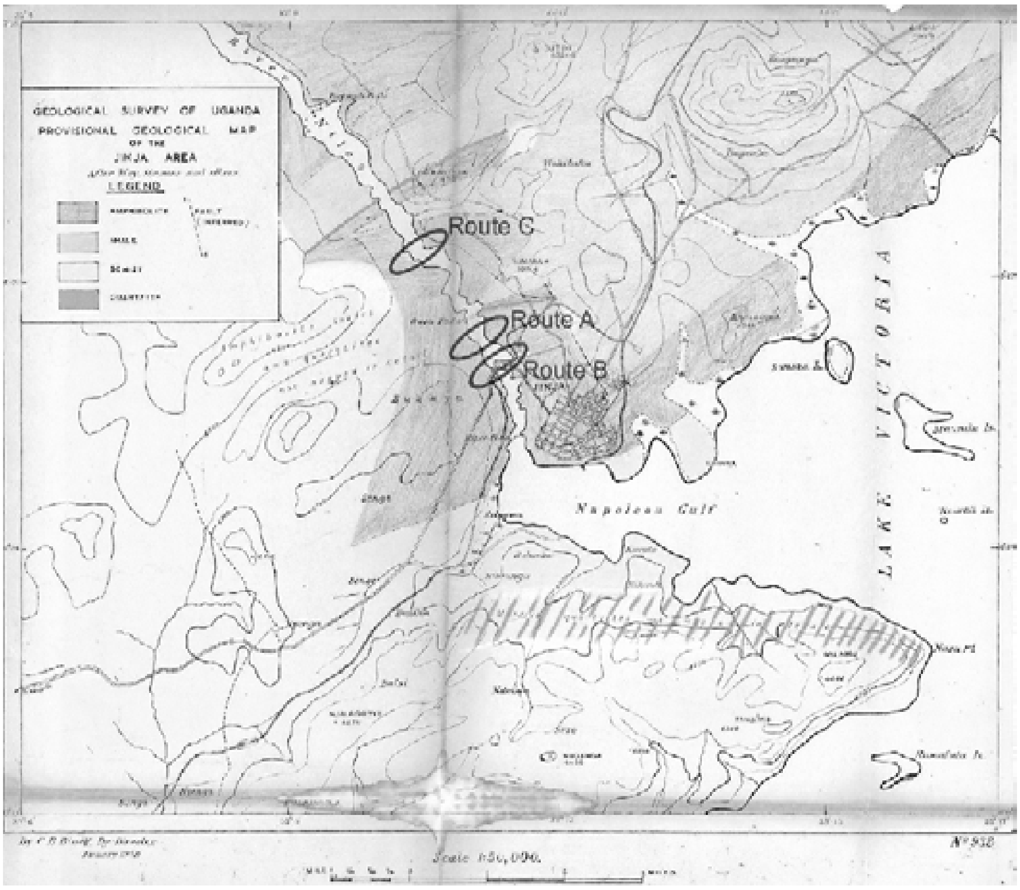
5.4.3 Local Geology

(1) Existing Information Relating to Nalubaale Dam

In cooperation with the Ministry of Energy and Mineral Development, the results of geological surveys which had been carried out prior to the construction of Nalubaale Dam, formerly called Owen Falls Dam, were provided.

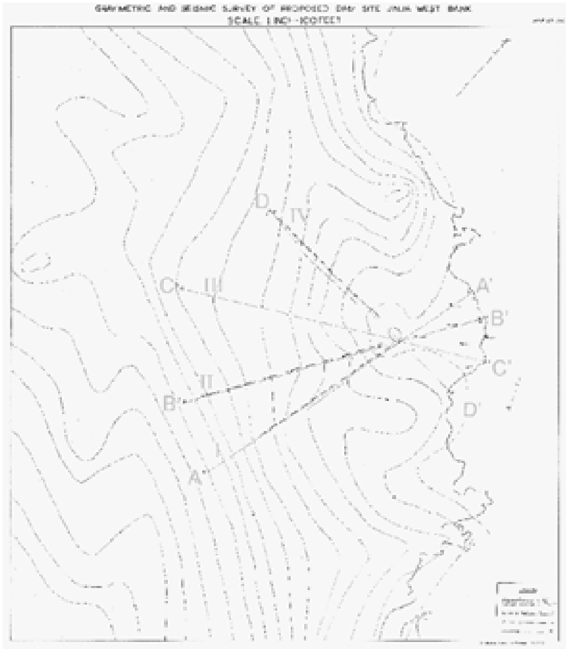
Figure 5.4.4 is a provisional geological map for the construction of Owen Falls Dam. According to this map, at the site where each proposed route crosses the River Nile, Amphibolite is expected to underlie both sides of the river bank.

Gravimetric and seismic surveys had been performed on the west bank of the River Nile as a part of the geological surveys for the dam site. Figure 5.4.5 shows the locations of survey lines. The results of the surveys were compiled into geological cross sections along with results of drillings on the west bank. Figure 5.4.6 collects the speculated geological sections. The notable point understood from these sections is the uniformity in thickness of soil and decomposed layers overlying fresh Amphibolite. Except around the river bank, soil and decomposed layers were speculated to be distributed with nearly-constant thickness, in other words, the top surface of fresh Amphibolite was expected to emerge at roughly the same elevation everywhere in the surveyed area, although it is assumed to have certain ranges of unforeseeable undulation.



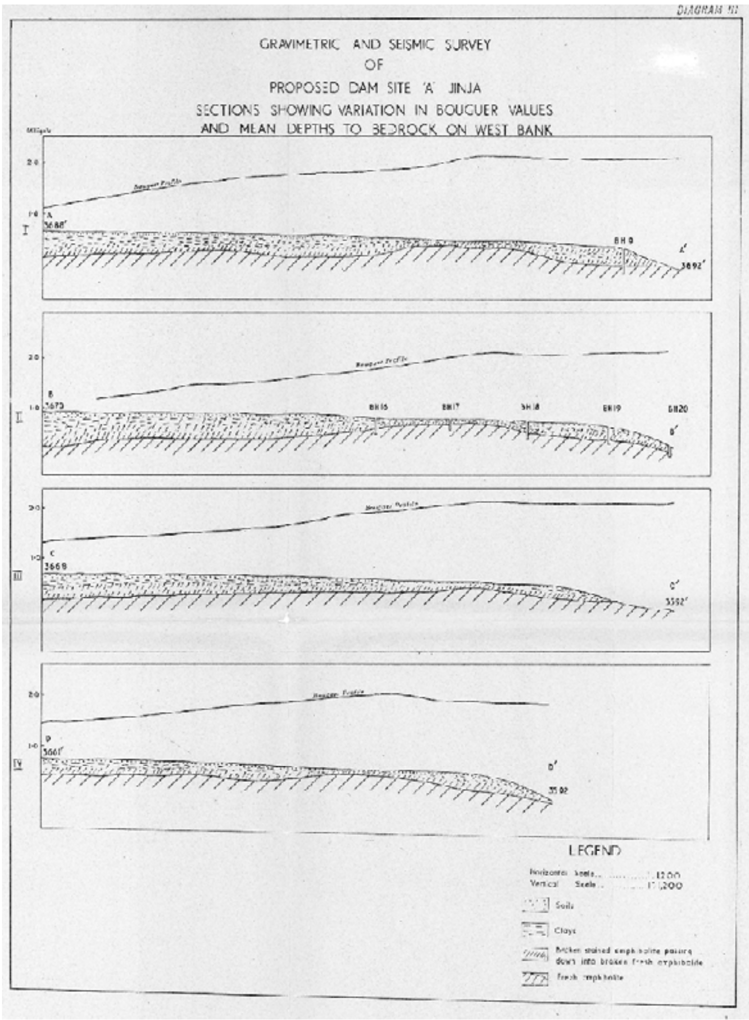
Source: Ministry of Energy and Mineral Development

Figure 5.4.4 Provisional Geological Map for the Construction of Owen Falls Dam



Source: Ministry of Energy and Mineral Development

Figure 5.4.5 Survey Lines of Gravimetric and Seismic Surveys



Source: Ministry of Energy and Mineral Development

Figure 5.4.6 Geological Sections Assumed from Gravimetric and Seismic Surveys

(2) Existing Information Relating to the Kiira Canal

The Kiira Canal was constructed during the late '90s in the east vicinity of Nalubaale Dam. It bypasses the dam with its head located around 300 meters upstream of the dam. Simultaneously constructed Kiira Dam occupies the end of the Kiira Canal which is positioned roughly 750 meters downstream of Nalubaale Dam.

In relation to the proposed route of the new bridge, the east bank of Route A is positioned close to the head of the Kiira Canal.

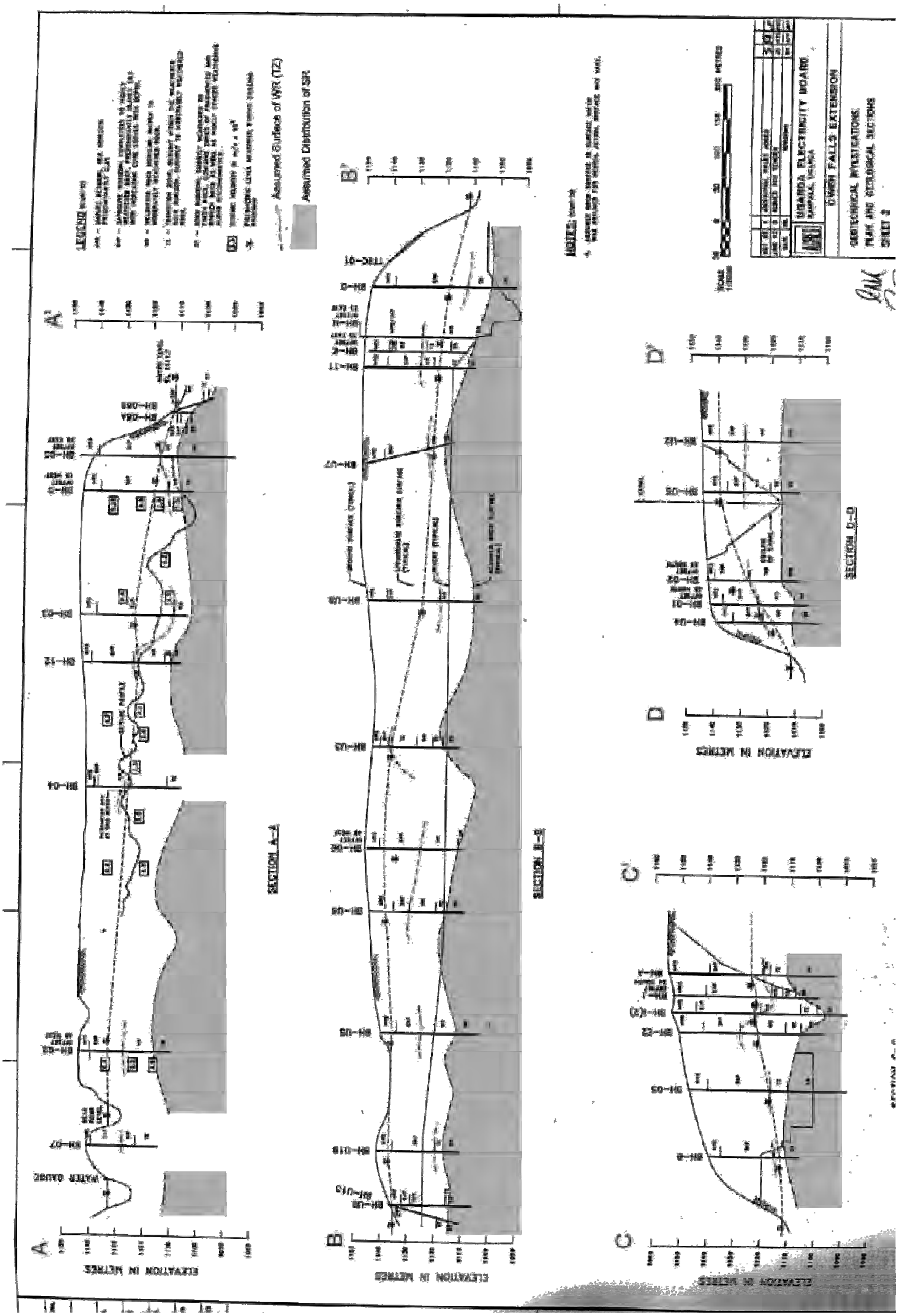
The Ugandan Electricity Generation Company Ltd (UEGCL) provided 4 geological sections which run along and cross the planned (and possibly realized) course of the Kiira Canal. Figure 5.4.7 shows the locations of geological sections which are summarised in Figure 5.4.8. Additional interpretations were added on Figure 5.4.8.

