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THE STUDY
ON
[illegible] BASIN HYDROELECTRIC
POWER POTENTIAL INVENTORY
PROJECT

VOLUME V

SUPPORTING REPORT

PREFEASIBILITY STUDY
ON
[illegible] AND
[illegible] HYDROELECTRIC SCHEMES

OCTOBER 1991

JAPAN INTERNATIONAL COOPERATION AGENCY
TOKYO, JAPAN

FEDERATIVE REPUBLIC OF BRAZIL

THE STUDY
ON
ITAJAI RIVER BASIN HYDROELECTRIC POWER POTENTIAL INVENTORY PROJECT

VOLUME V

SUPPORTING REPORT

PRE-FEASIBILITY STUDY
ON
SALTO PILÃO (1), DALBERGIA AND BENEDITO NOVO HYDROPOWER SCHEMES

OCTOBER 1991

JAPAN INTERNATIONAL COOPERATION AGENCY
TOKYO, JAPAN

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ABBREVIATIONS

(1) Organizations and Agencies

JICA	:	Japan International Cooperation Agency
ACARESC	:	Associação de Crédito e Assistência Rural de Santa Catarina
CASAN	:	Companhia Catarinense de Águas e Saneamento
CEDEC	:	Coordenação Estadual de Defesa Civil
CELESC	:	Centrais Elétricas de Santa Catarina S.A.
CEPA	:	Instituto de Planejamento e Economia Agrícola de Santa Catarina
CIDASC	:	Companhia Integrada de Desenvolvimento Agrícola de Santa Catarina
DNAEE	:	Departamento Nacional de Águas e Energia Elétrica
DNER	:	Departamento Nacional de Estradas de Rodagem
DER	:	Departamento de Estradas de Rodagem
DNOS	:	Departamento Nacional de Obras de Saneamento
ELETROBRAS	:	Centrais Elétricas Brasileiras S.A.
ELETROSUL	:	Centrais Elétricas do Sul do Brasil S.A.
EMATER	:	Empresa de Assistência Técnica e Extenção Rural
EMBRAPA	:	Empresa Brasileira de Pesquisa Agropecuária
EMPASC	:	Empresa de Pesquisa Agropecuária de Santa Catarina
FATMA	:	Fundação de Amparo à Tecnologia e Meio Ambiente
FGV	:	Fundação Getúlio Vargas
GAPLAN	:	Gabinete de Planejamento e Coordenação Geral
GCPS	:	Grupo Coordenador do Planejamento dos Sistemas Elétricos
IBDF	:	Instituto Brasileiro de Desenvolvimento Florestal
IBGE	:	Instituto Brasileiro de Geografia e Estatística
IBRD	:	International Bank for Reconstruction and Development
ITAG	:	Instituto Técnico de Administração e Gerência
MA	:	Ministério da Agricultura
MDUMA	:	Ministério do Desenvolvimento Urbano e Meio Ambiente
PORTOBRAS	:	Empresa Brasileira de Portos
SAMAE	:	Serviço Autônomo Municipal de Água e Esgoto
SUDEPE	:	Superintendência do Desenvolvimento da Pesca
ITAIPU BINATIONAL	:	Entity for hydropower development of Rio Parana, which was established based on the treaty between Brazil and Paraguay

(2) **Abbreviations of Measurement**

Length

mm : millimeter
cm : centimeter
m : meter
km : kilometer

Area

cm^2 : square centimeter
m^2 : square meter
ha : hectare
km^2 : square kilometer

Volume

cm^3 : cubic centimeter
l : liter
m^3 : cubic meter
MCM : million cubic meter

Weight

g : gram
kg : kilogram
ton : metric ton

Electricity

Hz : Hertz
kV : Kilovolt
MVA : Megavolt Ampere
kVA : Kilovolt Ampere
MW : Megawatt
kW : Kilowatt
GWh : Gigawatt hour
MWh : Megawatt hour
kWh : Kilowatt hour
V : Volt
W : Watt

Time

s or sec : second
min : minute
h or hr : hour
d : day
y or yr : year

Others

% : percent
℃ : degree centigrade
10^3 : thousand
10^6 : million
10^9 : billion

Derived Measure

m^3/s : cubic meter per second

Money

Cr$: Cruzeiro
US$: US dollar
¥ : Japanese Yen

(3) **Exchange Rate**

Official rate as of end of May 1991 : US$1 = Cr$285.5 = ¥ 140

(4) **Others**

Socio-economic Technical Terms

GDP	:	Gross Domestic Product
GRDP	:	Gross Regional Domestic Product
GVA	:	Gross Value Added
VA	:	Value Added
PV	:	Production Value

ANNEX I

TOPOGRAPHIC SURVEY

ANNEX I. TOPOGRAPHIC SURVEY

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1. INTRODUCTION

The photogrammetric mapping works to prepare the topographic maps at a scale of 1:10,000 with contour interval of 5 m for the proposed 3 schemes were carried out by local contractor under the supervision of JICA survey expert, in order to provide the basic data necessary for pre-feasibility study. These survey works comprise aerial photography, ground control point survey, field classification, aerial triangulation, restitution, and scribing. These works were performed during the period from October in 1990 to February in 1991. Location of the survey areas is shown in Fig.I.1.1.

