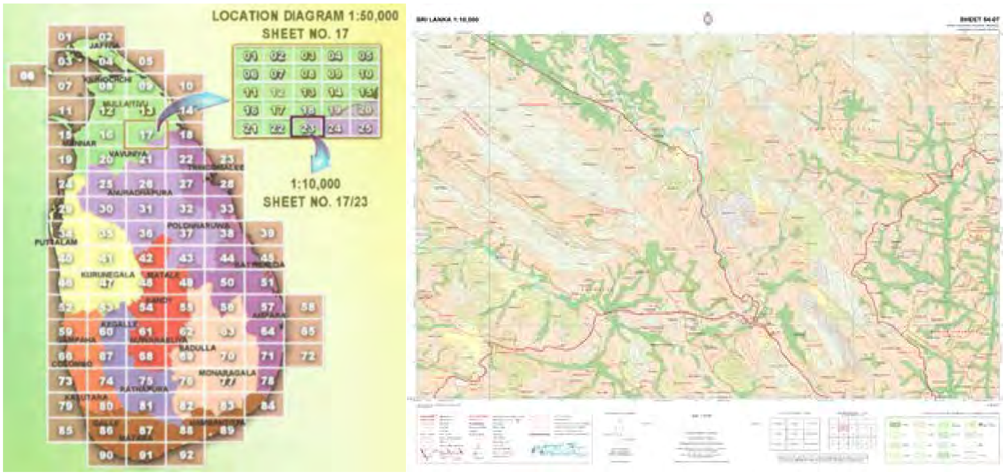


Figure A-1 Index of Topographic Map (left) and Example of 1:10,000 Topographic Map (right)

(Source: Survey Department)

3 Geology

3.1 Collection of Existing Geological Maps of the Project Area

Existing geological maps and maps for other projects of the project area are collected in order to understand the characteristics of the foundation and overlying strata of the tunnels and appurtenant structures.

3.2 Collected Geological Data and Published Documents (e.g. project reports)

Collected geological data shall be analyzed and used for preparation of a site investigation carried out in the next stage.

3.3 Studies on Surface Geology around the Tunnel Alignment

Preliminary site reconnaissance survey on the geology of the project area including the area of construction materials is generally carried out in this stage.

3.4 Data Collection

Geological maps are provided by the Geological Survey and Mines Bureau. A list of available geological maps and the index are shown in Table A-2 and Figure A-2, respectively.

Table A-2 List of Geological Map

Item	Remark	Data Provider
Geological map	- Hard copy & soft copy are available - 21 geological maps (1:100,000) cover entire country	Geological Survey and Mines Bureau 569, Epitamulla Road,Pitakotte (http://www.gsmb.gov.lk/)

(Source: JICA project)

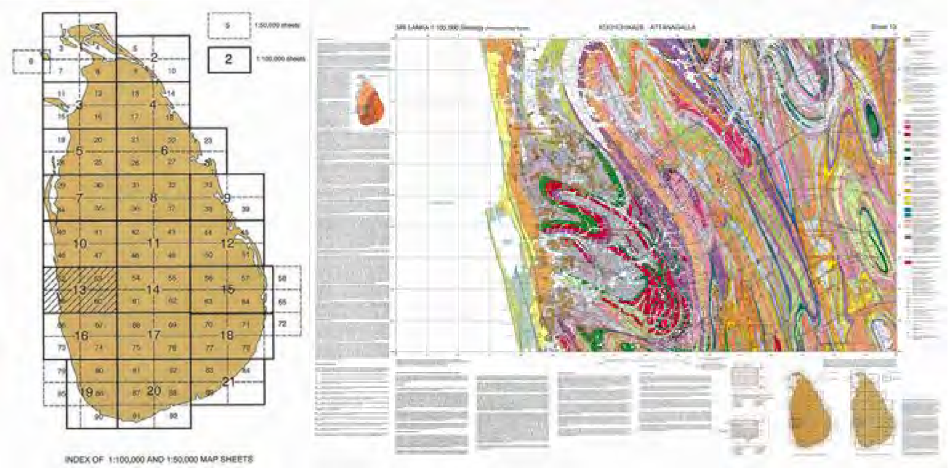


Figure A-2 Index of Geological Map (left) and Example of 1:50,000 Geological Map (right)

(Source: Geological Survey and Mines Bureau)

4 Meteorology

4.1 Meteorological Data

Meteorological data including temperature, wind speed rainfall and other related indicators observed at scattered meteorological stations in and around the project area is recommended to be collected.

4.2 Locations of Meteorological Stations

Location of meteorological stations shall be confirmed the beginning stage of the project.

4.3 Data Collection

The Department of Meteorology and Irrigation Department has provided the meteorological data. A list of meteorological data and location maps of meteorological stations are shown in Table A-3 and Figure A-3, respectively.

Table A-3 List of Meteorological Data

Item	Remark	Data Provider
Rainfall, temperature, wind speed, wind direction, etc	- Rainfall, temperature, wind and other factors are observed at 22 synoptic stations. Long term records are available. - 38 automatic weather stations record 1hour interval meteorological data, but long term records are not available. - Daily rainfall is observed at more than 400 rain gauge stations in the country. - 39 agro-met stations conduct general meteorological observations and agro-met observations.	Department of Meteorology 383, BauddhalokaMawatha, Colombo 07 (http://www.meteo.gov.lk)
Rainfall, evaporation	- Rainfall and evaporation are observed at gauging stations	Irrigation Department 230, BaudhalokaMawatha, Colombo07 (http://www.irrigation.gov.lk/)

(Source: JICA project)

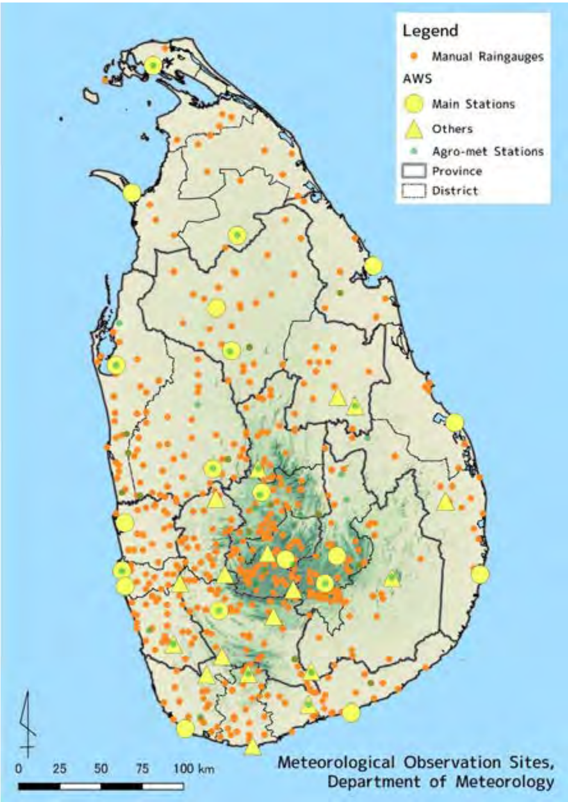


Figure A-3 Location of Meteorological Stations (Department of Meteorology)

(Source: JICA project)

5 Hydrology

5.1 Hydrological Data Observed at Gauging Stations

Analysis of the hydrological data in and around the project area is recommended to know the hydrological conditions around the project area, subsequently hydrogeological conditions of the area

5.2 Locations of Gauging Stations

Location of the hydrological station is recommended to be confirmed in the beginning stage of the project.

5.3 Data Collection

Historical records of river discharge are provided by the Irrigation Department. A list of hydrological data and location of gauging stations is shown in Table A-4 and Figure A-4, respectively.

Table A-4 List of Hydrological Data

Item	Remark	Data Provider
River discharge	- More than 30 gauging stations record 1 hour interval river discharge. Long term records are available. - Ca. 100 automatic gauging stations are operated, but long term records are not available.	Irrigation Department 230, BaudhalokaMawatha, Colombo07 (http://www.irrigation.gov.lk/)

(Source: JICA project)

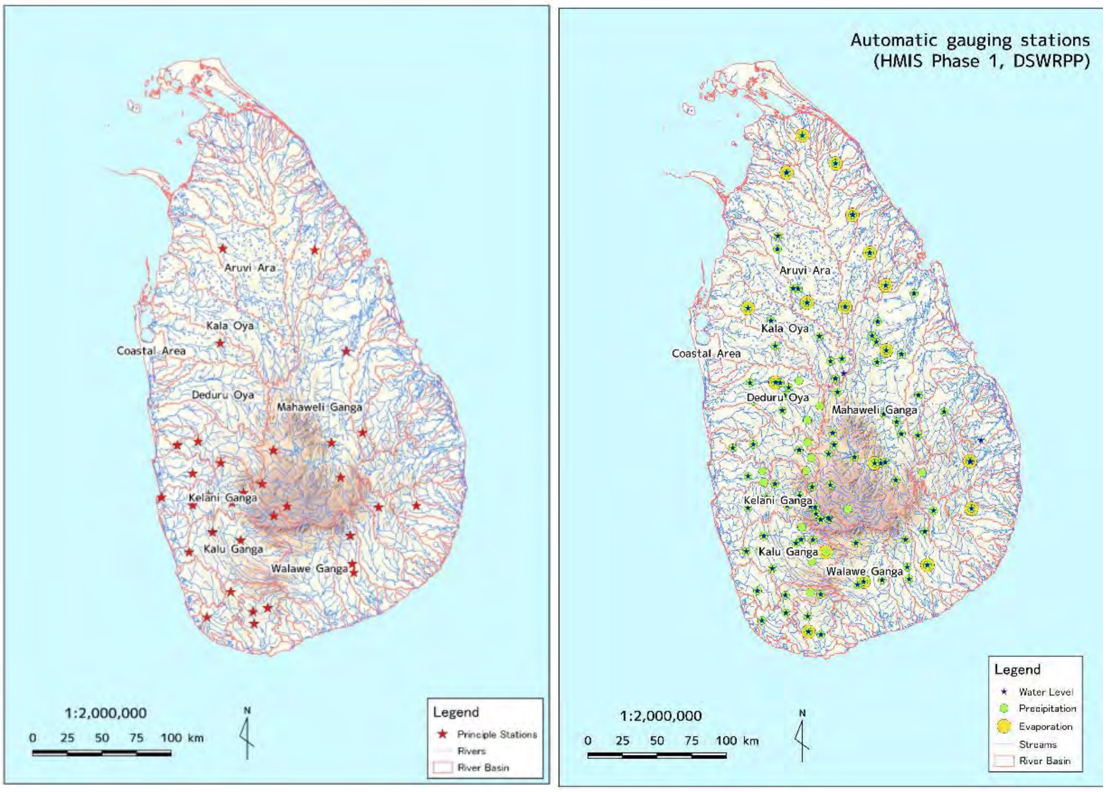


Figure A-4 Location of Gauging Stations (Irrigation Department, left: manual, right : automatic)

(Source: JICA project)

6 Engineering Design Criteria

6.1 The Codes, Standards, Criteria and Practices Related to the Project

The code, standards, criteria and practices related to design and construction of the project are collected. And, those of similar existing projects are collected and studied.

6.2 Specific and Detailed Information of Similar Projects

More specific and detailed information such as detailed design, specifications of material procurement and construction and other engineering related information are collected.

6.3 Project Information During Construction Work

Information obtained during the design and construction stage is collected and the actual condition of the project is analyzed for utilization in project planning.

6.4 Data Collection

Related agencies regarding engineering design are shown in Table A-5.

Table A-5 Related Agencies Regarding Engineering Design

Agency	**Remark**
National Building Research Organization 99/1, Jawaththa Road, Colombo 05 (http://www.nbro.gov.lk/)	- Advise for engineering design - Previous study / project
Central Engineering Consultancy Bureau No. 415, BauddalokaMawatha, Colombo - 7 (http://cecb.lk/)	

(Source: JICA project)

7 Environmental Hazards

7.1 The Planning and Implementation Aimed for Minimizing the Environmental Hazards

Tunneling disturbs the environment in several ways. The tunneling involves vibrations induced through blasting or ground cutting and drilling, producing an abnormal quantity of dust, interferes with the water supply and system of nearby areas. Therefore, the project plan should be considered for minimizing the environmental hazards in the populated areas and reservoirs.

7.2 Ecological and Socio Economic Surveys

An ecological and socio economic surveys are conducted along the reservoirs and entire stretch of tunnel. Impacts on environmental, ecological and socio-economic components will be identified.

7.3 Data Collection

Related agencies regarding environmental hazards are shown in Table A-6.

Table A-6 Related Agencies Regarding Environmental Hazards

Agency	**Remark**
Central Environmental Authority 104, DenzilKobbekaduwaMawatha,Battaramulla (http://www.cea.lk/)	- Environmental Impact Assessment
Department of Census and Statistics No.306/71,Polduwa Rd, Battaramulla (http://www.statistics.gov.lk/)	- Socio economic data
Local Government	- Socio economic data - Water resource management
Mahaweli Authority of Sri Lanka No. 500 T.B.JayathMawatha, Colombo 10(http://mahaweli.gov.lk/)	
Irrigation Department 230, BaudhalokaMawatha, Colombo07(http://www.irrigation.gov.lk/)	

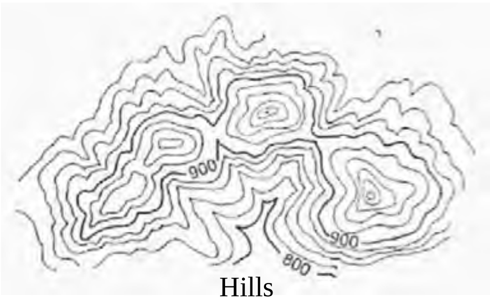
(Source: JICA project)

B. Topographic interpretation

1 Introduction

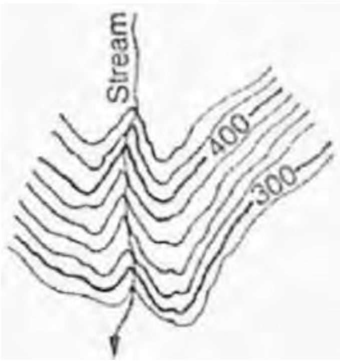
Interpretation of topographic maps and aero-photos is generally carried out to select road routes including the tunnel alignment as one of the topographic surveys in the planning stage. Topographic maps provide visualization of contours, altitudes, shapes, slopes and locations of landforms. Visualization of the three-dimensional shape of the terrain from the contour lines can be developed with the basis of compressive understanding of contour lines as information of continuous elevation.

Abnormalities of the contours are often recognized in the topographic map when there are some abnormalities in the subsurface geological conditions. Several cases of abnormalities of contours related to the assumed subsurface geological condition are shown in the following. Recommended alignment of a tunnel portal in different topographies, general conditions of the topography in Sri Lanka and other topography related subjects are also described.



Hills

Depression



Valley and Stream

Figure B-1 Topography and Contours

(Source: JICA project)

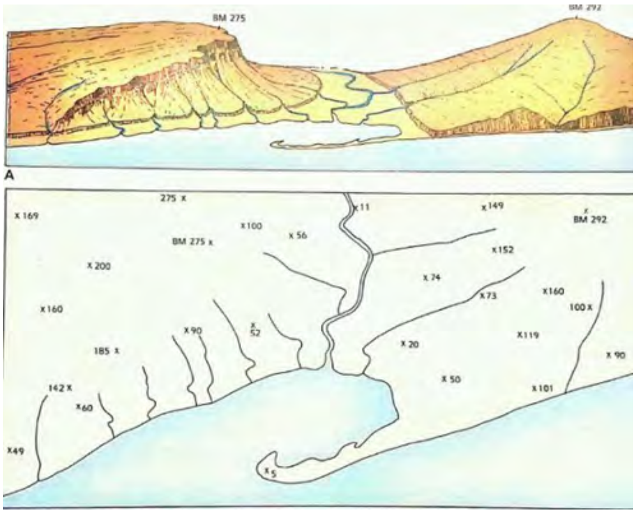


Figure B-2 Slope Gradient and Length of Streams

(Source: JICA project)

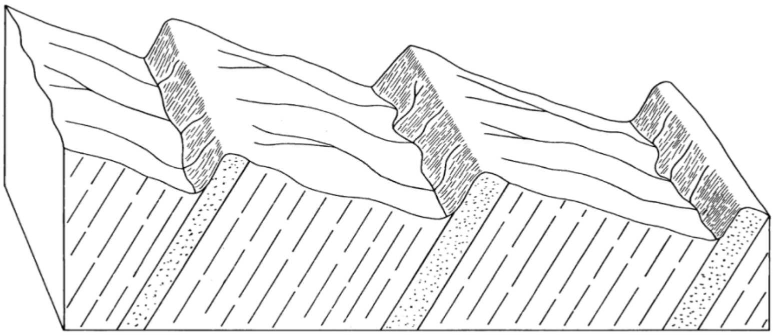


FIGURE 6.—Sketch showing long tributary streams on downdip side of strike valley in area of steep dips.

Long Tributary streams on down-dip side of strike valley in area of steep dips

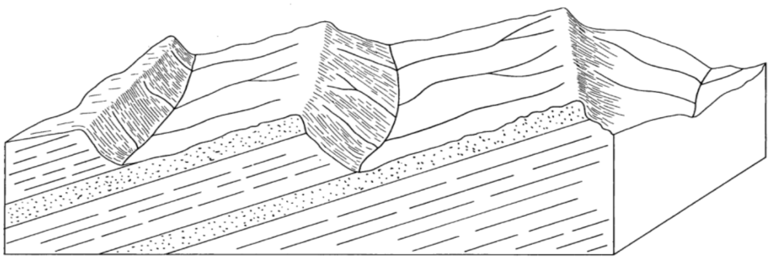


FIGURE 5.—Sketch showing long tributary streams on updip side of strike valley in area of low dips.

Long Tributary on up-dip side of strike valley in area of low dips

Figure B-3 Topography and Subsurface Geology

(Source: JICA project)

2 Topography and Geology of Sri Lanka

2.1 Topography

A mountainous area in triangular shape known as the Central Highlands occupies the south-central region of Sri Lanka, and is the heart of the country. This highland mass is surrounded by a diverse plain, the general elevation ranging from sea level to about 300 m. This plain occupies about five-sixths of the country's total area.

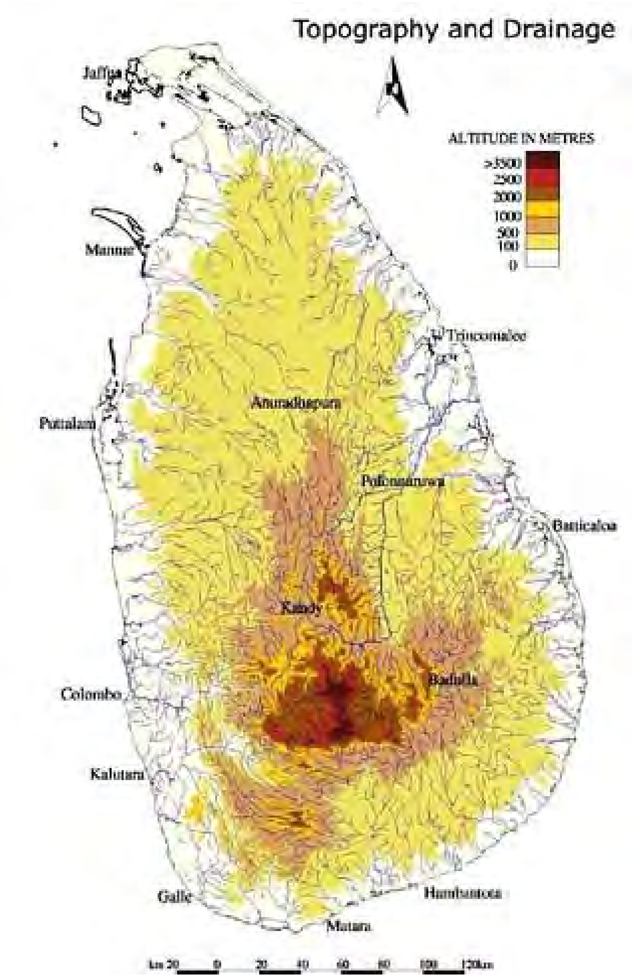


Figure B-4 Topography of Sri Lanka

(Source: JICA project)

The Central Highlands have a highly dissected terrain consisting of a unique arrangement of plateaus, ridges, escarpments, intermountain basins and valleys. Sri Lanka's highest mountain is located in this region at the highest peak of 2,524m above sea level. The highlands, except on their western and south-western flanks, are sharply defined by a series of escarpments.

The plains that surround the Central Highlands do not have entirely flat or featureless terrain. To the north and northeast of the highlands, the plains are traversed by low ridges that decrease in altitude as they approach the coast. The western and southwestern parts of the plains feature alternating ridges and valleys running parallel to the coast and increasing in elevation toward the interior to merge imperceptibly with the highland mass. Elsewhere the flatness of the plains is sporadically interrupted by rocky buttes and mounds, some of which reach elevations of more than 300 m. The plains are fringed by a coast consisting mostly of sandy beaches, spits and lagoons. Over a few stretches of the coast there are rocky promontories and cliffs, deep-water bays and offshore islets.

2.2 Geology

Geologically, the island of Sri Lanka is considered a southerly extension of peninsular India with which it shares a continental shelf and some of its basic lithologic and geomorphic characteristics. Sri Lanka is known as a part of East Gondwana, together with fragments of Antarctica, Australia, India, Madagascar, Mozambique and Tanzania. Sri Lanka acted as a bridge through which Antarctica and East Africa can be correlated. Thus, Sri Lanka reveals remarkable geological and geotectonic similarities to those of neighboring Gondwana fragments.

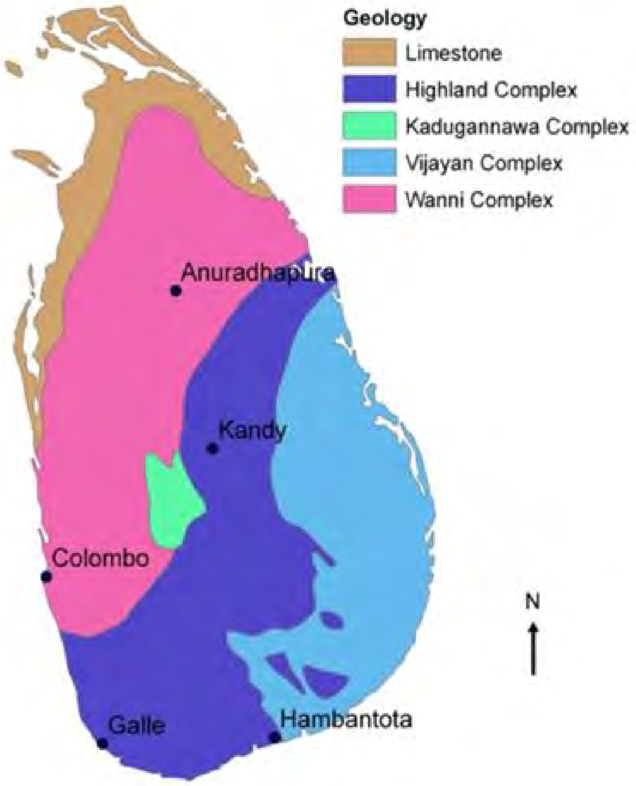


Figure B-5 Geological Map of Sri Lanka
(ESDAC - European Soil Data Centre)

The Proterozoic basement of Sri Lanka exposes substantial parts of the lower continental crust. Four different units were distinguished on the basis of isotopic, geochemical and petrological constraints, such as the Vijayan Complex in the east, the Highland Complex in the central, Wanni Complex in the west and the Kadugannawa Complex in between Highland Complex and Wanni Complex. The Vijayan Complex consists mainly of amphibolite-facies granitoid rocks, metadiorites, metagabbros and migmatites while the Highland Complex is composed of intercalated meta-sedi- mentary and meta-igneous rocks of pelitic, mafic such as quartzo-feldspathic granulites, charnockites, marble and quartzite. Most of the Highland Complex rocks have attained granulite-facies conditions whereas some contain ultra-high temperature assemblages. Rocks in the Wanni Complex are granitoid gneisses, granitic migmatites, scattered metasediments and charnockites, which are metamorphosed under upper amphibolite to granulite facies conditions. The dominant rocks of the Kadugannawa Complex are hornblende and biotite-hornblende gneisses with interlayered granitoid gneisses in the core, pink feldspar granitic gneisses at the inner rim and metasediments at the outer rim of the arenas. Rocks of the Kadugannawa Complex are metamorphosed under upper amphibolite to granulite facies conditions. Post-peak metamorphic magmatic and hy- drothermal activities are responsible for the formation of pegmatite, dolerite, carbonatite and granite bodies found in Sri Lanka.

Hard and compact crystalline rock formations are distributed in the area of about nine-tenths of the island.

3 Landform Process

3.1 General

The landform changes in respect of forms, process and stage are controlled by a number of factors. The land is rapidly worn away and the surface continually lowered towards base level when there is no major uplift. Over a long period of cyclic time, landform slowly gets loss of energy and mass, and the altitude decreased.

Secular changes and/or process of the landform can be described on the basis of phenomena observed in the topography of each stage as shown in Table B-1.

Stage I:

- Fault is formed and weathering proceeding
- No sliding landmass forms
- No open cracks

Stage II:

- Block glide occurs
- Open cracks formed
- Steps formed due to block glide of sliding land mass

Stage III:

- Block glide proceeds and landslide topography formed
- Large open cracks formed
- Steps formed due to block glide of sliding land mass
- Depression formed at top of Block glide

Stage IV:

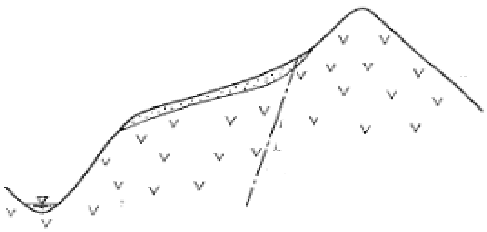
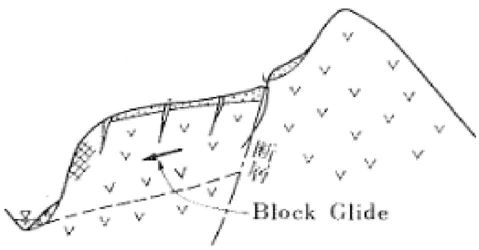
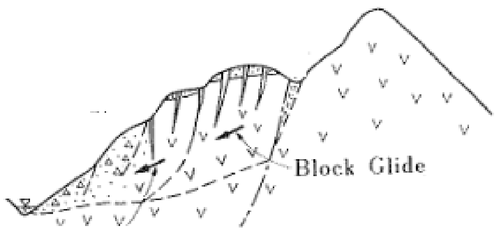
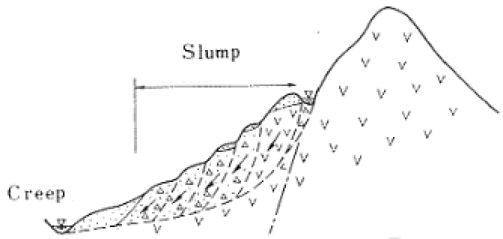
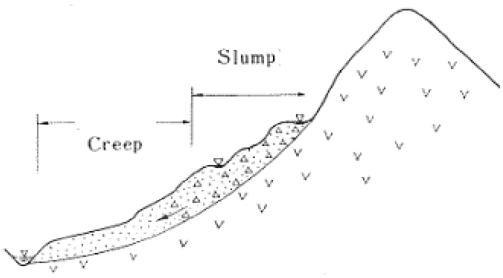
- Weathering proceeds
- Landslide topography formed
- Slump & creep portion formed, toe of sliding land mass becomes clayey
- Pond formed in the depression at top of sliding mass

Stage V:

- Weathering proceeds further

- Creep exceeds slump portion
- Sliding land mass becomes clayey in general
- Pond formed at places

Table B-1 Secular Changes in Topography of Landslide

Stage	Section Profile	Remarks
I		・Fault is formed and weathering proceeding ・No sliding landmass forms ・No open cracks
II		・Block glide occurs ・Open cracks formed ・Steps formed due to block glide of sliding land mass
III		・Block glide proceeds and landslide topography formed ・Large open cracks formed ・Steps formed due to block glide of sliding land mass ・Depression formed at top of Block glide
IV		・Weathering proceeds ・Landslide topography formed ・Slump & creep portion formed, toe of sliding land mass becomes clayey ・Pond formed in the depression at top of sliding mass
V		・Weathering proceeds further ・Creep exceeds slump portion ・Sliding land mass becomes clayey in general ・Pond formed at places

(Source: JICA project)

3.2 Other Landforms

Unstable landform of the tunnel route shall be carefully examined to prevent in difficulties to executing the tunnel project. Some of the unstable landforms are described below.

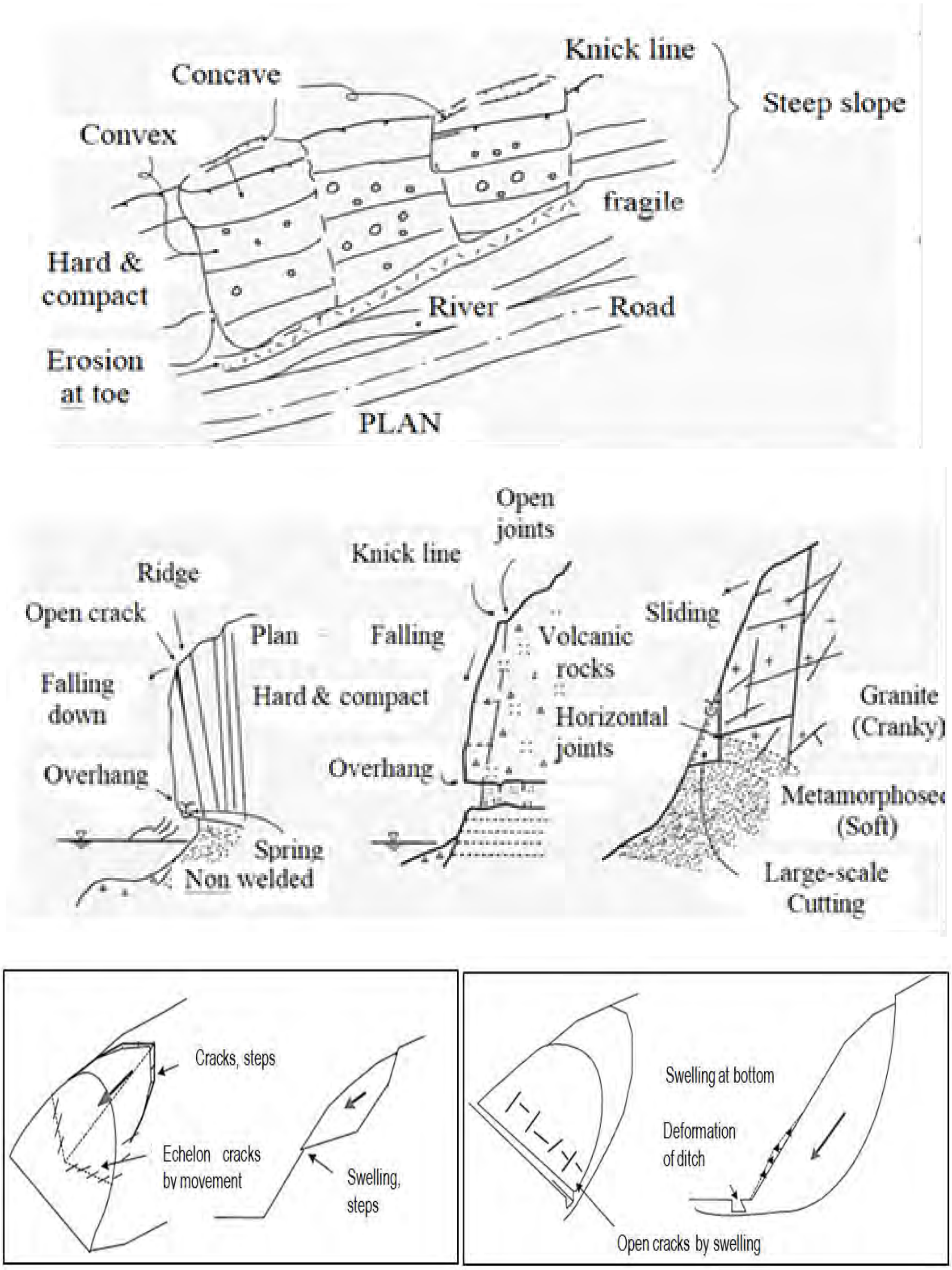


Figure B-6 Topographical and Geological Factors of Unstable Slope

(Source: JICA project)

4 Aerial Photo Interpretation

4.1 General

Aerial photographs interpretation is widely used to obtain both qualitative and quantitative geologic information. Techniques and procedures of geological interpretation of aerial photographs are based on the fundamental recognition of photographic tone, colour, texture and pattern, which is related to associated features, shape, and size. The scale of photographs, as well as the present vertical exaggeration in most stereoscopic models, are also significant in photo interpretation.

The amount of geologic information that may be obtained from aerial photographs is primarily dependent on the geological type of terrain (igneous, metamorphic or sedimentary), climatic environment, and stage of the geomorphic cycle. Because features are more readily recognized where strong differences exist in the erosional resistance of adjacent rocks, sedimentary terrain may be expected to yield the greatest amount of information from aerial photographs. Metamorphic terrain may yield the least information because metamorphic processes tend to eliminate differences that may have existed in the un-metamorphosed rocks. Combinations of criteria such as photographic tone, texture, pattern, and vertical exaggeration permit influences as to rock type and geologic structure, which is important in engineering geology.

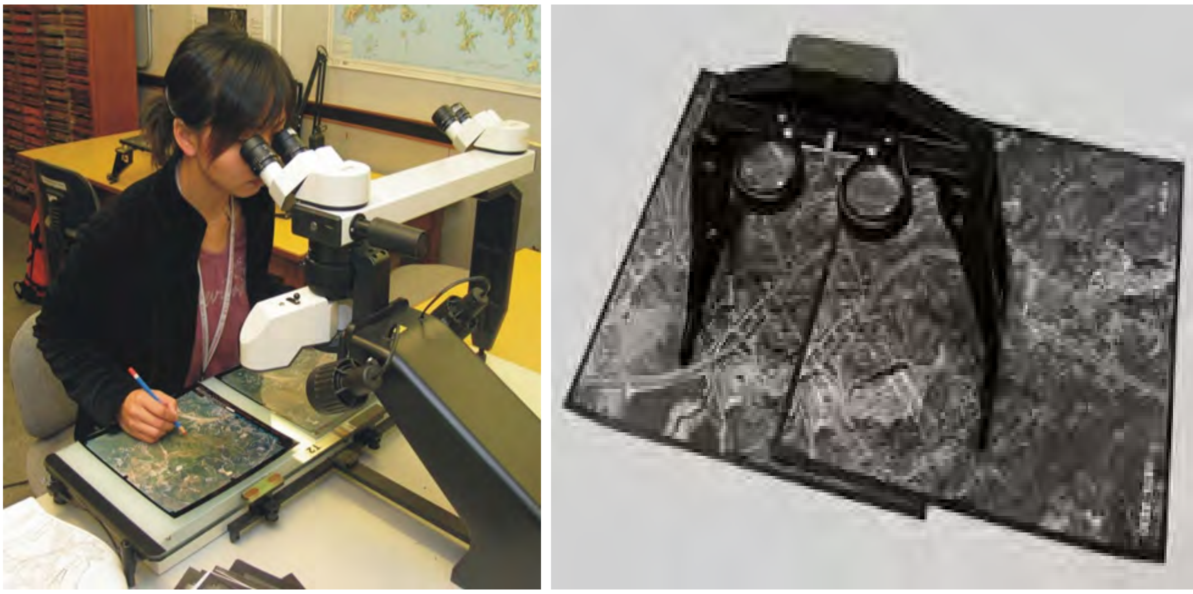


Figure B-7 Aerial Photo Interpretation

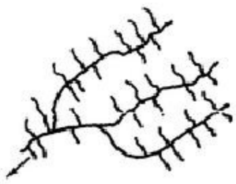
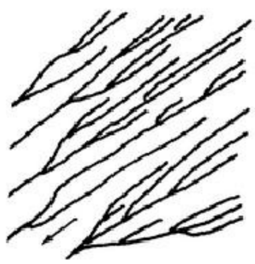
(Source: JICA project)

4.2 Drainage Pattern

Regional terrain characteristics determine the network pattern, so identifying drainage patterns can reveal attributes of the terrain [e.g., Howard, 1967]

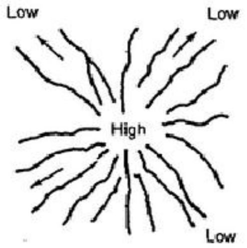
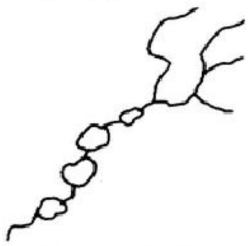
Fluvial network delineation is likewise proving useful in revealing information about the subsurface geology of the area. Typical drainage patterns and assumed geological conditions are summarised on the basis of existing publications.