The following tendencies are understood from Figure 5.4.8:

- At the upstream sides of section A-A' and B-B', an underlying rock horizon, which is defined as slightly weathered to fresh rock, can be assumed to emerge at the elevation of between approximately 1,110 and 1,120 m , with its top surface undulating irregularly. A rock horizon is shown in green colour with the symbol "SR".
- The top surface of the weathered rock horizon, which is shown in a broken red line, is assumed to have erratic variation in its emerging elevation. This speculation implies that the thickness of matured residual soil and sapprolite horizon varies erratically.
- At the upstream side of section B-B', which is shown in the left of the section, the top surface of the weathered rock horizon can be assumed to be close to the ground surface. Despite this speculation, the erratic variation in geological condition cannot assure the existence of the weathered rock horizon at a shallower depth, at the planned abutment of Route A.

Source: Ugandan Electricity Generation Company Ltd (UEGCL), annotated by JICA Study Team

Figure 5.4.7 Survey Lines of Geological Sections for the Construction of the Kiira Canal



Source: Ugandan Electricity Generation Company Ltd (UEGCL), annotated by JICA Study Team

Figure 5.4.8 Geological Sections for the Construction of the Kiira Canal

An excerpt of another report seemed to be related to Bujagali Dam construction was provided from the Ministry of Energy and Mineral Development. In the excerpt, some descriptions concerning seismicity and geology in and around the study area are found as follows:

1) Seismicity

In the excerpted report, it was noted that Owen Falls had probably not experienced peak acceleration on site greater than .05g associated with any event in the available record from 1850. This was attributed to the significant distance of Owen Falls from the major limbs of the rift system and relatively small number of recorded events not associated with the major limbs.

Also a seismic design factor is recommended in the excerpted report, shown as follows:

An earthquake event file for Uganda from 1850 to 1989 was compiled based on the literature source and information supplied by the United States Geological Survey data base. Return period for various accelerations experienced on the site were calculated, assuming following:

- *A maximum credible earthquake of M7.5*
- *A frequency-magnitude relationship based on seismic records from 1850 to 1989 for within 200km of Owen Falls (i.e., seismic activity along the major limbs of the rift system was ignored)*
- *A 'floating' event, i.e., one that could occur anywhere within 200km of Owen Falls*
- *Attenuation relationships for hard foundations and either shallow or deep earthquakes as derived by Krinitzsky and Nuttli (1988)**
- *An acceleration factor for use in design of two thirds of peak calculated acceleration.*

Based on the above assumptions, the acceleration factor for design of structures on rock foundations should be approximately 0.18g to match a return period of 1-in-10,000 years.

**: Krinitzsky E.L., Chang F.K. and Nuttli O.W., "Magnitude – Related Earthquake Ground Motion", Bulletin Association of Engineers and Geologists, Vol XXV, No.4, 1988, pp399-423.*

Source: Ministry of Energy and Mineral Development

2) Geology

Within the description of geological information of the excerpted report, the following description can be noted.

- *Top of sound rock may be at a depth of 30m or more and is generally found at about river elevation. The weathering profile is quite variable, probably due to relatively complex and narrow nature of interbanded rock types. As such, depth of weathering can apparently fluctuate tens of metres within a similar range of horizontal distance.*
- *The Nile has eroded through the weathered material and runs along the top of sound rock. The more resistant rock beds form the set of rapids seen in the River Nile, such as Ripon Falls and Bujagali Falls. An example of this kind of unweathered, very strong and competent rock outcrop can be seen at existing tailwater levels downstream of the (Nalubaale) dam.*

Source: Ministry of Energy and Mineral Development

(3) Existing Information Relating to the Under Construction Bujagali Dam

The currently under construction Bujagali Dam is located about 8km north of Nalubaale Dam. Although the distance between Bujagali Dam and the proposed routes for a new bridge is too far to make it reasonable to use its geotechnical data directly to the design of a new bridge, the geotechnical data obtained through the construction of Bujagali Dam is valuable since it can be used as a reference to the general geotechnical condition in the surrounding area.

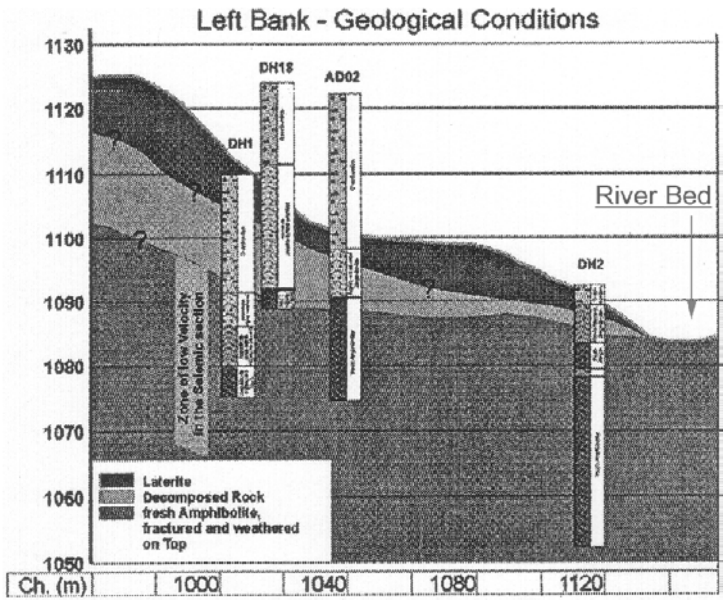
According to the geotechnical report obtained from Bujagali Hydro Power Project, the typical strata profile is described as follows:

Overburden, generally representing lateritic weathering with variable thickness of decimetres up to 25m approximately. Within the lateritic weathering profile, clayey laterite, completely decomposed Amphibolite (up to several meters thick) and highly weathered, fractured Amphibolite are present.

- *At the bottom of lateritic profile, fresh (locally slightly weathered on top) Amphibolite.*
- *Typical lateritic layers on top of the Amphibolite are missing in the river bed.*
- *At Dumbell Island the lateritic profile is very thin (maximum some decimetres).*

Source: Bujagali Hydro Power Project

Figure 5.4.9 speculates the geological condition at the left bank of Bujagali Dam.



Source: Bujagali Hydro Power Project, annotated by JICA Study Team

Figure 5.4.9 Geological Conditions at the Left Bank of Bujagali Dam (currently under construction)

In the same report, massive coarse grained Amphiboite is reported as strong to extremely strong; on the other hand, fresh rock of finer grained Amphibolite is described as generally strong to very strong in a confined state.

Table 5.4.1 is quoted from the same report. It summarises the laboratory test results of rock samples collected from drillings for the construction site of Bujagali Dam

Table 5.4.1 Summary of Uniaxial Compressive and Point Load Test Results in Bujagali Dam

Serial No.	Sample No.	Uniaxial Compression Tests			Point Load Tests			
					Parallel Axis		Perpendicular to Axis	
		ρ [g/cm^3]	σ_{max} [MPa]	$Ev_{,40\text{-}60}$ [MPa]	$I_{s(50)}$ [MPa]	Relation $\sigma_{max}/I_{s(50)}$	$I_{s(50)}$ [MPa]	Relation $\sigma_{max}/I_{s(50)}$
1	A001-4	3,05	256	85401	19,0	13,5		
					21,0	12,2		
2	A002-1	2,93	142,9	63963	20,0	7,1	9,0	15,9
					15,1	9,5		
3	A002-3	3,01	279,1	85026	19,9	14,0	12,7	22,0
					20,8	13,4	14,0	19,9
4	A005-1/1	3,01	208,4	64199	15,8	13,2	11,0	18,9
					15,3	13,6	9,7	21,5
5	A005-1/2	3,06	186,6	64613	15,8	11,8	11,0	17,0
					15,3	12,2	9,7	19,2
6	A005-3	3	210	59713	17,0	12,4		
					14,9	14,1		
7	A007-1/1	3,03	261	82870	18,4	14,2		
					20,2	12,9		
8	A007-1/2	3,05	272	83478	18,4	14,8		
					20,2	13,5		
9	A007-2	3,01	275,6	78944	27,2	10,1		
					19,4	14,2		
10	A001-1		232,4		13,1		17,4	
					20,0			

Source: Bujagali Hydro Power Project

Based on the information cited above, the following characteristics can be assumed as general geological and geotechnical trends on the River Nile in the area:

- In the riverbed, fresh rock with fractured and weathered state on top can be assumed to emerge.
- The strength of the fresh rock of Amphibolite is assumed to be strong to very strong in general. For the massive coarse grained Amphibolite, it can be strong to extremely strong.
- On the islands in the River Nile, it is assumed that the lateritic profile overlying the rock layer is very thin.

5.4.4 Results of Geo Investigation

The following studies, investigations, and tests were planned and executed for this feasibility study:

a) Geological Reconnaissance
b) Drilling Investigation
c) Standard Penetration Test (SPT)
d) Trial Pit Excavation on the Island of the River Nile
e) Laboratory Test (using soil and rock samples recovered through drilling)
f) Alignment Trial Pit Laboratory Investigation (incl. CBR Test)
g) Dynamic Cone Penetration Test (DCP)
h) Investigations for Possible Quarry and Borrow Pit (incl. Lab Test)

Amongst them, most study items excluding “a) Geological Reconnaissance” were subcontracted to COWI Uganda limited after going through the compulsory procurement procedures to select the Subcontractor.

(1) Geological Reconnaissance

Prior to the drilling investigation, a geological reconnaissance of the site was performed. The results of the reconnaissance are shown in Figures 5.4.10 and 5.4.11.

The actual locations of the drilling investigations are also shown in these maps. The results of drilling investigations are described in section 5.4.3 (2).

Preliminary information of geological conditions in and around the proposed site for each route is summarised below:

- Route A

Sedimentary layers are considered to have spread over onto both sides of the River Nile.

The distributions of Black Cotton soil were confirmed on the both sides of the river.

- Route B

Weathered Phyllite and Green Schist were observed at outcrops on a passage of the west bank, whereas an outcrop of lightly weathered Shale was seen on the back of the western abutment of the railway bridge, and an outcrop of Amphibolite was confirmed at the narrow peninsula of the east bank.

The observed strikes of the bedding show approximately NE-SW trends, with southward dips inclining 40-80 degrees.

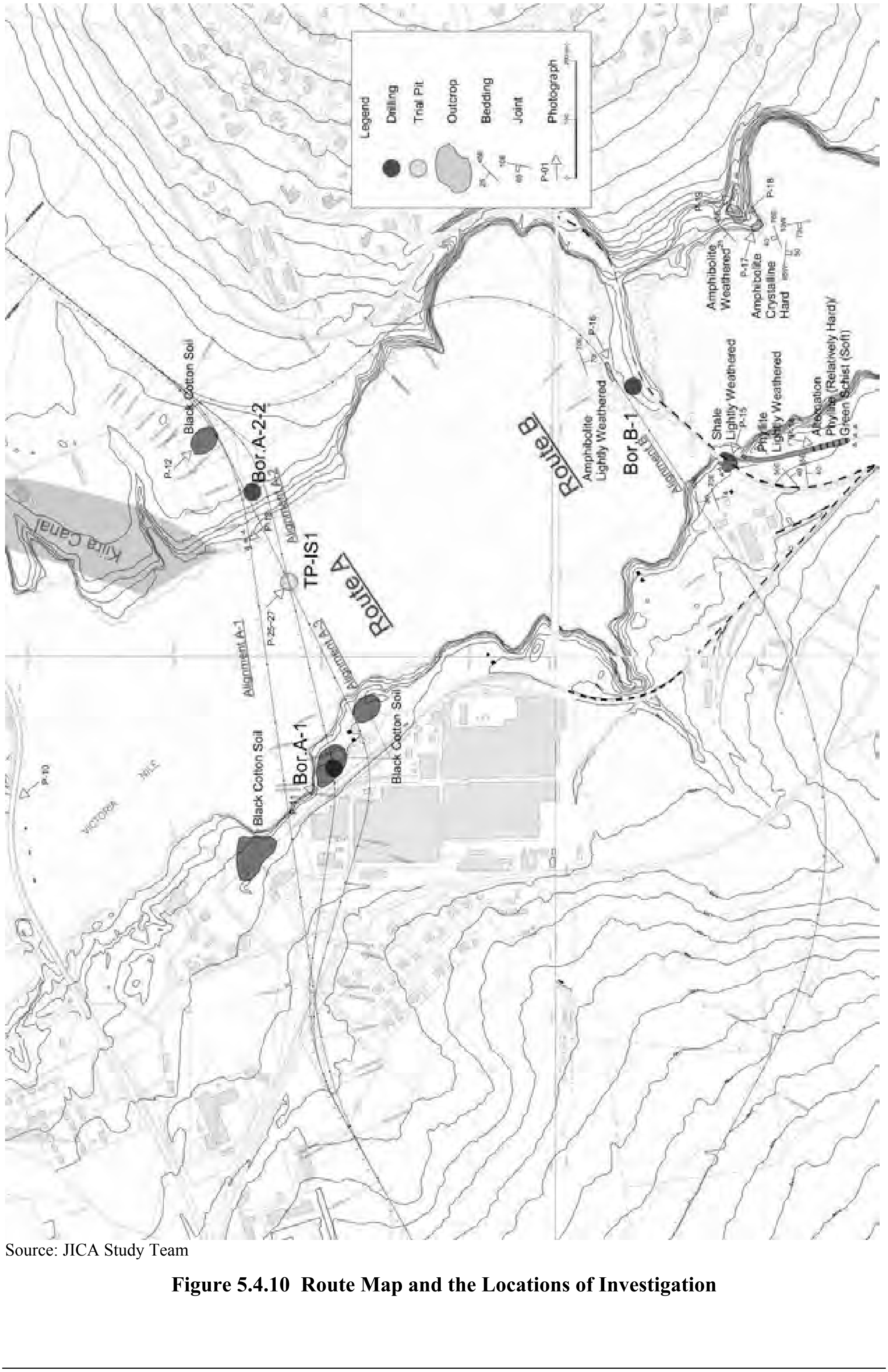
Weathered Schist was relatively soft in comparison with Amphibolite and Phyllite. The alternation of Phyllite and Schist was observed on the upper portion of the passage of the west bank. Interbedded Schist can form fragile parts inside the hard rock.

An outcrop of hard Amphibolite was confirmed at the small tongue of the east bank, 300 m to the south of the railway bridge. The riverbank line of the River Nile has plenty of irregularities with protruding promontory and narrow peninsulas along with concave coves and creeks. This fluctuated shoreline seems to reflect the variations in the hardness of the base rock. Areas of harder materials are considered to have endured fluxing action, then formed islands, promontory, and peninsulas.

- Route C

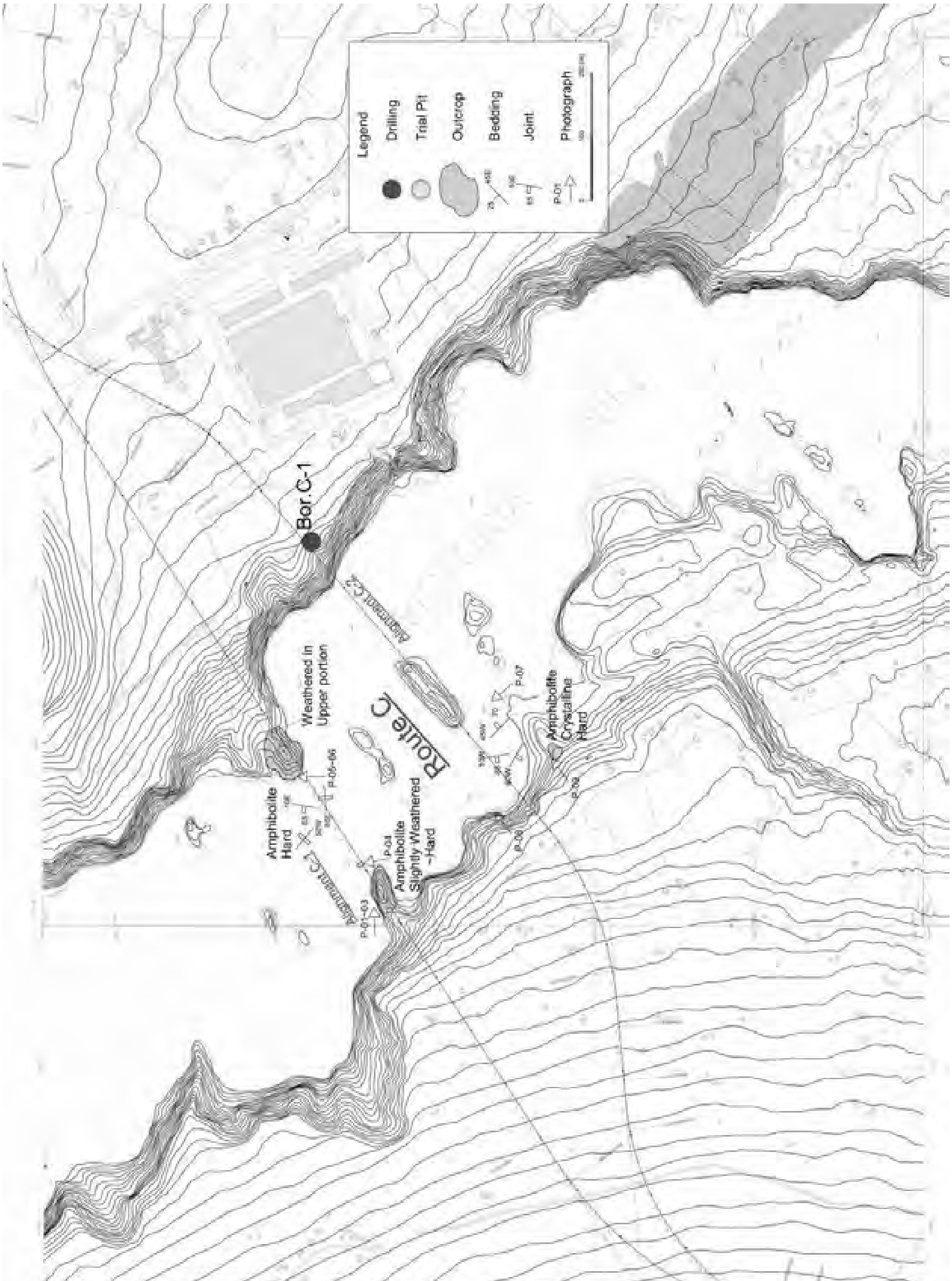
Jointed Amphibolite was observed at outcrops of a peninsula on the west bank and a promontory on the east bank. Another outcrop of Amphibolite was also confirmed on the east bank. These outcrops were located near the level of river water, and each outcrop was relatively hard.

Layers of residual soil and weathered rock are assumed to cover unweathered parts of base rock; the thicknesses of these layers are assumed to be more than 15 m in total.



Source: JICA Study Team

Figure 5.4.10 Route Map and the Locations of Investigation



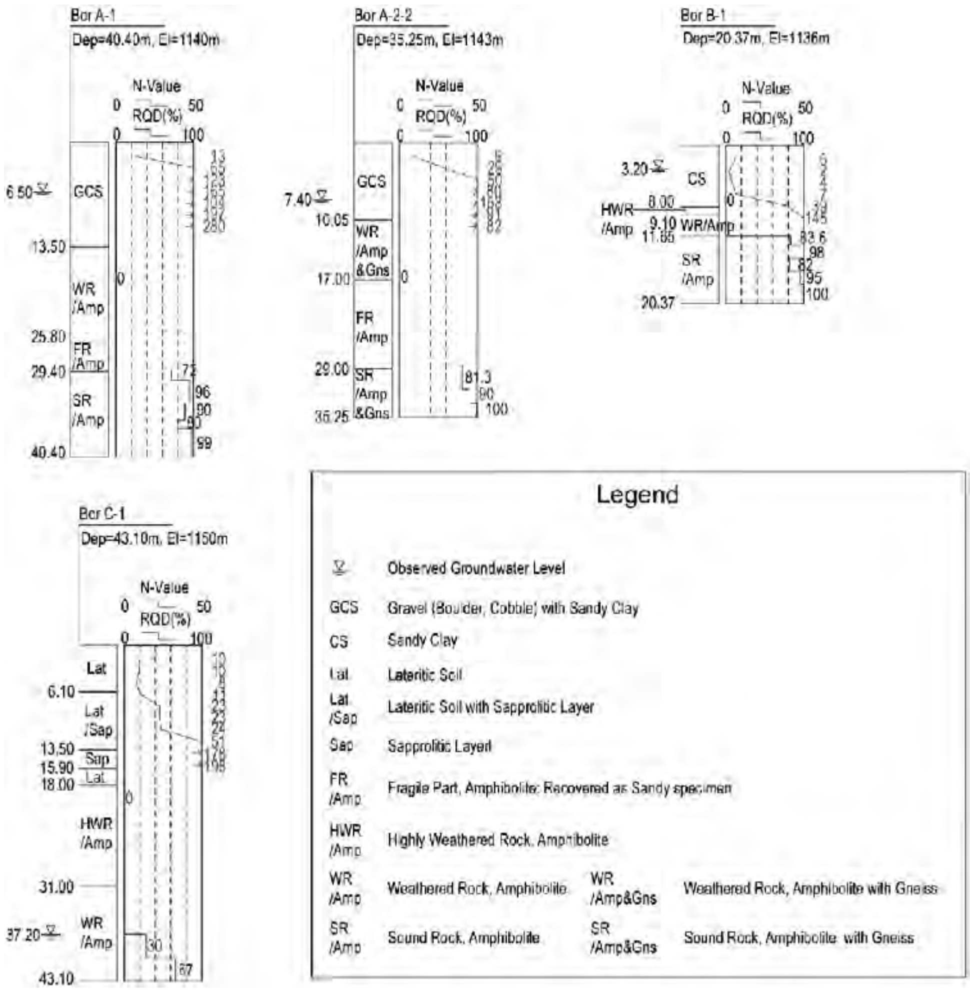
Source: JICA Study Team

Figure 5.4.11 Route Map and the Locations of Investigation

(2) Drilling Investigation and Geological Profile

The locations of drilling investigations are shown in Figures 5.4.10 and 5.4.11. Drilling investigation was executed by 2 different types of drill rig, due to the limited capabilities of drill rigs available in Uganda at the time of the study. A wire-line type drill rig which was incapable of performing a Standard Penetration Test (SPT) was mobilised to obtain core sample, whereas a drill rig which is specialized in performing SPT was mobilised to carry out the investigation. Thus 2 drilling logs for each investigated site were prepared. Drilling logs and photos of core boxes are organized in Appendix 4.

Figure 5.4.12 shows the columnar sections which summarises the results of each drilling with a geological interpretation.



Source: JICA Study Team

Figure 5.4.12 Columnar Sections of Each Drilling Site

Table 5.4.2 organises the geological stratification speculated to underlie each route. For the part of rock mass, engineering classifications were evaluated in accordance with the classification system shown in Figure 5.4.13. This classification system was formulated by the Japanese Geotechnical Society with reference to ISO 14689. Table 5.4.3 shows the representative appearance of core samples obtained from each stratum.