Table B-2 Drainage Pattern (1/2)

Drainage Pattern	Example
Dendritic This is a randomly developed, treelike pattern composed of branching tributaries and a main stream. It is the most common drainage pattern and is characteristic of essentially flat-lying and/or relatively homogeneous rocks and impervious soils.	
Rectangular This is sometimes called rectangular dendritic or angular dendritic. It is a modified version of the dendritic pattern, characterized by abrupt, close to 90-degree, changes in stream direction and distinct obtuse or acute angles of stream juncture. This pattern is usually caused by jointing or faulting of the underlying bedrock. It is usually associated with massive, intrusive igneous and metamorphic rocks.	
Trellis This is a modified version of the dendritic pattern. It forms in areas of folded rock strata. The main streams are parallel, following the lowlands, and they receive tributaries at right angles from adjacent ridges.	
Parallel This drainage pattern is characterized by major streams trending in the same direction. Tributaries usually join the main stream at approximately the same angles. Parallel streams are indicative of gently dipping beds or uniformly sloping topography. For example, sloping basalt flows and young coastal plains exhibit parallel drainage.	

(Source: Engineer Reference and Training Manuals)

Table B-3 Drainage Pattern (2/2)

Drainage Pattern	Example
Radial Radial drainage is composed of streams radiating outward from a central peak, dome, or volcanic cone.	
Annular Primary streams develop in the concentric, circular joints surrounding an uplifted dome of sedimentary rocks. Fractures may also control the flow of tributaries, which are generally at right angles to the main stream.	
Internal There is very little surface expression displayed with internal drainage. Instead, there is a subterranean drainage system that is characterized by caves and sinkholes. Internal drainage is generally developed in areas underlain by soluble rocks, such as carbonates.	
Deranged This is a poorly defined drainage system resulting from a high water table and a flat or gently undulating topographic surface. These conditions are normally met in areas of glacial till plains.	

(Source: Engineer Reference and Training Manuals)

4.3 Lineament

Lineament is a linear topographic feature of regional extent that is considered reflecting underlying crustal structure. Lineaments related to steep slopes, straight valley segments, abrupt changes in vegetation coverage and sudden bends along river courses have been evaluated as potential faults.

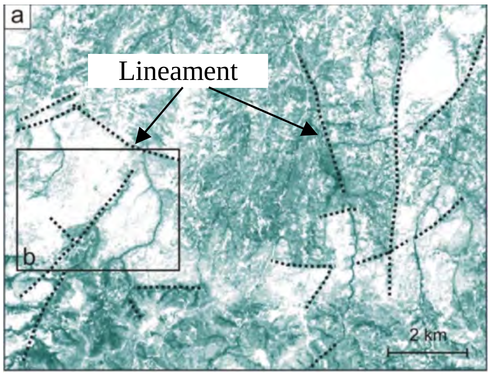


Figure B-8 Lineament

(Source: JICA project)

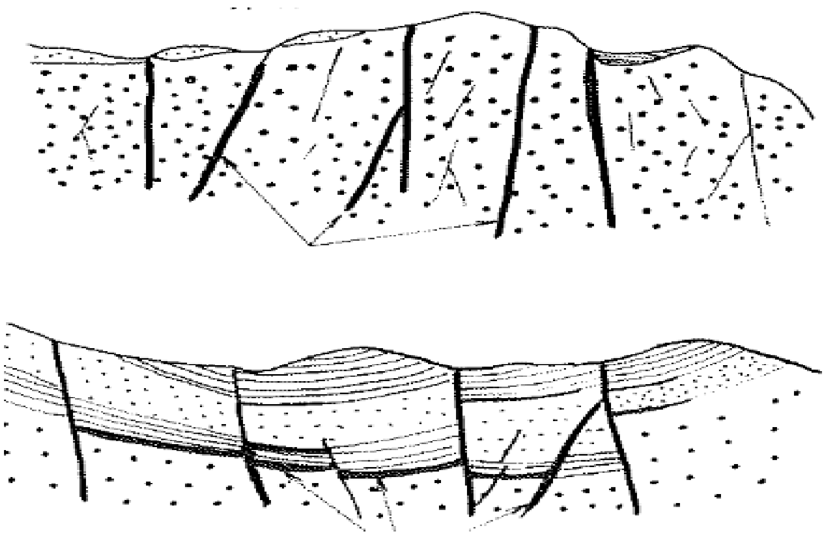


Figure B-9　Faults and Lineaments

(Source: JICA project)

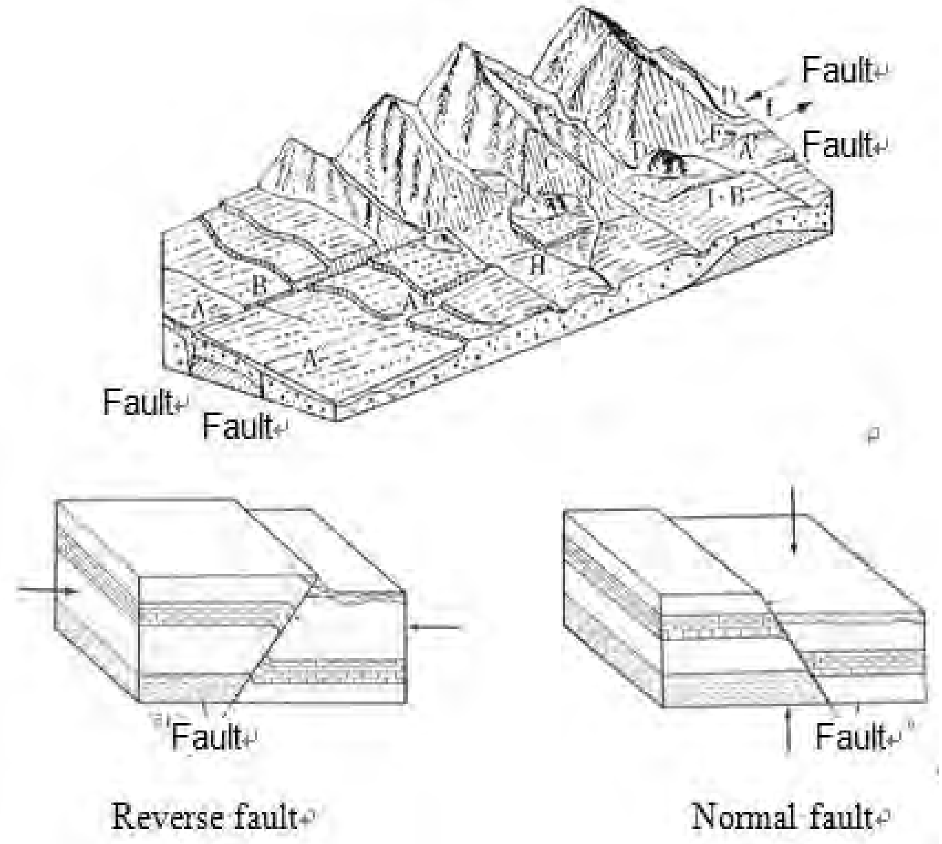


Figure B-10　Fault Topography and Major Fault System

(Source: JICA project)

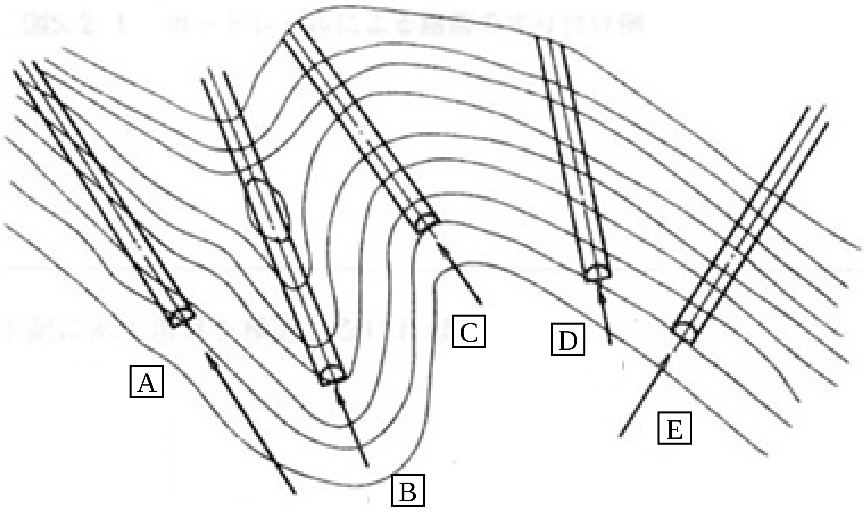
5 Tunnel and Topography

5.1 Location of Portal

Road tunnels are one of the alternative transportation systems to surface roads, bridges etc. to shorten the travel time and distance in addition to increase the transportation capacity through mountains or open waters. The portal and adjacent area of the tunnel shall be stable since the portal provides access to the tunnel, and it is excavated at the beginning of tunnelling work. Special attention shall be paid to the location of the portal.

Topography around the portal shall be carefully studied in order to evaluate subsurface geological conditions to affect workability of the tunnelling. Appropriateness of selection of a portal in different topographic features is described in Table B-4. In this figure, the most desirable location of portal is E.

Table B-4 Location of Portal and Topographic Features



	Judgement	Remarks
A	Undesirable	Attention must be paid on unbalanced in-situ stress.
B	Good in general	Thin ridge and thick weathering are assumed
C	Relatively good	High groundwater table, Risk of mudflow are anticipated
D	Undesirable	Attention must be paid on unbalanced in-situ stress.
E	Most desirable	Attention must be paid on layout of construction yard.

(Source: JICA project)

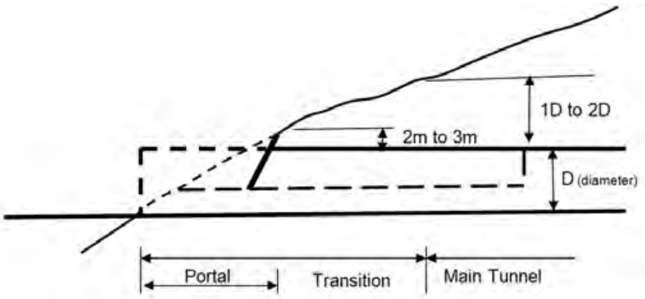


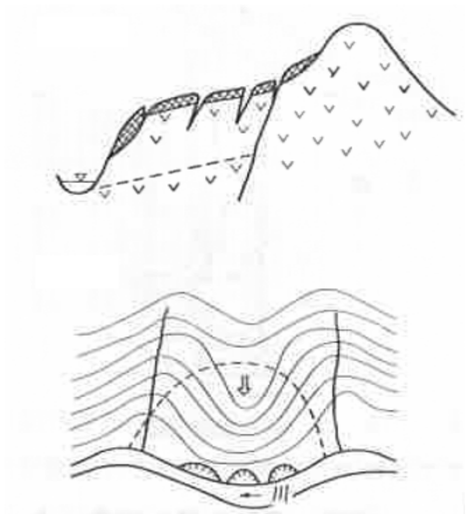
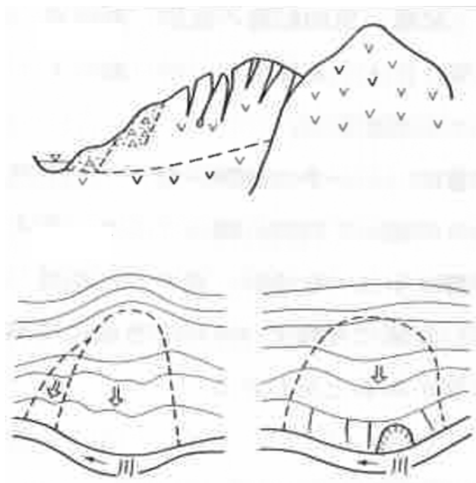
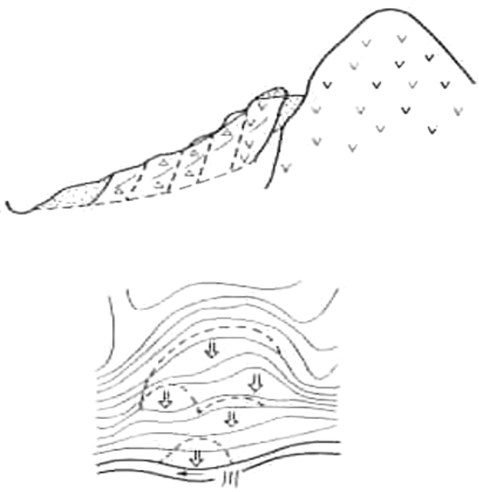
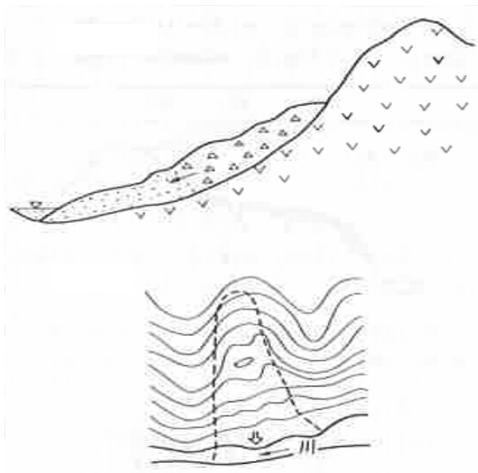
Figure B-11 General Layout of Tunnel Portal

(Source: JICA project)

5.2 Landslide

Landslides are usually continuous and recurrent, and therefore, exhibit specific morphological characteristics. The above morphological characteristics, which are recognisable by topographic interpretation, can be used as indicator for identifying and estimating a landslide area and its direction. The morphological characteristics and contours are shown in Table B-5.

Table B-5 Profiles of Landslides and Contour patterns

Convex-ridge type (Stage II)	Convex-plateau type (Stage III)
	
・Block glide (slide) formed ・Beginning phase of Landslide	・Proceeding phase of Block gliding (sliding) ・Weathering proceeded
Convex-depression type (Stage IV)	**Convex-hollow type (Stage V)**
	
・Landslide formed ・Toe of sliding landmass becomes clayey	・Proceeding phase of landslide ・Weathering proceeds

(Source: JICA project)

These morphological characteristics are related to Table B-1 ” Secular Changes in Topography of Landslide”. Contour pattern of each type of landslide can also defined as secular changes of landslide topography.

5.3 Tunnel and Landslide

Road tunnels are planned to construct to shorten the travel time and distance through barriers such as mountains and terrains, and to avoid surface congestion, improve air quality, reduce noise, or minimize surface disturbance. In a tunnel route study, topographic and geologic considerations shall be paid in addition to above mentioned issues. A landslide is one of the serious phenomena related to topography and geology of the project area. Portal and alignment of the tunnel shall be out of landslide mass. However it is required to deal with landslide when the tunnel drives into the landslide mass in an unforeseeable circumstances.

Countermeasures for landslides, when the tunnel drives into landslide mass, are studied in the following conditions.

A. Tunnel running perpendicular to movement of landslide mass

B. Tunnel running parallel to movement of landslide mass

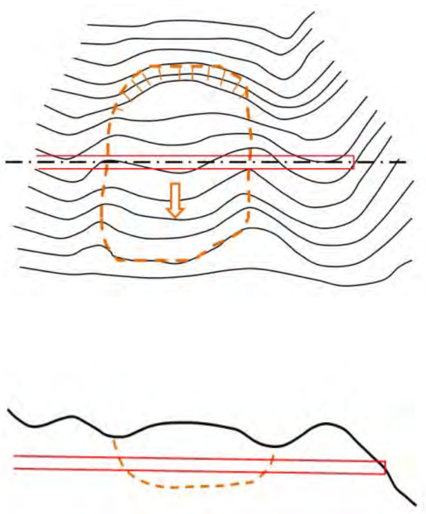
As the results of study, the following conditions are presumed and the results are summarised in Table B-6.

- Tunnel drives crossing the slip surface of the slide, therefore the friction coefficient decrease due to reduction of slip surface.
- Sliding force may change in case of cutting of sliding landmass.

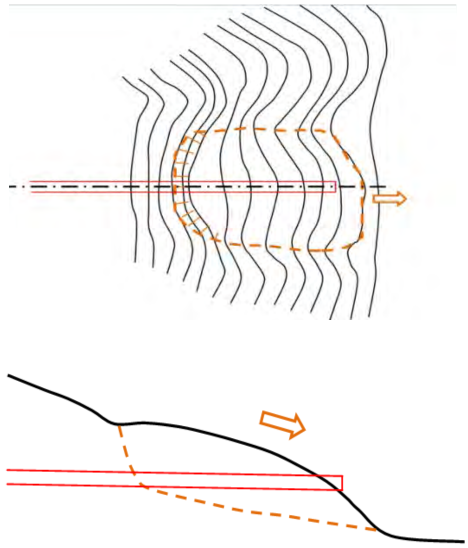
Safety factor increases in case of cutting at upper and placing of embankment at lower portion of the sliding mass. On the contrary, the safety factor decreases in case of cutting at lower and placing of embankment at upper portion of the sliding mass (See Table B-7).

Table B-6 Tunnel and Landslide

A. Tunnel running perpendicular to movement of landslide mass

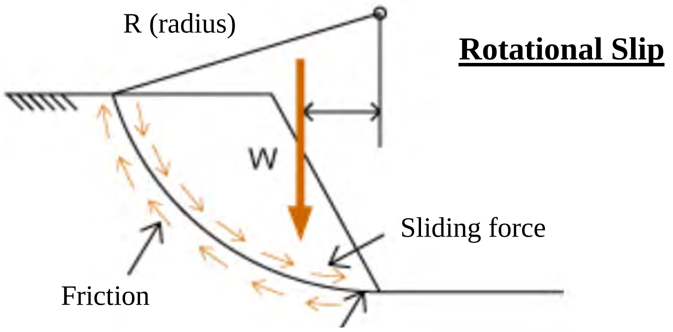


B. Tunnel running parallel to movement of landslide mass

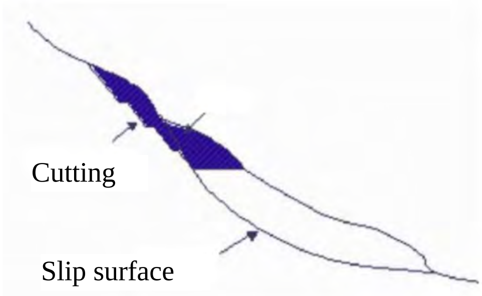


・Tunnel drives crossing the slip surface of the slide, therefore the friction coefficient decrease in some amount due to reduction of slip surface.

・ Sliding force may change in case of cutting of sliding landmass.

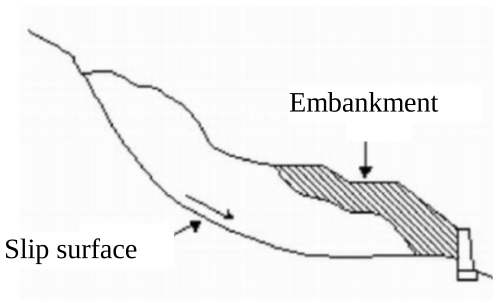


Safety factor increases in case cutting at upper and Embankment at lower portion of Sliding.



Cutting at **Upper** portion of Sliding landmass

Safety factor **increases**

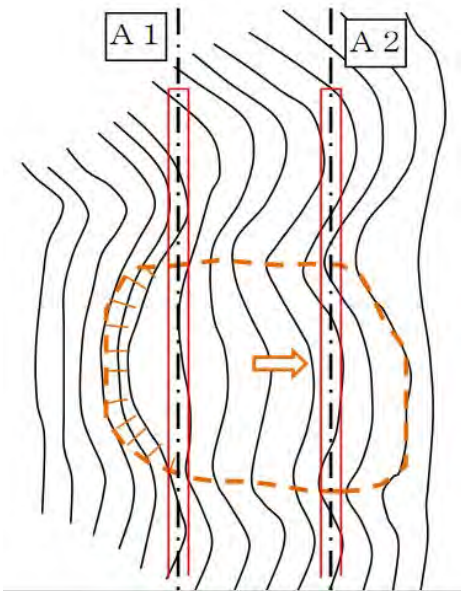
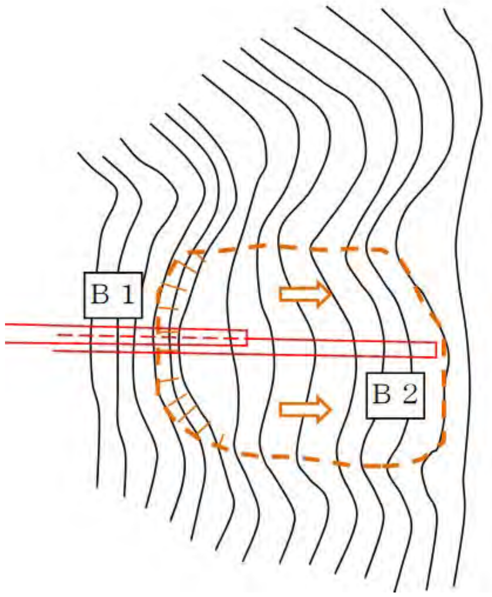
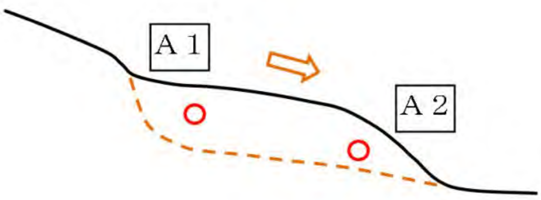
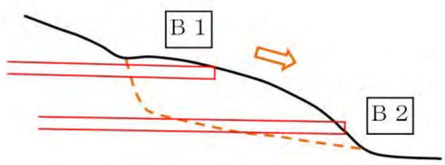


Cutting (Removal of Embankment) at **Lower** portion of Sliding landmass

Safety factor **decreases**

(Source: JICA project)

Table B-7 Tunnel Alignment and Landslide

Tunnel running perpendicular to movement of landslide mass	Tunnel running parallel to movement of landslide mass
	
A1: Tunnel at Upper of sliding landmass A2: Tunnel at Lower of sliding landmass	B1: Tunnel at Upper of sliding landmass B2: Tunnel at Lower of sliding landmass
	
A1: Safety factor increases. However reinforcement of the tunnel support is required in the section running in the sliding landmass. Special attention shall be paid on groundwater, which may be encountered in a open crack.	B1: Safety factor increases. However, reinforcement of the tunnel support is required in the section running in the sliding landmass of fragile due to abundant cracks.
A2: Safety factor decreases. Counter weight against sliding force decreases, therefore countermeasures to sustain the movement of sliding landmass is required. Additional countermeasures might be required to cope with stress likely emerge around the portal.	B2: Safety factor decreases. Counter weight against sliding force decreases, therefore countermeasures to sustain the movement of sliding landmass is required. Additional countermeasures might be required to cope with stress likely emerge from mountain side.

(Source: JICA project)

C. Geological Mapping

1 Introduction

The many types of geological maps are prepared by geological mapping to deal with different types of the project. When the geological mapping is conducted, special equipment is required. Subsequently, many types of geological maps are prepared for one project. Making a geological map is one of the challenging tasks.

A guideline for what to do in the field and how to collect the evidence in the field to obtain geological conclusions is described in the following chapters. (Reference Section 2)

The methods and techniques used in geological mapping, including a description of photo geology, which is the use of aerial photographs in interpreting geology on the ground, are described in chapter B and C. Methods to locate the observation points on a map are also described and the advice is given on what to do if no topographic base maps are obtainable. (Reference Section 3)

A description follows the different kinds of topographic base maps which may be available for geological observations in the field.

Regional geological maps should be plotted on a reliable base map. Unfortunately, in some cases, geological mapping area outstrips topographic coverage when there is a sudden interest in a specific area from a view point of economic efficiency of road construction; geologists have to carry out topographic survey by themselves.

An accurate geological map cannot be prepared if an adequate topographic base is not available. Regional geological mapping conducted on the ground may be supported by systematic photo geology, and it should be emphasized that photo geological evidence is not inferior to information obtained on the ground although it may differ in character. (Reference to Section 4)

Some geological features seen in aerial photographs cannot even be detected on the ground while others can be conveniently followed on photographs than in surface exposures. All geological mapping should incorporate any techniques which can help in plotting the geology, including geophysics, pitting, core drilling and even the use of satellite images. (Reference Section 5)

The office work covers methods of drawing, cross-sections and the preparation of other diagrams to help the geological interpretation. (Reference to Section 6) Part of the office work may have to be done at field camp,

Advice is also given on preparing of the geological map which shows interpretation of the data from the field map in this chapter. However, preparation of a geological map is not an end of the work. The whole purpose of geological mapping is to explain the geology of the area, and the map of preparing at site is only a part of that process. The report is also necessary to explain the geological history of the area and the sequence of geological events. (Reference Section 7)

2 Geological Mapping

A geological mapping is the basic professional work normally conducted by the geologists. A geological mapping is a systematic investigation of the geology of an area. It reflects the geology and structure beneath the ground. Any geological investigation method employs several techniques including the traditional traverses, site reconnaissance surveys, studying the exposures, outcrops and landforms.

Geological investigations include some intrusive methods such as core drilling and geophysical investigations including seismic refraction prospecting and the electrical resistivity survey. Geological investigations also use remote sensing methods, such as aerial photography and satellite imagery.

Geological investigations are normally undertaken by private agencies, state government departments of mines and geology, and national geological survey organizations. They maintain a geological inventory of various formations, mineral deposits and resources. Then geological mapping becomes a common technique in the civil engineering field.

Geological mapping is part of the geological investigations, and it involves certain procedures.

2.1 Objectives of Geological Mapping

There are several reasons based on which a geological field mapping is carried out. They are all entailed in collecting variable amounts of field data. The main reason is to delineate the geological conditions including natural mineral and other resources. The geological mapping for civil engineering including mineral and oil exploration always proceeds in the field mapping. Geological mapping is usually the first task in any reconnaissance study. Geophysical investigations are carried out to answer the question of the extent of the system under the subsurface. Geochemical investigations are also used to estimate parameters such as the geotechnical condition of the system. Exploitation of the area for civil engineering including all mineral resources requires the appreciation of basic geology and optimum utilization of a potential area. In addition, understanding of the spatial distribution and deformation of rock units, at the surface, is critical in order to develop a 3-dimensional model of the subsurface geology.

2.2 Field Equipment and Tools

Geological mapping requires a lot of small field equipment and tools are:

a) Hammer and chisels
b) Compasses, clinometers and camera
c) Hard lenses and Tapes
d) Map cases and Field note books
e) Scales and Protractors
f) Acid Bottles and Hand gloves
g) GPS, pedometers and altimeters
h) Stereo net and stereoscopes.

In addition to these pencils, erasers and a jack- knife are needed in the field.

Hammers and Chisels

Geologists need a hammer and some chisels .These are used to break the rock and get samples. A 1 kg hammer is the most useful one in a resistive hard rock, the hammer should be fitted with a good wooden or fiber glass handle or a steel shaft. Normally a 45cm chisel with 2.5 cm cutting edge is used in the field. Chipping a rock sample should be proceeded carefully. People should wear safety glasses.

Compasses and clinometers

Geologists use a clinometer compass which is helpful instrument to detect the directions using a magnetic needle that swings freely on a pivot, in a horizontal plane.
One end of this needle always points to magnetic north. The compass is normally made of brass or an alloy, so that it is not susceptible to any magnetic influence. The circular dial of the compass is graduated into 360 degrees, In normal equipment this graduation is in a clock-wise direction. The east–west cardinal points are juxtaposed in order to enable the user to get direct bearing of the readings by drawn in the reverse direction.

Method of measuring the dip

The compass is useful in determining the dip direction of any dipping strata. The clinometers kept inside of the case is a free-fall type unit, and it always denotes the vertical position by orienting the axis of the compass along the dipping strata. The dip angle between the vertical plane and direction of the strata could be determined.

Clinometer

The clinometer is a simplified model of a compass containing the essential parts for determining the dip of the strata inclination of joints and angle of slope of an embankment, and/or determining the direction and location of objects through forward or backward bearing. All readings are made with reference to the north direction. In all geological mapping, a high degree of accuracy in recording the directions or bearing is essential. Dip, strike and direction are the three major measurements made using the Clinometer.

Sample bags and safety clothing

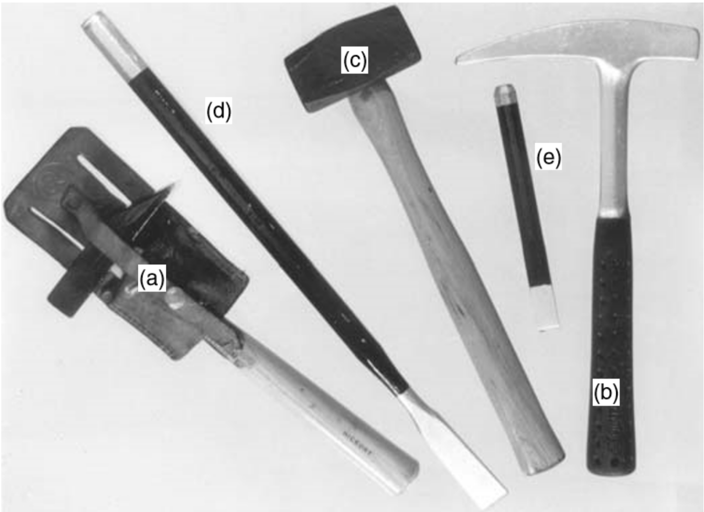
Sample bags which is the best suited for keeping all the geological samples such as a canvas in fabric are an essential item during surveys. Plastic bags may be used where the sample is soft, disintegrated or wet. Safety clothing is a necessary. The clothing includes sturdy shoes, thick outfits such as jeans, hat and glasses. Safety glasses and gloves are important especially when hammering rock samples.

First aid kit

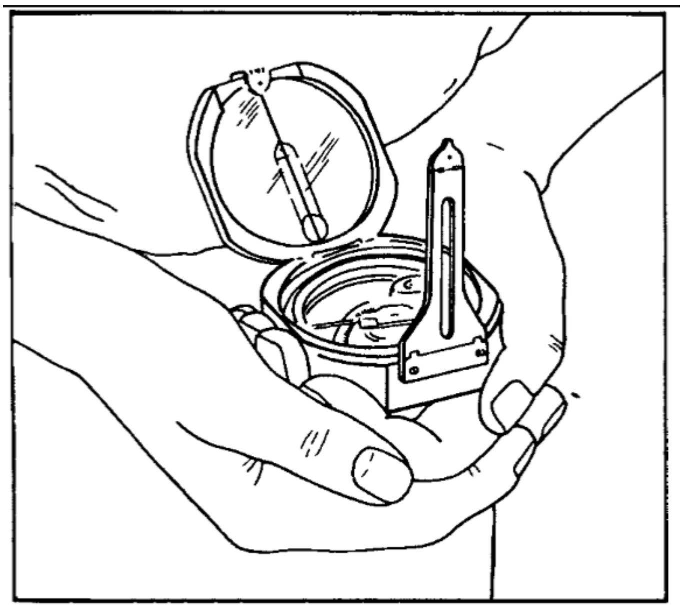
Safety in the field is always a priority; nevertheless accidents cannot be ruled out. A first aid kit should be prepared. It is crucial that at least one person in the field crews received the training for first aid.

GPS

A global positioning system is necessary for conducting geological investigations. It helps in detecting the latitude, longitude, altitude etc. Geospatial parameters with precision and accuracy, A GPS needs a line-of-sight to several satellites for getting the way-points. Grid references of topographic sheets are accurately indicated with other benchmarks for locating outcrops during geological mapping works.



Hammers and Chisels



Clinometer

GPS Garmin

Figure C-1 Mapping Equipment and Tools

(Source: Studies for students; the method of multiple working hypotheses)

2.3 Mapping Techniques

There are many kinds of geological maps, from small-scale reconnaissance surveys to large-scale detailed underground caverns and engineering site plans, and each mapping needs a different technique. However, concerning only with the rudiments of geological mapping, the intention is to provide basic knowledge which can be built upon.

Most suitable mapping techniques and methods of the geological mapping can be established on the basis of the stimulation of imagination obtained through experiences of field works on prevailing field conditions, as well as the scale and quality of topographic base maps.

A geologist must also remember that accurate geological maps are the basis of all geological work, even laboratory work; it is meaningless to make a detailed investigation of a specimen of which provenance is uncertain.

There is no substitute for the geological map and section. Basic geology still must come first– and if it is wrong, everything that follows will be wrong in most cases.

The comparison of mapping techniques is described in Table C-1.

Table C-1 Comparison of Mapping Techniques

Mapping technique	Ideal scales	Indications	Advantages	Disadvantages
Pace and compass	1:100–1:1,000	Rough prospect map. Infill between survey points	Quick. No assistance and minimal equipment needed	Poor survey accuracy, especially on uneven ground
Tape and compass	1:100–1:1,000	Detailed prospect maps. Linear traverse maps. Mine mapping	Quick. Good accuracy. No preparation needed	May need assistance. Slow for large equidimentional areas
Pegged grid	1:500–1:2,500	Detailed maps of established prospects	Fair survey accuracy. Relatively quick. Same grid controls/ correlates all exploration stages	Expensive. Requires advance preparation. Poor survey control in dense scrub or hilly terrain
Plane table	1:50–1:1,000	Detailed prospect mapping in areas of complex geology. Open cuts	High survey accuracy. No ground preparation required	Slow. Requires assistance. Geological mapping and surveying are separate steps
GPS and DGPS	1:5,000–1:25,000	Regional and semi-regional mapping. First pass prospect mapping	Quick, easy downloadable digital survey data. Good backup for other techniques at similar scales	Encourages geological mapping as collection of point data
Topographic map sheet	1:2,500–1:100,000	Regional mapping and reconnaissance. Areas of steep topography. Mine mapping. Base for plotting GPS observations	Accurate georeferenced map base. Height contours	Difficulty in exact location. Irrelevant map detail obscures geology. Not generally available in large scales
Remote sensed reflectance imagery	1:500–1:100,000	Preferred choice. Ideal geological mapping base at all scales	Geological Interpretation directly from image. Stereo viewing. Easy feature location	Scale distortion (air photos). Expensive if new survey needs to be acquired

(Source: Studies for students; the method of multiple working hypotheses)

3 Method of Geological Mapping

A geological investigation can be undertaken using a number of methods depending on the size of a region and the amount of information required. Different types of methods are involved in geological investigations.