Table 5.4.2 Order of Geological Stratification on Each Route

Route	Type	Symbols	Rock Mass Classification*	Rock (Soil) Type		Description
Route A	Deposit	GCS	-	Gravel (Boulder Cobble) with Sandy Clay		Gravels (Boulders, Cobbles) are irregularly weathered. It might be a part of residual soil, but obtained core samples show erratic appearances that Imply rather accumulated soil than residual soil.
	Base Rock	WR/Amp, WR/Amp&Gns	HM-DV/w3	Weathered	Amphibolite, Amphibolite with Gneiss (Partially)	Weathered part is rich in cracks and affected by weathering actions. Fragile part was recovered as sand. It is thought to be very rich in cracks. Weaker strength of rock pieces can be assumed, presumably due to the effects of alteration. Sound part is less in cracks, the effects of weathering is hardly seen, obtained in the shape of a rod.
		FR/Amp	HM-FVI/w3	Fragile		
		SR/Amp, SR/Amp&Gns	HM-CIII/w1	Sound Rock		
Route B	Former Filling Material	CS	-	Sandy Clay		It is thought to be a remnant of filling materials used to build the embankment which had initially connected to the railway bridge.
	Residual Soil	Lat	-	Lateritic Soil		Not confirmed by the drilling. Supposed to be distributed on the east bank of the River Nile.
	Base Rock	HWR/Amp	HM-FV/w4	Highly Weathered	Amphibolite	Highly weathered part was obtained in the shape of rock pieces heavily affected by weathering. Weathered part is rich in cracks and affected by weathering actions, obtained in the shape of rock pieces to a short rod. Sound part is less in cracks, the effects of weathering is hardly seen, obtained in the shape of a rod.
		WR/Amp	HM-DV/w3	Weathered		
		SR/Amp	HM-CIII/w1	Sound Rock		
Route C	Talus Deposit	Td	-	Sandy Clay with Gravel		Not confirmed by the drilling. Supposed to be distributed on the gentler slopes on both sides of the River Nile.
	Residual Soil	Lat	HM-FVI/w6	Lateritic Soil		Thick layer of residual soil. Lateritic soil is dominant, whereas Sapprolitic layers are interbedded erratically into Lateritic soil.
		Lat/Sap		Lateritic Soil with Sapprolitic Layer		
		Sap		Sapprolitic Layer		
	Base Rock	HWR/Amp	HM-FV/w4	Highly Weathered	Amphibolite	Highly weathered part was obtained in the shape of rock pieces, heavily affected by weathering. Weathered part is rich in cracks and affected by weathering actions, obtained in the shape of rock pieces to a short rod. Sound part is less in cracks, effects of weathering is hardly seen. This part was not confirmed by the drilling.
		WR/Amp	HM-DV/w3	Weathered		
		SR/Amp	HM-CIII/w1	Sound Rock		

*: Refer to Figure 5.4.13

Source: JICA Study Team

First-step Classification	Second-step Classification	Third-step Classification	Sub-step Classification
Hard Rock Mass [H]	Massive {M}, Foliated {F}	Table H	Sub-step table H
Soft Rock Mass [S]	Massive {M}	Table S-M	Sub-step table S-M
	Rudaceous {R}	Table S-R	Sub-step table S-R
	Interbedded {B}	Table S-B	Sub-step table S-B

Hard Rock Mass [H] — Massive {M} / Foliated {F}

Rock Material Strength MPa		Discontinuity Spacing mm (2000, 600, 200, 60, 20) I	II	III	IV	V	VI
	A	A I	A II	A III	A IV	A V	A VI
100	B	B I	B II	B III	B IV	B V	B VI
50	C	C I	C II	C III	C IV	C V	C VI
25	D	D I	D II	D III	D IV	D V	D VI
10	E	E I	E II	E III	E IV	E V	E VI
5	F	F I	F II	F III	F IV	F V	F VI

No.	Weathering degree	Number of discontinuity sets	Discontinuity aperture	Discontinuity roughness	Existence of filling material
1	w_1	n_1	a_1	r_{sr}	f_1
2	w_2	n_2	a_2	r_{sm}	f_2
3	w_3	n_3	a_3	r_{ss}	f_3
4	w_4	n_4	a_4	r_{ur}	
5	w_5	n_5	a_5	r_{um}	
6	w_6		a_6	r_{us}	
7				r_{pr}	
8				r_{pm}	
9				r_{ps}	

Soft Rock Mass [S] — Massive {M}

Rock Material Strength MPa		Dominant Grain Size mm (2, 0.063, 0.002) I	II	III	IV
	D	D I	D II	D III	D IV
10	E	E I	E II	E III	E IV
5	F	F I	F II	F III	F IV
1	G	G I	G II	G III	G IV

No.	Weathering degree	Discontinuity spacing
1	w_1	s_1
2	w_2	s_2
3	w_3	s_3
4	w_4	s_4
5	w_5	s_5
6	w_6	s_6

Soft Rock Mass [S] — Rudaceous {R}

Matrix Strength Mpa		Fragment Content % (50, 20, 10) I	II	III	IV
	D	D I	D II	D III	D IV
10	E	E I	E II	E III	E IV
5	F	F I	F II	F III	F IV
1	G	G I	G II	G III	G IV

No.	Weathering degree	Dominant size of matrix particles	Large fragments content	Dominant fragment size	Fragment strength
1	w_1	p_1	b_1	g_1	h_1
2	w_2	p_2	b_2	g_2	h_2
3	w_3	p_3		g_3	h_3
4	w_4	p_4		g_4	h_4
5	w_5				h_5
6	w_6				h_6
7					h_7

Soft Rock Mass [S] — Interbedded {B}

Difference of layer strength class	Percentage of Weak Layer % (10, 30, 50, 80) I	II	III	IV	V
Same	Example DD I	Example DD II	Example DD III	Example DD IV	Example DD V
Level 1	Example DE I	Example DE II	Example DE III	Example DE IV	Example DE V
Level 2	Example DF I	Example DF II	Example DF III	Example DF IV	Example DF V
Level 3	DG I	DG II	DG III	DG IV	DG V

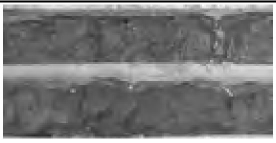
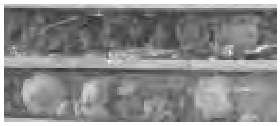
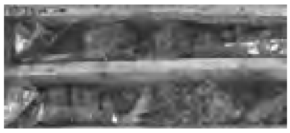
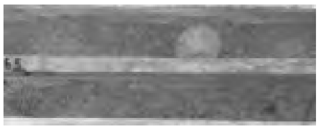
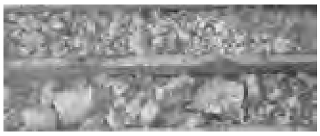
No.	Weathering degree	Average thickness of weak layer
1	w_1	t_1
2	w_2	t_2
3	w_3	t_3
4	w_4	t_4
5	w_5	t_5
6	w_6	

*If necessary, the symbol SM class is designated

Source: Japanese Geotechnical Society

Figure 5.4.13 The System for Engineering Classification of Rock Mass (JGS, 2004)

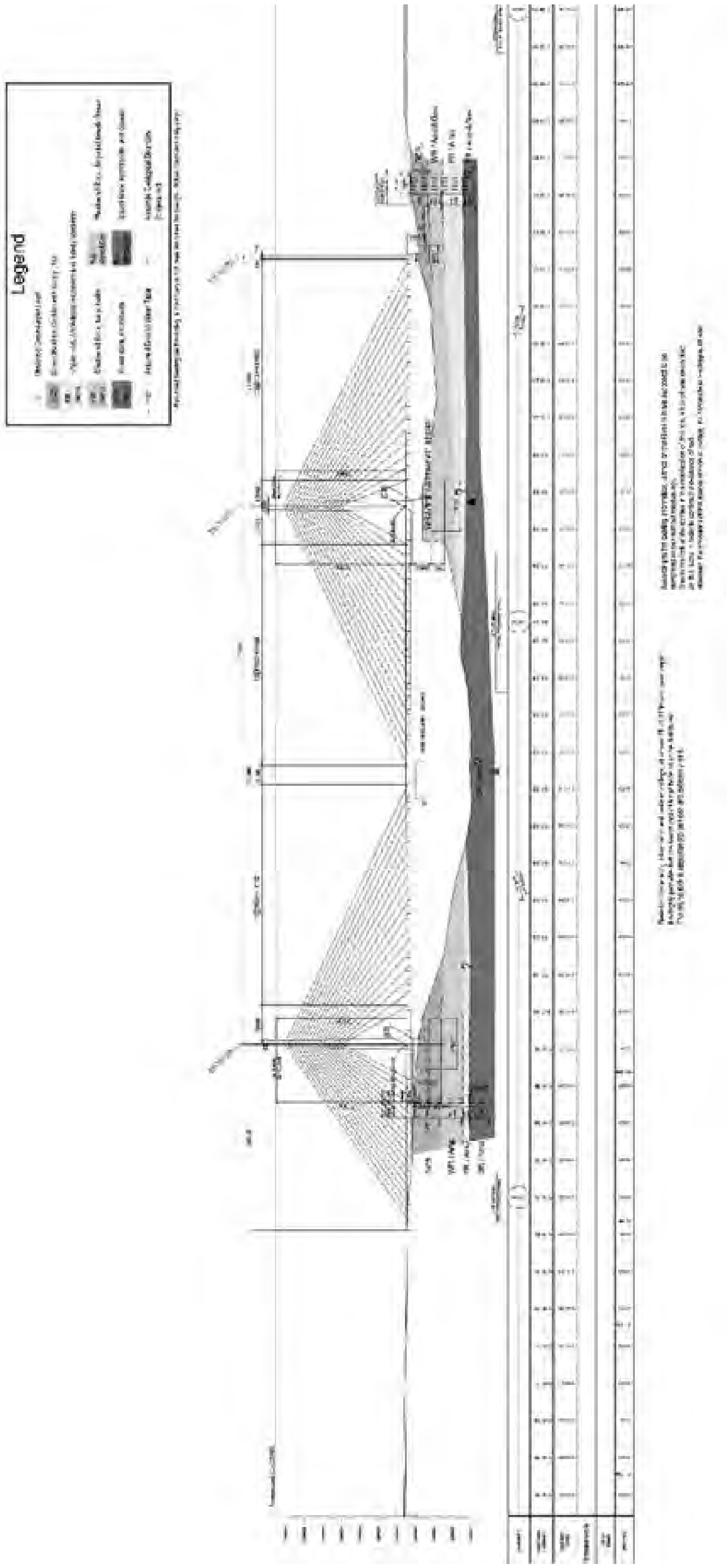
Table 5.4.3 Representative appearance of core samples

Type	Symbols	Rock (Soil) Type		Appearance
Former Filling Material	CS	Sandy Clay		
Deposit	GCS	Gravel (Boulder Cobble) with Sandy Clay		
Residual Soil	Lat	Lateritic Soil		
	Lat/Sap	Lateritic Soil with Sapprolitic Layer		
	Sap	Sapprolitic Layer		
Base Rock	FR/Amp	Fragile	Amphibolite	
	HWR/Amp	Highly Weathered		
	WR/Amp	Weathered		
	SR/Amp	Sound Rock		