Geological mapping is conducted to obtain and provide basic knowledge about the prevailing field conditions, not only through direct observations but also by collecting and analyzing rock, mineral and sediment samples. A geologist conducts field surveys and prepares accurate geological maps by collecting samples and measuring the geometrical aspect of outcrops.
There is no substitute for a geological map. Geological mapping is normally conducted in a project mode with people in a team, a set of special equipment, and a topographic base map. Careful observations are executed during the geological mapping.

Geological mapping is the process of making observations of geology and structure in the field and recording them on a base map and reproducing. Then, it is the form of a geological map. The information recorded must be factual and thorough objective examination of rocks and exposures. There are several methods adopted during this process as traversing.
Following contacts and exposure mapping is the methodology adopted. This involves mapping of poorly exposed regions with indications of rocks from soils, vegetation guides, topography and geomorphology and structures. Contours are the major lines of trace involved during surface geological mapping.

3.1 Surface Mapping

Mapping superficial deposits, drilling, logging and geophysical measurements, underground mapping, pitting, trenching, augering and loaming are some of the subsurface mapping method adopted during surface geological mapping. The features recorded during geological mapping are as follows.

- Rock types and Contacts
- Shape of the rock bodies
- Note on the sequence and relative ages
- Note on the primary porosity and permeability
- Note on the weathering and patterns
- Note on the depositional or magmatic flow features
- Structures including
    * Folding - dip, strike, deformation, orientation of grains
    *Joints – attitude, size, open or closed c) Faults – look for slickensides, fault gouge, breccia and their visible displacements

3.2 Boundary of Formations and Outcrops

The primary purpose of mapping the geology of an area is to trace the contacts between different rock formations, groups and types, and to show where they occur on a map. One way of doing these activities is to follow the contact on the ground and trace it on the map. In some places contacts may be visible and easy to trace. In some places, the contact may not be continuously exposed or the contacts may be beneath superficial deposits. In such cases, stratum contours are considered for tracing using geometric alignments and visual interpolations. When a contact is concealed by alluvium or scree, it should be shown by a dotted line.

3.3 Outcrops

Mapping by exposures is the major method of geological mapping. It is done on the scale of 1:10000 and larger. Mapping of outcrops shows the factual evidence on which interpretations are made.It shows what has been seen and inferred. A form line map is prepared based on the interpretation of the form of geological structure.

3.4 Limited Exposure of Outcrops

In some places, rocks are poorly exposed .They are mostly covered and hidden by vegetation. They may show poor exposure due to coverage by weathered regolith of all rocks. Mica schist forms the poorest exposure but show evidence along the footpaths. In some places, when trees are uprooted due to storm events, rocks are exposed for observation. In addition, road cuttings, railway cuttings and other man made /animal–made excavations, rock types are expected to be exposed for mapping.

3.5 Soil

Soil reflects the parent rocks existing beneath if the soil is not transported. Sandy soil indicates that parent rocks contain more quarts; clayey soils are from Kankan and weathered Dolerites; other basic rocks tend to produce distinctive red-brown soil. More to acidic igneous rocks form lighter colored soil in which mica may be visible. Characteristics of any soil depends on parent rock, climate age and other factors. The association with specific rocks may help in mapping the rock bodies.

3.6 Vegetation

Plants are good indicators of some elements which present in the rocks beneath them. Some are typical around lime stones, on some acid rocks and on serpentinous rocks. Floras are capable of indicating the varieties of rocks in an indirect way, when they are natural. Some plants can indicate metallic ores too. There are many copper indicative plants, uranium indicative plants and even gold indicative plants.

3.7 Topography and Geomorphology

Geomorphology is the science of landscapes. Geologists always look into the rock and relief exposed in various places. In many places, resistive rock bodies stand well above the land surfaces. Remnants, residual hills, pediment zones, flood plain deposits, alluvial cones and fans, drumlins, dunes, boulder clays, and other fluvio-marine deposits show typical relief features as landforms. Volcanic rocks always show very unique exposures. Typical drainage patterns exist in different rock types and relief zones. Landslides bring down a major land mass and expose the basement rocks.

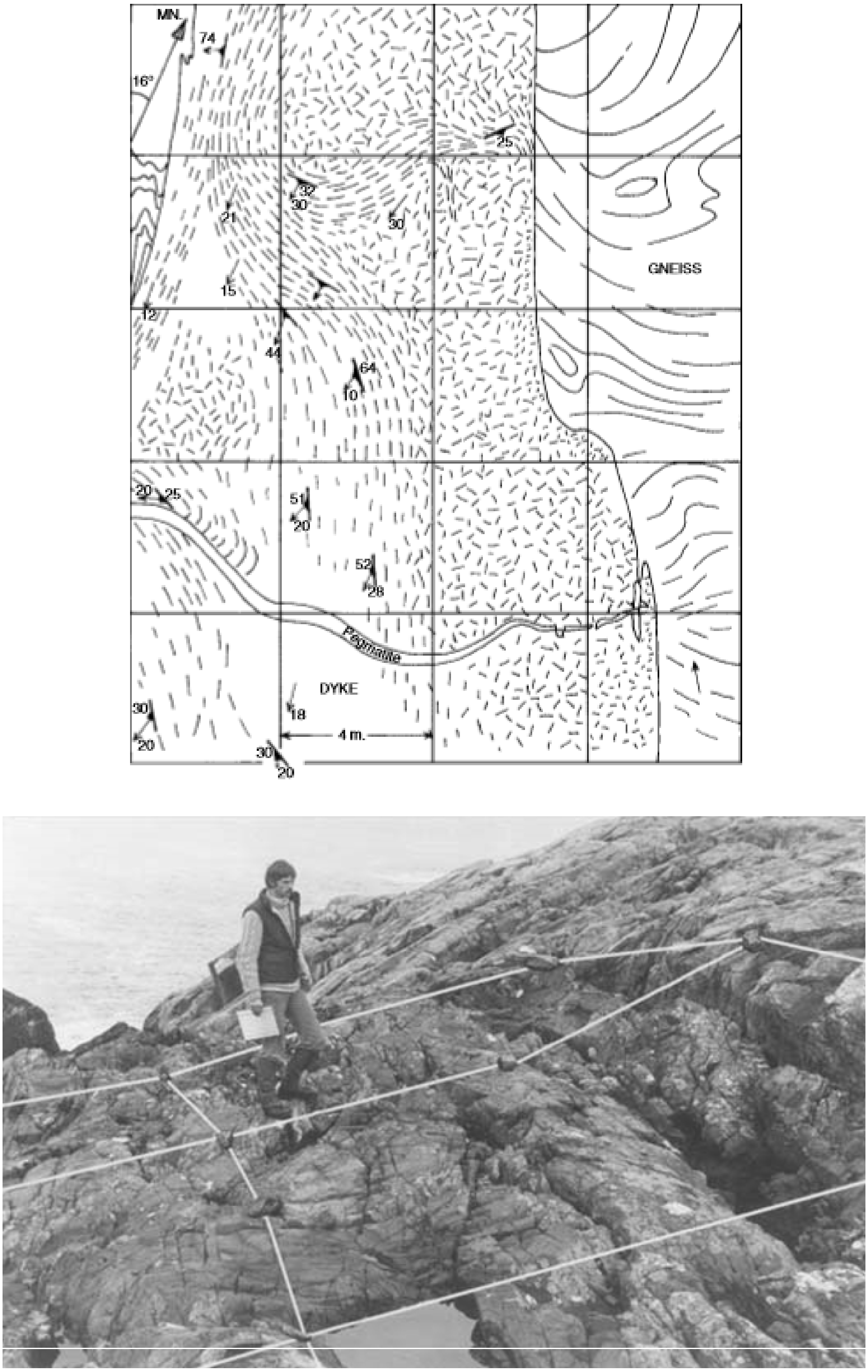


Figure C-2　Mapping a Large Exposure of Outcrops

(Source: Studies for students; the method of multiple working hypotheses)

4 Preparation of Geological Maps

Geological methods rely on the identification of rocks and minerals. They are done for getting an understanding of the environment in which they are formed. Geological investigations aim to find what rock types occur at or close to the surface and how these rock types are related to each other. Surveys orient to know about the disposition of rocks and minerals, their boundaries, ages, and structures. At the end of the investigations, geologists prepare geological maps.

4.1 Interpreting Geological Maps

Once a geological map is complete, the most important task is interpreting it correctly. A map is basically a visual summary of an entire report, and the two should complement each other. When the interpretation process is at hand, it is crucial to have group discussions with all those involved in the actual fieldwork. It is also deemed important to have discussions with those who are experienced in the field of geology as well as other related scientific fields in the area.

4.2 Quality and Interpretations

Basically, the quality of a geologic map will depend upon the accuracy and the precision of the field work. The interpretation of a geological map depends on adequate training, interest and the techniques used. It is necessary to visualize the scenarios that might have been involved during the formation processes of geological features displayed in maps while analyzing the geological maps. The ability to form a three-dimensional (3-D) image from a two dimensional map, is in real sense, a major part of geologic map interpretations.

4.3 Utilization of Geological Maps

A Geologic map helps to protect groundwater resources. Geologic maps are also used for habitat prediction. Geologic maps are used to evaluate the mineral resources. Geologic maps are also used to delineate landslide prone areas. Geologic map guides to delineate the earthquake-prone areas and help in damage prediction. A geologic map delineates volcanic hazards and aids in the mitigation of earthquake damage, cyclone damage, tsunami damage and others. Geologic maps show the locations of exploring sand and gravel resources. Geologic maps identify the economic resources and mining areas. A Geologic map guides transportation planning.

4.4 Preliminary Mapping

Reconnaissance-level surficial maps are prepared by examining gravel pits and other readily accessible exposures of surficial earth materials, and gathering information on remote areas through the interpretation of aerial photographs in the preliminary stage.

Sand, gravel, clay, and other unconsolidated materials that overlie bedrock are known as surficial deposits. These deposits cover most of surface of the land and include sediments deposited by water, wind, and glacial ice.

Surficial geologic maps differ significantly from soil maps because they identify the deeper earth materials that lie between the soil zone and the underlying bedrock (soil commonly develops by weathering of the uppermost part of these materials). In addition to the distribution of unconsolidated surficial deposits, the maps also show landforms, directions of the mass movement, and other specific features. The map explanation describes the composition of each deposit, its characteristic topography, and its origin.

Surficial geologic maps are compiled from field work, well and test hole data, and aerial photograph interpretation. In the field, the geologist examines as many exposures of surficial materials as time and access allow, recording information on the type, thickness, texture, and structure of the sediments exposed.

Reconnaissance-level surficial maps are prepared by examining gravel pits and other readily accessible exposures of surficial earth materials, and gathering information on remote areas through the interpretation of aerial photographs.

4.5 Detailed Mapping

Detailed-level mapping involves more comprehensive field work, together with compilation of data from wells, test borings, seismic surveys, and other sources. Well and test hole information is important in determining the thickness of surficial deposits and mapping the extent of older deposits covered by younger materials.

4.6 Uses of Surficial Geology Information

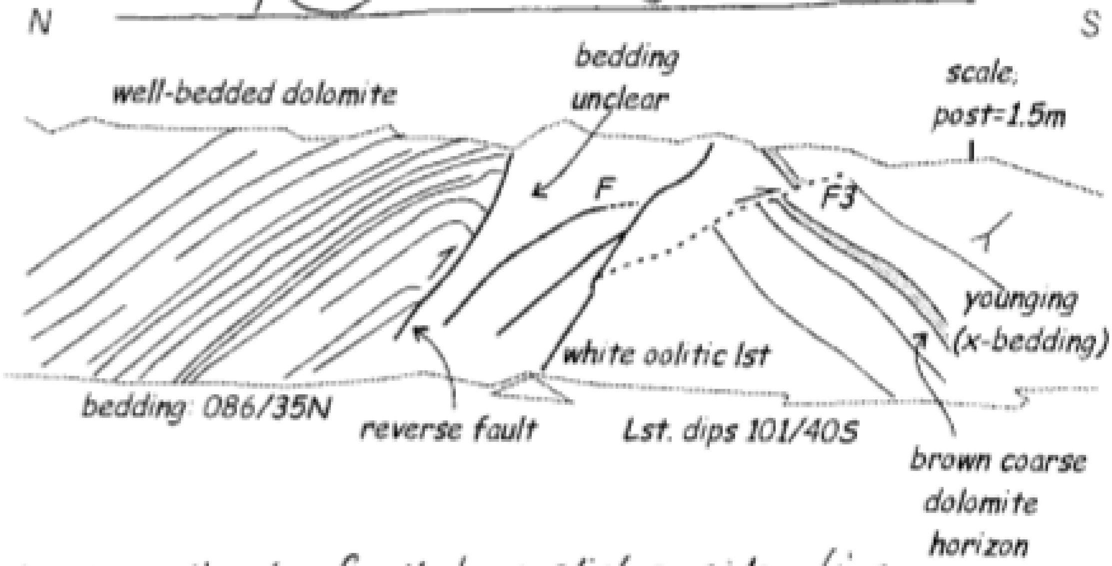
Identification of surficial materials is critical for making a number of land use decisions, including determining the suitability of an area for development, planning major construction projects, or looking for sources of ground water. Surficial geologic maps also provide information on the location and extent of sand and gravel deposits. Large scale (1:24,000 or larger) maps are most useful for investigations of particular sites.

Obtaining maps and publications can be the subject of the geographic area. Search results are linked to online publications. To order printed materials see the ordering instructions.

in greater detail than on 07/04/03.

A set of faults is seen south of loc (108) near the Junction between Langland Oolite Fm. and the Mumbles Dolomite Fm.

Locality (179): S of bathing huts (567 129)



Most northerly fault has slickenside lins (calcite fibres) on fault surface (at beach level) These lineations pitch at 66°E and suggest reverse movement.

Oolitic Lst. to S of faults are right way up (X-bedding). Need to revise ideas of last week (12/04) in Langland Bay — implies simple upright fold not overfold. Spec (28) – deformed ooids ? near to fault

Figure C-3　Sketch Notes of Geological Mapping

(Source: Studies for students; the method of multiple working hypotheses)

5 Remote Sensing for Geological Mapping

Today, the availability of aerial photography and remote sensing from satellite imagery, as well as the computer capability for storage, recovery and evaluation of data are used for geologic mapping and other purposes. These methods have almost replaced many old methods of geologic data collection, plotting, and interpretation. Remote sensing technology and satellite products provide fast access to all geospatial data. Also, a greater and finer resolution of data and images is readily available in planimetric and 3-D mode at any desired scale and time. These data can be integrated with Geographic Information Systems (GIS) for vertical and horizontal comparison. Maps can be combined with layers of information on topography, minerals, water, energy and the environment. These technological advances have increased the usefulness of and public access to geologic maps.

5.1 General

Light from the sun reflects from the earth's surface and radiates in all directions, including (provided it is not blocked by clouds) back into space. Any system, which can record the intensity and wavelengths of the reflected light, and reproduces the data as an image, is known as reflectance imagery. The instrument that does this can be mounted on either an aircraft or satellite. The word photograph is specifically used for images recorded onto photographic film by a camera lens system. This section deals primarily with aerial photographs – i.e. photographs taken looking vertically down from an aircraft – but most of the comments apply equally to the handling and use of hard copy satellite images. Details about how satellite images are acquired and presented, and how they can be used as a remote sensing geophysical tool (spectral geology), will be found in Chap. 8. In aerial photography, a camera mounted in an aircraft takes a series of photographs as the plane flies in regular parallel passes over the terrain. Aerial photographs have the advantage of being relatively cheap to collect and, since they are taken at low altitude, can show great detail. Overlapping adjacent photographs along the flight path enables subsequent stereoscopic (3-D) viewing. Aerial photographs typically offer a resolution of ground features that range in size from a few centimeters upwards, depending on the height of the aircraft above the ground and the quality of the camera optics used. Film is an analog method of recording data that offers exceptionally high resolution that is ultimately limited only by the grain size of the chemical emulsion on the film. The resolution of the film used for aerial photography is an order of magnitude greater that is currently achievable with electronic recording methods. Aerial photographs are typically collected for normal viewing at scales of from 1:500 to 1:100,000, but, unlike digital images, they can be enlarged many times without losing resolution.

5.2 Acquiring Aerial Photographs

Many governments have acquired aerial photo coverage of their territories and these can usually be purchased from the relevant government agency. The aerial photo is provided by the Survey Department in Sri Lanka (Table A-1). Needless to say, the quality and coverage of this product varies enormously, but since it is a cheap resource, it is always worth checking to see what is available. In areas where there has been a high level of mineral exploration, surveys flown by previous explorers may also be available. If none of these avenues yields a useful product, it is possible to commission the survey. This is comparable in cost to purchasing high resolution satellite imagery for the same area and gives the opportunity to specify a scale and coverage that will suit the project.

5.3 Geological Interpretation

Aerial photographs (along with other similar remote sensed products such as satellite and radar imagery) provide both a mapping base on which to record field observations and an integrated view of landscape on which map-scale patterns of lithology and structure can be directly observed or interpreted. Where available at a suitable scale and resolution, they are the pre-eminent medium upon which to construct a geological map. For any geological mapping programme making use of remote sensed imagery, image interpretation represents the idea-generating, integrative, control and planning phases of that programme. The initial interpretation made from the images will provide

- Definition of areas of outcrop and areas of superficial cover;
- Preliminary geological interpretation based on topographic features, drainage patterns, colours and textures of rocks, soils and vegetation, trend lines of linear features, etc.;
- Geological hypotheses for field checking;
- Selection of the best areas to test these hypotheses;
- Familiarity with the topography and access routes to assist in logistic planning of the field programme – access roads and tracks, fording points for streams and gullies, potential helicopter landing sites, and others.

Aerial photo or satellite image interpretation needs to be carried out before, during and after the field phases of the mapping process. Obviously, detailed interpretation making use of stereo viewing can be most conveniently done at an office desk, but, as ideas change or evolve, interpretation of photo features will have to be attempted in the field as well. The ability to use a pocket stereoscope on the outcrop is an essential skill to acquire. Since making and interpreting geological observations on the photo and outcrop are two aspects of the same process, they should ideally be carried out by the same person. Whenever possible, the field geologist should do his own interpretation. Geological interpretation of remote sensed imagery complements field mapping and should never be regarded as an adequate substitute for it. Skills required for the geological interpretation of remote sensed imagery are very much the same as those needed for field mapping. However, some practical techniques need to be learned in order to turn aerial photo observations into usable geological maps. The next section describes some of these techniques.

6 Collection of Supporting Data

When it is essential to investigate the rock lying beneath any overburden, pits and trenches are made to study them. Many contacts could be well identified from trenches. In many cases, identifiable fragments of weathered rocks can be from shallow anger-hole drilling method loaming is a method of mapping in poorly exposed and deeply weathered regions.

6.1 Test Drilling

Test drilling is commonly employed to locate formation at depths. It is also conducted to confirm their presence when there is a gap in other information, and also when there is a need to find out the details of structure and their geometry. There are two kinds of drilling methods adopted which are percussion and rotary drills. By percussion drills, rocks are fully crushed and powdered or drilled using a ring like drill bit to obtain a core sample. By rotary drills, only crushed products will come up for identification

6.2 Preparing a Geological Map

A geological map is a spatial representation showing the distribution of rock units and structures across a region. It is drawn on a plane surface. A map showing the occurrence of structural features across a region, the distribution of rock units, and their types and relationship in age is termed a geological map. A geological map is expected to show all the rock types of a region, their structures, geological formations, geothermal manifestations, age relationships, distribution of mineral ore deposits and fossils. All these features are to be superimposed over a topographic map or a base map. The amount of detail shown in a map depends largely on the scale and a smaller scale will naturally disclose finer detail.

6.3 Mapping of Groundwater Sources

The aim of our mapping of groundwater resources is to provide an overview of the locations of large groundwater aquifers and of potential abstraction volumes. The aim is also to show the direction of groundwater flows and any areas where the groundwater may contain high concentrations of salt or fluoride. SGU also gathers information on Swedish drinking water wells, energy wells, major water supply sources, springs and groundwater chemistry.

7 Reporting

Geologic reports and maps prepared to assist in public decision making. Geologic maps and reports can be used in regional-scale environmental management and resource management planning documents to assist geologists, engineers and land-use planners in making decisions that affect public health and safety, critical environmental habitats, water quality, uses of public lands, and help identify areas where more detailed geologic studies are needed. It is often said that a report is as good as its data collected from field work. It is necessary that there is a need to collect very clear and accurate data. Nothing can be overemphasized. Ultimately, they are taken back to the laboratory for sorting, interpretation and analysis when all possible available data are collected. This phase is the most challenging phase. Any wrong analysis or misinterpretation of data could lead to an inaccurate report, and in consequence, to misinformation. Geological symbols used for geological mapping are shown in Figure 7.1.

8 Conclusion

Geologic map is useful tool. It provides much information for economic resource discovery and development. It is useful in the design of buildings, canals, roads and drainage of farmland as well as environmental planning, and development. It is essential to understand the procedures of conducting geological investigations and mapping the earth's features. There are lot more aspects to learn in this topic alone. Geological mapping is fundamental for every earth scientist.

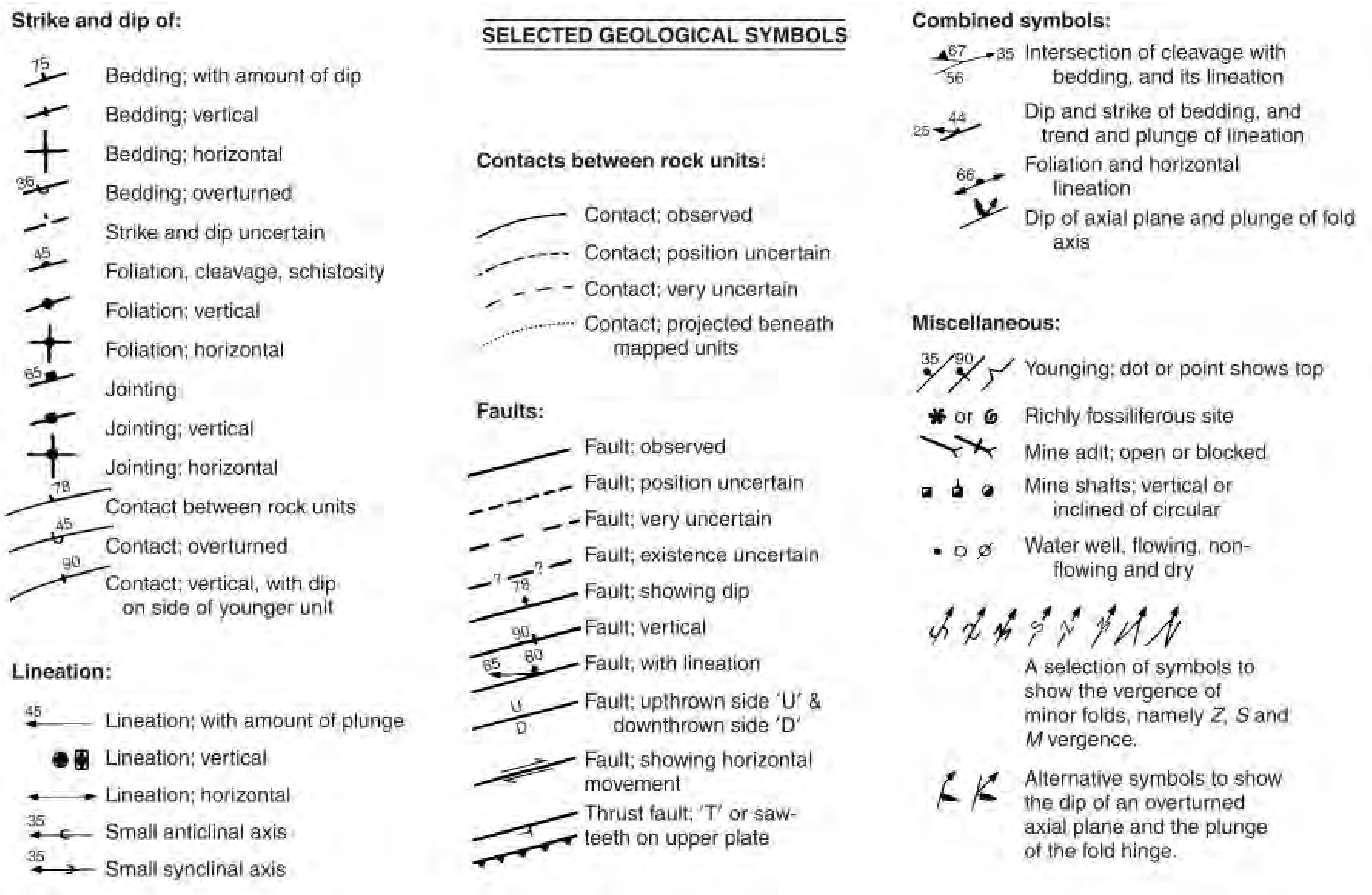


Figure C-4 Geological Symbols used for Geological Mapping

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Proceedings of a Workshop on Digital Mapping Techniques: Methods for Geologic Map Data Capture, Management, and Publication
Convened by the Association of American State Geologists and the United States Geological Survey
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Hosted by the Kansas Geological Survey
U.S. GEOLOGICAL SURVEY OPEN-FILE REPORT 97-269

D. Core Drilling

1 Introduction

1.1 General

Rock core drilling is the most commonly used method for subsurface exploration of a rock tunnel project. In rock coring, a core barrel is advanced through rock by the application of downward pressure during rotation. Circulating water removes ground-up material from the borehole while also cooling the bit. The rate of advance is controlled to obtain the maximum possible core recovery.

This chapter provides a summary of commonly used core drilling techniques and recent developments. Furthermore, this chapter presents the basic planning criteria for the locations and depth of core drillings as well as the interpretation and application of core drilling data to be obtained.

Additional information is explained in ASTM D 2113 (Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation) or AASHTO T225 (Standard Method of Test for Diamond Core Drilling for Site Investigation).

1.2 Purpose of Core Drilling

Core drilling (or rock coring) is the primary method used to obtain intact samples of rock for testing purposes and for assessing rock quality and structure. Boreholes resulting from rock coring are also used to obtain groundwater data, to perform in-situ tests and to install instruments. The objectives of rock core drillings for a rock tunnel project are detailed as follows:

1) To investigate the geological structure and subsurface stratigraphy as well as materials/zones of complex geology;
2) To obtain core samples for observation, classification and examination of rock mass structure (discontinuities) for geological core logging and rock mass quality assessment;
3) To collect rock core samples (intact) for laboratory tests;
4) To provide holes for in-situ tests, such as the Lugeon test;
5) To provide holes for installation of geotechnical instrumentations, such as piezometer for hydrogeological measurement; and
6) To provide holes for other geotechnical/geological survey, such as the borehole geophysical survey.

In addition to the above-mentioned objectives, the findings of core drillings are commonly used to geologically and geotechnically interpret and correct the results of the seismic refraction survey – one of the most commonly used methods for subsurface exploration of a rock tunnel.

2 Plan and Location of Boreholes for Rock Tunnels

The number, depth and spacing of core drillings required for the subsurface investigation of a rock tunnel project are generally dependent mainly upon the type and size of the tunnel structure and the complexity and variation of local subsurface conditions because no general rules are applicable in all situations.

The following general guidelines are given with respect to location, spacing, depth and direction of coring drilling.

2.1 Borehole Locations

Core borings are located to focus on the following locations and sites:

1) Along the tunnel alignment;
2) In portal slopes;
3) In access areas; and
4) Complex geological zones and areas, such as major shear zone or fault.

Portal slopes or areas should be investigated in detail. The portal slope generally has lower overburden thickness, which means a zone of high weathering and high permeability with an increase in water flow. These factors can cause greater deformability and lower strength than in the rest of the tunnel. In addition, landslides and slope instability are the main potential geological hazards to be considered in any portal slope.

Complex geological zones or areas provide unfavorable subsurface conditions contributing to the instability of the rock masses around tunnel structures during and after construction, consequently leading to construction delays and construction cost overruns.

These zones should be also investigated and defined at the design stage. These zones or areas include below:

1) Faults and tectonic zones;

2) High in-situ stress zones;
3) Weak, expansive, squeezing and swelling materials areas;
4) Aggressive and abrasive materials areas;
5) Zones or areas of potentially high seepage rates and large inflow water; and
6) Zones or areas connected with underground gases and high temperature.

2.2 Borehole Spacing

A general guideline from FHWA Road Tunnel Manual (2009) has been given for determining the spacing of boreholes for road tunnel projects, as shown in Table D-1 below.

Table D-1 Pressure Magnitudes Typically Used for Each Test Stage

Tunnel Type	Ground Condition	Typical Borehole Spacing (feet)	Typical Borehole Spacing (m)
Cut/Cover Tunnel	-	100 to 300	30 to 100
Rock Tunnel	Adverse	50 to 200	15 to 50
	Favorable	500 to 1000	150 to 300
Soft Ground Tunnel	Adverse	50 to 100	15 to 30
	Favorable	300 to 500	100 to 150
Mixed Face Tunnel	Adverse	25 to 50	10 to 15
	Favorable	50 to 75	15 to 20

(Source: Technical Manual for Design and Construction of Road Tunnels, FHWA-NHI-10-034, 2009)

Note: Adverse = Area with complex geology or very variable lithology, Favorable = Area with more uniform geology

The above guideline can be used as a starting point for determining the number and locations of borings. However, especially for a long tunnel through a mountainous area, it may not be economically feasible or the time sufficient to perform core drillings accordingly. Therefore, engineering judgment will need to be applied to establish the investigation program together with other subsurface exploration, such as a seismic refraction survey to reduce the quantity of boreholes.

In addition, a minimum of one borehole is commonly required for each portal slope. Two to three boreholes should be added to the portable slopes where a potential landslide is suspected and identified within such a portal slope. For major shear zone and fault crossing the tunnel route, additional boreholes are recommended to observe and evaluate the rock mass conditions.

2.3 Drilling Depth

In principle, Core drilling should be extended to at least 1.5 times of the tunnel diameters below the proposed tunnel invert. If there is uncertainty regarding the final profile of the tunnel, especially at the planning and preliminary design stages, the core drillings should be extended at least two or three times of the tunnel diameter below the preliminary tunnel invert level.

2.4 Borehole Diameter

The diameter of boreholes should be determined in view of core quality and rock core sample size required for laboratory tests. It is preferable to perform rock coring with as large a core barrel as possible to optimize core recovery and to minimize core damage due to drilling action. In general, large-diameter (60–90 cm) boreholes are used to obtain high-quality cores and high core recovery.

The laboratory test specimen should be a rock cylinder of length-to-width ratio (H/D) of 2. ASTM methods use a HQ core diameter of 2.5 inches (64mm).

In addition, the rock quality designation (RQD) is an expression of intact core lengths greater than a threshold value of 100 mm along any borehole, and is the method most commonly used for characterizing the degree of jointing in borehole cores. For the RQD measurement, the International Society for Rock Mechanics (IRSM) recommends a core size of at least NX (core size 54.7mm) (refer to Table D-2 below).

Accordingly, the diameter of boreholes to be drilled should be generally not less than 76 mm.

Table D-2 Diameters of Core and Borehole

Size	Core Diameter (mm)	Borehole Diameter (mm)
EX	21.5	37.7
AX	30.1	48.0
BX	42.0	59.9
NX	54.7	75.7
HQ	63.5	96.3

(Source: JICA project)

2.5 Drilling Direction

Three directions of core drillings are generally used to explore tunnels: vertical, directional (inclined) and horizontal. In general, vertical and inclined drilling should be planned and conducted to provide a continuous record of ground conditions and information which is directly relevant to the tunnel alignment. However, for a deep mountainous tunnel alignment, horizontal core drillings are economical and thus recommendable to avoid unnecessary drilling of overburden materials and disruption to the ground surface activities, local community and industries.

In addition, inclined and horizontal drillings (or boreholes) are prone to fail or collapse, and should be thus stabilized with drilling fluid and casings.

3 Equipment and Procedure for Core Drilling

3.1 Equipment and Material Used for Core Drilling

Rock core drilling exploration is intended to obtain continuous and high-quality cores and samples, as well as a high percentage of core recovery. Some techniques for obtaining good core quality and recovery have been developed, especially in equipment and drilling fluid, as well as experiences in treating problematic geological and groundwater conditions, as described in the following sections.

(1) Equipment

A rotary drilling machine, drill rods, a core barrel to receive the core, and a cutting bit are needed. Hydraulic-rotary core drilling method is commonly used to collect rock core samples. The rock core samples are obtained by using core barrels equipped with diamond or tungsten carbide tipped bits. Although the basic method of coring drilling has changed little, coring and sampling tools and methods have advanced for obtaining good core quality and recovery. There are several basic types of core barrels: single-tube, double-tube, and triple-tube, as summarized in Table D-3 and Shown in Figure D-1. Single tube core barrel generally provides poor recovery rates, whereas double-tube and triple-tube barrels obtain good recovery rates and are thus recommended for the use of subsurface exploration for rock tunnels.

Table D-3 Coring Tools and Methods

Coring Tool	Description and Application
Single-tube core barrel	· Coring hard rock where high recovery is not necessary. · Circulating water washes out soft, weathered, or fractured rock.
Double-tube core barrel	· Coring most rock types where high recovery is not necessary and rock is not highly fractured or soft. · Recovery often low in soft or fractured rocks and unconsolidated materials.
Double-tube swivel-type core barrel	· Superior to double-tube core barrel above, particularly useful to obtain high recovery in friable, highly fractured rocks and unconsolidated materials. · Barrel is more costly and complicated than others mentioned above. · Not needed in good-quality rock.
Triple-tube core barrel	· Like the double tube above, but has an additional inner liner, consisting of either a clear plastic solid tube or a thin metal split tube, in which the core is retained. · This barrel best preserves fractured and poor-quality rock cores.