Source: JICA Study Team

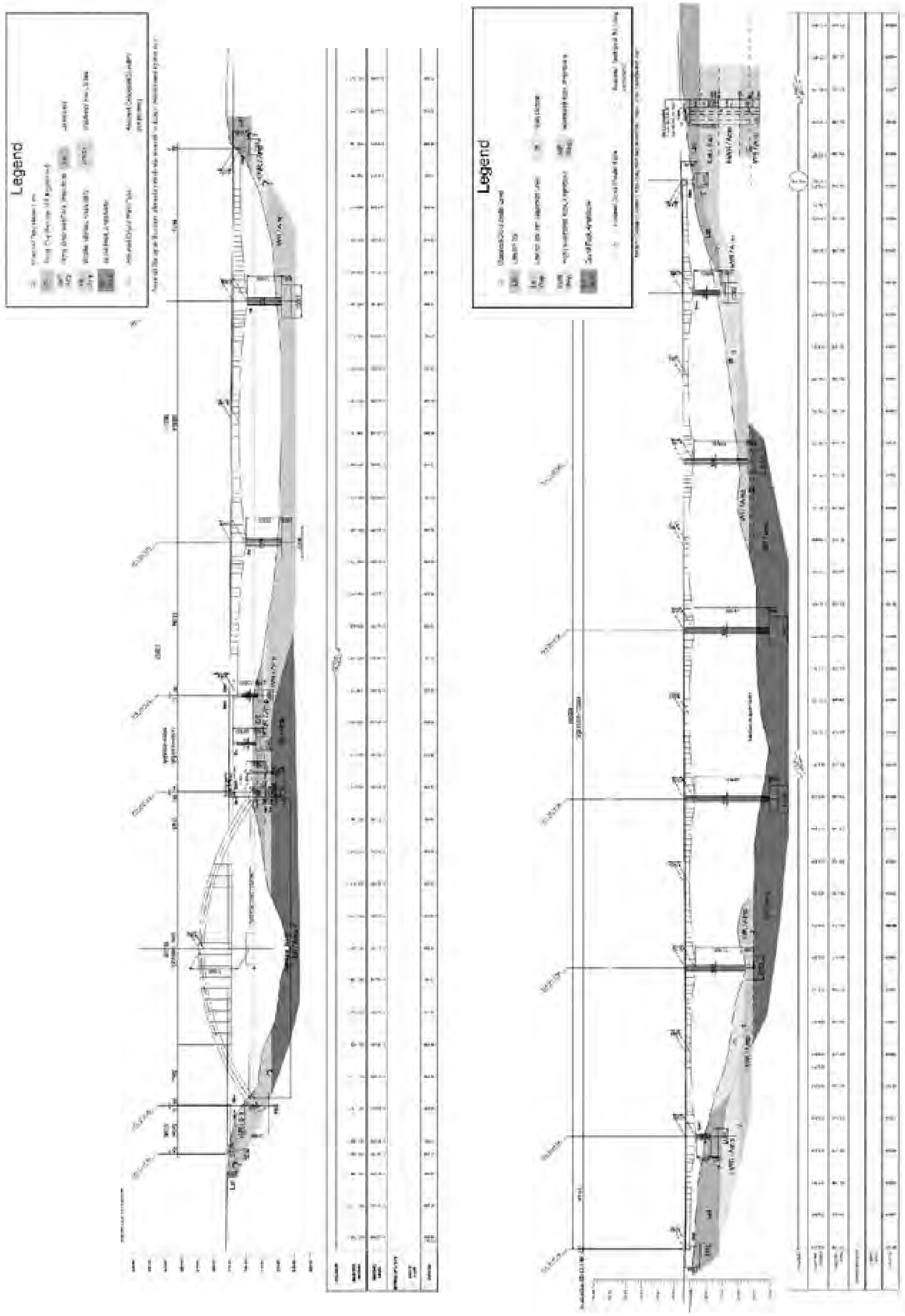
Geological information obtained from drilling investigations, geological reconnaissance and other studies was interpreted into geological profiles assumed for each route.

- Route A: Figure 5.4.14
- Route B & C: Figure 5.4.15



Source: JICA Study Team

Figure 5.4.14 Assumed Geological Profiles (Route A)



Source: JICA Study Team

Figure 5.4.15 Assumed Geological Profiles (Route B & C)

(3) Standard Penetration Test (SPT)

Standard penetration tests were conducted in accordance with British Standard BS 5930. Details of the N-values are recorded in the drilling log attached in Appendix 4 along with soil description and consistency or relative density. Correlation curves between depth and N-values are attached to the columnar sections shown in Figure 5.4.12. Table 5.4.4 summarises the results of the SPT along with soil classification based on laboratory tests.

Table 5.4.4 SPT Values and Unified Soil Classification of Strata

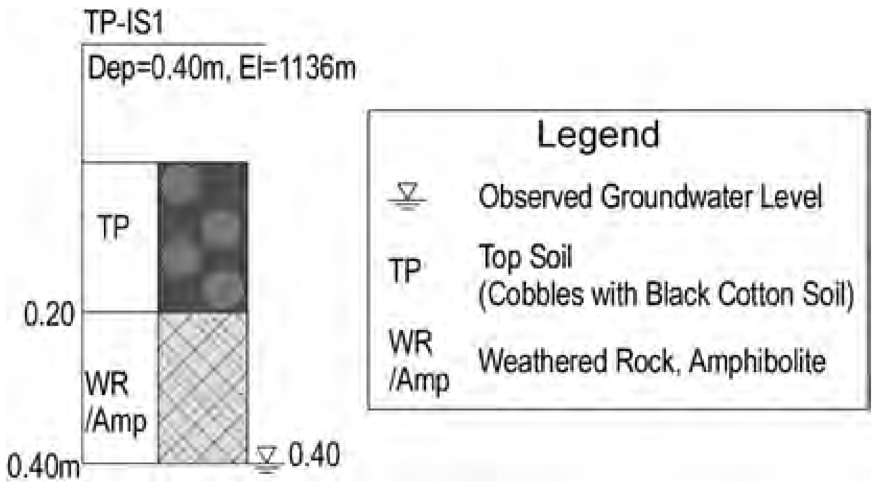
BH	Sampling Level	Depth (m)	Range of SPT blows	Design N-value	Consistency	Material Description
Bor.A-1	I	1.50	5-8	13	Medium dense	Silt (ML)
	II	3.00	9-41	65	Very stiff	Lean clay (CL)
	III	4.50	16-76	123	Very stiff	Lean clay (CL)
	IV	6.00	16-100	165	Very stiff	Lean clay (CL)
	V	7.50	24-54	104	Very dense	Lean clay (CL)
	VI	9.00	16-100	197	Very dense	Clayey sand (SC)
	VII	10.50	34-150	280	Very dense	Silty sand (SM)
Bor.A-2-2	I	1.50	2-5	9	Loose	Silt (ML)
	II	3.00	7-20	29	Medium dense	Silt (ML)
	III	4.50	15-27	50	Very stiff	Lean clay (CL)
	IV	6.00	22-43	80	Very stiff	Sandy fat clay (CH)
	V	7.50	19-100	153	Very stiff	Lean clay (CL)
	VI	9.00	24-56	91	Very stiff	Silty sand (SM)
	VII	10.50	13-64	82	Very stiff	Lean clay (CL)
Bor.B-1	I	1.50	2-4	6	Loose	Elastic silt (MH)
	II	3.00	1	2	Very soft	Sandy fat clay (CH)
	III	4.50	1-3	4	Soft	Sandy fat clay (CH)
	IV	6.00	1-4	7	Firm	Sandy fat clay (CH)
	V	7.50	7-30	39	Very stiff	Sandy fat clay (CH)
	VI	9.00	40-80	145	Very stiff	Sandy fat clay (CH)
Bor.C-1	I	1.50	3-6	10	Stiff	Sandy fat clay (CH)
	II	3.00	2-6	10	Stiff	Sandy fat clay (CH)
	III	4.50	3-5	8	Loose	Elastic silt (MH)
	IV	6.00	4-7	11	Medium dense	Elastic silt (MH)
	V	7.50	8-14	23	Very stiff	Sandy fat clay (CH)
	VI	9.00	8-13	23	Very stiff	Sandy fat clay (CH)
	VII	10.50	9-14	24	Very stiff	Sandy fat clay (CH)
	VIII	12.00	11-32	51	Very stiff	Sandy fat clay (CH)
	IX	13.50	28-130	178	Very stiff	Sandy fat clay (CH)
	X	15.00	24-142	196	Very stiff	Sandy fat clay (CH)

Source: JICA Study Team

(4) Trial Pit on the Island of River Nile

To confirm the geological condition of the island on the River Nile, a trial pit was excavated on the island, designated as TP-IS1. Excavation was done by hand using shovels and picks. The actual location of the trial pit is shown in Figure 5.4.10.

Figure 5.4.16 shows the columnar section observed at TP-IS1.



Source: JICA Study Team

Figure 5.4.16 Columnar Section of TP-IS1

Top soil covered the island surface with thickness of 20cm. The Top soil was composed of semi-round cobble of Amphibolite and Black Cotton soil. Under the top soil, weathered Amphibolite underlay the island. Even though weathered Amphibolite was partially argillised and cracks were filled with fine materials, rock pieces obtained from the excavation site retained their shapes firmly and apparently remained rock structures. In some cases, a single hammer hit could break rock pieces easily, but others resisted firmly.

Groundwater table was confirmed at depth of 40 cm and excavation was terminated due to the absence of a drainage system. The level of the ground water table was apparently the same as that of the River Nile. Around the ground water table, argillation of weathered Amphibolite was notable in some limited places, at the same time the rest of rock pieces maintain their hardness supposedly with medium strength. The cracks between rock pieces were open and had some space to allow water to intrude into the ground of the island. As described above, the weathering condition was varied by location, so the intervals of discontinuities also extended up to 20 cm.

On the western shore of the island, an outcrop of probably Amphibolite was able to be seen below the water surface. It looked hard to some degree from its shape and the density of the cracks observed through murky water.

(5) Laboratory Test

Lab tests intended for soil specimens were done using disturbed samples obtained through SPT. Lab tests intended for rock specimens were carried out using core samples recovered by core drilling. Table 5.4.5 lists the Standard Test Methods applied for the soil tests. The results of soil tests are included in Appendix 4, and the key index properties of the soil materials are presented in Table 5.4.6.

Table 5.4.5 Laboratory tests carried out on SPT samples

Name of Test	Standard Test Method	Sample Status
Moisture content	BS 1377: Part 2: 1990	Disturbed
Particle size distribution	BS 1377: Part 2: 1990	Disturbed
Liquid Limit	BS 1377: Part 2: 1990	Disturbed
Plastic Limit	BS 1377: Part 2: 1990	Disturbed
Plasticity Index	BS 1377: Part 2: 1990	Disturbed

Source: JICA Study Team

Table 5.4.6 Summary of Soils Index Properties

Borehole	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Natural Moisture Content (%)
Bor.A-1	1.5	43	28	15	15
	3.0	43	21	22	7
	4.5	45	18	27	7
	6.0	39	17	22	11
	7.5	36	16	20	16
	9.0	42	14	28	8
	10.5	33	NP	-	11
Bor.A-2-2	1.5	45	31	14	27
	3.0	46	29	17	21
	4.5	42	24	18	13
	6.0	51	23	28	12
	7.5	49	21	28	12
	9.0	39	26	13	17
	10.5	32	17	15	14
Bor.B-1	1.5	66	34	32	25
	3.0	70	27	43	39
	4.5	66	26	40	43
	6.0	79	30	49	39
	7.5	65	25	40	41
	9.0	63	27	36	40
Bor.C-1	1.5	62	30	32	23
	3.0	64	27	37	23
	4.5	61	34	27	24
	6.0	59	30	29	30
	7.5	76	31	45	18
	9.0	80	26	54	13
	10.5	81	23	58	18
	12.0	79	28	51	16
	13.5	75	27	48	15

Laboratory tests aimed at the engineering properties of the rock samples were conducted for the following items:

- Specific gravity, Bulk Density
- Water Absorption
- Unconfined Compressive Strength (UCS)

Table 5.4.7 summarises the results of the rock tests.

Table 5.4.7 Results of Laboratory Tests of Rock Samples

BOREHOLE, BOX No. & DEPTH	SPECIFIC GRAVITY	BULK DENSITY (Mg/m^3)	WATER ABSORPTION (%)	UNCONFINED COMPRESSIVE STRENGTH (MPa)	Geological Interpretation	Remarks
BHA-1 BOX # 1 10.5-13.5	2.85	3.02	0.35	Not Applicable (Cores had cracks)	GCS	The specimen seems to be a boulder included in GCS.
BHA-1 BOX # 1 16.62-18.10	2.76	2.91	0.27	Not Applicable (Cores had cracks)	WR/Amp	
BHA-1 BOX # 2 25.08-29.16	2.94 2.84 2.85	3.05 3.20 3.12	0.20 0.24 0.19	54.2 39.1 48.2	WR/Amp~FR/Amp	Incredible results. Should be omitted.
BHA-1 BOX # 3 36.8-39.80	2.98	3.31	0.21	72.2	SR/Amp	
BHA 2-2 BOX # 2 11.65-12.87	2.78 2.76	3.03 3.02	0.34 0.31	Not Applicable (Cores had cracks)	WR/Amp&Gns	Suspicious results. Caution needed.
BHA 2-2 BOX # 2 16.7-19.65	2.87 2.84	3.01 3.02	0.19 0.20	Not Applicable (Cores had cracks)	WR/Amp&Gns~FR/Amp	Suspicious results. Caution needed.
BHA 2-2 BOX # 2 25.65-28.65	2.77 2.76	3.03 3.02	0.17 0.17	75.3 75.3	FR/Amp	Incredible results. Should be omitted.
BHA 2-2 BOX # 2 28.65-31.65	2.84	3.26	0.18	39.1	(FR/Amp~) SR/Amp&Gns	
BHA 2-2 BOX # 3 33.45-35.25m	2.99 2.81 2.99	3.40 3.23 3.43	0.22 0.24 0.23	51.2 51.2 57.2	SR/Amp&Gns	
BH-B1 BOX # 2 11.65-12.87	3.04 3.05	3.34 3.35	0.19 0.17	57.2 75.3	SR/Amp	
BH-B1 BOX # 3 17.77-20.37	2.98 2.98 3.03	3.26 3.26 3.29	0.29 0.27 0.26	39.1 48.2 72.2	SR/Amp	
BH-C1 BOX # 3 37.10-40.30	2.93	3.00	0.15	Not Applicable (Cores had cracks)	WR/Amp	
BH-C1 BOX # 3 40.3-43.10	2.95	3.21	0.18	24.1	WR/Amp	

Source: JICA Study Team

(6) Alignment Trial Pit Laboratory Investigations (CBR test)

To grasp the basic information for pavement design, 4 trial pits were excavated and the laboratory tests shown in Table 5.4.8 were executed using the samples from the trial pits. The locations of the trial pits are shown in Figure 5.4.17.

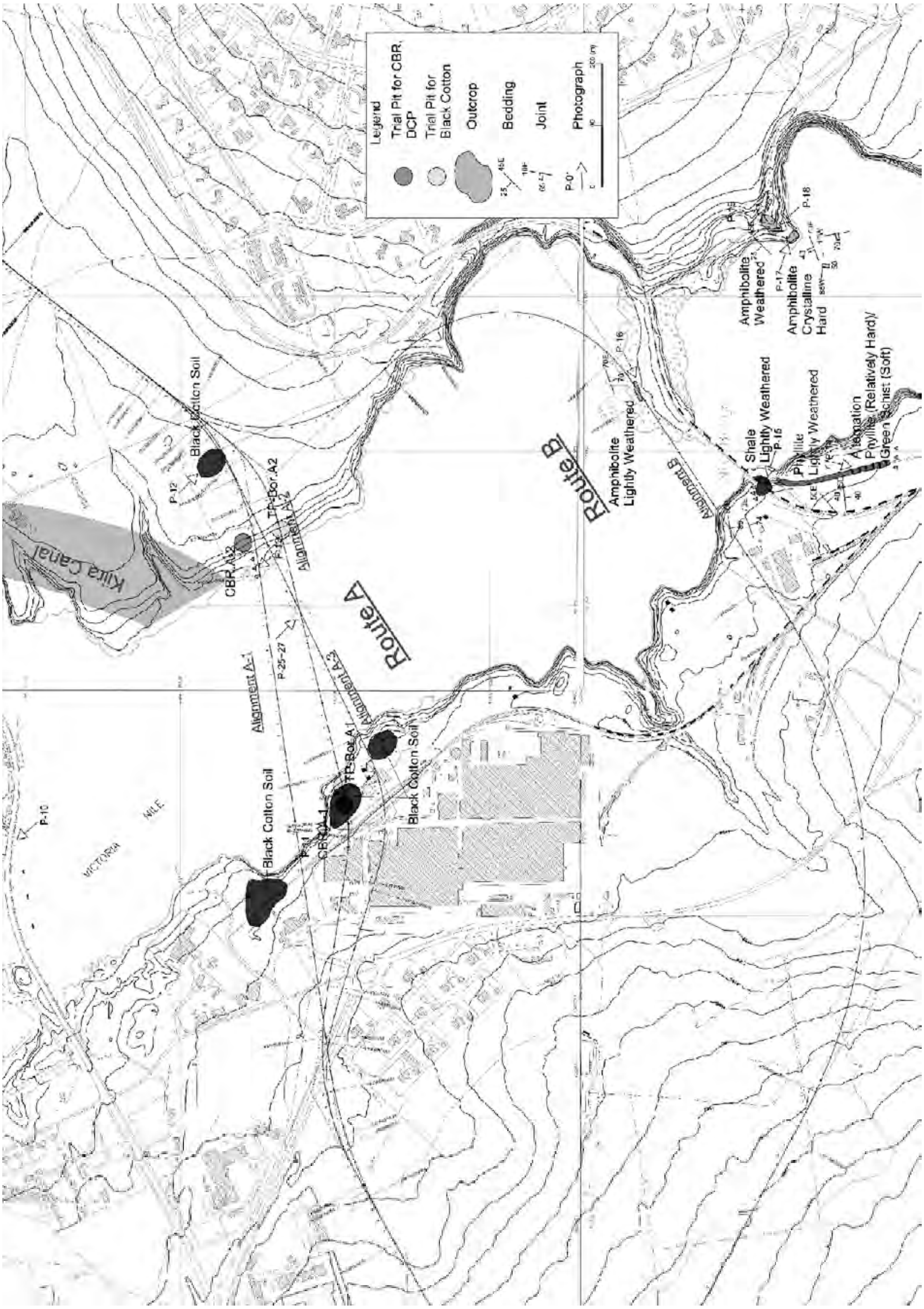
Table 5.4.8 Laboratory Tests on Alignment Test Pit Materials

Test Description	Standard
In-Situ Moisture Content	AASHTO T265
Liquid Limit	AASHTO T89
Plastic Limit & Plasticity Index	AASHTO T90
Specific Gravity	AASHTO T100
Particle Size Distribution (Wet Sieving)	AASHTO T311
5-point compaction (Modified Proctor)	AASHTO T180, Method D
3-point CBR (4 days soaking)	AASHTO T193

Source: JICA Study Team

Initially, 2 trial pits (CBRA-1, CBRA-2, shown in green colour) had been planned and 2 trial pits (TP-Bor.A-1, TP-Bor.A-2, shown in yellow colour) were added after the existence of probable Black Cotton soil had been identified.

The Trial pit excavation records and summary table of lab tests are attached in Appendix 4. (Note: In the appendices, TP-Bor.A-1 and TP-Bor.A-2 are designated as Bor.A-1 and Bor.A-2 respectively.)

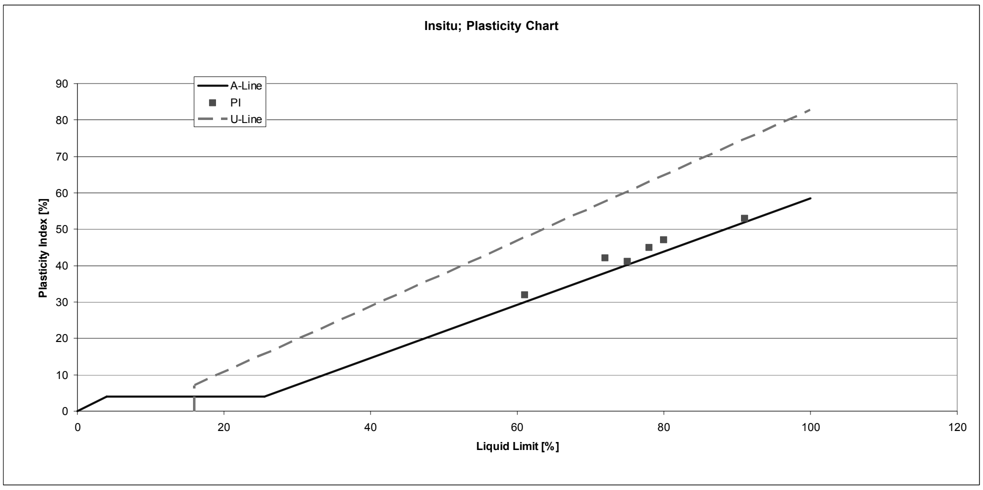


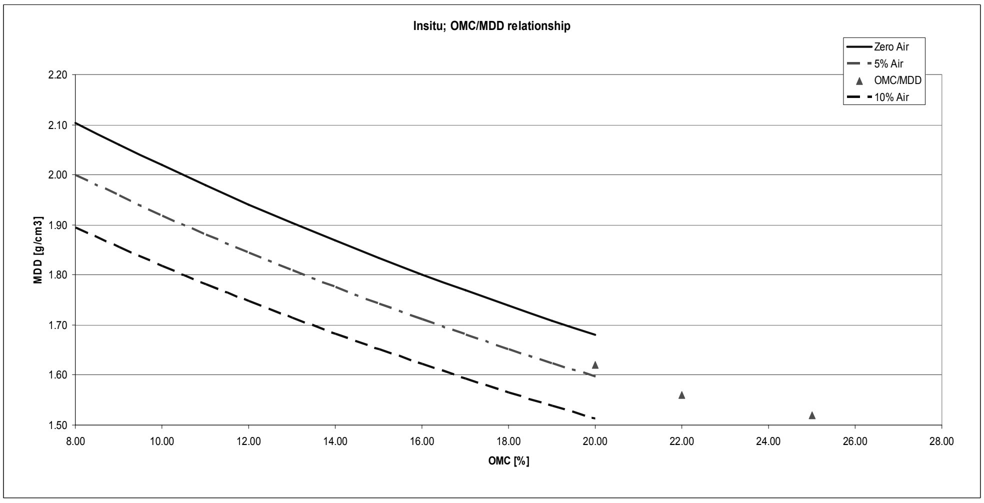
Source: JICA Study Team

Figure 5.4.17 Realised locations of Alignment Trial Pits

The in-situ samples have been classified according to AASHTO. Most of them excluding sample TP-Bor.A2 are classified as A-7-5, whereas the sample TP-Bor.A2 is classified as A-7-6.

Figure 5.4.18 shows the graphs of the plasticity chart of the samples from the trial pits along with MDD/OMC relationships for the in-situ soils along Route A. As seen from the plasticity chart, the file-grained portion of the tested samples is classified as Clay, as they fall above the A-line that divides Clay and Silt.





Source: JICA Study Team

Figure 5.4.18 Plasticity Chart of the Samples from Alignment Trial Pit

(7) Dynamic Cone Penetration Test (DCP)

Dynamic Cone Penetration Tests (DCP) was executed in the same locations with the 2 alignment trial pits shown in Figure 5.4.17.

The equipment used was standard TRL DCP equipment (8 kg hammer, 575 mm free fall, 20 mm diameter 60° cone).