(Source: JICA project)

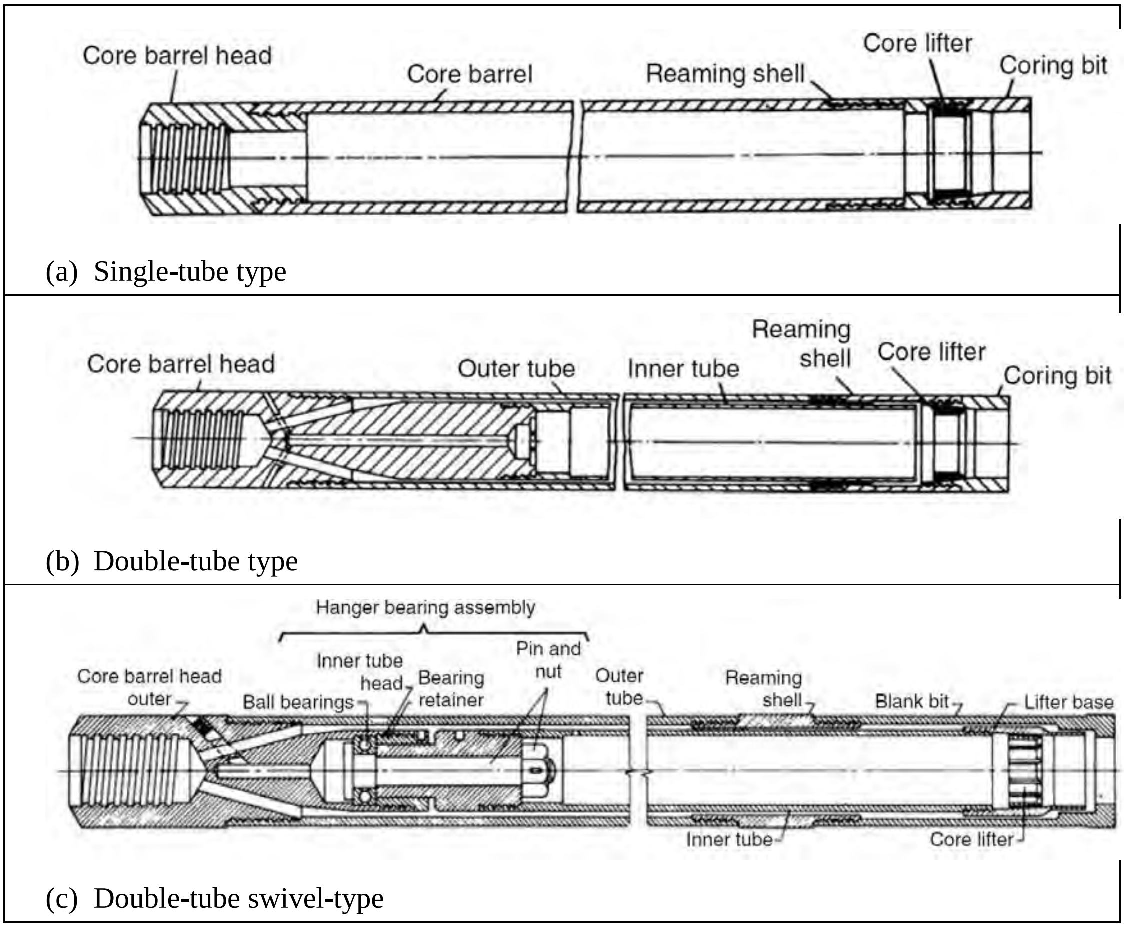


Figure D-1 Typical Core Barrels

(Source: Modified from Geotechnical Engineering Investigation Handbook, Roy E. Hunt, 2005)

The core drilling machine, drilling-mud mixing equipment and drilling-fluid circulating pump may be the same for the different type of tubes. However, use of double-tube swivel-type core barrel systems ensure borehole stability and good core recovery while drilling and coring. The double-tube swivel-type core barrel consists of an outer tube equipped with a diamond-set reaming shell and core bit at the lower end, a retrievable locking inner-barrel assembly equipped with a core retainer at the lower end, a swivel-type ball-bearing head at the upper end of the inner barrel, and a locking head and spear above the ball-bearing swivel assembly. Bits for coring unconsolidated or friable rocks are of the recessed, bottom-discharge types. Because of the recessed waterway, the drilling fluid does not come in contact with the core. Thereby practically core erosion is eliminated.

In addition, the minimum core barrel to be used should be an NW or NQ to cut a 3-inch (or 76 mm) hole and retrieve a 2-inch (51 mm) diameter rock core.

(2) Drilling Fluid

Drilling fluid is circulated to carry cuttings out of the hole, cool the bit, lubricate the rotating drill pipe, and so forth. High-quality coring drilling uses foam surfactants or equivalent materials as drilling fluid to reduce the friction at the cutting edge and remove the cuttings, allowing a reduction in fluid pressure from that when using water or mud and obtaining high-quality cores and samples.

In addition, it is highly recommended that the utilization of a strictly controlled drilling-fluid program whenever hydraulic-rotary coring of unconsolidated material is done. When coring unconsolidated materials, water or other thin drilling- fluid mixtures cannot be used. Viscosity of the drilling fluid should be high and the weight must be low. Drilling mud having these restrictive properties are made using very high-yield (low solids) bentonites. Low-solid polymers can be added to the bentonites mixture to hold the weight down, while still increasing viscosity. When coring unconsolidated materials, such drilling fluid forms a quick, thin filter cake on the borehole wall, as well as on the exterior of the cores, so that little or no filtrate invasion or erosion of the drilled core occurs.

3.2 Coring Procedure in Rock

(1) Drilling and Casing through Overburden

The bedrock is generally covered with overburden materials, and therefore, in most hydraulic-rotary coring programs for rock coring, the overburden must be supported so rock coring can be accomplished without danger of the unconsolidated material caving or falling into the borehole. Casing out or supporting the overburden is usually accomplished in one of the following ways:

1) A heavy-wall drive casing with drive shoe attached is driven until resisted by means of a heavy drive hammer. If refusal is reached prior to encountering bedrock because of boulders, gravel, stiff clay, and so forth, a drill rod with a chopping bit or cutting bit attached is used to chop or drill the material out of the inside of the drive casing, and the cuttings are carried out of the drive pipe with the circulated drilling fluid. This chopping or drilling out may proceed some distance ahead of the drive casing, depending on the types of material drilled. The drill rod is removed from inside the drive casing, and driving of the casing proceeds as above by the addition of necessary sections of drive casing; alternate washing out and driving is continued until the drive shoe is seated in bedrock.

2) A drilling-in method of installing the casing is used, particularly when the overburden is fine-grained material. A diamond cutting bit on the bottom section of the drill casing, and circulating drilling fluid is used to drill the casing through the overburden and down to consolidated material. If this method is used, a chopping bit may be needed to clean out accumulated debris from inside the drill casing. This drilling-in of the casing is preferred when alternate hard and soft formations are anticipated.
3) A popular method for installation of casing through overburden is to use a mud-rotary drill through the overburden using a drag bit or roller-cone bit, and viscous drilling mud to build a filter cake and support the wall of the borehole so the casing may be installed after the drill pipe is removed.
4) This method for supporting the overburden is the same as that described in method 3) above, except no casing is installed in the hole. The filter cake and hydrostatic head of the drilling fluid in the borehole is relied on to hold the overburden in place. This is the least-preferred method for supporting the overburden because of concern about borehole caving and the possibility of pebbles or gravel falling to the bottom of the hole. These pebbles or gravels can result in considerable damage to the diamonds in the coring bit.

(2) Coring Rock

After the overburden is stabilized using casing or drilling fluid, a diamond core barrel consisting of a double-tube barrel, a diamond-set reaming shell, a non-rotating inner barrel equipped with a core catcher and shoe, and a diamond cutting bit that meets the requirements for cutting the material are attached to the drill pipe and lowered to a point just off the bottom of the borehole. The spindle rod is connected to the top of the drill pipe by means of a mechanical or hydraulically tightened fastening chuck. Fluid is pumped through the drill pipe to establish circulation.

Starting of the core barrel is accomplished using minimal down pressure and slow turning. After the core barrel is started, the down pressure and rotational speed are increased for obtaining the optimum penetration rate in the rocks being drilled. Down pressure and the rotational speed are variable, depending on the hardness of the rock and the diameter of the core barrel. Although down pressure and rotational speed are primarily dictated by the experience of the driller, the following observations can be made:

1) The down pressure and rotational speed working together control the penetration rate and bit life. In the case of hard rock coring, fairly high down pressure can be applied, if the rotational speed is compatible. When coring hard rock with an NW size core barrel (inside

dimeter:76 mm), the core barrel can safely be rotated at 600 r/min using a down pressure of as much as 2,000 lb, assuming the drill pipe is straight and no undue vibrations or chattering of the drill pipe result. The rotational speed must vary accordingly with the diameter of the core barrel used. In practice, under ideal conditions using a straight drill pipe in a straight borehole, maintaining proper drilling-fluid conditions, rotational speeds as much as 2,500 r/min for AW core barrels (inside dimeter: 48 mm), and 1,500 r/min for NW core barrels, can be achieved in coring hard, competent rock. These speeds can be determined by an experienced driller, who recognizes that any vibration of the drill pipe will cause vibration and chattering of the bit, resulting in bit blockage, broken core, and poor core recovery.

2) If coring is performed in abrasive, fractured, or friable rock, the rotational speed and down pressure must be decreased accordingly to maintain smooth running of the drill pipe. If the coring bit penetrates these materials too fast, it overdrills, which breaks out pieces of rock and results in bit blockage and poor core recovery. After the increment of core has been cut that corresponds to the core barrel length, the core must be broken off at the bottom of the core bit. The core retainer will slide down slightly in the beveled shoe, imparting an ever-increasing grip on the core, and the core will almost always break off in the hole at or very near the bottom of the core bit. After the core has been broken loose and prior to pulling the drill pipe for removal of the core, circulate the drilling fluid for several minutes to clear the cuttings from the borehole.

(3) Rock Core Sample Storage

The drilled rock cores are the sample record for the subsurface geology at the borehole location within the project area, and therefore must be properly stored and preserved without loss and damage for observing and sampling, once required at any time till the completion of the project.

In general, these rock cores should be stored in core boxes (Figure D-2) in which wooden spacers are placed along the core to identify the depth of run for core observation. They should be placed in the core box from left to right, with the top to the left, bottom to the right, starting at the top of the box so the core reads like a book.

Figure D-2 An Example of Core Box Storage

(Source: JICA project)

The ends and top of the box should be marked with black enamel paint or an indelible felt pen. Core blocks, which mark the depths, are placed between each run and the depth marked. Data on the outside of the left end of the box should include the project name, borehole number, box number, and depth interval in the box. Filler blocks (spacers) are necessary to properly record information.

3.3 Causes of Low Recovery

(1) Coring Equipment

Worn bits, improper rod sizes (too light), improper core barrel and bit, and inadequate drilling machine size all result in low recovery. In one case, in practice, coring to depths of 30 to 50 m in a weathered to sound gneiss with light drill rigs, light “A” rods, and NX double-tube core barrels resulted in 40 to 70% recoveries and 20 to 30% RQD values. When the same drillers re-drilled the boreholes within a 1m distance using heavier machines, “N” rod and HX core barrels, recovery increased to 90 to 100% and RQDs to 70 to 80%, even in highly decomposed rock zones, layers of hard clay, and seams of soft clay within the rock mass.

(2) Coring Procedure

Inadequate drilling fluid quantities, increased fluid pressure, improper drill rod pressure, or improper rotation speed all affect core recovery.

(3) Rock Conditions and Groundwater Conditions

Fractured or decomposed rock and soft clayey seams cause low recovery. Rock quality can vary substantially for a given location and rock types. Some typical geological and groundwater conditions that may cause low-quality and loss of cores and samples during drilling are listed below:

1) Highly weathered or broken rocks
2) Faults and joints (or sheared and jointed rocks)
3) Sensitive or quick clays
4) Unstable ground (landslide clay)
5) Very dense glacial soils
6) Boulders and cobbles
7) Loose, granular soils
8) High groundwater and saturated soils
9) Sampling below water table

4 Interpretation and Presentation of Core Drillings

4.1 Geological Log

Geological logs are generally prepared to provide complete documentation on the drilling, sampling, and coring operations and on the materials and other aspects of the subsurface encountered, including groundwater conditions, in-situ test depth and result. They provide the basis for analysis and design, and therefore complete documentation as well as clear and precise presentation of all data are necessary. Normally, two sets of logs are required: field logs and final logs, each serving a different purpose.

4.2 Geological Section along Tunnel Alignment

The finding or results of core drillings together with data from the various exploration methods are used as a basis for typical geologic sections to illustrate subsurface geological conditions. The objective of the geological section preparation is to clearly illustrate the problems of the geological environment influencing design and construction and to characterize the subsurface geotechnical conditions for the design of tunnel supports.

For rock condition evaluations and rock tunnel engineering design, it is often useful to prepare two dimensional sections on which are plotted all of the key engineering property data as measured in the field and in the laboratory along the tunnel alignment, as listed below:

1) Overburden thickness
2) Groundwater level
3) Rock types and geological structures including shearing and fracturing zones
4) Highly permeable materials and potential seepage zones
5) Rock mass classification

E. Seismic Exploration

1 Introduction

Elastic waves include longitudinal waves, traverse waves and surface waves. Comparing the propagation speeds of these waves, the longitudinal wave is the fastest, followed by the traverse wave and the surface wave. Therefore, the longitudinal wave takes the initial letter of "primary" in the sense of the wave arriving at "primary" and is called the P-wave. The traverse wave is said to be an S-wave taking the acronym for "Secondary" following it.
P-waves are widely used in investigations of civil engineering fields such as tunnels and dams. On the contrary, S-waves and surface waves are used for soil foundation in flat topography conditions.
Generally speaking, seismic refraction prospecting refers to measuring initial wave of the P-wave.

The seismic refraction prospecting produces an artificial seismic wave by a blasting or a large hammer. The seismic wave is refracted at the underground geological boundary and returns to the surface. This refracted wave (P-wave) is measured by the surface instrument.
It will give an overall picture of the subsurface foundation condition and detect depth of solid rock, locations of weak zones, faults and others.

2 Planning and Preparation

In addition to negotiations to enter private lands, it is necessary to explain the work contents of seismic refraction prospecting to landowners and neighboring residents. Also, when using dynamite for vibration, it is necessary to apply for permission to use explosives.

The exploration plan determines the length of survey line, the vibrating point, the receiving point interval and others. in consideration of the conditions such as the purpose of the survey, the topography of the survey site and the geology.
In order to develop a good plan, it is necessary to have knowledge about measurement and analysis methods. In addition, it is necessary to obtain as much information as possible of the geology of the surveyed area in advance. If this is insufficient, data necessary for analysis is insufficient, leading to incorrect analysis results.

2.1 Length of Survey Line

To determine the length of a line, first determine the depth of exploration. In general, the line length shall be 5 to 7 times the depth of exploration.
In the case of the tunnel survey, the length of the longitudinal line is generally set to be the tunnel length plus 30 to 60 m. Therefore, the survey lines will extend at least 30 m from the

tunnel portal headway.

2.2 Interval of Receiving Point and Seismic Source Point

2.2.1 Receiving Point Interval

The distance between the receiving points of the civil engineering geological survey is either 5 m or 10 m. In general, it is carried out at intervals of 5 m, but long tunnel surveys may be carried out at intervals of 10 m if the depth of exploration is deep.

2.2.2 Seismic Source Point Interval

The finer interval between the vibration points improves the analysis accuracy. However, if the intervals between the vibration points are too fine, the working efficiency will be poor. Generally, the distance between the vibration points is 30 m to 50m when the receiving point interval is 5 m. When the receiving point interval is 10 m, it is set to 60 m to 80m.

The shooting for each spread shall be made for at least at six (6) locations on the traverse line or its extension; i.e., two at both ends of a spread, two at the middle points of the spread and the other two for remote shootings beyond the ends of the spread.

2.3 Seismic Sources

The accuracy of analysis of seismic refraction prospecting is greatly affected by the quality of acquired records. And the quality of the record is greatly affected by the magnitude of the vibration. Therefore it is common to use dynamite for the vibration source.

Administrative procedures for using explosives and detonators are follows. A permission from Geological Survey and Mines Bureau (GSMB) is required to utilize explosives and detonators for seismic prospecting. Furthermore, surveyors should send applications to Ministry of Defense and Police following GSMB’s instruction and get permissions to keep and utilize explosives. Experienced/licensed engineers for using explosives must be assigned and control the prospecting work.

A guideline for vibration and sound noise issued by Central Environmental Authority (CEA) also restricts using explosives. Surveyors should apply and conduct seismic prospecting according to the guideline.

In the case of a long tunnel survey, there are cases where blasting is carried out in a mountain stream. Also, a boring depth of about 5 m to 20 m may be dug and sometimes blasted within the borehole.

Vibration caused by dynamite has the advantage that a large signal can be obtained. On the other hand, application procedures are necessary, there are places that cannot be used, and there are disadvantages such as noise affecting the environment.

If dynamite cannot be used due to various circumstances, vibrate by hammer hitting is used. Hammer hitting is possible anywhere, and there is little influence on the environment. However, there is a disadvantage that the obtained signal is small.

There is a stacking method as a method to compensate for the disadvantage that the vibration of hammer striking is small. This is a method of adding signals by hitting the same place several times (ground noise is averaged and decreases). There is also a way to drop heavy weight (63.5 kg) using a tripod.

The following figures show differences in receiving distance by vibration source.

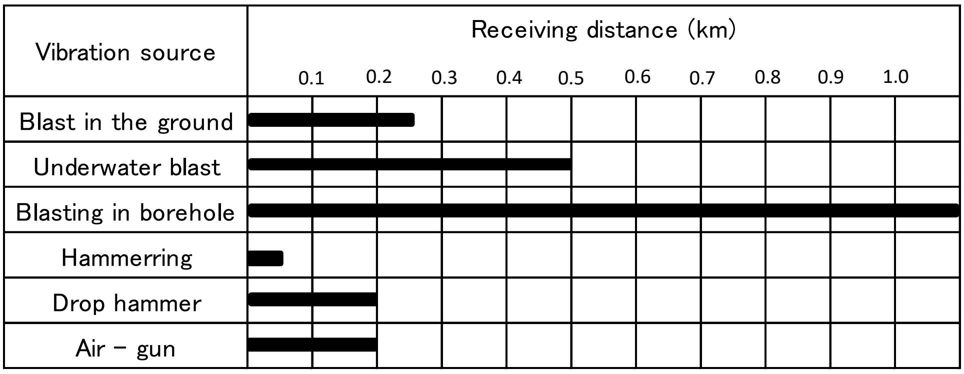


Figure E-1 Difference in Receiving Distance by Vibration Source

(Source: Outline on investigation plan of seismic refraction prospecting in civil engineering geological survey, 1979, Society of Exploration Geophysicists of Japan)

In Sri Lanka, the application procedure to utilize the explosives is complicated and strict. Therefore, hammers are generally utilized for the seismic refraction prospecting in Sri Lanka. The ease of the hammering method is an advantage. However, the exploration depth by the hammering is limited. Although geological condition in Sri Lanka is relatively good, it is difficult to obtain good results by hammering for more than 50m exploration depth. Thus, it is recommended to utilize explosives for tunnel survey.

3 Measuring Method

3.1 Measuring Instrument

Table E-1 shows the equipment use in the measurement.

Table E-1 Measuring Instrument

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

Measuring instrument-1

Measuring instrument-2

(Source: JICA project)

3.2 Measurement Work

Figure E-2 shows the procedure of seismic refraction prospecting

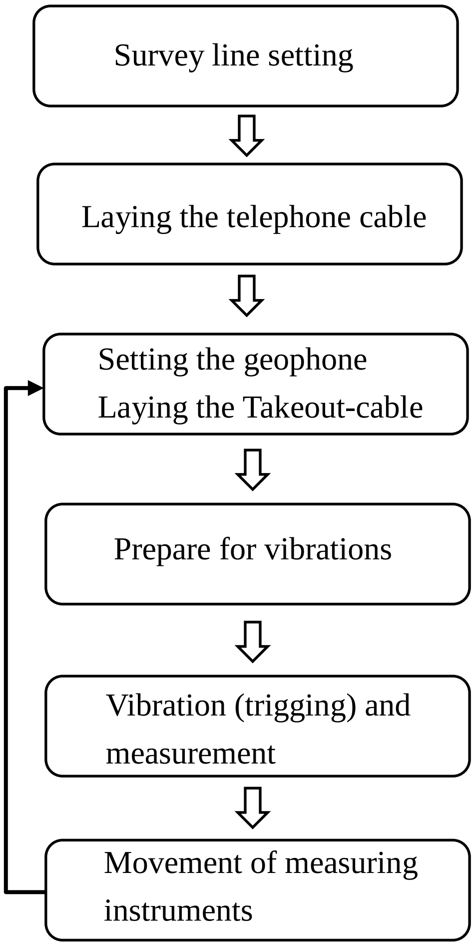


Figure E-2 Procedure of Seismic Refraction Prospecting

(Source: JICA project)

3.2.1 Setting of Survey Lines

Since the measurement length is expressed by the horizontal distance, it is necessary to measure the distance between the reception points by the horizontal distance.

Specifically, set a wooden pile that has a number (distance from the starting point) at all the receiving points.

For easier walking when measuring, it is recommended to cut vegetation with a width of about 1 m. The grass and tree branches should be cut down; but thick trees are not necessary.

Surveying

Wooden stake

3.2.2 Laying the Telephone Line

It is necessary when using explosives.

It is necessary to lay the telephone line on all the survey lines so that measurement headquarters and blasters can make telephone contact.

The telephone line also serves to send a shot mark from the blasting machine to the recorder.

3.2.3 Setting the Geophone (laying the cable)

Geophones (number of the geophones depends on the equipment model) will be installed from the starting point of the survey line with the interval of 5 m.

(1) Set the geophone closely beside the measurement point pile.

(2) Fix the geophone firmly so that it will not shake.

(3)For places where the geophone cannot be installed (roads, structures, rocks, etc.), place it in a possible location and report the distance and height moved to the measurement headquarters.

(4) Connect geophones to measuring instruments with a take-out cable.

(5) Pay attention when the survey line crosses the road.

1) Do not disturb the traffic of cars and people.

2) Take measures to prevent the cable from being cut by cars.

3) Set a lookout guard to prevent trouble.


Setting the Geophone


Connection of geophone and take-out cable

3.2.4 Preparation for Seismic Sources

Prepare seismic sources according to the seismic sources plan. If the blasting (triggering) point is different from the plan, contact the measurement headquarters.

(1) When Using Gunpowder

Confirm that the blasting point is safe for housing and roads.

Dig a hole of about 1 m in the ground with a steel bar, and load the dynamite (with an electric detonator).


Digging a blast hole


Load dynamite

(2) When not Using Gunpowder

Attach the triggering switch to the hammer.

When trigging, instead of hitting the ground directly, place an iron plate and hit the iron plate.

In the case of a drop hammer, let it fall from a position as high as possible.

Triggering

Drop hammer

3.2.5 Seismic Sources and Measurement

Preparation for measurement

(1) Set the measurement conditions.

(2) Check whether all geophones operate normally.

(3) Confirm that the shot mark from the hammer (or blast machine) operates.

The gain of the amplifier is set according to the level of the ground noise. Normally, in seismic refraction prospecting, the gain of amplification is made as large as possible to obtain good recording. Therefore, it is sensitive to noise, and it will adversely affect the measurement when someone moves near a geophone. People who have completed preparatory work should move away from the geophone as far as possible and keep quiet.
(Ground noise - Noise of the person walking near the geophones, The tree shaking by the wind, Traffic noise and others.)

When the ground noise is small, send a signal of ‘GO!’ to the person triggering.

When any record is not clear or questionable, the triggering (blasting) and recording shall be made again.

Recorder screen

Measurement

The following figures show as the exploration conception diagram.

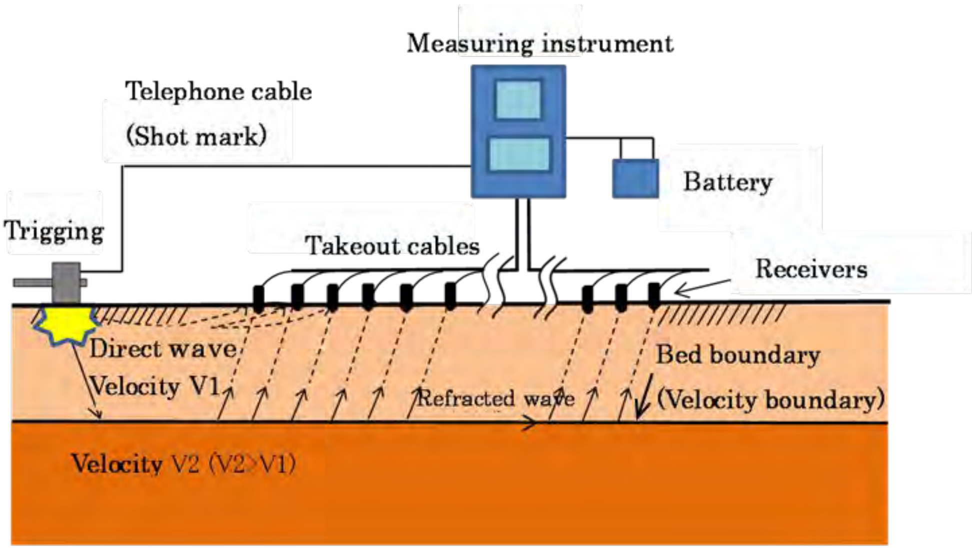


Figure E-3 Exploration Conception Diagram

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

3.2.6 Move to the Next Span

When the measurement with a span is finished, the operator instructs the seismograph installer and the vibrator to "Move". Then move on to the next span.

The geophone at the end in the direction of travel is kept installed, and in the next span, it is set as the first receiving point.

4 Analysis

Figure E-4 shows the analysis procedure.

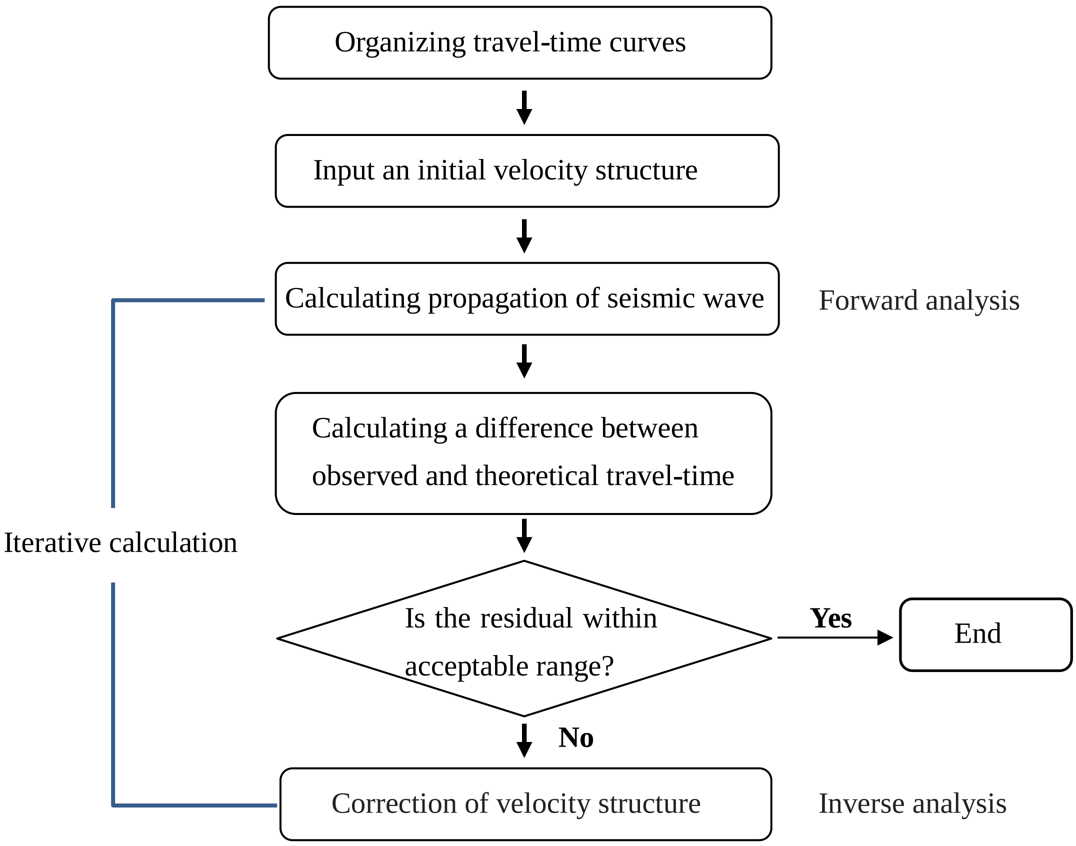


Figure E-4 Procedure of Analysis
(Source: JICA project)

In order to improve the analysis accuracy of seismic refraction prospecting, the following needs to be done.

(1) Obtain good measurement record.

(2) Make a vibration plan to obtain a travel-times curve propagating through the basement rock.

(3) Obtain accurate basement rock velocity.

4.1 Preparation of Travel-time Curve Diagram

Read the first movement of the measurement data and create a travel-time curve diagram.

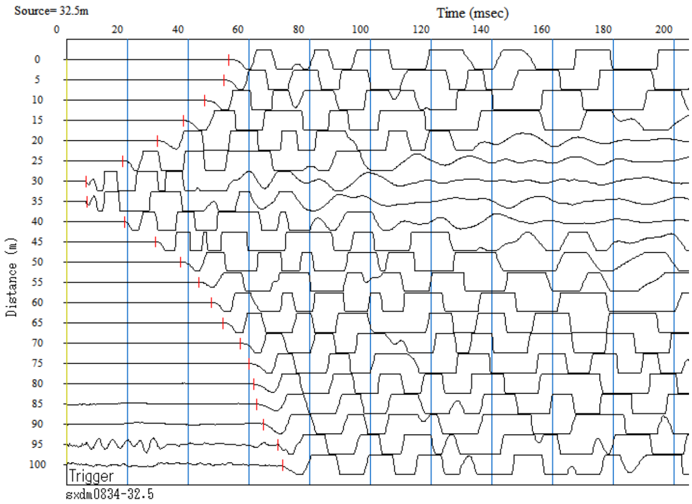


Figure E-5　Read the First Movement

(Source: JICA project)

In general, a reading error always occurs. However, the travel- time curve theoretically has to satisfy the following conditions.

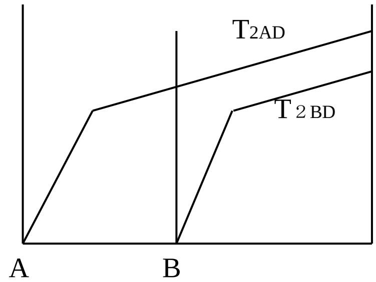
Modify the travel-time curve to satisfy the following conditions. If big correction is necessary, try to reread the measurement data again.

Analyze after finishing the correction of the traveling curve that satisfies the following conditions.

The travel-time graphs and the analysis result are shown in Figure E-7 and Figure E-8.

(1) Travel-time parallelism

Travel-time difference does not increase



(2) Reciprocal travel-time

Reciprocal travel-time should be same

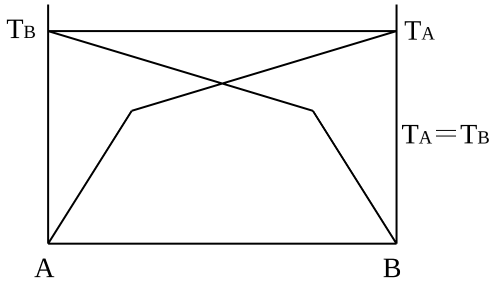


Figure E-6　Notes on the Traveling Time Curve

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

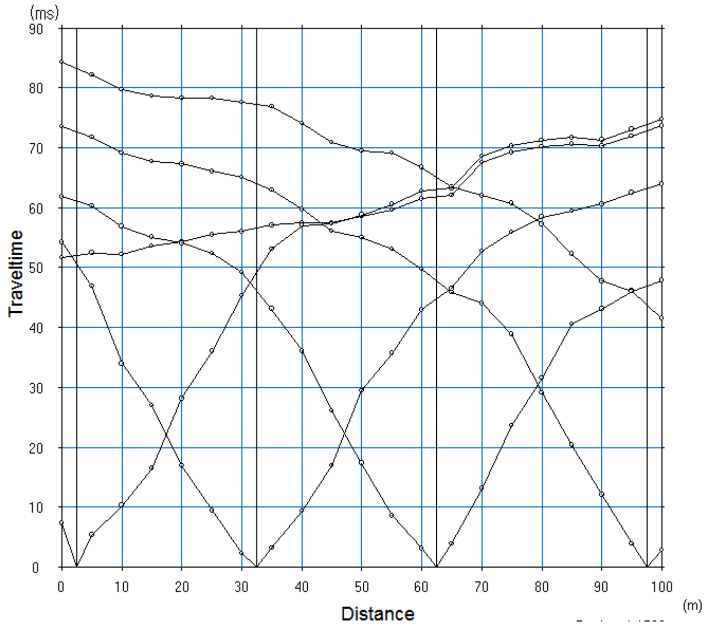


Figure E-7 Travel-time Curve Diagram

(Source: JICA project)

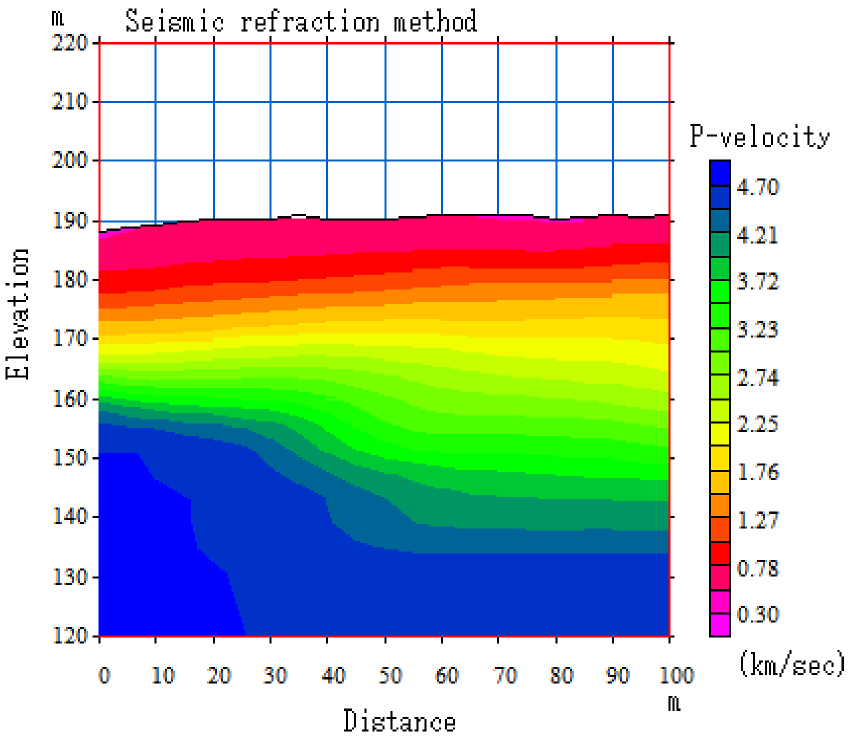


Figure E-8 Analysis Result (velocity cross section)

(Source: JICA project)

In Japan, Hagiwara's method has been used for analysis for a long time. (Figure E-9) Tomographic analysis (Figure E-10) is currently being conducted, but it may be used in conjunction with Hagiwara's method.

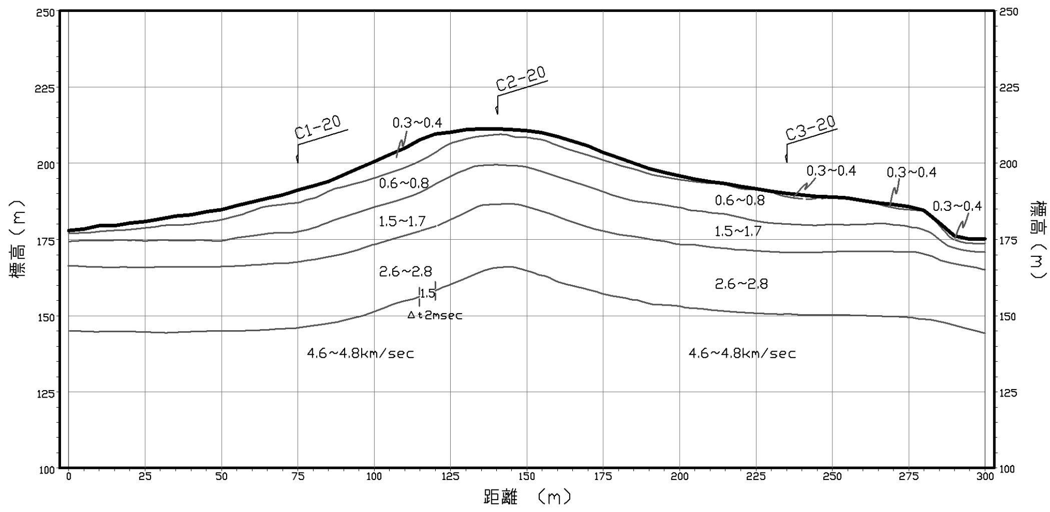


Figure E-9 Hagiwara's Method

(Source: JICA project)

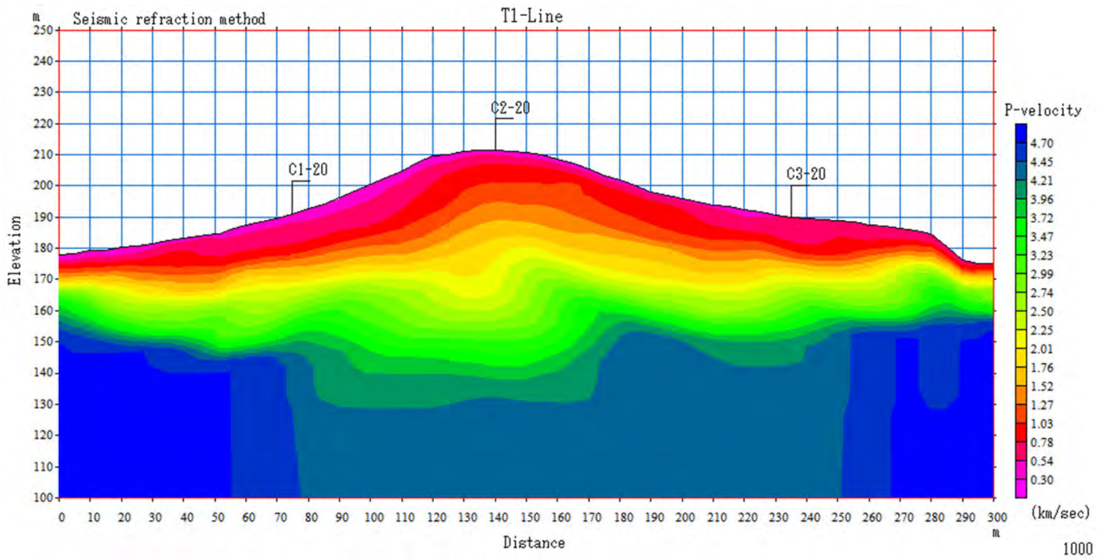


Figure E-10 Tomographic Analysis

(Source: JICA project)

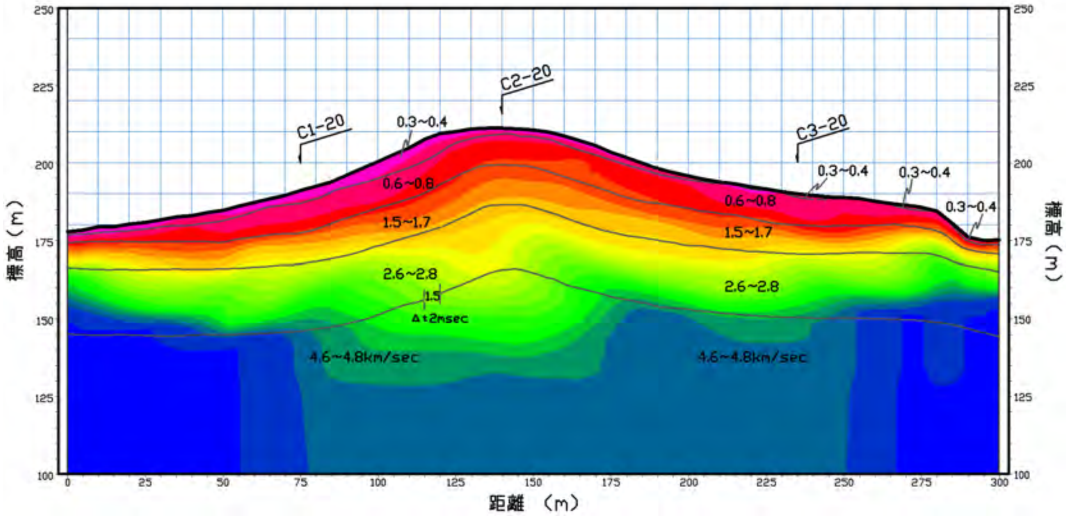


Figure E-11 Tomographic Analysis + Hagiwara's Method

(Source: JICA project)

4.2 Interpretation of Analysis Result

In the analysis of seismic refraction prospecting, there are cases where the same velocity value is obtained in different kinds of geology. In addition, even with the same rock type, the speed value may be greatly different depending on the weathering difference. From this, it is important to make judgments by carefully referring to geological exploration, drilling and other geological survey data.

Elastic wave velocity is used for the design and construction of civil engineering structures and civil engineering works. Three examples are shown below.

(1) Determination of Excavation Difficulty

The optimum excavation machine can be selected if it is possible to determine the difficulty of drilling in advance when constructing a construction plan in the cutting work of earth and rock.

(2) Determination of Slope of Cut Surface

Useful information for designing slope gradient can be obtained from the geology of the site where cutting is to be made and the elastic wave velocity,

(3) Use in Rock Mass Classification

In the geological survey of the tunnel, rock masses in the planned construction area are classified into several types.-(Rock mass classification)

Actual design and construction are carried out based on the results of this rock mass classification.

The elastic wave velocity is adopted as one criterion of rock mass classification criteria.

In order to utilize the elastic wave velocity, not only the speed value itself but also the geology and the rock condition must be grasped correctly.

Figure E-12 shows P wave velocities of strata and rocks, and Table E-2 shows the relationship between P waves and ground conditions.

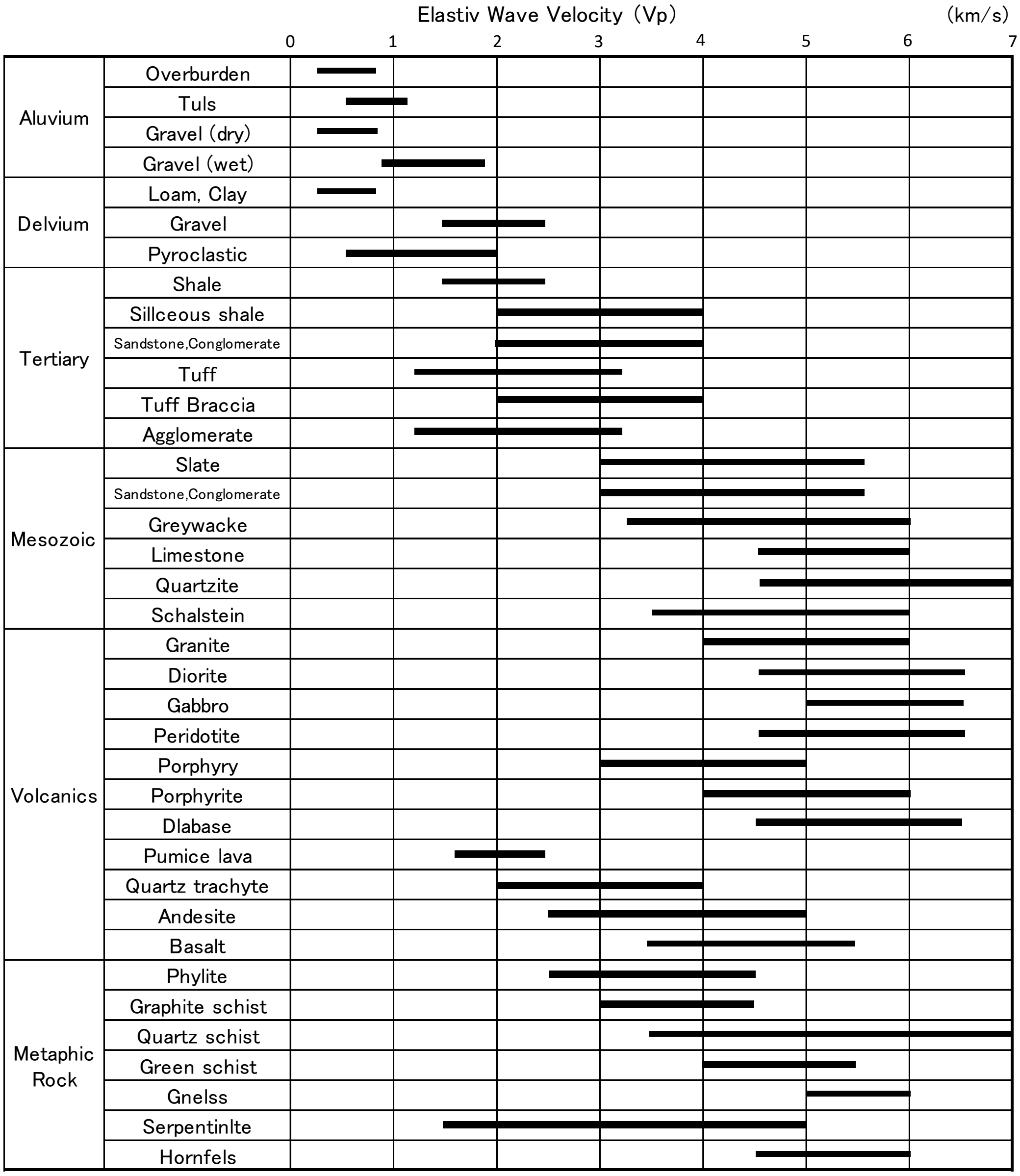


Figure E-12 Seismic Velocity of Rocks

(Source: Technical Standard for Road Tunnel (Structure), 1989, Nihon Geophysics exploration Co. Ltd)

Table E-2 Relation between the Ground and Seismic Velocity

Factor	P-Wave Velocity
	Small ——— Large
Soil property	Cohesive soil - Sandy soil - Gravel soil - Bedrock
Gradation	Small ——— Large
Porosity	Large ——— Small
Degree of saturation	Small ——— Large
Weathering · Alteration	Large ——— Small
Concretion grade	Loose ——— Solid
Geologic time	New ——— Old
Ground pressure	Small ——— Large

(Source: modified after Guidance on Application of Geophysical Exploration (Civil Engineering Field), 1989, Society of Exploration Geophysicists of Japan)

5 Reporting

The report of seismic refraction prospecting includes the following items.

1 Locations and arrangements of the prospecting traverse lines.

2 Work quantity and period.

3 List of equipment actually used, material consumed and personnel engaged.

4 Methodology adopted to interpretation.

5 Time-distance graphs.

6 Seismic wave velocity layer profiles.

7 Technical comments on field work, the interpretation, the geological condition and others.

F. Electrical Resistivity Survey

(Two Dimensional: 2-D)

1 Introduction

The purpose of electrical surveys is to determine the subsurface resistivity distribution by making measurements of the ground surface. From these measurements, the true resistivity of the subsurface can be estimated. Ground resistivity is related to various geological parameters such as the mineral and fluid content, porosity and degree of water saturation in the rock.

The resistivity method of quantitative interpretation, conventional sounding surveys originated in the 1920's, and for approximately 60 years, since then quantitative interpretation, conventional sounding surveys were normally used. A more accurate model of the subsurface is a two-dimensional (2-D) model where the resistivity changes in the vertical direction, as well as in the horizontal direction along the survey line. In this case, it is assumed that resistivity does not change in the direction that is perpendicular to the survey line.

Electrical resistivity surveys have been used for many decades in hydrogeological, mining and geotechnical investigations. More recently, it has been used for environmental surveys. Resistivity measurements are normally made by injecting current into the ground through two current electrodes (C1 and C2 in Figure 1), and measuring the resulting voltage difference at two potential electrodes (P1 and P2). An apparent resistivity (pa) value is calculated from the current (I) and voltage (V) values.

$$pa = k\, V / I$$

Pa: apparent resistivity, V: voltage, I: current, k : geometric factor

The geometric factor depends on the array of the four electrodes. Figure F-1 shows the common arrays used in resistivity surveys.

Resistivity meters normally give a resistance value, R = V/I, so in practice the apparent resistivity value is calculated by

$$pa = k\, R$$

The calculated resistivity value is not the true resistivity of the subsurface, but an "apparent" value which is the resistivity of a homogeneous ground which will give the same resistance value for the same electrode array. The relationship between the "apparent" resistivity and the "true" resistivity is a complex relationship. To determine the true subsurface resistivity, an inversion of the measured apparent resistivity values using a computer program must be carried out.

2 Array of Electrodes

The typical setup for a 2-D survey with a number of electrodes along a straight line attached to a multi core cable. Normally a constant spacing between adjacent electrodes is used. A multi core cable is attached to an electronic switching unit which is connected to a laptop computer. The sequence of measurements, the type of array to use and other survey parameters is normally entered into a text file which can be read by a computer program in a laptop computer.

One of the new developments in recent years is the use of 2-D electrical imaging/tomography surveys to indicate A 2-D section profile of moderately complex geology. Such surveys are usually carried out using a large number of electrodes, 25 or more, connected to a multi-core cable. A laptop microcomputer together with an electronic switching unit is used to automatically select the relevant four electrodes for each measurement.

The measurement of electric resistivity uses a pair of current electrodes and another pair of potential electrodes that are installed in the surface of the earth. Direct and indirect current are injected into the ground through current-electrode, and the resulting voltage difference at two potential electrons is measured. Depending on the purpose and efficiency of the investigation, several methods are generally applied. They are shown in Figure F-1.

The dipole-dipole method (4 pole method) and the pole-dipole method (3 pole method) can obtain a high resolution, however these methods require a large amount of current to obtain sufficient data. These methods are effective in the depth of investigation of less than 30 meters below the ground surface.

In order to secure the same investigation depth, the length of the array is increased 2 times in the pole-dipole and dipole-dipole methods, and 3 times in the Wenner array. Besides, the investigation range of both ends of the exploration line becomes significantly narrow in the pole-dipole, dipole-dipole, and Wenner arrays.

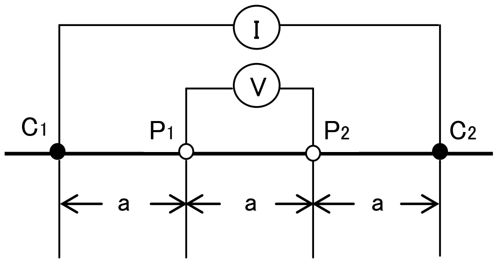
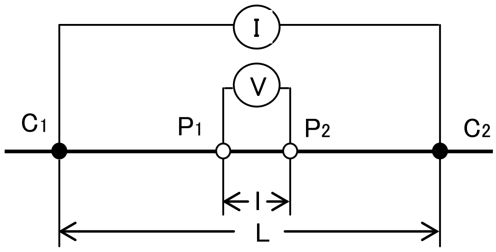
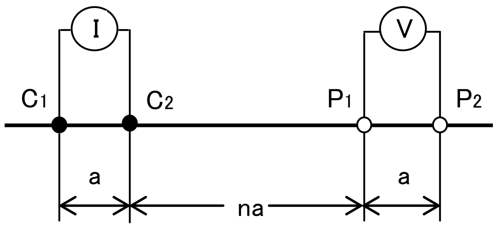
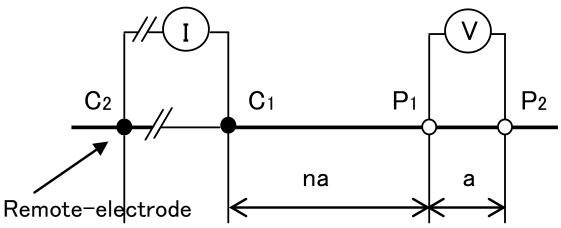
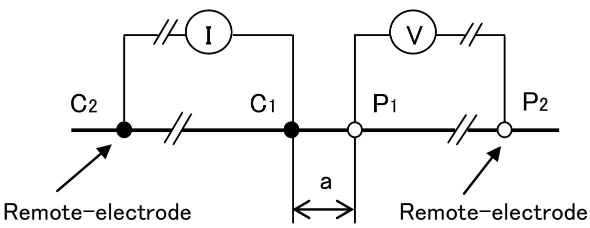
Name of arrangment	Arrangment diagram	Coefficient electrode array
Four-electrode array method (Wenner array)		$2\pi a$
Four-electrode array method (Schlumberger array)		$\frac{L^2-l^2}{4l}\pi$
Four-electrode array method (dipole-dipole array)		$n(n+1)(n+2)\pi a$
Three-electrode array method (pole-dipole array)		$2n(n+1)\pi a$
Two-electrode array method (pole-pole array)		$2\pi a$

Figure F-1 Common Arrays Used in Resistivity Surveys

(Source: JICA project)

As a result of discussion on arrays of the electrode, pole-pole array is judged to be the most effective in the civil engineering field even if the length of the array increases.

The advantage of the pole-pole array is described below.

1) A large resulting voltage can be obtained. (It is easy to obtain reasonable results.)
2) A shorter length of the array can be applied in comparison with other methods in order to secure a certain depth of investigation. A large investigation range is also secured when the length of the array is same.
3) Efficiency of the site survey can be secured when a remote-electrode is installed.
4) Since the response of the potential to underground structure is relatively simple, computerized analysis can be applied without difficulties.
5) The data of other methods can be applied in combination with data of the bipolar method. (Reverse is not possible.)

3 Plan of Survey

The survey line shall be planned to avoid power transmission lines, steel structures and others. in order not to cause abnormal data.

If lineaments or faults are expected due to topographical interpretation or geological reconnaissance, it is desirable to provide a survey line perpendicular to these. It is also important to establish a plurality of survey lines and check their direction.

3.1 Depth of Investigation and Length of Survey Line

Since the data density decreases at the bottom of the survey area, accuracy of the analysis decreases. Thus, it is necessary to proceed further down to the necessary depth. For example, if it is planned to drill a well up to 80 m in depth, the measurement shall be up to 120 m in depth. A depth of 1.5 times the required depth is recommended to be measured, and double the required depth is preferable.

Reliability of data and accuracy of analysis decrease when the investigation depth generally exceeds 300m, therefore in alternative investigation method is recommended when investigation depth exceeds 300m.

3.2 Spacing of Electrodes

Generally, when the investigation depth is less than 100m, the measurement spacing shall be 5m. Measurement spacing shall be 10m when the investigation depth is more than 100m.

When investigation depth is less than 50m, 2.5m of spacing can be applied.

4 Measuring Instrument

The main measurement apparatuses are shown in the following Table F-1.

Table F-1 Measuring Instrument

Name	Type	Specification	Quantity
Observation equipment	McOHM profiler 4 XP MODEL-2140D-XPE	Maximum voltage: 400V The maximum current: 1000mA (in case of the power booster using)	1 set
Electrode change machine	SCANNER64 MODEL-2141	The number of channels: 64	1 set
Booster	McOHM PROFILER4 POWER BOOSTER MODEL-2142	Maximum voltage: 200V Conduction current: 250,500,750,1000mA	1 set
Electrode rod	Stainless steel steel-manufacture stick	φ=12 mm･L=50cm	52 sets
Observation line	Exclusive cable	10m interval and 16 ingredient	3 sets
The electric wire	Single line	Electric strength: 600V	3000m
Transceiver			5 sets
Battery	vehicle battery	12V 74A	2 sets

Figure F-2 Measuring Instruments at Survey Site

(Source: JICA project)

5 Measuring Method

The procedure of measurement is shown below.

Setting of survey line

↓

Setup of base for measurement

↓

Remote Electrode
Install the electrode rod into the ground → Setup of an electric wire → Check of cable and ground resistivity

↓

Electrode rods are installed in the measurement points

↓

Setup of observation cable

↓

Check of cable and ground resistivity

↓ ← Adjustment of reducing ground resistivity

Measurement

Figure F-3 Work Flow of Measurement

(Source: JICA project)

5.1 Setup Base for Measurement

The base for measurement shall be set up to secure that the measurement apparatuses work properly for observation. Direct sunlight to the apparatuses shall cause troubles.

Figure F-4 Base for Measurement
(Source: JICA project)

5.2 Installation of Remote Electrodes

(1) The distance of the remote electrode is recommended to be set to a point that is more than 10 times the depth of investigation. Even if the distance of the remote electrode cannot be secured 10 times or more depending on circumstances, it shall be at least five times.

(2) The location of each remote electrode shall be secured at a sufficient distance from each other.

(3) Insulation of a remote electrode shall be in a place where ground resistance is as low as possible.

(4) In order to lower ground resistivity, it is recommended to connect two or more electrode rods in parallel.

Figure F-5 Installation of Remote Electrode

5.3 Installation of Electrodes

The measured potential of the ground is proportional to the strength of the current flowing in the ground. The intensity of the current is proportional to the applied voltage and inversely proportional to the resistivity between the current electrodes (the sum of the ground resistivity of both electrodes).

Since there is a limit to the voltage that can be applied, it is most important to reduce the ground resistivity of the electrode in order to obtain a signal of a larger potential difference.

Reasonable results can be obtained without difficulties when the surface layer shows a low resistivity due to existence of a large amount of clay or high moisture content layer. However, no reasonable results can be obtained when the surface layer shows high resistivity due to dry condition of the layer of gravel and exposed weathered rocks.

In the arid zone, it is effective to spray water and salt water around the electrode, and it is highly recommended to spray it in a considerably large amount in order to improve the effect.

In addition, it is effective to connect electrode rods in parallel and use them as one electrode rod. Attention shall be paid on installation of electrode rods to obtain reasonable results in the arid zone.

Figure F-6 Placement of Electrode Rods

(Source: JICA project)

Figure F-7 Close view of Electrode Switch and Rod

(Source: JICA project)

5.4 Measurement

The measurement procedure of the pole-pole array is explained in the following.

In the pole-pole array method, two current electrodes and two potential electrodes, in total four electrodes, are utilized. One current electrode and potential electrode in the four electrodes are used as fixed electrodes (remote electrodes). The remaining two poles are sequentially moved for measurement. In the actual work, electrodes are set at 32 to 48 points at the same time. The measurement work is carried out while automatically switching electrodes with measuring equipment.

The measurement is carried out that a current electrode and a potential electrode are moved one by one. The following figures show the measurement procedure.

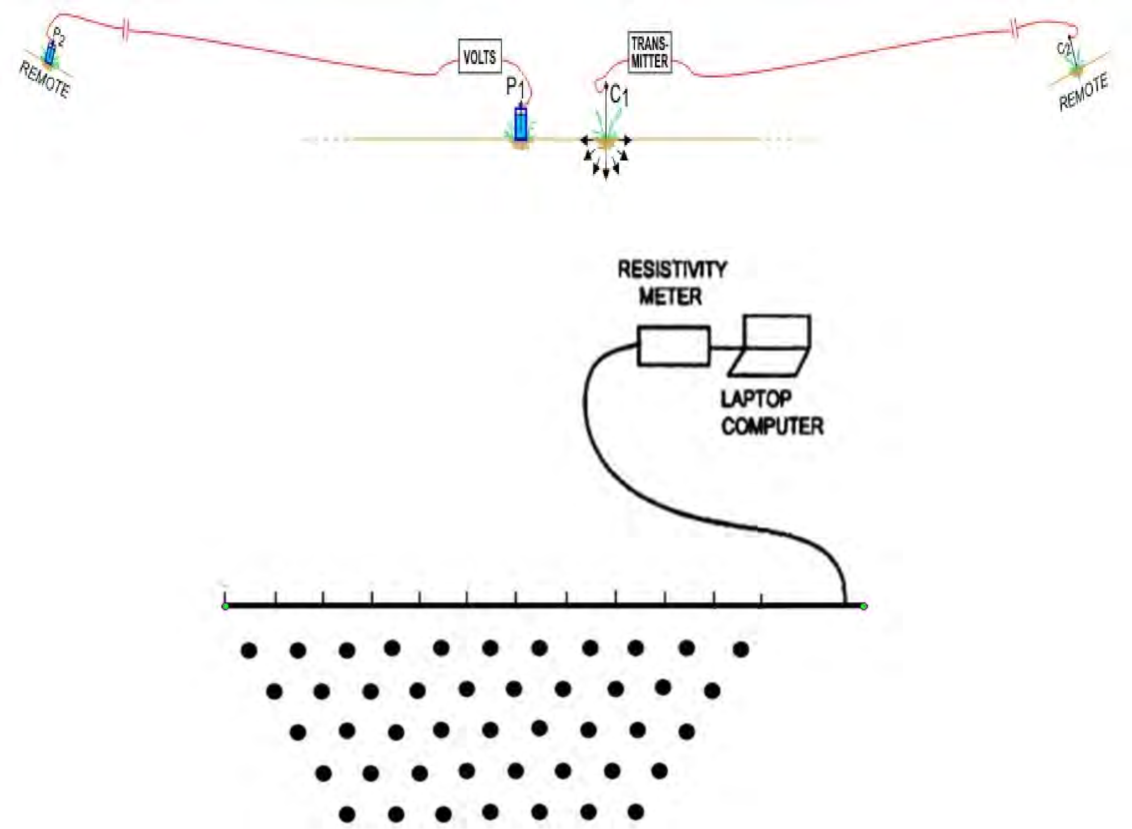


Figure F-8 Two (2) Dimensional Resistivity Profiling (pole-pole array)

(Source: JICA project)

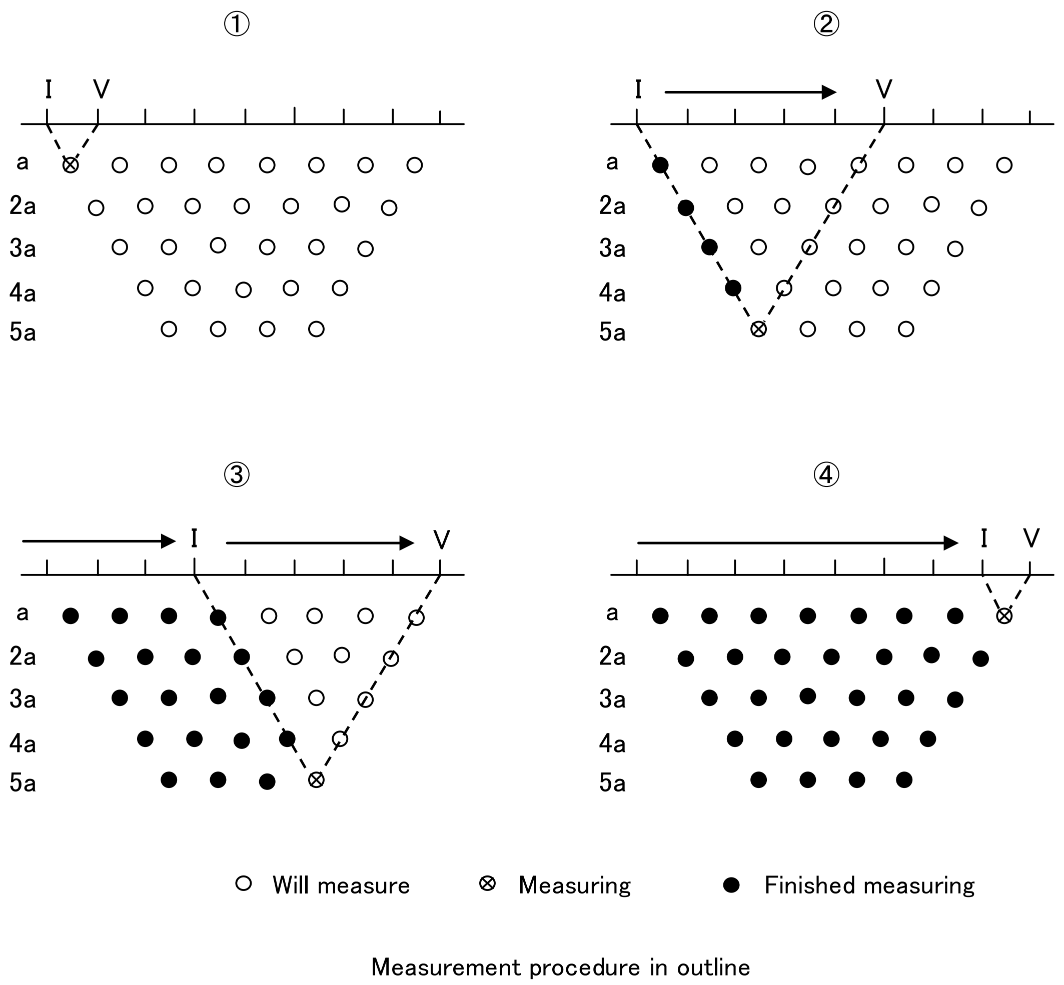


Figure F-9 Measurement Procedure

(Source: JICA project)

(1) The current electrode and the potential electrode are set at the first and second measurement points respectively, and measurement is implemented.

(2) Measurement is performed in the condition of fixing the current electrode and sequentially moving the potential electrode, then measurement is continued until the potential electrode reaches the last measurement point.

(3) Measurement is continued when the current electrode is moved to the next measurement point and the potential electrode placed at the next measurement point.

(4) The above measurement procedure is sequentially repeated until the current electrode moves to the point prior to the last measurement points and the potential electrode reaches the last measurement point, then the measurement is completed.

Attention and experience are required to obtain useful data that can be used for analysis and reflects the actual condition of the underground.

Measurement shall be repeated when a high degree of variation is confirmed in the values (standard deviation is high) obtained as a result of stacking. Measurement shall also be repeated in case that no decay of the potential is confirmed even if the distance increases.

Re-measurement is required after eliminating causes of these occurrences since the problem seems to exist in the measurement system.

Cut of the multi core cable connecting the remote electrode, or disengaging of the electrode frequently occurs.

Cases of troubles at the work site and the concept of the decay curve are described below..

1) The measured value is far from the value expected as a whole..
   - The electric wire of the remote electrode on the electric potential side has been disconnected or about to disconnect.
   - The remote electrodes are too close to each other.

2) Even if the distance increases, the potential does not decay.
   - Current leakage from the measuring cable occurs.
   - The resistivity of ground around the electrode is high.
   - The electrode and the measuring cable are not connected.

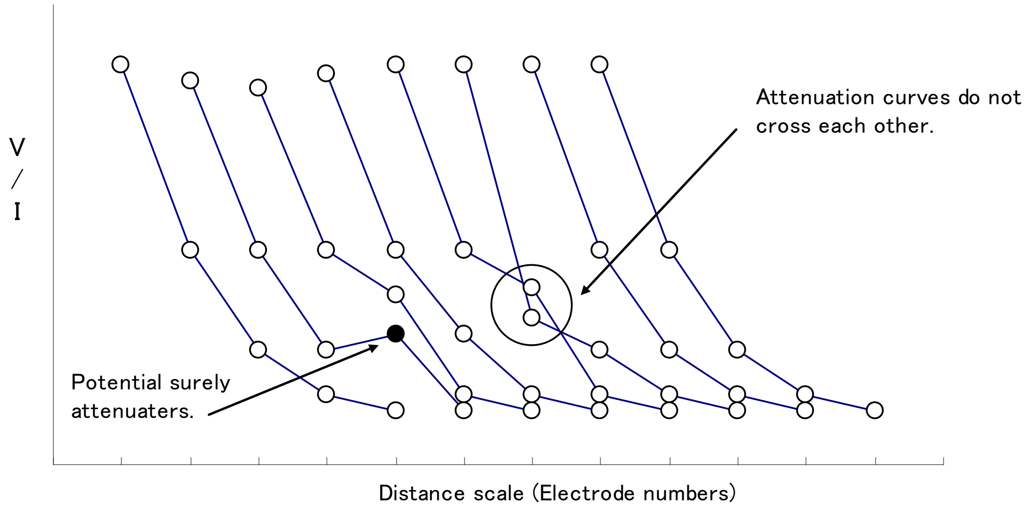


Figure F-10 Concept of the Attenuation Curves

(Source: New electric exploration method for construction, disaster prevention and environment - resistivity imaging method, Kokon Shoin (in Japanese))

1) Potential is decreased along with the increase in distance.
2) Attenuation curves do not cross each other.
3) Potential changes smoothly.

6 Analysis

A laptop microcomputer together with an electronic switching unit is used to automatically select the relevant four electrodes for each measurement. The type of array and other survey parameters is normally entered into a text file which can be read by a computer program in a laptop computer. Procedure of analysis is shown in Figure F-11.

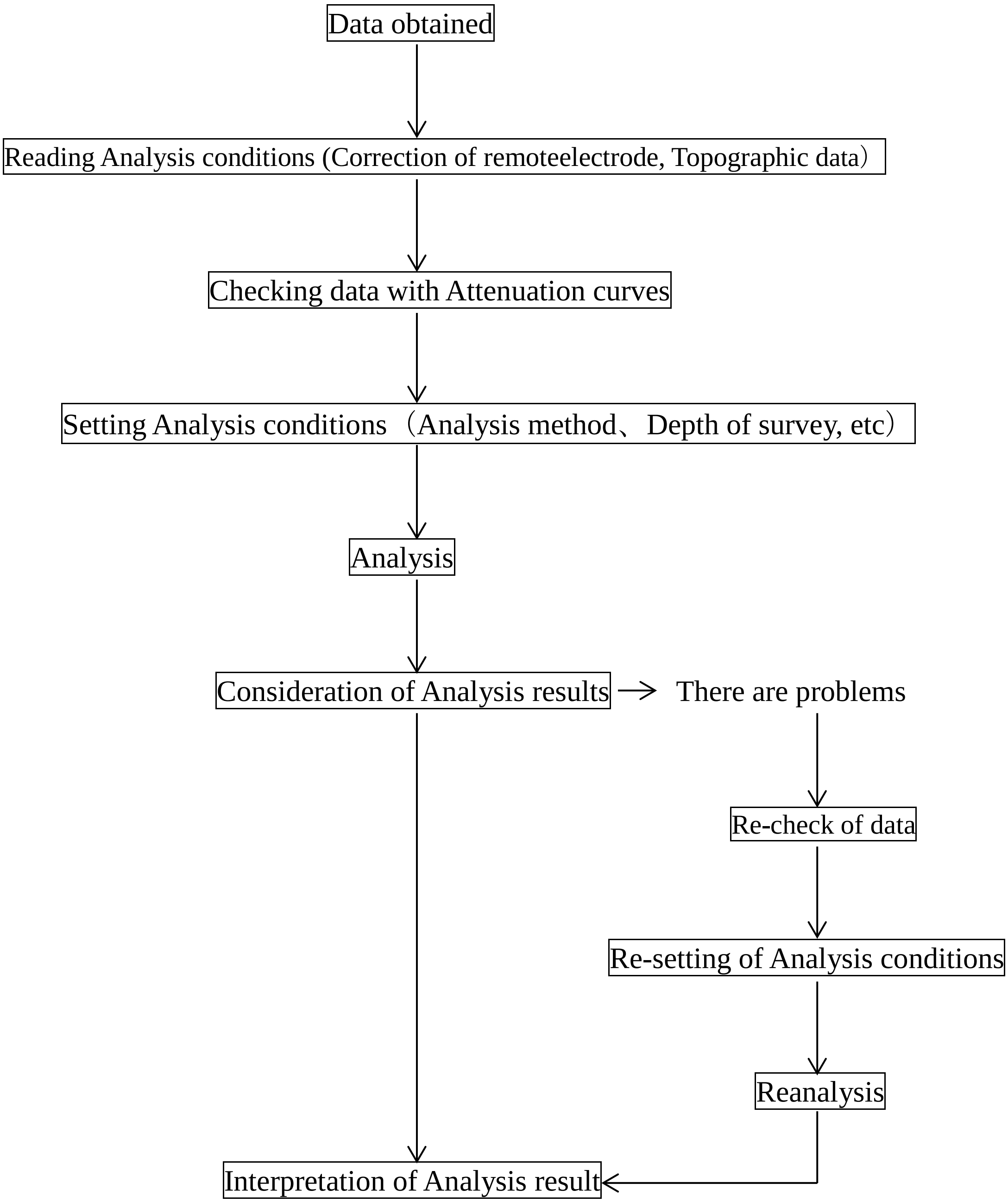


Figure F-11 Flow of Data Processing

(Source: JICA project)

After reading the control file, the computer program then automatically selects the appropriate electrodes for each measurement. In a typical survey, most of the fieldwork is in laying out the cable and electrodes. After that, the measurements are taken automatically and stored in the computer.