Penetration was recorded between 1 to 5 blows; the higher number of blows required sufficient penetration. The data were analysed using two different models to convert the DCP-data into CBR-values:

- The first model is the Kleyn and Van Heerden DCP-CBR relationship:

$$\mathrm{Log}_{10}(\mathrm{CBR}) = 2.632 - 1.28\,\mathrm{Log}_{10}(\mathrm{mm/blow})$$

- The second equation is provided in TRRL Road Note 8

$$\mathrm{Log}_{10}(\mathrm{CBR}) = 2.48 - 1.057\,\mathrm{Log}_{10}(\mathrm{mm/blow})$$

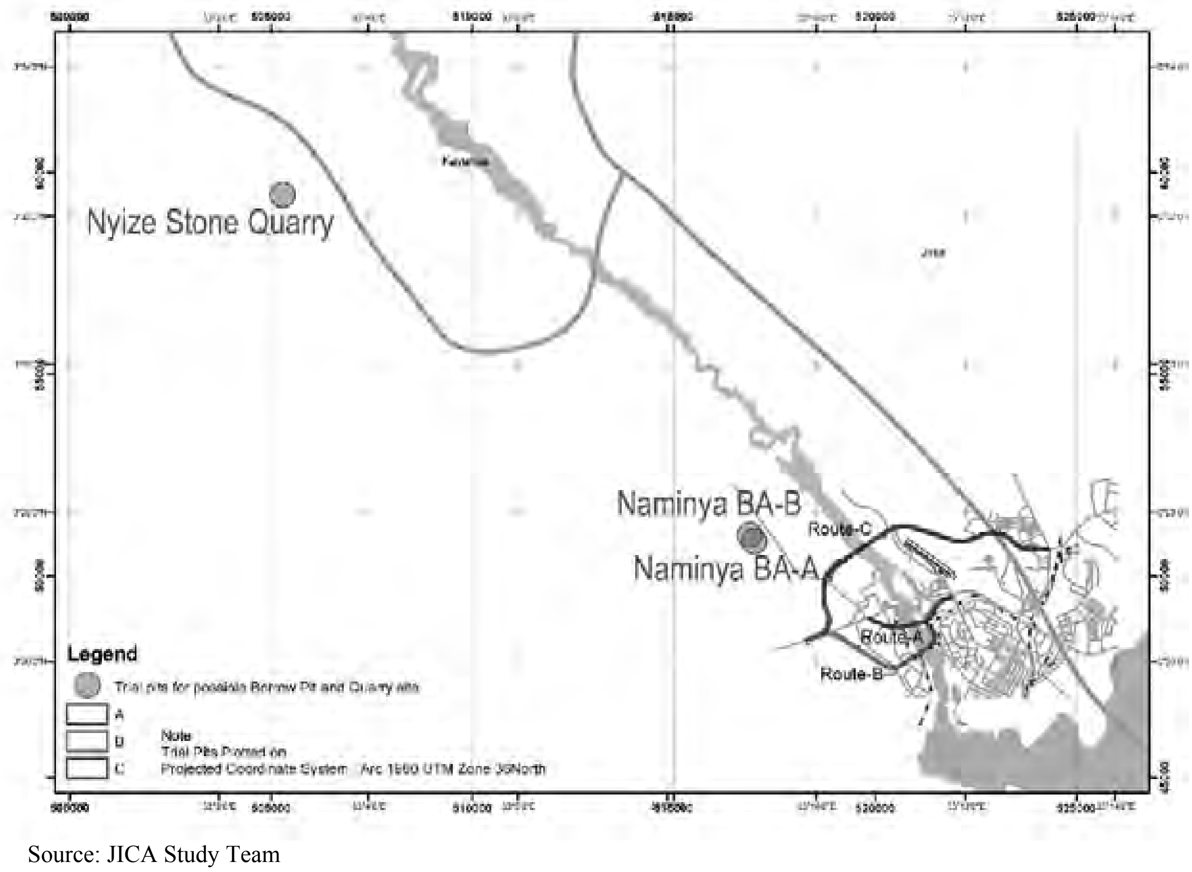
The Kleyn and Van Heerden relationship resulted in larger CBR-values at low penetration per blow values as compared with the TRRL equation (The Kleyn and Van Heerden relationship, resulted to higher CBR-values than the TRRL equation for CBR-values in excess of 58%). At the lower end of the spectre, the TRRL equation resulted to slightly higher CBR-values which were considered acceptable for the current investigation considering that the subgrade material was the former road base course. It was assessed that the Kleyn and Van Heerden relationship resulted to a very high CBR-values at the upper end of the DCP-CBR spectre and therefore the TRRL relationship was chosen for the detailed analysis.

At each location, the layer thickness and corresponding CBR strength can be assessed from the DCP measurements and the actual registered soil layer thicknesses as recorded during the test pit excavations.

As it can be seen, the DCP-CBR strengths vary from 1% to 54%. Summaries of the DCP analysis data are attached in Appendix 4.

(8) Investigations for Possible Quarry and Borrow Pit

Possible sources of construction materials were explored, and as a result, 1 possible quarry site and 2 possible borrow pits were investigated. Figure 5.4.19 shows the approximate locations of the quarry and borrow pits investigated.



Source: JICA Study Team

Figure 5.4.19 Locations of Possible Quarry and Borrow Pits

1) Borrow Pits

The features of the borrow pits that was investigated are summarised in Table 5.4.9.

Table 5.4.9 Details of Investigated Borrow Pits

Name of Borrow Area	Distance from Bridge (km)	Potential exploitable area (m^2)	Thickness of exploitable material (m)	Estimated exploitable quantity (m^3)	Usage of Material	Remarks
Naminya – A	0.7	10,000	5	50,000	G15	Fill or subbase material
Naminya – B	0.6	2,850	3	8,550	G15	Fill or subbase material
Total				58,550		

Source: JICA Study Team

Site Descriptions of the investigated sites is shown in Appendix 4.

Lab tests shown in Table 5.4.10 were carried out on the samples extracted from the investigated borrow areas.

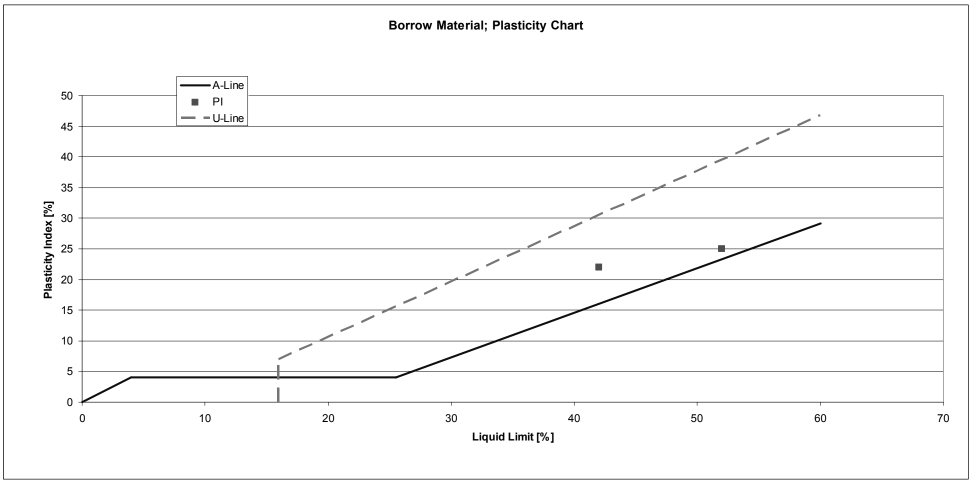
Table 5.4.10 Laboratory Test Programme for Construction Materials from Bas

Test Description	Standard Test Designation
Borrow Materials:	
Liquid Limit	AASHTO T89
Plastic Limit & Plasticity Index	AASHTO T90
Particle Size Distribution – Wet sieving	AASHTO T311
Compaction, 5-point Modified Proctor [1]	AASHTO T180
CBR (3 point 4 days soaked) [1]	AASHTO T193
Specific Gravity	AASHTO T100

Source: JICA Study Team

The summaries of the laboratory tests are attached in Appendix 4.

Overview of the plasticity results and need for stabilising the borrow area samples when used for earthworks and pavement layers are shown graphically in Figure 5.4.20.



Source: JICA Study Team

Figure 5.4.20 Plasticity Chart and Stabilisation Need for Borrow Area Materials

As can be seen from the above graphs, the fine-grained portion of the identified borrow area material is mainly classified as clay and can be stabilised using either lime or cement.

2) Stone Quarry

One possible rock quarry as source of crushed aggregate, asphalt concrete and concrete aggregates was identified and material samples from already crushed aggregates of sizes ranging from 10 to 20 mm was taken for laboratory test. The estimated volume available is more than 25,000 m3.

The lab tests shown in Table 5.4.11 were carried out on the samples taken from the quarry source.

Table 5.4.11 Laboratory Test Programme for Construction Materials

Test Description	Standard Test Designation
Quarry Materials:	
Aggregate Crushing Value (ACV)	
Los Angeles Abrasion (LAAV)	AASHTO T96
Asphalt Stripping	
Aggregate Stripping Resistance	

Source: JICA Study Team

Laboratory test results of the samples taken from the quarry are shown in Appendix 4.

5.4.5 Geotechnical Consideration

(1) Assumed Design Parameters

Available information consists of the following:

a) Field observations:

- geological surface survey data
- borehole logs examination

- core boxes examination
- standard (SPT) penetration test results (NSPT).

b) Laboratory investigation results:

- index characteristics (grain-size distribution, Atterberg limits)
- compression resistance from uniaxial compression tests.

Table 5.4.12 organises the assumed design parameters. Due to limited availability of data and dispersion in the results, the design parameters should be considered as indicative values for the assessment of a range of acceptable limits.

Table 5.4.12 List of Assumed Design Parameters

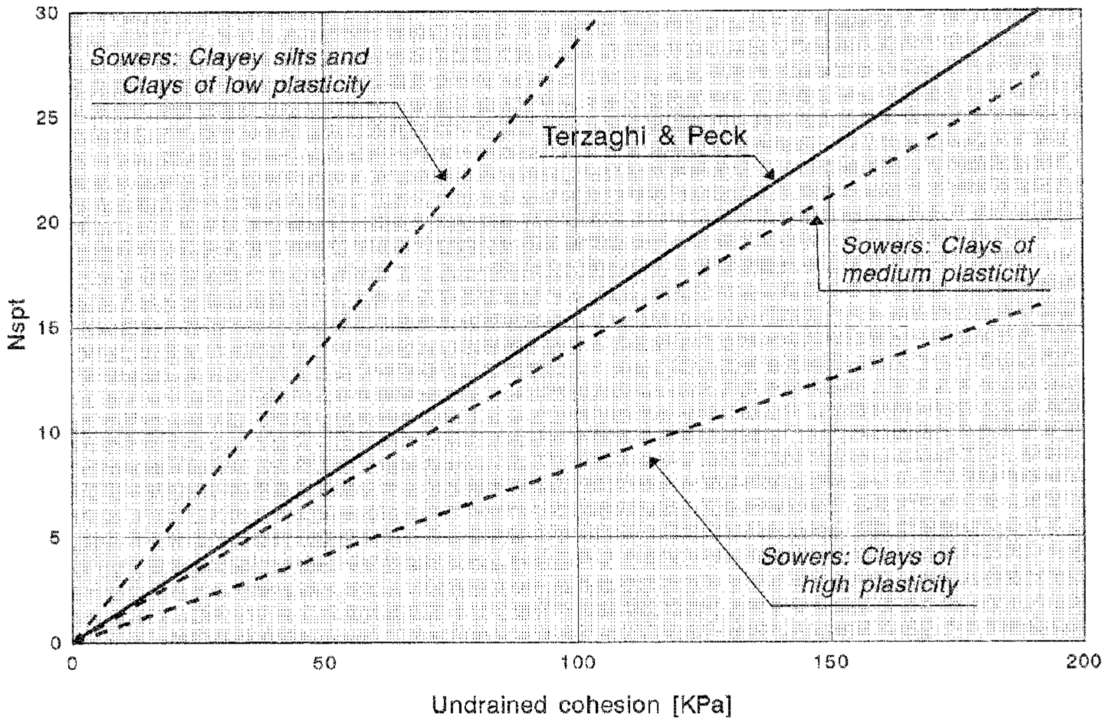
<table>
<tr><th>Symbols for Strata</th><th colspan="2">Rock (Soil) Type</th><th>γ
Mg/m³</th><th>C
kN/m²</th><th>φ
°</th><th>Remarks</th></tr>
<tr><td>CS</td><td colspan="2">Sandy Clay</td><td>1.8</td><td>50</td><td>-</td><td>Average values from NSPT</td></tr>
<tr><td rowspan="2">GCS</td><td colspan="2" rowspan="2">Gravel (Boulder Cobble) with Sandy Clay</td><td rowspan="2">2.1</td><td>-</td><td>35</td><td>Assuming Inter Rocking Gravelly Layer</td></tr>
<tr><td>120</td><td>-</td><td>Assuming Matrix Supported Gravelly Layer, Average values from NSPT</td></tr>
<tr><td>Lat</td><td colspan="2">Lateritic Soil</td><td>2.0</td><td>110</td><td>-</td><td>Average values from NSPT</td></tr>
<tr><td>Lat/Sap</td><td colspan="2">Lateritic Soil with Sapprolitic Layer</td><td>2.0</td><td>200</td><td>-</td><td>Average values from NSPT</td></tr>
<tr><td>FR/Amp</td><td>Fragile</td><td rowspan="4">Amphibolite</td><td>2.6</td><td>50</td><td>8</td><td>RMR10 (BIENIAWSKI)
qu<5Mpa (Supposed)</td></tr>
<tr><td>HWR/Amp</td><td>Highly Weathered</td><td>2.6</td><td>50</td><td>8</td><td>RMR10 (BIENIAWSKI)
qu<5Mpa (Supposed)</td></tr>
<tr><td>WR/Amp</td><td>Weathered</td><td>3.0</td><td>195</td><td>22</td><td>RMR39 (BIENIAWSKI),
qu=24Mpa (Average Valuses from Lab Test)</td></tr>
<tr><td>SR/Amp</td><td>Sound Rock</td><td>3.0</td><td>265</td><td>29</td><td>RMR53 (BIENIAWSKI)
qu=50Mpa (Average Valuses from Lab Test)</td></tr>
</table>