The 2-D forward modeling program calculates the apparent resistivity pseudosection through the defined 2-D subsurface model. With this program, the finite-difference or finite-element method can be selected to calculate the apparent resistivity values. In the program, the subsurface is divided into a large number of small rectangular cells. The program might also assist the user in choosing the appropriate array for different geological situations or surveys.

The program supports almost all arrays such as the Wenner, Wenner-Schlumberger, pole-pole, pole-dipole and dipole-dipole in general. Each type of array has its advantages and disadvantages.
An inversion model where the arrangement of the model blocks directly follows the arrangement of the pseudosection plotting points. This approach gives satisfactory results for several arrays where the pseudosection point falls in an area with high sensitivity values.

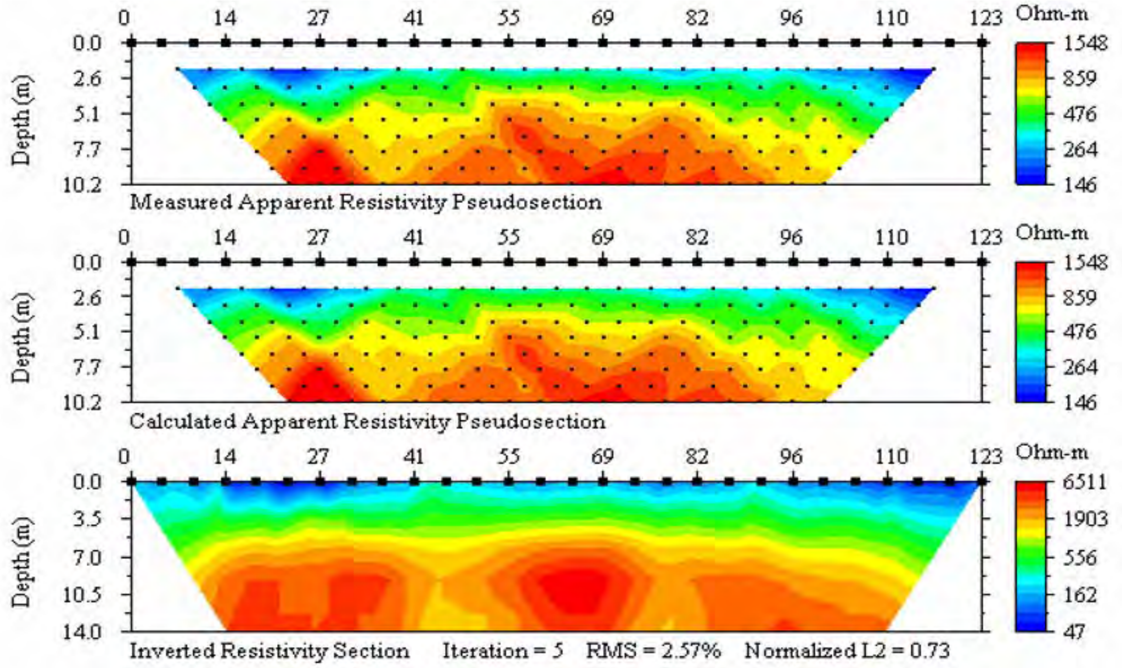


Figure F-12 Apparent Resistivity Pseudosection and Inverted Resistivity Section

(Source: JICA project)

7 Interpretation of an Analysis Result

As shown in the Figure F-13 in the following section, the resistivity of the ground takes various values depending on the substances and conditions constituting the ground. Judgments of the state of rock mass and evaluation of the rock mass of different location based only on the resistivity value is not recommendable due to large variation of the value of each rock.

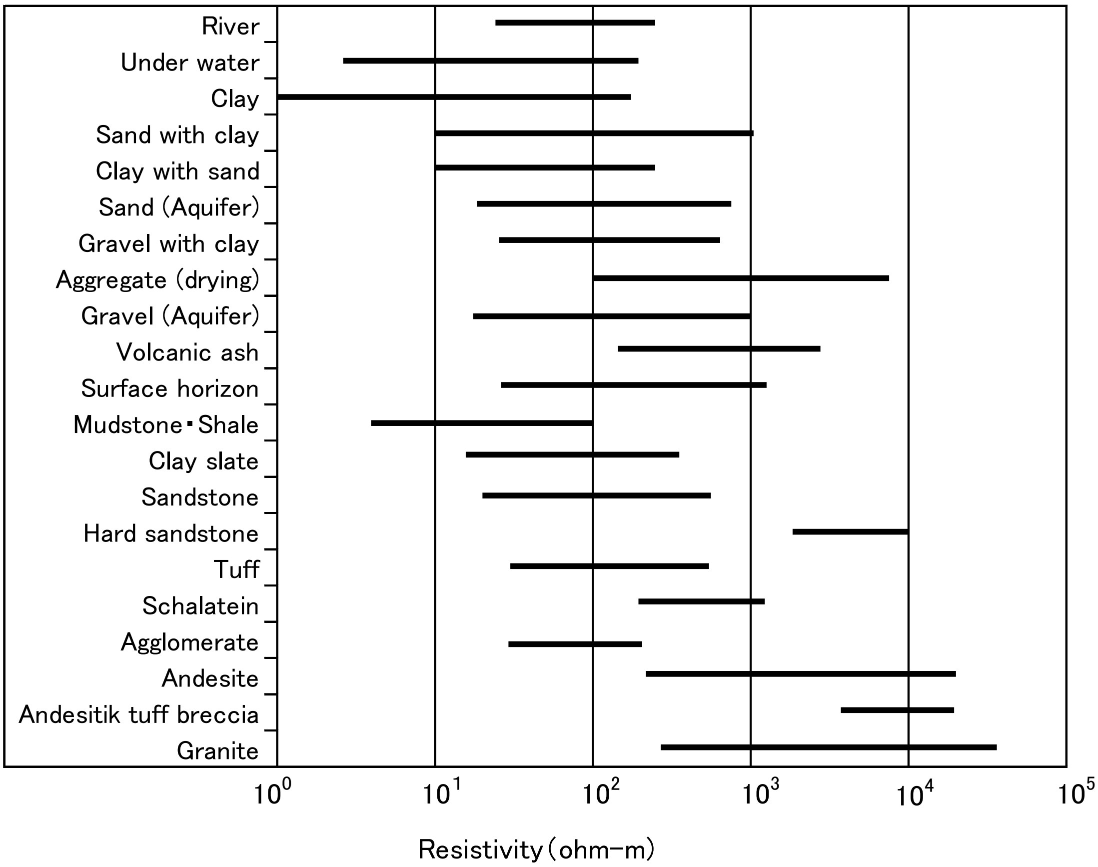


Figure F-13 Relationship between Resistivity and Geology

(Source: modified after New electric exploration method for construction, disaster prevention and environment - resistivity imaging method, Kokon Shoin (in Japanese))

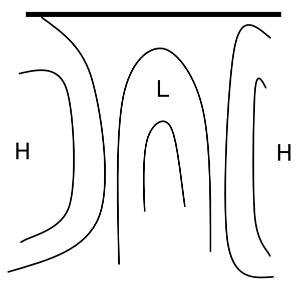
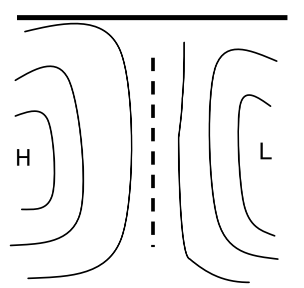
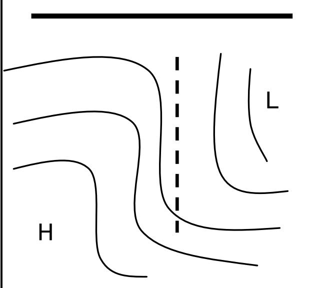
However, the resistivity distribution indicates the geological and geotechnical conditions such as sedimentary rocks, igneous rocks, faults / fracture zones, weathered rocks and others. The geological structure can be assumed on the basis of the resistivity distribution.

Factors affecting the resistivity of the ground and typical resistivity distribution pattern are shown in Table 7.1 and Figure 7.2.

Table F-2 Relationship between the Ground and Resistivity

Factor		Resistivity of ground Low ⟷ High	Associated phenomenon with ground
Degree of prosity	Saturation	Large ⟷ Small	Weathering , Fault
	Dring	Small ⟷ Large	
Dgree of saturetion		Large ⟷ Small	Groundwater level
Moisture content of volume		Large ⟷ Small	Weathering , Crush zone
Content of clay mineral		Many ⟷ Few	Weathering , Alteration
Resistivity of groundwater		Low ⟷ High	Concentration of salt
Ground temperature		High ⟷ Low	Geothermy、Warm Water

(Source: Factors affecting the resistivity of rocks, 2001 (in Japanese))

Pattern			
Characteristic	Belt-Like low resistivity part with verticalness	The boundary between a high resistivity part and a low resistivity part are straight line-like.	Resistivity distribution has a level difference like stairs.
Geological structure	Fault(Crush zone)	Fault	Fault
Besides		There is correspondence with springwater.	The inconsistency of a stratum is shown.

H Hight Resistivity
L Low Resistivity

Figure F-14 Pattern of Resistivity Distribution

(Source: JICA project)

G. Permeability Test

1 Introduction

1.1 General

Permeability, the capacity of a geological material to transmit water, is generally quantified by the material characteristics termed the coefficient of permeability, k (or coefficient of hydraulic conductivity), and expressed in terms of Darcy's law which is valid for laminar flow in a saturated and homogeneous material, as

$k = q/iA$ (cm/sec)

where q is the quantity of flow per unit of time (cm^3/sec), i is the hydraulic gradient, i.e., the head loss per length of flow h/L (a dimensional number) and, A is the area (cm^2).

For rock tunnels, k values of a rock mass (or in-situ rock) relate mainly to fracture characteristics (concentration, opening width, nature of filling) and degree of saturation, as well as level and nature of imposed stress form (compressive or tensile). k values are commonly estimated through the Lugeon test or packer test.

The Lugeon test, also called the packer test or water pressure test, is an in-situ constant head permeability testing method commonly carried out in a portion of a borehole isolated by inflated packers. The test is named after Maurice Lugeon (1933), a Swiss geologist who first introduced the test. The test, originally introduced and suggested as a test for assessing the need for foundation grouting at dam sites, presently has been widely used to estimate the average hydraulic conductivity (or permeability) of rock mass. The test is a routine and common in-situ method of measuring hydraulic conductivity of rock mass for a tunnel project.

This chapter briefly presents the Lugeon test procedure and test result interpretation method, which is based on international standards, widely accepted methods and engineering experiences.

1.2 Purpose of Lugeon Test

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

1) To provide in-situ permeability of the different rock formations in the region of the tunnel alignments and portal areas

2) To aid design of rock excavation, tunnel support and lining, as well as dewatering systems for the proposed tunnel
3) To check the effectiveness of grouting through measuring of the impermeability of grouted rocks around the tunnel opening

1.3 Definition

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

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

1.4 Relevant Standards and References

The following documents have been consulted and referred in preparation of this chapter:

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

2 Test Equipment and Procedure

2.1 Test Equipment and Material

The Lugeon test is conducted in a portion of a borehole isolated by pneumatic packers as the borehole progresses. The water is injected into the isolated portion of the borehole using a slotted pipe which itself is bounded by the inflated packers. The packers can be inflated using a gas compressor on the ground surface, and so they can isolate and seal that portion of the borehole. A pressure transducer is also located in that portion to measure the pressure with a help of a reading station on the surface. Figure G-1 illustrates the general arrangement of a Lugeon test and test equipment required.

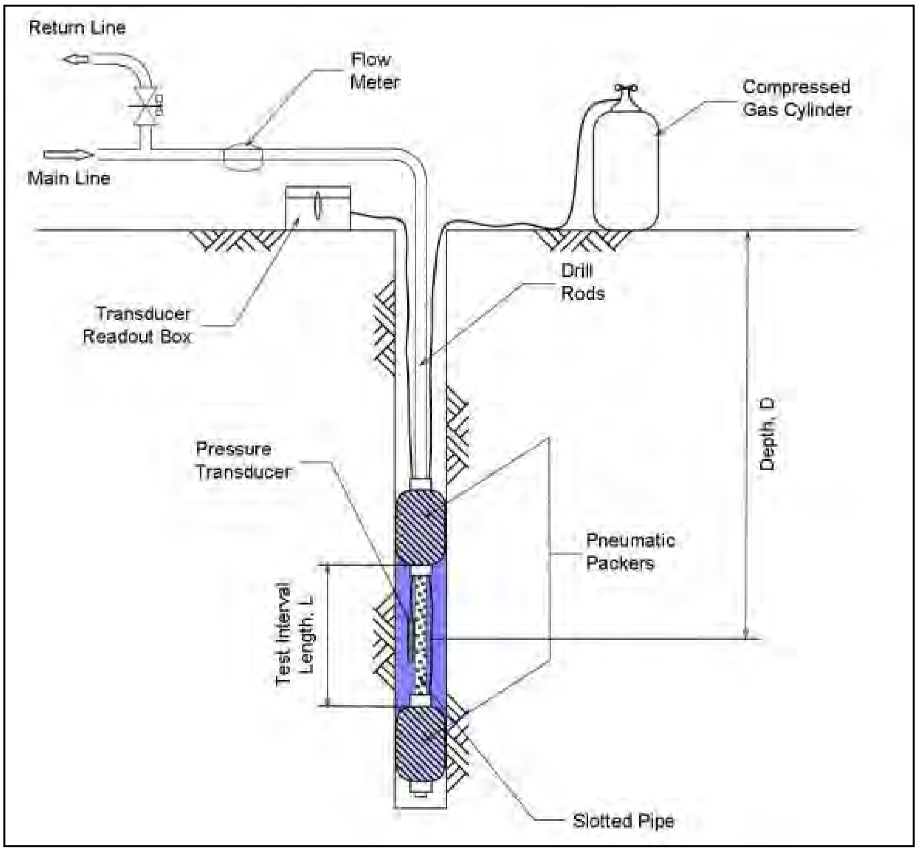


Figure G-1 Lugeon Test Schematic

(Source: Quiñones-Rozo, 2010)

(1) Packer System

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

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

(2) Pump

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

(3) Pressure Gauge and Flow Meter

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

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

(4) Others

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

2.2 General Requirements

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

(1) Maximum Test Pressure

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

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

$$P_{\max} = D \times \frac{1psi}{1ft} \approx 20 \times D(m) \text{ in } kPa$$

$$\approx 0.02 \times D(m) \text{ in } kg/cm^2$$

In addition, according to ASTM D 4630-96, the test pressure is typically determined between 300 and 600 kPa (50 to 100 psi).

(2) Test Stage

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

Table G-1 Pressure Magnitudes Typically Used for Each Test Stage

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

(Source: Modified after Quiñones-Rozo, 2010)

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

(3) Test Section Length

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

2.3 Test Procedure

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

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

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

(1) Wash the borehole with clean water

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

(2) Expand the packer with air pressure.

A packer is installed at the top of the test section and then inflated to a pressure sufficient to ensure a water tight seal against the borehole wall, without damaging the rock formation. After inflation of the packer the water level in the borehole above the packer is monitored with an electric dip meter when the first water pressure is applied and then at 5 minute intervals. If the water level rises during the initial stages of the test the packer should be re-seated and the test repeated.

(3) Introduce water under a given pressure into the borehole

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

(4) Record elapsed time and volume of water pumped

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

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

(5) Test at five pressures

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

3 Analysis and Interpretation

3.1 Calculation

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

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

where,

Q = Flow rate [lit/min]

L = Length of the test section [m]

P0 = Reference pressure of 1 MPa [MPa] (equivalent to 10 bar or 145 psi)

P = Test pressure [MPa] (at the specific stage)

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

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

$$P = H \times pg$$

where,

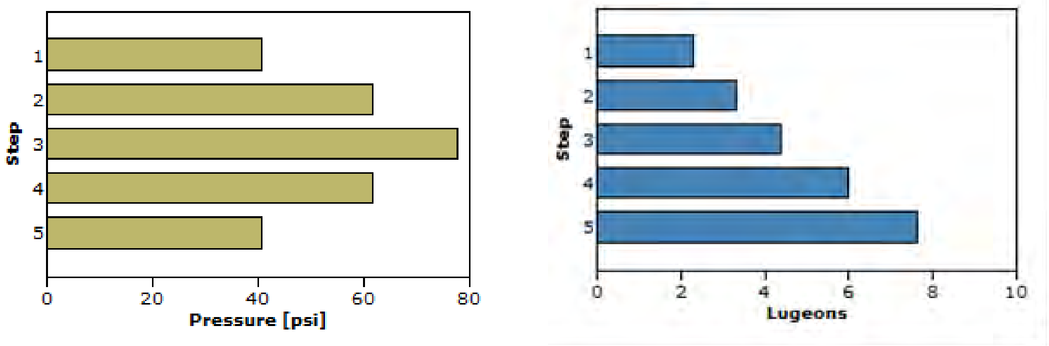
g = acceleration due to gravity (g = 9.81 m/s²)

p = density of water (p = 999.7 kg/m³)

3.2 Data Interpretation

(1) Pressure and Lugeon Diagrams

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



(a) Pressure Diagram (b) Lugeon Diagram

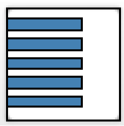
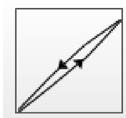
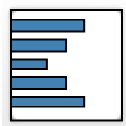
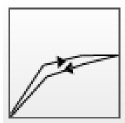
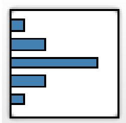
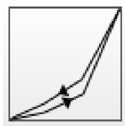
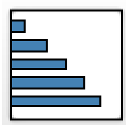
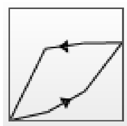
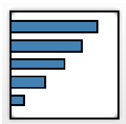
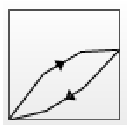
Figure G-2 An Example of Pressure and Lugeon Diagrams

(Source: Modified after Quiñones-Rozo, 2010)

(2) Typical Lugeon Behaviors

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

Table G-2 Lugeon Interpretation Pressure Following Water Loss vs Pressure Pattern

Behavior	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)

These typical Lugeon behaviors are summarized:

1) Laminar Flow: The hydraulic conductivity of the rock mass is independent of the water pressure employed. This behavior is characteristic of rock masses with low hydraulic conductivities and relatively small seepage velocities.
2) Turbulent Flow: The hydraulic conductivity of the rock mass decreases as the water pressure increases. This behavior is characteristic of rock masses exhibiting partly open to moderately wide cracks.
3) Dilation: Similar hydraulic conductivities are observed at low and medium pressures; however, a much greater value is recorded at the maximum pressure. This behavior – which is sometimes also observed at medium pressures – occurs when the water pressure applied is greater than the minimum principal stress of the rock mass, thus causing a temporary dilatancy (hydro-jacking) of the fissures within the rock mass.
4) Wash-Out: Hydraulic conductivities increase as the test proceeds, regardless of the changes observed in water pressure. This behavior indicates that seepage induces permanent and irrecoverable damage on the rock mass, usually due to infillings wash out and/or permanent rock movements.
5) Void Filling: Hydraulic conductivities decrease as the test proceeds, regardless of the changes observed in water pressure. This behavior indicates that either: (a) water progressively fills isolated/non-persistent discontinuities, (b) swelling occurs in the discontinuities, or (c) fines flow slowly into the discontinuities building up a cake layer that clogs them.

4 Application and Correlation of Test Results

4.1 Estimation of Water Inflow and Pressure

Water inflow inside a tunnel is one of the principal problems and also uncertainties in tunnels that cross either highly fractured or karstified rock masses. In most cases, the most appropriate method for estimating discharge into a tunnel is to use mathematic flow models, but sufficient data are not always available for their application. Alternatively, approximate estimation, such the Goodman method (Goodman et al., 1965), can be used to calculate the tunnel inflow, as follows:

$$Q = 2\pi \cdot K \frac{\Delta h}{ln \frac{2 \cdot \Delta}{r}}$$

where,

Q = Tunnel inflow (m^3/s).

K = Hydraulic conductivity (m/s) in the formation.

Δh = distance between the center of the tunnel and the groundwater level (m).

r = tunnel radius (m).

Goodman's formula is the most commonly used approximation for calculating early tunnel inflow rates. The Lugeon value of the rock mass relating to the subject tunnel can be used to obtain its hydraulic conductivity or permeability coefficient: therefore, the Lugeon value is one of the main inputs for estimating tunnel inflow.

In addition, the hydraulic load on a tunnel caused by water pressures is evaluated according to the water pressure inside the rock mass.

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

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

Table G-3 Effect of Discontinuity Characteristics on Rock Mass Permeability

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

(Source: Bell, 1992)

Table G-4 Estimation of Rock Mass Permeability from Discontinuity Frequency

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

(Source: Bell, 1992)

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

Table G-5 Discontinuity Spacing

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

(Source: Modified from ISRM, 1981)

Table G-6 Discontinuity Aperture (Opening)

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

(Source: Modified from ISRM, 1981)

4.3 Lugeon Test in Rock

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

Table G-7 Indicative Rock Mass Permeability from Lugeon Value

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

(Source: Modified after Quiñones-Rozo, 2010)

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

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

5 Recommendation

The boreholes, depth, section length and rock types to be tested should be determined after a complete review of the project requirements and features as well as the subsurface condition of the project site.

The water pressure at the depth of the tunnel is generally required to measure prior to the permeability test when high water pressure is applied.

6 Limitations

The Lugeon test affects a limited volume of rock around the borehole to be tested. It has been reported that the effect of the Lugeon tests is generally restricted to an approximate radius of

30 feet around the borehole with a test section length of 10 feet (Bliss and Rushton, 1984). This indicates that the hydraulic conductivity obtained by the test method is only representative for a cylinder of rock delimited by the length of the test section and the radius as mentioned above.

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

H. Laboratory Test

1 Introduction

Phenomena observed in the rock mass which have effect on the works are usually caused by the load exceeding the shearing resistance and deformations. Therefore, mechanical properties of the rock mass have been studied to identify important parameters of shear strength, angle of internal friction, deformation coefficient and others. The parameters of angle of internal friction and cohesion are elements of internal resistance of solid, plastic and loose rocks.

Properties of the larger-scale rock mass can be evaluated on the basis of the properties of intact specimens of tested in the laboratory. The intact specimen can also be considered as a part of the rock mass of little joints, fissures, and discontinuity features, and without water pressures and in-situ stress. (Refer to Figure H-1)

Basic methods of determination of parameters of shear strength within solid rock masses depend on the results of rock testing performed in laboratories or "in -situ". In addition to this testing. Some simple index tests are mentioned in this guideline.

The laboratory tests introduced in this guideline are mainly composed of relatively simple methods which can be carried out at sites or outside of the laboratory except for some tests.

Simple index tests are highly recommended to be performed using the same specimen of laboratory test and in-situ rock testing in order to obtain a correlation among the test results. The simple index test can be carried out in a wide area at low cost, and the test result is highly useful in some cases.

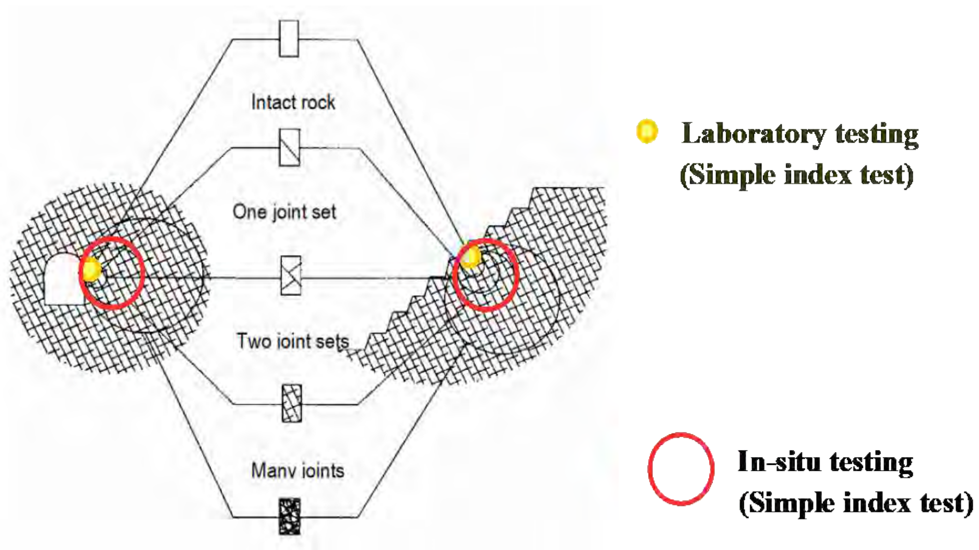


Figure H-1 Specimen for Laboratory Testing and Rock Mass for In-situ Testing

(Source: JICA project, Hoek and Brown, 1997 modified)

2 Laboratory Testing

Laboratory rock testing is performed to determine the strength and elastic properties of intact specimens and the potential for degradation and disintegration of the rock material. The derived parameters are used in part for the design of tunnels, and the assessment of cut slopes, shallow and deep foundations.

Properties including strength and deformation of the larger-scale rock mass can be evaluated on the basis of the properties of intact specimens tested in the laboratory. The intact specimen can also be considered as a part of the rock mass of little joints, fissures, and discontinuity features (spacing, roughness, orientation, infilling), and without water pressures and in-situ stress.

Standards and Procedures of frequently utilized tests for laboratory rock testing are described below. The main tests are summarized in Table H-1.

Table H-1 Standards and Procedures for Laboratory Rock Testing

Test Category	Name of Test		Test Designation	
			AASHTO	ASTM
Compressive Strength	Compressive strength (qu = Fu) of core in unconfined compression (uniaxial compression test)	*		D 2938*
	Triaxial compressive strength without pore pressure		T 226	D 2664
Tensile Strength	Direct tensile strength of intact rock core specimens	*		D 3936
	Splitting tensile strength of intact core (Brazilian test)			D 3967
Direct Shear	Laboratory direct shear strength tests - rock specimens, under constant normal stress	*		D 5607*
Deformation and Stiffness	Elastic moduli of intact rock core in uniaxial compression	*		D 3148*
	Elastic moduli of intact rock core in triaxial compression -			D 5407
	Pulse velocities and ultrasonic elastic constants in rock	*		D 2845*
Point Load Strength	Method for determining point load index (Is)	*		D 5731*

Note; ＊Standards and Procedures are described in this manual.

(Source: Swts and Zeitlinger Lisse, 2001)

2.1 Uniaxial Compression Test (ASTM D 2938)

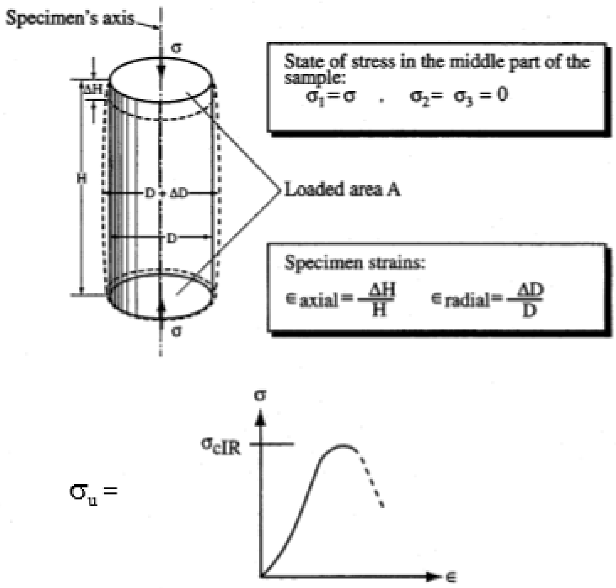
Purpose

To determine the uniaxial compressive strength of rock (qu = Fu = FC).

Procedure

In this test, cylindrical rock specimens are tested in compression without lateral confinement. The test procedure is similar to the unconfined compression test for soils and concrete. The test specimen should be a rock cylinder of length-to-width ratio (H/D) in the range of 2 to 2.5 with flat, smooth, and parallel ends cut perpendicular to the cylinder axis. Originally, specimen diameters of NX size were used (D = 2c in. = 44 mm), yet now the standard size is NQ core (D = 1f in. = 47.6 mm).

(a)



(b)

Figure H-2 Uniaxial Compression Test on Rock with

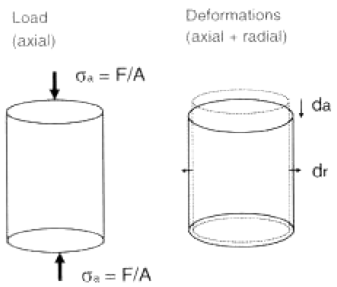
(a) Definitions of stress conditions and strains,

(b) Derived stress-strain curve with peak stress corresponding to the uniaxial compressive strength (qu = Fu)

(Source: AASHTO, ASTM (FHWA NHI Subsurface Investigations))

Commentary

The uniaxial compression test is the most direct means of determining rock strength. The results are influenced by the moisture content of the specimens, and thus should be noted. The rate of loading and the condition of the two ends of the rock will also affect the final results. Ends should be planar and parallel per ASTM D 4543. The rate of loading should be constant as per the ASTM test procedure. Inclined fissures, intrusions, and other anomalies will often cause premature failures on those planes. These should be noted so that, where appropriate, other tests such as triaxial or direct shear tests can be required



$$\sigma_a = \frac{F_a}{A}$$

(Failure load)

(Specimen cross sectional area)

Figure H-3 Uniaxial Compression Strength

(Source: JICA project, Hoek and Brown, 1997 modified)

Determination of the Uniaxial compressive strength of cylindrical intact rock specimens (load up 2000kN). The load rate is kept constant using a servo-hydraulic control unit.

Before failure

After failure

Figure H-4 Uniaxial Compression Test

(Source: JICA project, Hoek and Brown, 1997 modified)

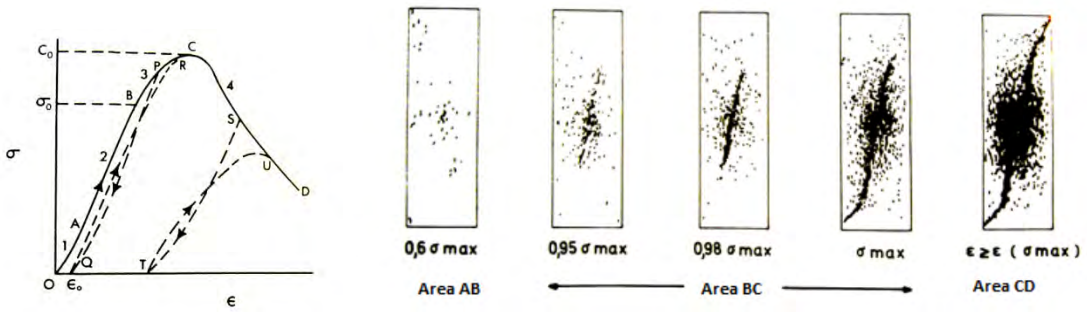


Figure H-5 Development of Microcracks at Uniaxial Compression Stress

(Source: AALTO University, Finland)

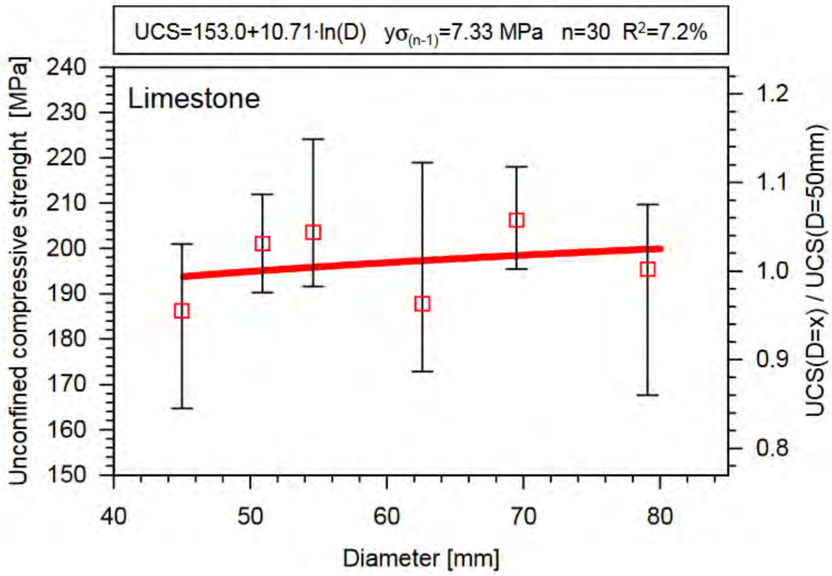


Figure H-6 Uniaxial Compression Strength and Size of Specimen

(Source: Swts and Zeitlinger Lisse, 2001)

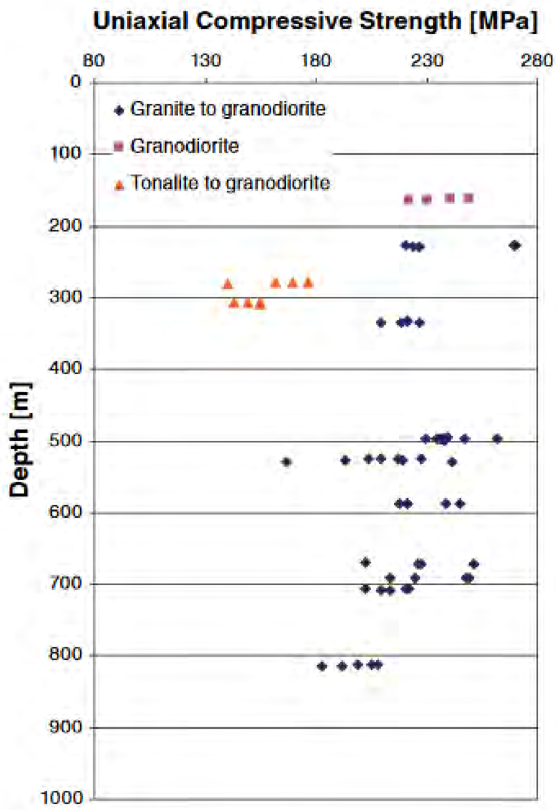


Figure H-7 Variation of the Indirect Uniaxial Compression Strength of the Intact Rock with Depth

(Source: Rock Mechanical Model, Golder Associates, 2005)

2.2 Splitting Tensile (Brazilian) Test for Intact Rocks (ASTM D 3967)

Purpose

To evaluate the (indirect) tensile shear of intact rock core, FT.

Procedures

Core specimens with length-to-diameter ratios (L/D) of between 2 to 2.5 are placed in a compression loading machine with the load platens situated diametrically across the specimen. The maximum load (P) to fracture the specimen is recorded and used to calculate the split tensile strength.

Commentary

The Brazilian or split-tensile strength (FT) is significantly more convenient and practicable for routine measurements than the direct tensile strength test (T0). The test gives very similar results to those from direct tension (Jaeger & Cook, 1976). It is a more fundamental strength measurement of the rock material, as this corresponds to a more likely failure mode in many situations than compression. Also, note that the point load index is actually a type of Brazilian tensile strength, which is correlated back to compressive strength.

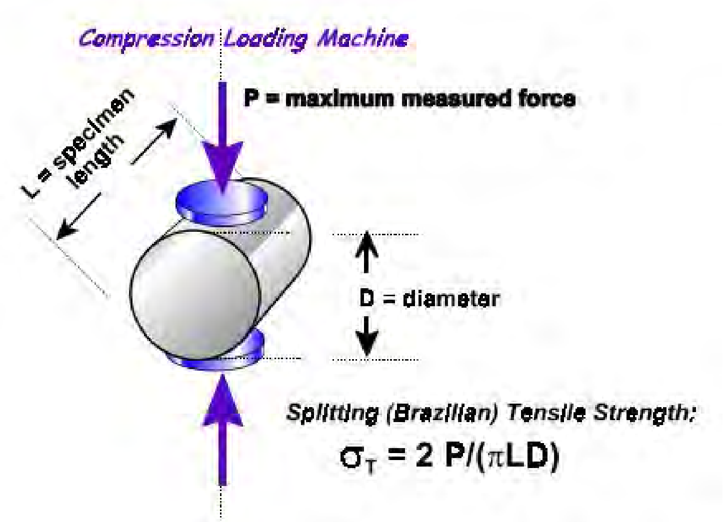


Figure H-8 Setup for Brazilian Tensile Test in Standard Loading Machine

(Source: AASHTO, ASTM (FHWA NHI Subsurface Investigations))

Brazilian test machine

Load

Figure H-9 Setup for Brazilian Tensile Test and Standard Loading Machine

(Source: JICA project, Hoek and Brown, 1997 modified)

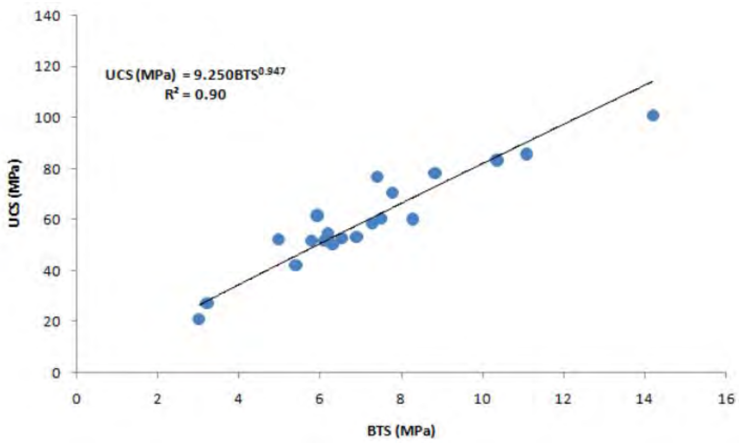


Figure H-10 Proposed Correlations between Unconfined Compressive Strength and Brazilian Tensile Strength

(Source: Swts and Zeitlinger Lisse, 2001)

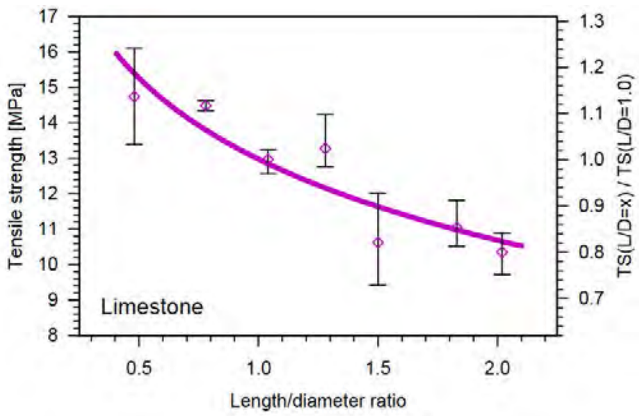


Figure H-11 Tensile Strength and Size of Specimen

(Source: Swts and Zeitlinger Lisse, 2001)

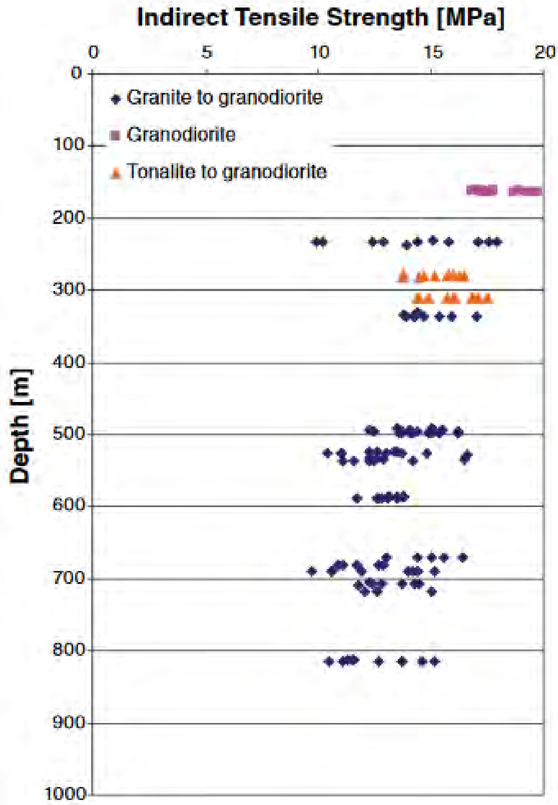


Figure H-12 Variation of the Indirect Tensile Strength of the Intact Rock with Depth

(Source: Rock Mechanical Model, Golder Associates, 2005)

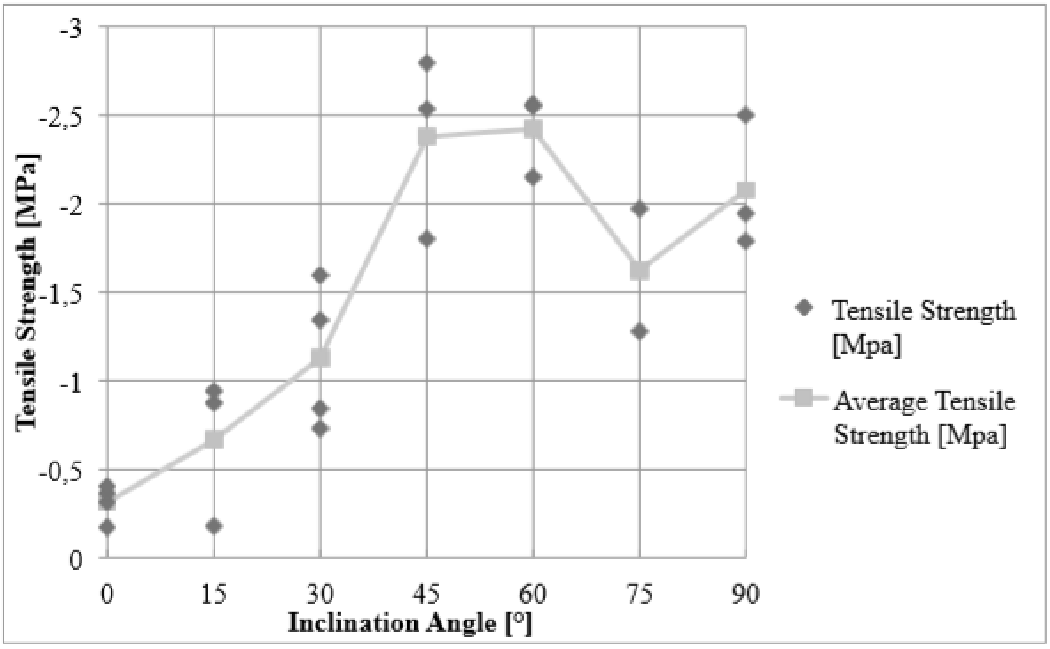


Figure H-13 Tensile Strength for All Inclination of Bedding

(Source: Norwegian University of Science and Technology)

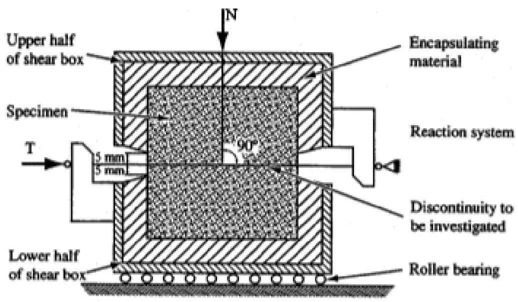
2.3 Direct Shear Strength of Rock (ASTM D 5607)

Purpose

To determine the shear strength characteristics of rock along a plane of weakness

Procedure

The laboratory test equipment is shown below in Figure H-14. The specimen is placed in the lower half of the shear box and encapsulated in either synthetic resin or mortar. The specimen must be positioned so that the line of action of the shear force lies in the plane of the discontinuity to be investigated. The normal force acts perpendicular to this surface. Once the encapsulating material has hardened, the specimen is mounted in the upper half of the shear box in the same manner. A strip approximately 5 mm wide above and below the shear surface must be kept free of encapsulating material. The test is then carried out by applying a horizontal shear force T under a constant normal load, N.



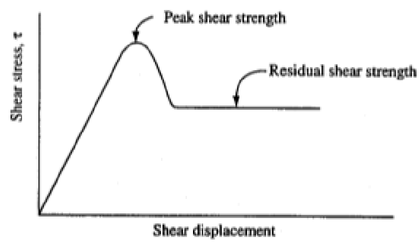


Figure H-14 (a) General Set-up for Direct Shear Strength Testing of Rock (Wittke, 1990)
(b) Derived Shear Stress vs. Shear Displacement Curve. (ASTM D 5607, 1995)
(Source: AASHTO, ASTM (FHWA NHI Subsurface Investigations)

Commentary

Determination of shear strength of rock specimens is an important aspect in the design of structures such as rock slopes, foundations and other purposes. Pervasive discontinuities (joints, bedding planes, shear zones, fault zones, schistosity) in a rock mass, as well as genesis, crystallography, texture, fabric, and other factors can cause the rock mass to behave as an anisotropic and heterogeneous discontinuum. Therefore, the precise prediction of rock mass behavior is difficult.

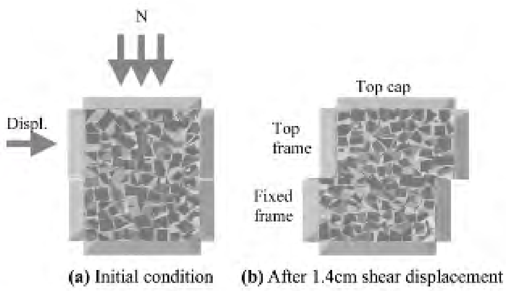
For nonplanar joints or discontinuities, shear strength is derived from a combination base material friction and overriding of asperities (dilatancy), shearing or breaking of the asperities, rotations at or wedging of the asperities (Patton, 1966). Sliding on and shearing of the asperities can occur simultaneously. When the normal force is not sufficient to restrain dilation, the shear mechanism consists of the overriding of the asperities. When the normal load is large enough to completely restrain dilation, the shear mechanism consists of the shearing off of the asperities.

Using this test method to determine the shear strength of intact rock may generate overturning moments that induce premature tensile breaking. Thus, the specimen would fail in tension first rather than in shear.

Rock shear strength is influenced by overburden stresses; therefore, the larger the overburden stress causes the larger the shear strength.

In some cases, it may be desirable to conduct tests in-situ rather than in the laboratory to more accurately determine representative shear strength of the rock mass, particularly when design is controlled by discontinuities filled with very weak material.

Shear test on rock discontinuities



Shear of rock discontinuity

Figure H-15 Portable Direct Shear Test

(Source: JICA project, Hoek and Brown, 1997 modified)

$$A = \frac{\pi D^2}{4\cos\Theta}$$

where:
D = core diameter, and
Θ = angle of tip.

Calculate the following engineering stresses:

$$\text{Apparent normal stress } \sigma = \frac{P_n}{A}$$

$$\text{Apparent shear stress } \tau = \frac{P_s}{A}$$

where:
P_n = normal load,
P_s = shear load, and
A = nominal initial cross-sectional area

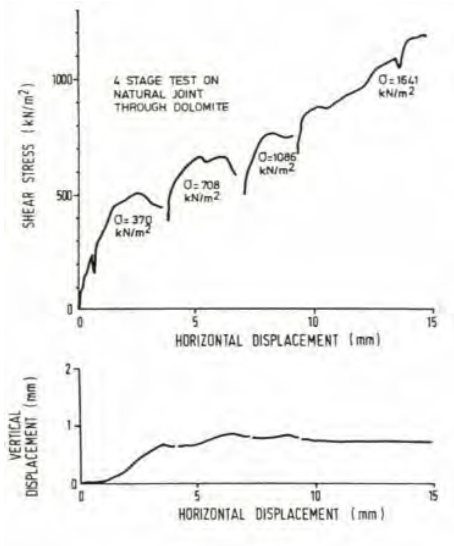


Figure H-16 Calculation of Stress and Raw data of Direct Shear Testing

(Source: Ground Endineering, 1989)

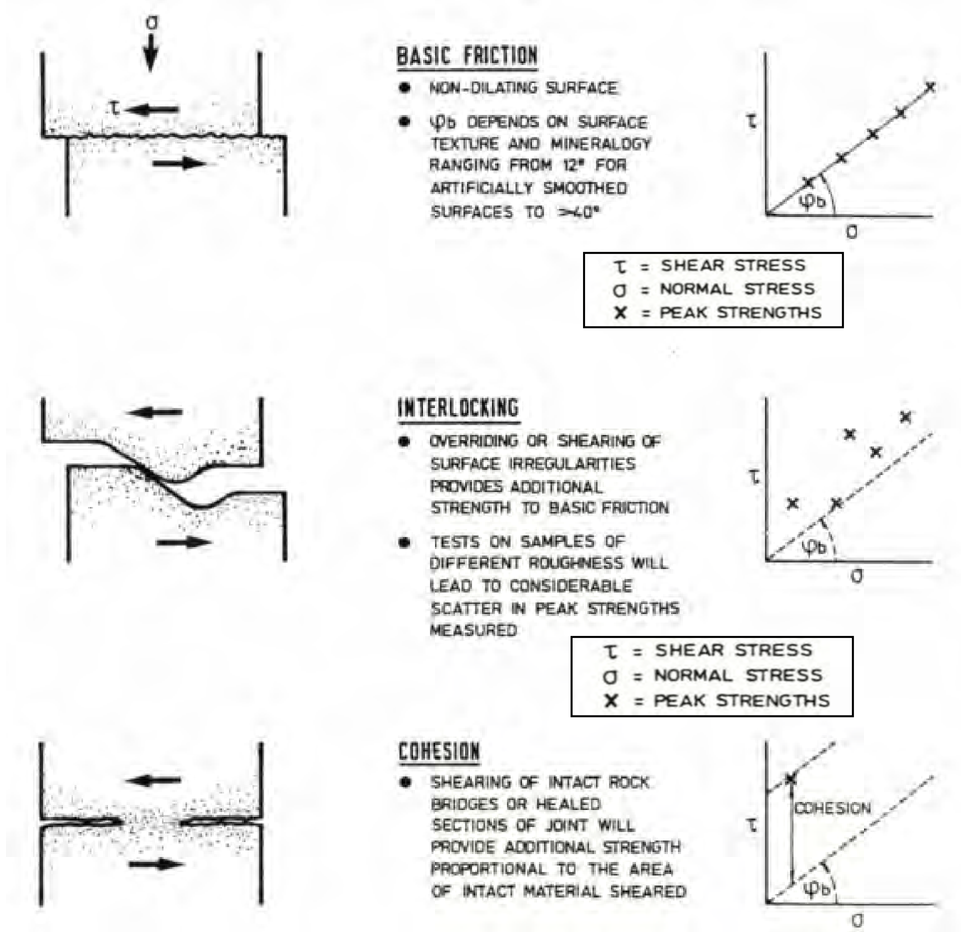


Figure H-17 Factors Contributing to Shear Strength

(Source: Ground Endineering, 1989)

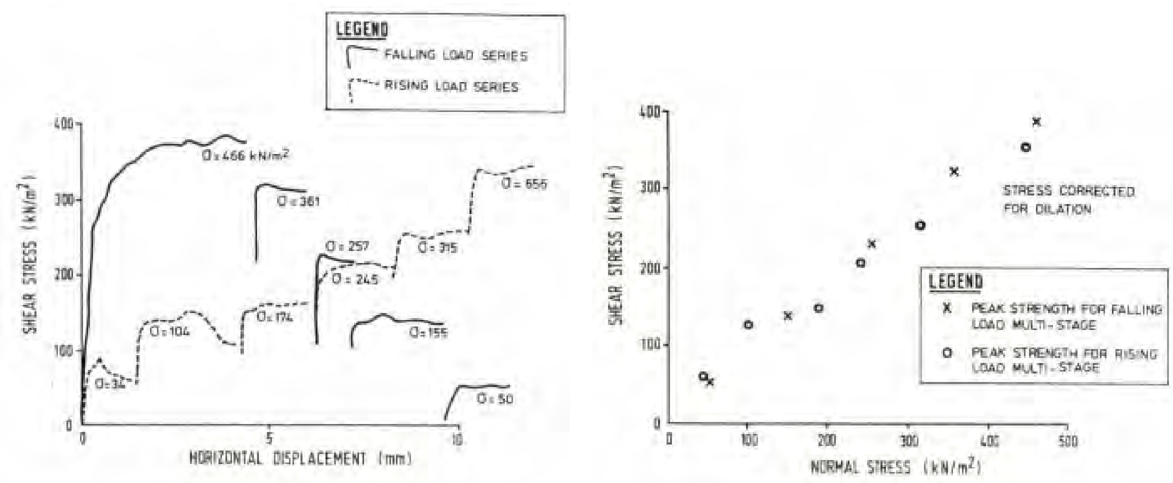


Figure H-18 Examples of Results from Rising and Falling Load Multi Stage Tests

(Source: Ground Endineering, 1989)

2.4 Elastic Moduli (ASTM D 3148)

Purpose

To determine the deformation characteristics of intact rock at intermediate strains and permit comparison with other intact rock types.

Procedure

This test is performed by placing an intact rock specimen in a loading device and recording the deformation of the specimen under axial stress. The Young's modulus, either average, secant, or tangent moduli, can be determined by plotting axial stress versus axial strain curves.

Commentary

The results of these tests cannot always be replicated because of localized variations in each unique rock specimen. They provide reasonably reliable data for engineering applications involving rock classification types, but must be adjusted to take into account rock mass characteristics such as jointing, fissuring, and weathering.

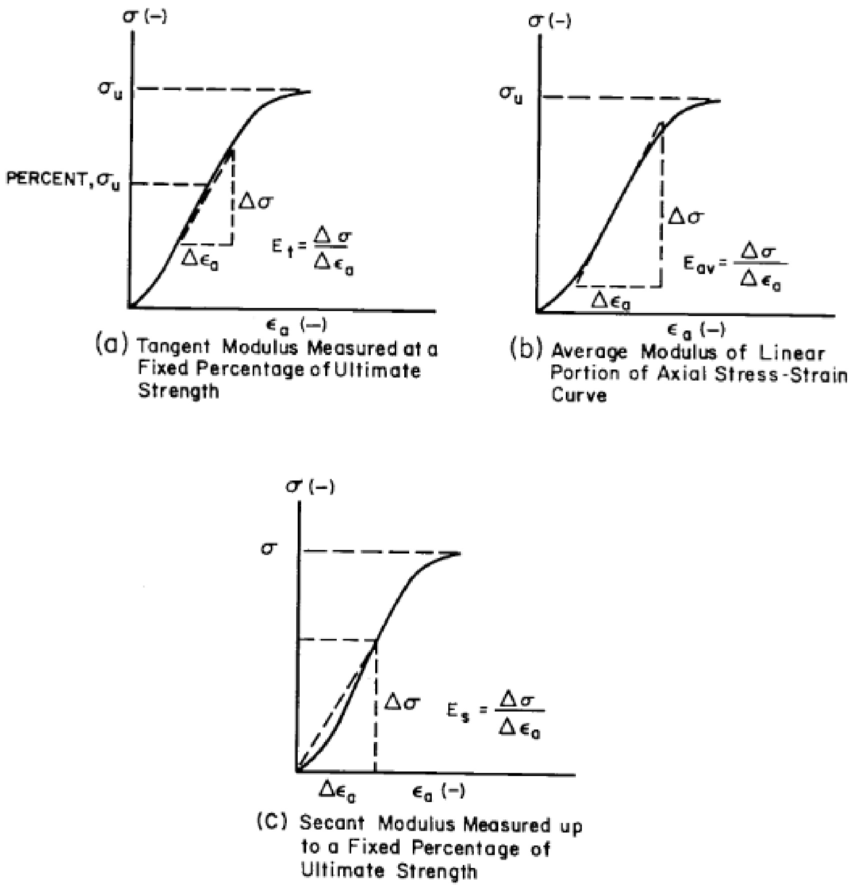


Figure H-19 Method for Calculating Young's Modulus from Axial Stress-Axial Strain Curve

(Source: AASHTO, ASTM (FHWA NHI Subsurface Investigations))

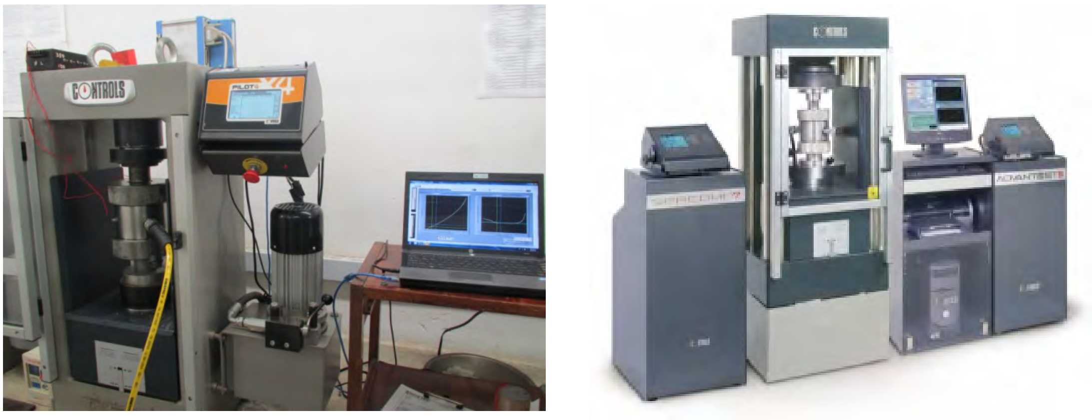


Figure H-20 Equipment of Measurement for Deformation

(Source: JICA project, Hoek and Brown, 1997 modified)

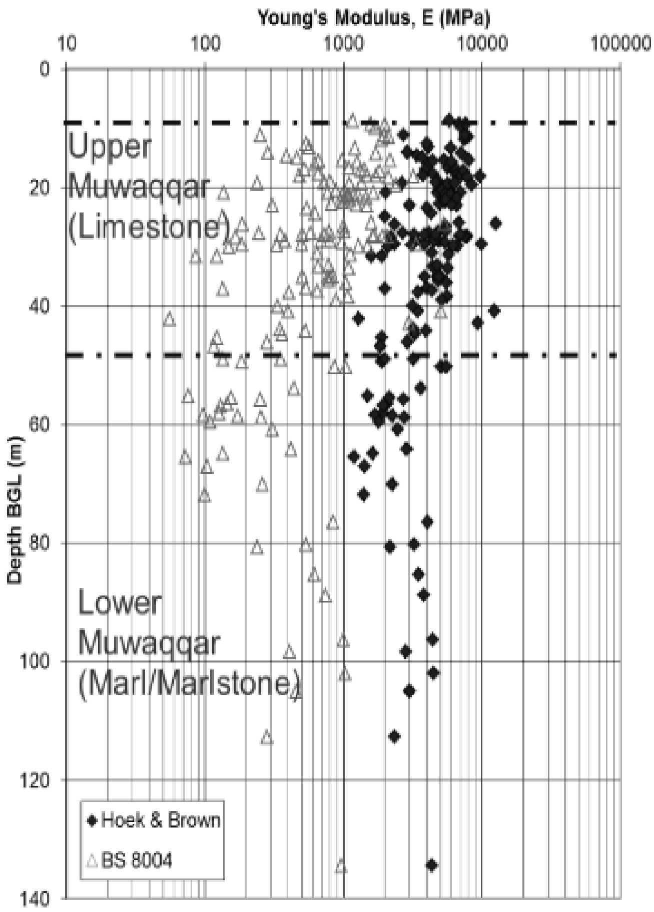


Figure H-21 Relationship between Deformation Modulus and Sampled Depth

(Source: Arab Center for Engineering Studies)

2.5 Ultrasonic Testing (ASTM D 2845)

Purpose

To determine the pulse velocities of compression and shear waves in intact rock and the ultrasonic elastic constants of isotropic rock.

Procedure

Ultrasound waves are transmitted through a carefully prepared rock specimen. The ultrasonic elastic constants are calculated from the measured travel time and distance of compression and shear waves in a rock specimen. Figure H-21 shows a schematic diagram of a typical apparatus used for ultrasonic testing.

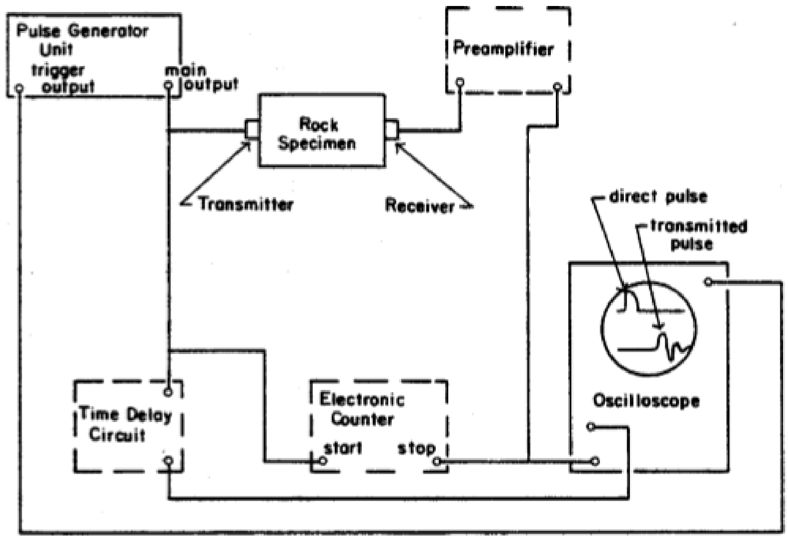


Figure H-22 Schematic Diagram of the Ultrasonics Apparatus (ASTM D 2845)

(Source: AASHTO, ASTM (FHWA NHI Subsurface Investigations))

Commentary

The primary advantages of ultrasonic testing are that it yields compression (P-wave) and shear (S-wave) velocities, as well as ultrasonic values for the elastic constants of intact homogeneous isotropic rock specimens. Elastic constants for rocks having pronounced anisotropy may require measurements to be taken across different directions to reflect orthorhombic stiffnesses and moduli, particularly if pronounced foliation, banding, layering, and fabric are evident.

The ultrasonic evaluation of elastic rock properties of intact specimens is useful for rock classification purposes and the evaluation of static and dynamic properties at small strains (shear strains < 10-4 %). Older equipment only provides ultrasonic P-waves measurements, while new designs obtain both P- and S-wave velocities. When compared with wave velocities obtained from field geophysical tests, the ultrasonic results provide an index of the degree of fissuring within the rock mass. This test is relatively inexpensive to perform and is nondestructive, it thus may be conducted prior to strength testing of intact cores to optimize data collection.

Determination of the ultrasonic velocity of longitudinal and shear waves in cylindrical rock specimens is conducted by calculating the travel time through them as an index to degree of fissuring.

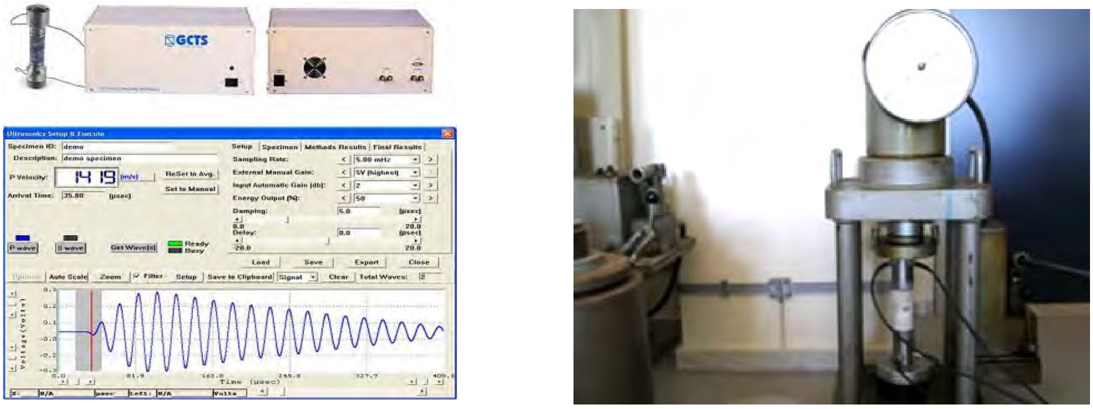


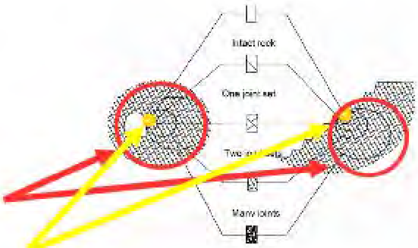
Figure H-23 Ultrasonic Measurement Apparatus and P and S wave Recorder

(Source: JICA project, Hoek and Brown, 1997 modified)

$$Cr = 1 - (VF / VL)$$

When

Cr : seismic velocity ratio
VF : seismic velocity of rock mass
VL : seismic velocity of rock sample



Condition of Rock Mass	
Cr > 0.80	Poor
Cr < 0.25	Good

Figure H-24 Seismic Velocity Ratio and Condition of Rock Mass

(Source: JICA project, Hoek and Brown, 1997 modified)

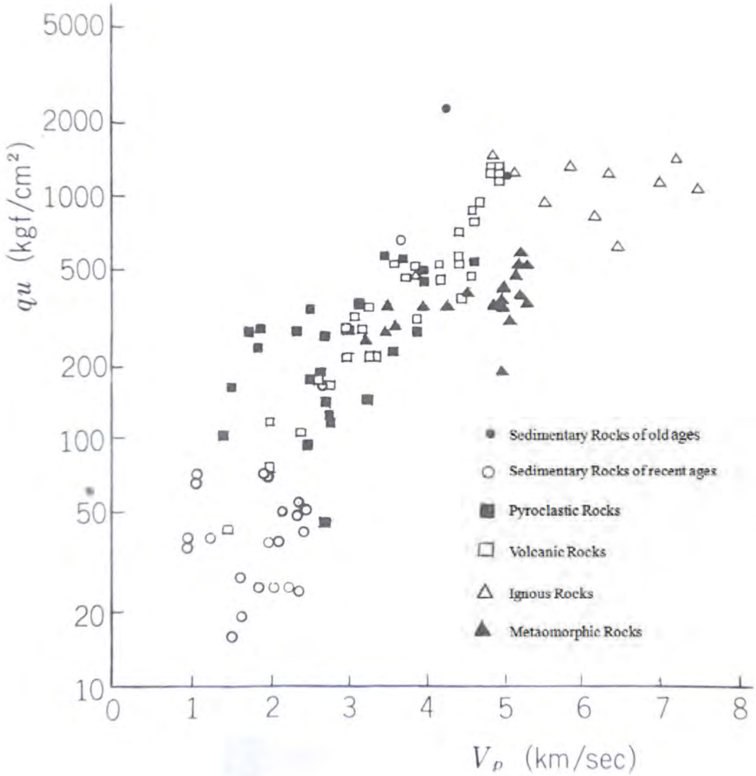


Figure H-25 Geology and S wave Velocity

(Source: Rock Mass Classification, Doboku Kogakusya, 1989)

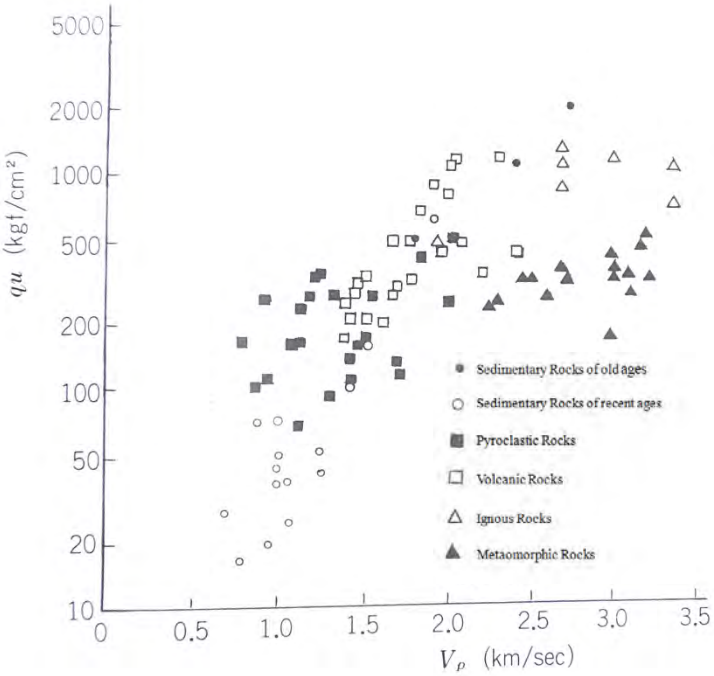


Figure H-26 Geology and P wave Velocity

(Source: Rock Mass Classification, Doboku Kogakusya, 1989)

2.6 Point Load Index (Strength) (ASTM D 5731)

Purpose

To determine strength classification of rock materials through an index test.

Procedure

Rock specimens in the form of core (diametral and axial), cut blocks or irregular lumps are broken by application of concentrated load through a pair of spherically truncated, conical platens. The distance between specimen-platen contact points is recorded. The load is steadily increased, and the failure load is recorded.

There is little sample preparation. However, specimens should conform to the size and shape requirements as specified by ASTM. In general, for the diametral test, core specimens with a length-to-diameter ratio of 1.0 are adequate while for the axial test core specimens with a length-to-diameter ratio of 0.3 to 1.0 are suitable. Specimens for the block and the irregular lump test should have a length of 50±35 mm and a depth/width ratio between 0.3 and 1.0 (preferably close to 1.0). The test specimens are typically tested at their natural water content. Size corrections are applied to obtain the point load strength index, $Is_{(50)}$, of a rock specimen. A strength anisotropy index, $Is_{(50)}$, is determined when $Is_{(50)}$ values are measured perpendicular and parallel to planes of weakness.