Source: JICA Study Team

The GCS stratum was not clear whether it was gravelly layer, but clayey constituent seems to be rich, and should thus be considered as “Inter Rocking Gravel” or “Matrix Supported Gravel”. Behaviour of a gravelly layer depends on its internal structures. The mechanical features of a matrix supported gravel, in which gravels are supported by the clayey or sandy matrices, can be determined by the characteristics of the matrices, whereas for an inter rocking gravel, the structure is supported by contacts of gravels, which can be determined by the internal friction angles of the enclosed gravels.

Field observation could not provide the decisive factor to determine the internal structure of GCS, since the appearance of the outcrop varies by location, but on the other hand the appearance in core samples, boulders and cobbles can be observed alternately with matrices.

For cohesive soils, undrained cohesion for each stratum was estimated from the average values of N_{SPT} measured at the stratum. Figure 5.4.21 shows the commonly accepted correlation between undrained cohesion and N_{SPT}.



Source: NAVFAC 7.01 (modified)

Figure 5.4.21 Determination of Cohesion with NSPT

For rock strata, the RMR method was applied to assess cohesion and angle of shear resistance. The table for RMR method is shown in Table 5.4.13. The rating for SR/Amp is RMR=53 and that for WR/Amp is RMR=39, whereas that for HWR/Amp is assumed at RMR=10. The same rating is applied to FR/Amp.

From RMR ratings, cohesion and angle of shear resistance were calculated based on the following formula:

$$C = 5 \times RMR$$
$$\phi = (5 + RMR)/2$$

where

C = cohesion (kN/m^2)

ϕ = angle of shear resistance (°)

RMR = RMR rating

Table 5.4.13 Rock Mass Rating System

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence)	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m
Rating	6	4	2	1	0
Separation (aperture)	None	< 0.1 mm	0.1 - 1.0 mm	1 - 5 mm	> 5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Ratings	6	5	3	1	0

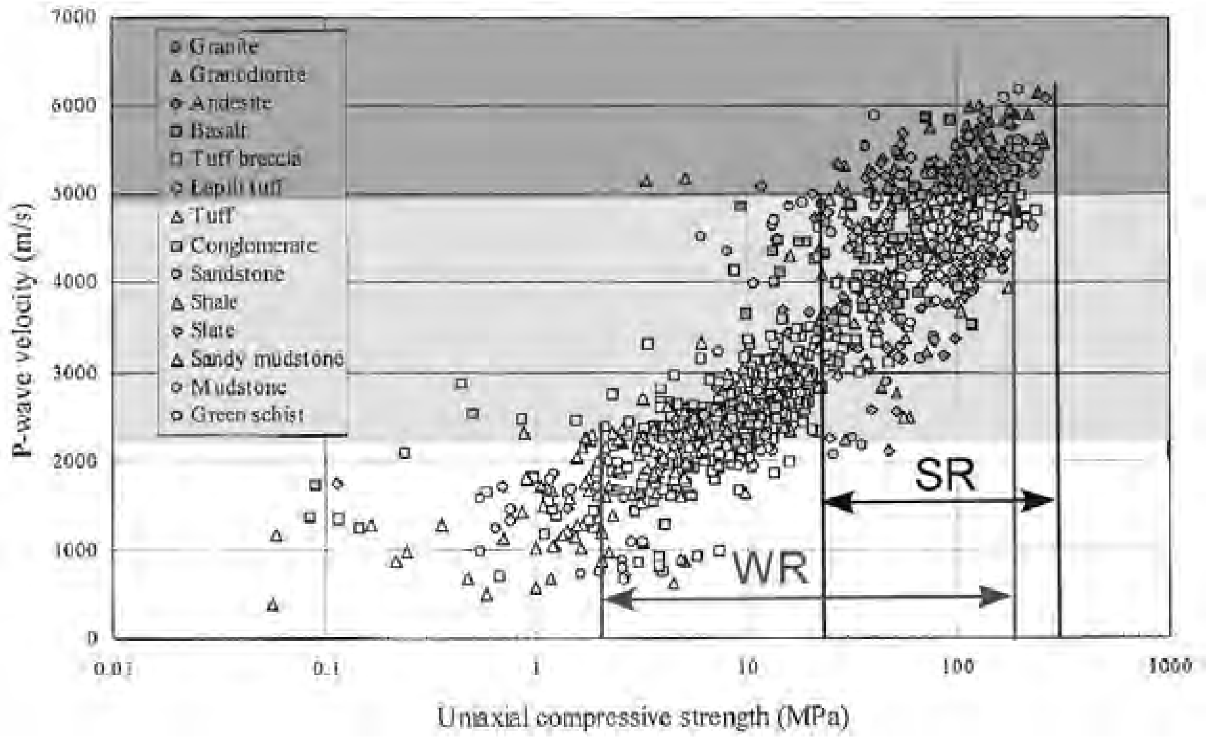
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.
** Modified after Wickham et al (1972).

Source: BIENIAWSK 1989

Figure 5.4.22 shows the commonly accepted correlation between uniaxial compressive strength and P-wave velocity. According to Figure 5.4.8, P-wave velocity assumed for WR/Amp is more than 2.2 km/sec, whereas the SR/Amp is more than 4.9 km/sec. The assumed ranges for uniaxial compressive strength (qu) are also shown in Figure 5.4.22. The lab test results for WR/Amp (qu=24 MPa) falls in the middle of the assumed range as shown in Figure 5.4.22. Moreover, the average lab test result for SR/Amp (qu=50 Mpa) fall slightly lower than the middle of the range. Amphiboite is a sort of hard rock in general, thus in relation with P-wave velocity, a somewhat higher uniaxial compressive strength than tested in the lab might well be expected.



Source: Japanese Geotechnical Society (arranged)

Figure 5.4.22 Correlation between Uniaxial Compressive Strength and P-wave Velocity

(2) Soil bearing capacity

Soil bearing capacity was estimated by using the N-values. The details of the estimation are attached in Appendix 4. The maximum pressures the soils are capable of resisting have been estimated from the N-values obtained from field measurement during the SPT tests based on empirical equation relations. For purposes of computing the soil bearing capacity using the field N-values, a 1.0 m square footing was assumed. Further assumptions included the following:

- The Peck (et al) relationship between N-values and unconfined compressive strength is valid for clayey soils
- The maximum allowable settlement in non-cohesive soils if any is 25mm
- Failure mechanism is by local shear and the factor of safety against failure is 3.0
- The design N-values are derived from statistical average of all values within a depth zone equal to the footing width below the founding depth.

For evaluation of the Unconfined Compressive Strength qu for cohesive soils, the relationship

- qu = 13.1 x Design N-value was used,

the cohesion was therefore calculated as follows:

- Cu = qu/2; and
- qall = 5.14 x Cu.

qall was evaluated using a factor of safety of 3

Based on Terzaghi and Peck published in 1967, the Allowable Bearing Capacity with limited settlement to approximately 25mm for cohesionless soils is determined directly from the Chart.

The evaluations are summarised in Table 5.4.14.

Table 5.4.14 Estimated Allowable Bearing Capacities of Sub Soils Based on SPT N-Values

BH. No	Depth (m)	N-value	Allowable Bearing Capacity (kPa) From SPT - N Values
Bor.A-1	1.50	13	260
	3.00	65	>700
	4.50	123	>700
	6.00	165	>700
	7.50	104	>700
	9.00	197	>700
	10.50	280	>700
Bor.A-2-2	1.50	9	180
	3.00	29	580
	4.50	50	561
	6.00	80	>700
	7.50	153	>700
	9.00	91	>700
	10.50	82	>700
Bor.B-1	1.50	6	120
	3.00	2	22
	4.50	4	45
	6.00	7	79
	7.50	39	438
	9.00	145	>700
Bor.C-1	1.50	10	112
	3.00	10	112
	4.50	8	160
	6.00	11	220
	7.50	23	258
	9.00	23	258
	10.50	24	269
	12.00	51	572
	13.50	178	>700
	15.00	196	>700

Source: JICA Study Team

(3) Bearing Layer for Foundations

Based on the assumed geological profiles shown in Figures 5.4.14 and 5.4.15, the probable bearing layers for each route are organised in Table 5.4.15.

Table 5.4.15 Probable Bearing Layers for Each Route

Route	Probable Bearing Layer	Problematic Points	Necessary Surveys and Points of Concerns for Detailed Dasign
A	WR	Fragile Part (FR) is assumed to exist below Weatherd Rock (WR) on both sides of the river.	On east bank, the thickness of FR is speculated to be about 10m,whereas that of west bank is speculated to be about 4m. Drilling investigations are needed at the exact points of abutments and piers to grasp the characteristics of FR.
		Conditions under the island are unclear.	It must be noted that the mobilisation of a drill rig to the island requires extra cost and time. The most recomendable method of mobilization can be a construction of a makeshift jetty, that can allow a drill rig to be installed steadily in order to make it possible to drill the island.
B	WR	Conditions of overall alignment are unclear.	Assumed geological profile is based on only 1 drilling log and preliminary speculations from geological reconnaissance and existing information. Drilling investigations are needed at the exact points of abutments and piers to grasp the geotechnical conditions of the bearing layer.
		Instabilities of the slopes on east bank.	Planned abutment is located on the eroding front of the slope where Lateritic soil is speculated to be distributed. Taking the brittleness of Lateritic soil against erosion into consideration, instabilities of the slope could jeopardise the abutment.
C	WR (HWR)	Conditions of overall alignment are unclear.	Assumed geological profile is based on only 1 drilling log and preliminary speculations from geological reconnaissance and existing information. Drilling investigations are needed at the exact points of abutments and piers to grasp the geotechnical conditions of the bearing layer.
		Depth to the bearing layers	Recommended bearing layer is Weathered Rock (WR), whereas Highly Weathered Rock (HWR) could be a bearing layer in certain conditions. However the depth to them can be a problem. WR is speculated to emerge slightly above of the water level and HWR overlies WR. The depth from the surface to HWR is assumed to be nealy 20m at some planned locations of abutment and piers; At the same time that of WR is speculated to be about 30m.

Source: JICA Study Team