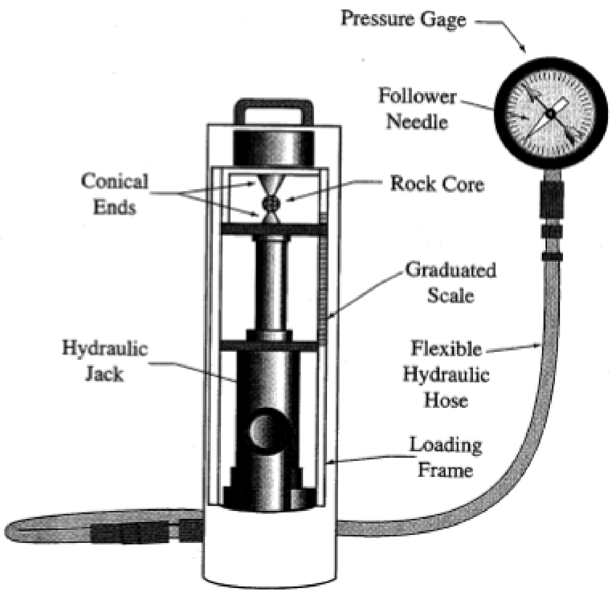


Figure H-27 Point Load Test Apparatus. (adopted from roctest)

(Soource: AASHTO, ASTM (FHWA NHI Subsurface Investigations))

Commentary

The test can be performed in the field with portable equipment or in the laboratory (Figure 2.12). The point load index is used to evaluate the uniaxial compressive strength (Fu). On the average, Fu = 25 $Is_{(50)}$, however, the coefficient term can vary from 15 to 50 depending upon the specific rock formation, especially for anisotropic rocks. The test should not be used for weak rocks where Fu < 25 MPa

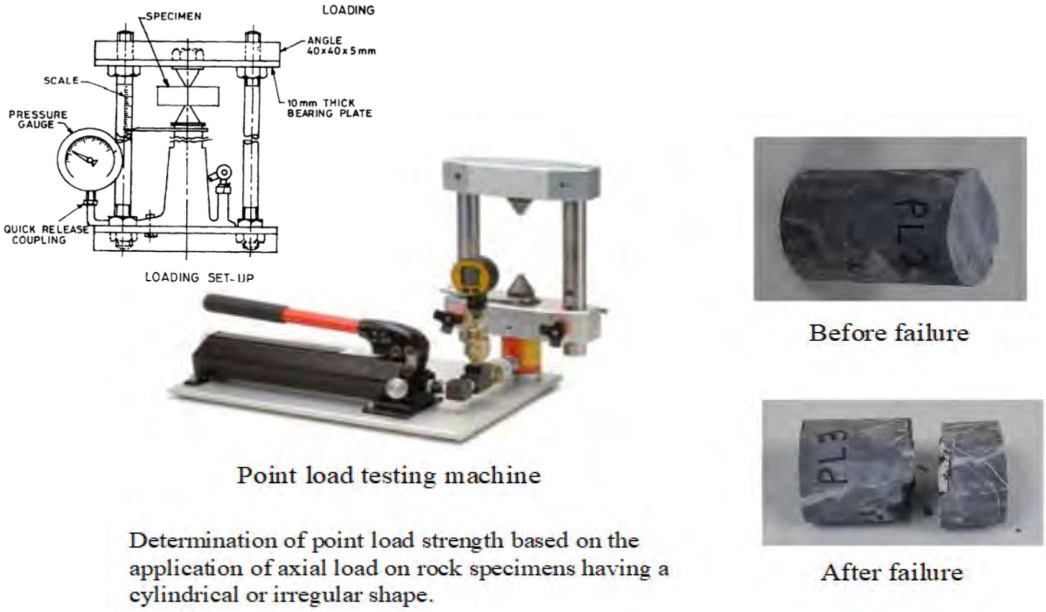


Figure H-28 Equipment and Specimen of Point Load Index

(Source: JICA project, Hoek and Brown, 1997 modified)

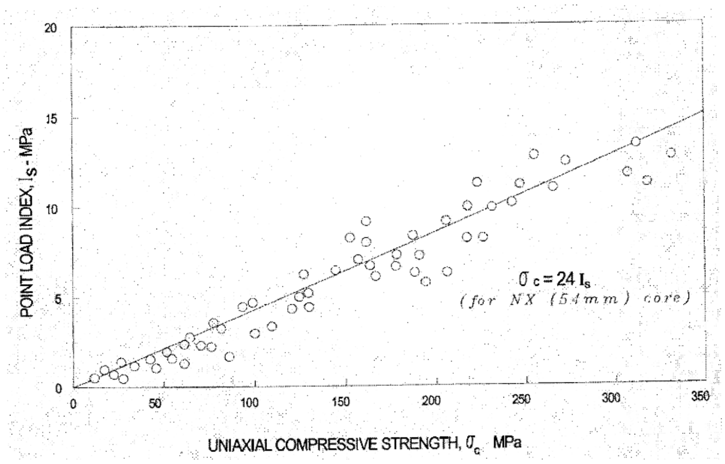


Figure H-29 Example of Correlations between Point Load and Uniaxial Compressive Strength (ISRM)

(Source: International Society of Rock Mechanics)

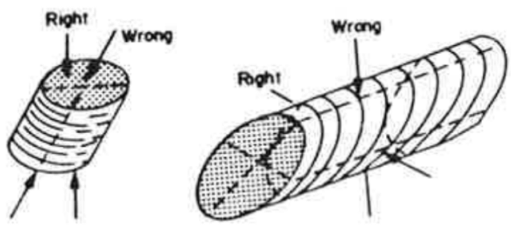


Figure H-30 Right and Wrong Application of the Point Loads on Drilled Core Samples Atan Oblique Angle to Foliatiom and Bedding

(Source: Talor & Francis Group, London 2006)

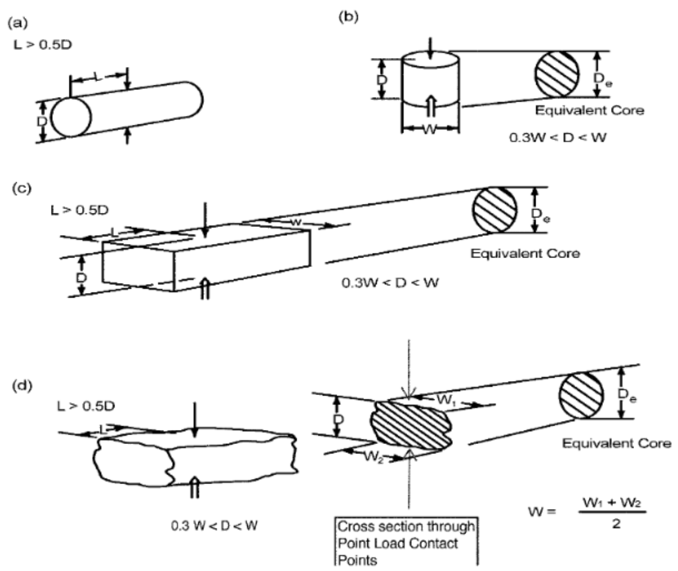


Figure H-31 Load Configuration and Specimen Shape for (a) the Diametral Test, (b) the Axial Test, (c) the Block Test, (d) The Irregular Lump Test

(Source: AASHTO, ASTM)

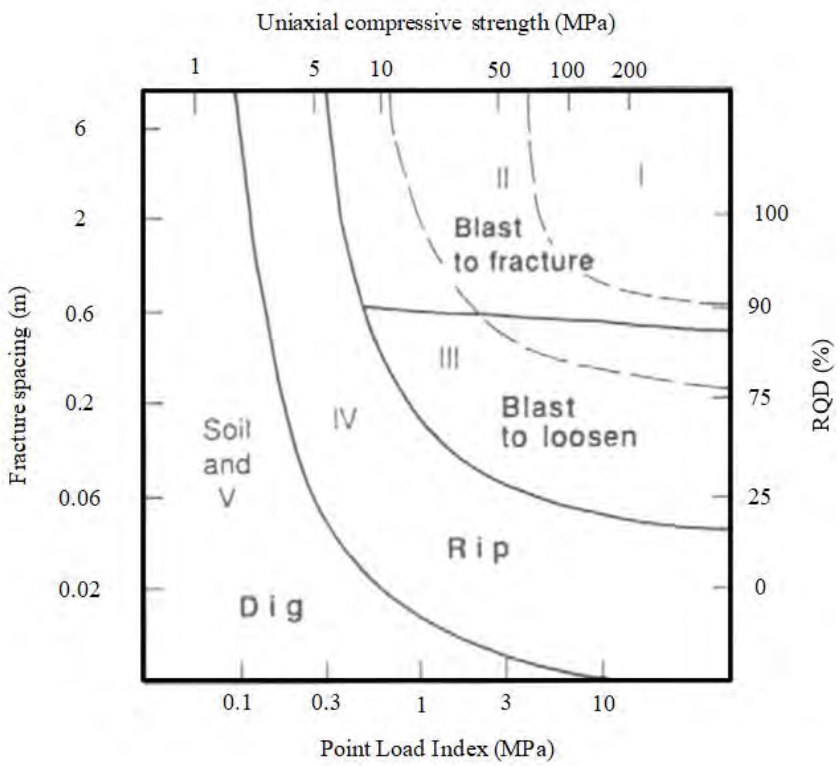


Figure H-32 Rock Excavatability and Point Load Index

(Source: Advanced Geotechnical Engineering, M. A. M. Ismail)

3 In-situ Testing

In-situ Rock Tests are done to determine in-situ stresses and deformation characteristics, as well as shear strength of jointed rock mass or critically weak seams within the rock mass, and residual stresses within the rock mass. In-situ tests are often the best means for determining the engineering properties of subsurface materials; in some cases, the in-situ tests may be the only way to obtain meaningful results.

Large-scaled in-situ tests tend to average out the effect of complex interactions. In-situ tests in rock are frequently expensive and should be reserved for projects with large and concentrated loads. Well-conducted tests may be useful in reducing overly conservative assumptions. Such tests should be located in the same general area as a proposed structure. Test loading should be applied in the same direction as the proposed structural loading.

In-situ deformation testing and shearing testing are mainly carried out for confirmation of properties of the rock mass. The general layout, equipment and tools together with analysis procedures are shown below.

3.1 Plate Loading Testing

In-situ plate loading testing is carried out to determine the in-situ deformation characteristics of the rock mass. In-situ tests are said to be the best means for determining the engineering properties of the rock mass. Large-scaled in-situ tests tend to average out the effect of complex interactions. In-situ tests in rock are frequently expensive and should be reserved for projects with large, concentrated loads.

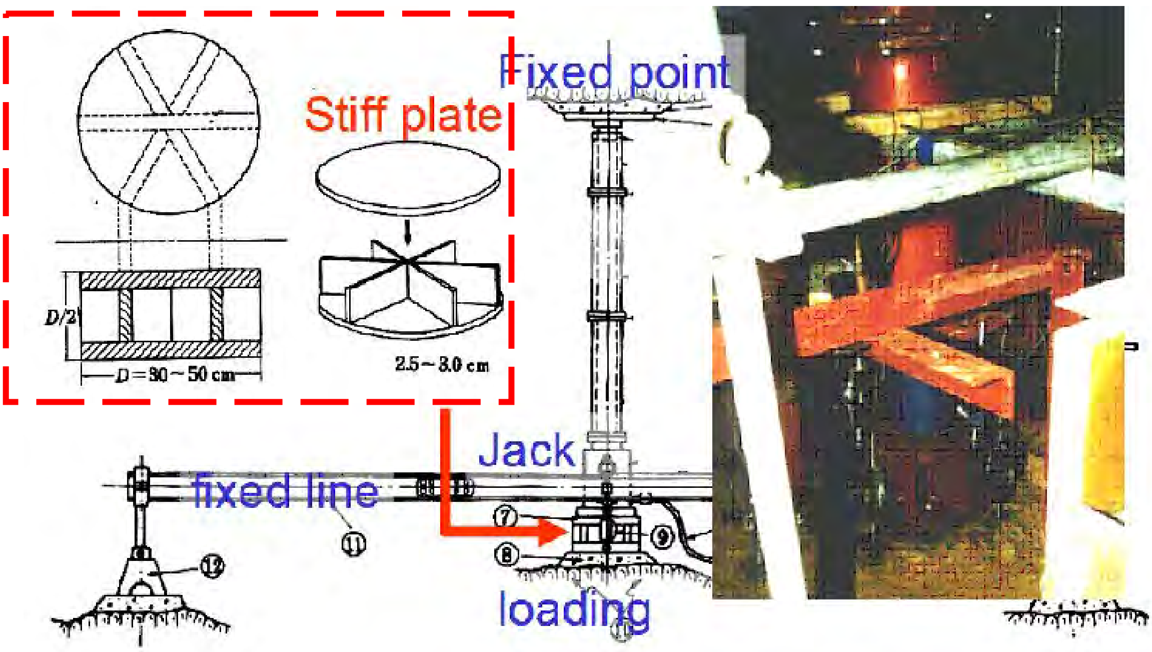


Figure H-33 Tools for In-situ Deformation Test

(Source: JICA project, Rock Mass Classification, Doboku Kogakusya, 1989)

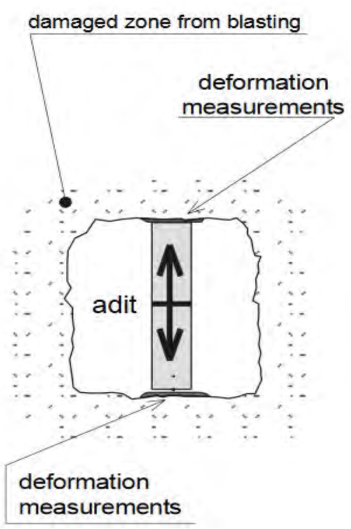
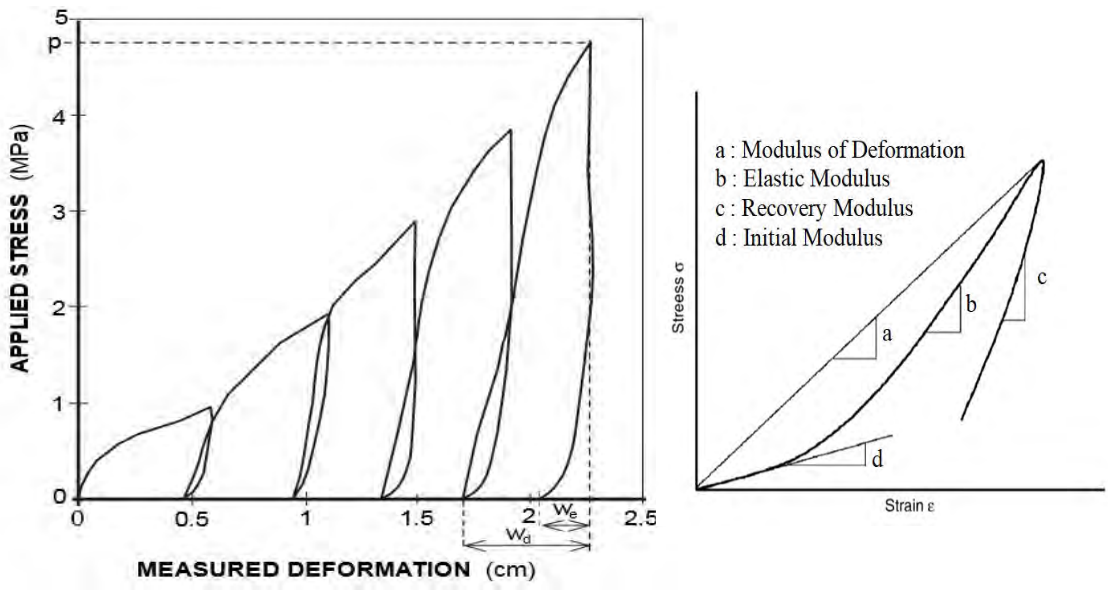


Figure H-34 Analysis for In-situ Deformation Test

(Source: JICA project)



$$E = m\frac{\left(1-\mu^2\right)}{\delta}\frac{P}{\sqrt{A}}$$

Where,

E = Elastic/deformation modulus in kg/cm^2

m = 0.96 for circular plate and 0.95 for square plate

δ = Incremental displacement (deformation or settlement) of bearing plate in one loading cycle in cm

P = Incremental applied load in kg

A = Area of test plate in cm^2

μ = Poisson's ratio of rock mass

Figure H-35 Calculation Equation of In-situ Deformation Modulus

(Source: JICA project)

3.2 Shearing Testing

An in-situ shearing test is done to determine in-situ stress including deformation characteristics of the rock mass. The shear strength often depend on joints, critically weak seams within the rock mass, and residual stresses within the rock mass. In-situ tests are often the best means for determining the engineering properties of the rock mass, however high cost is often required in comparison with laboratory tests and simple index tests. Large-scaled in-situ tests tend to average out the effect of complex interactions. In-situ tests in rock mass are frequently reserved for projects with large, concentrated loads.

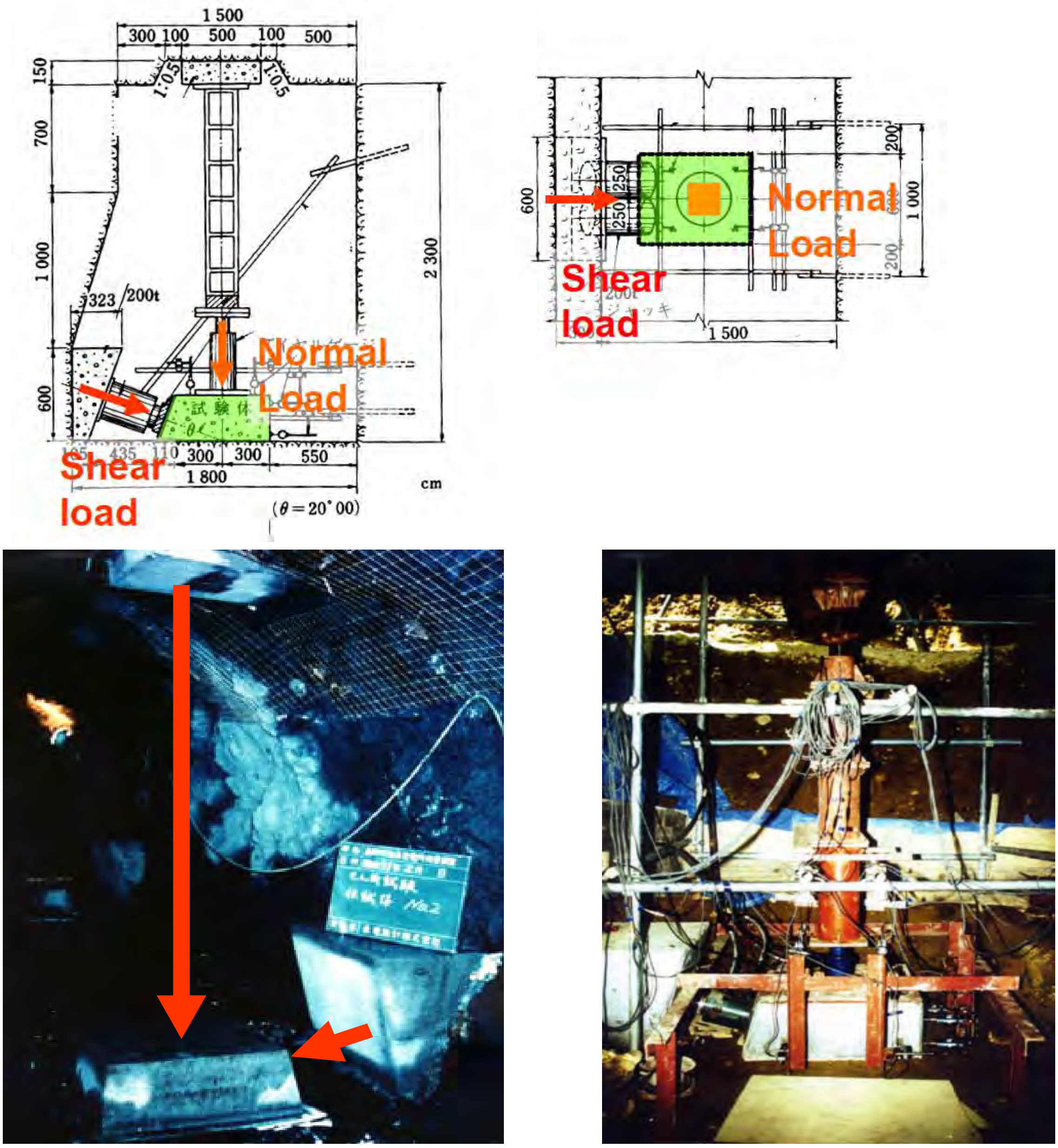


Figure H-36 In-situ Rock Shear Testing

(Source: JICA project, Rock Mass Classification, Doboku Kogakusya, 1989)

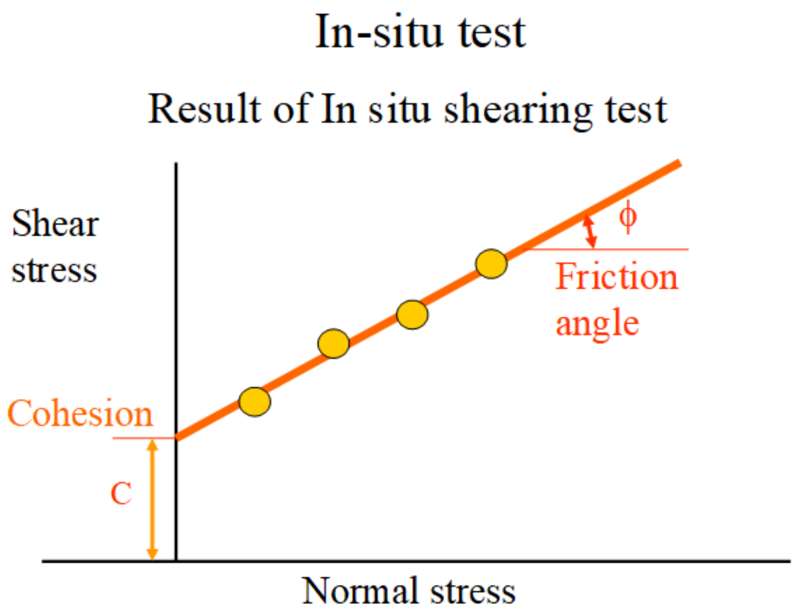


Figure H-37 Analysis of In-situ Rock Shear Testing

(Source: JICA project)

4 Simple Index Testing

A simple index test is carried out to measure the properties of the rock materials by several measurement methods designed for simplicity with less cost. A correlation between the measured properties such as hardness and the uniaxial compressive strength are established from various and numerous data. Various applications of the test to civil engineering practice are designed and performed.

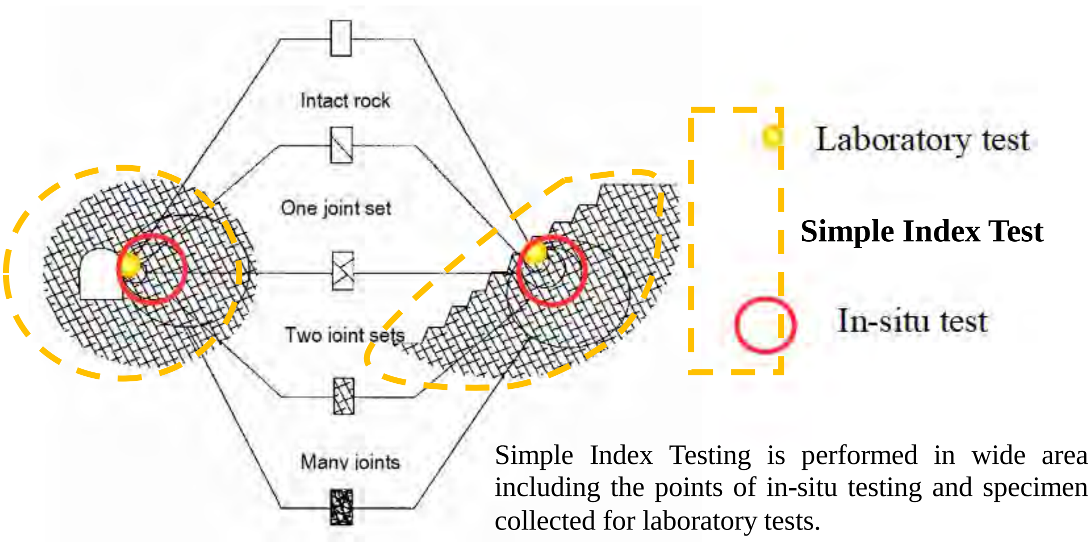


Figure H-38 General Concept of Simple Index Test

(Source: JICA project, Hoek and Brown, 1997 modified)

4.1 Schmidt Test Hammer

The Schmidt hammer is one of the most commonly used simple index testings at sites. Many types of hammer are utilized depending on the condition of outcrops at the site. The equipment of the Schmidt hammer and application of the testing results are shown hereinafter.

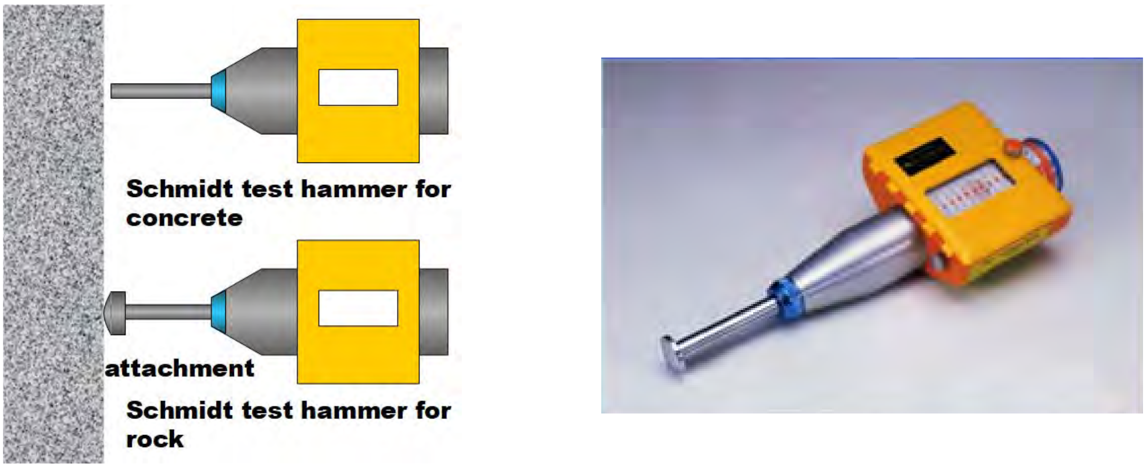


Figure H-39 Equipment and Tools for Schmidt Hammer

(Source: JICA project)

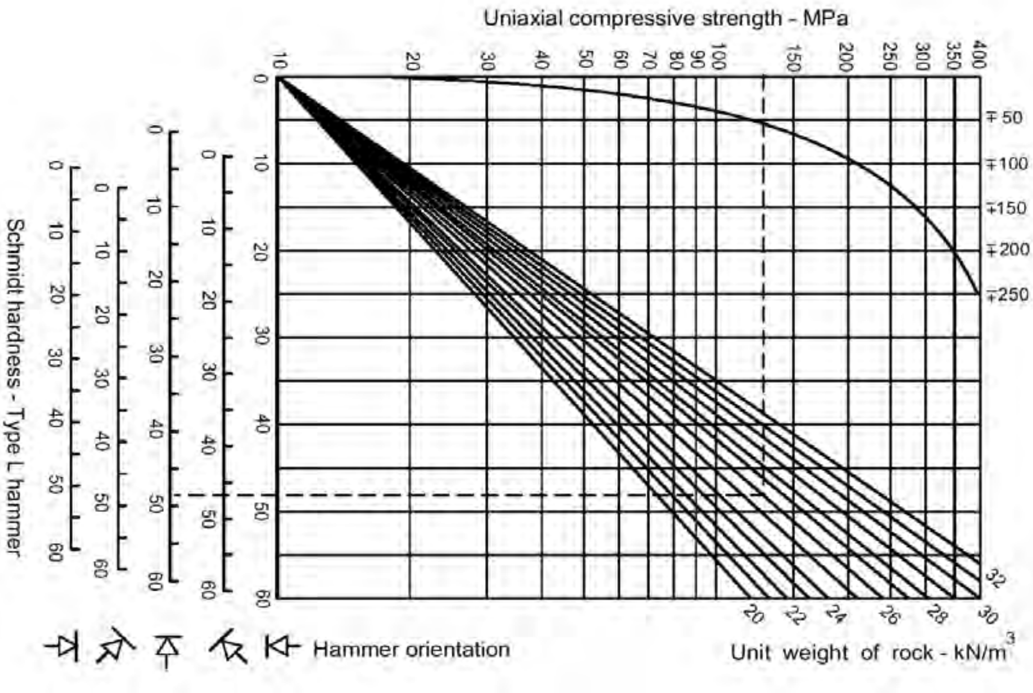


Figure H-40 Average Dispersion of Strength

(Source: Correlation chart for Schmidt hammer and Uniaxial strength, Miller 1965 modified)

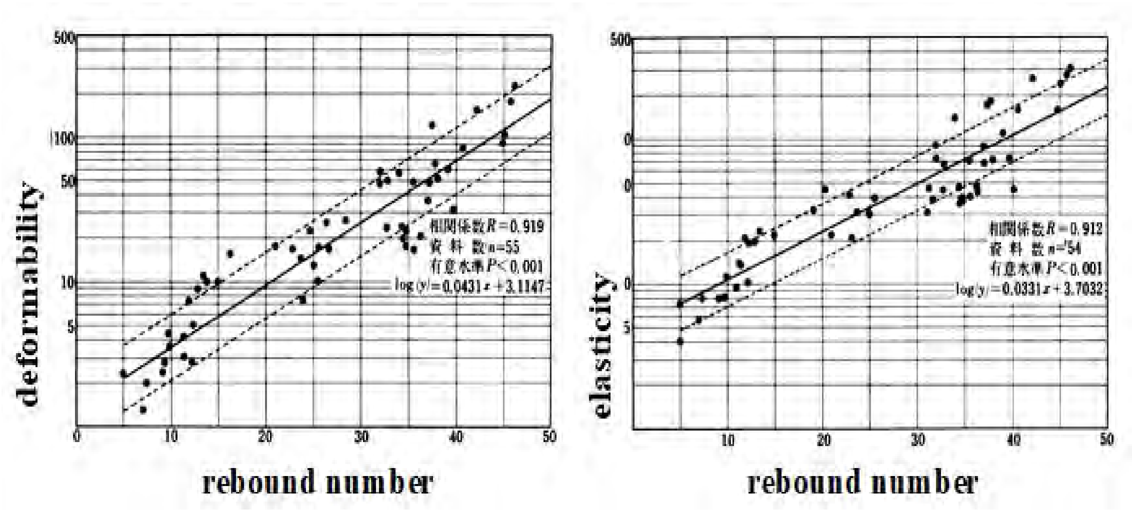


Figure H-41 Relationship between Schmidt Hammer Results and Deformability and Elasticity

(Source: Schmidt Rock Hammer, FTS Co.Ltd.)

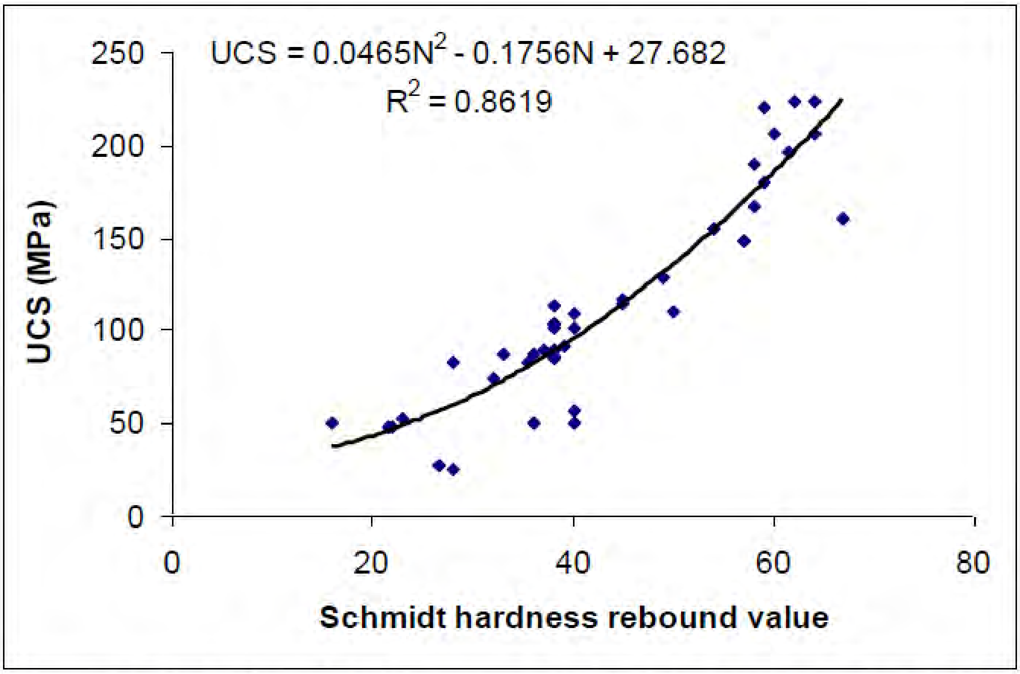


Figure H-42 Relationship between Schmidt Hammer Results and Unconfined Compression Strength

(Source: Swts and Zeitlinger Lisse, 2001)

4.2 Ultrasonic Measurement

Ultrasonic measurement, as ultrasonic testing described in the laboratory test, is one of the simple index testings of non-destructive testing techniques based on the propagation of ultrasonic waves in the rock material or rock mass tested. Very short ultrasonic pulse-waves with center frequencies ranging from 0.1-15 MHz are generally transmitted into the rock materials to detect internal flaws to characterize the rocks. Equipment and tools for ultrasonic measurement and the relationship between typical longitudinal wave velocity and geology are shown as follows.

Figure H-43 Equipments and Tools for Ultrasonic Mesurement (testing)

(Source: JICA project)

Table H-2 Relationship between Rock and Longitudinal Wavws

Rock	Longitudinal waves (m/s)
Gabbro	7000
Basalt	6500-7000
Limestone	6000-6500
Dolomite	6500-7000
Sandstone and quartize	6000
Granitic rocks	5500-6000

(Source: Swts and Zeitlinger Lisse, 2001)

Reference

American Society for Testing And Materials (ASTM) Standards for materials, goods, services and systems.

American Association of State Highway and Transportation Officials (AASHTO) standards and publications.

THE DEFORMATION MODULUS OF ROCK MASSES
Tunnelling and Underground Space Technology, Vol. 16 No. 3, 2001,
comparisons between in situ tests and indirect estimates Arild Palmström, Ph.D., Norway et al,

Laboratory and in-situ rock testing (Advanced Geotechnical Engineering)

I. Hydrogeological Survey

Tunneling works possibly affect surrounding groundwater and surface water environment. Therefore, investigation and analysis are necessary to forecast the impacts on the environment and to take measures against the impacts.

For this reason, a guideline for hydrogeological survey, named as "Guideline for Environmental Impact Study (Groundwater)", is developed. The guideline is bound separately. For detailed information about the hydrogeological survey, please refer to the separate volume of the guideline.