2. PHOTOGRAMMETRIC MAPPING

2.1 Aerial Photography

The available aerial photographs in the project areas were shot in 1978. Since then, topographic conditions in the project areas have been changed due to construction of houses, roads and bridges and so on. To obtain updated topographic information and photogrammetric maps, new aerial photographs at a scale of 1:25,000 were taken in this time under supervision of CELESC. Flight course and photo number are illustrated in Figs I.2.1 to I.2.3.

2.2 Ground Control Point Survey

The ground control point survey for the aerial triangulation was carried out by traverse survey and leveling.

The datum of horizontal and vertical control are;

(i)	Vertical	:	Imbituba, Santa Catarina,
(ii)	Horizontal	:	SAD-69 (South American Data in 1969), and
(iii)	Project	:	Universal Transverse Mercator (UTM)

The results of ground control point survey are listed in Table I.2.1, and the location of these ground control points is shown in Figs I.2.1 to I.2.3.

2.3 Field Classification

To obtain the topographic information necessary for the mapping works, the field classification was performed for the project areas. The data obtained through this field classification were edited on the enlarged aerial photographs.

2.4 Aerial Triangulation

The aerial triangulation based on ground control point survey was executed by means of analytical method, PAT-M-43. The following equipment were used for the aerial triangulation :

Point transfer device	:	PVG-4 (Swiss)
Observation	:	Autograph A-10 (Swiss)
Computer	:	VAX 11/730 (American)

2.5 Restitution

The topographic mapping works at a scale of 1: 10,000 and contour interval of 5 m were carried out based on the aerial triangulation. The topographic features were plotted and edited on the polyester bases using Autograph A-10(Swiss) and Stereo plotter A-8 (Swiss).

2.6 Scribing

The topographic map manuscripts at a scale of 1:10,000 were produced by the scribing. The scribing was carried out by establishing direct continuity to adjoining sheets.

2.7 Delivered Final Products

The following final products were delivered from the local contractor.

2.7.1 Aerial Photography

(i)	Negative film	1 set
(ii)	Positive film	1 set
(iii)	Contact print	2 sets
(iv)	Photo index	1 sheet

2.7.2 Topographic maps

(i)	Topographic map, Original (polyester base)	1 set
(ii)	Topographic map, Copy (polyester base)	1 set
(iii)	Description of ground control points	1 set
(iv)	Field notes and calculation data of ground control point survey	1 set
(v)	Calculation data of aerial triangulation	1 set

TABLES

Table I.2.1 RESULTS OF GROUND CONTROL POINT SURVEY (1/2)

Point number	Coordinates		Elevation (m)
	X	Y	
1. Salto Pilão Area			
VB-09	6,987,885.918	635,620.073	625.97
VB-10	6,988,459.882	631,378.777	635.35
VB-11	6,993,359.230	632,053.316	716.44
VB-12	6,995,544.970	640,074.938	732.44
VB-13	6,993,948.764	646,159.322	546.77
VB-14	7,001,461.034	647,977.368	434.15
HV-01	6,991,632.415	626,255.755	421.35
HV-02	6,988,079.677	623,628.084	398.72
HV-03	6,987,876.482	627,329.617	389.52
HV-07	6,986,476.089	626,531.864	541.99
HV-10	6,985,625.451	631,007.497	422.63
HV-11	6,989,729.341	635,983.452	337.13
HV-13	6,990,960.255	631,331.781	354.10
HV-14	6,988,685.501	634,628.449	377.07
HV-14A	6,986,733.325	632,660.000	412.98
HV-15	6,993,722.705	632,539.015	679.64
HV-19	6,996,728.887	636,779.758	355.02
HV-22	6,993,839.745	640,995.002	334.35
HV-23	6,998,363.567	640,688.672	453.62
HV-28	6,998,355.490	646,148.787	368.87
HV-31	7,003,893.892	647,357.848	169.71
HV-34	6,983,355.176	629,704.268	411.05
HV-35	6,979,277.174	631,997.425	500.21
HV-40	6,985,541.571	637,851.751	447.83
HV-44	6,990,074.352	643,961.602	505.33
HV-47	6,995,390.884	648,830.745	498.84
HV-52	7,003,473.566	654,378.471	113.26
HV-53	6,999,536.879	654,667.652	213.16
2. Dalbergia Area			
VB-04	7,002,302.382	650,676.038	438.89
VB-05	7,004,821.649	649,170.979	311.88
VB-06	7,006,825.958	646,453.875	313.87
VB-07	7,008,317.265	640,946.214	326.74

Table I.2.1 RESULTS OF GROUND CONTROL POINT SURVEY (2/2)

Point number	Coordinates		Elevation (m)
	X	Y	
VB-08	7,006,856.209	646,508.561	316.69
HV-50	7,003,174.345	651,996.341	375.99
HV-54	7,013,698.273	646,880.857	346.46
HV-55	7,012,129.052	640,705.188	387.37
HV-56	7,014,135.410	644,893.931	401.12
HV-57	7,010,813.930	641,499.583	318.65
HV-58	7,012,928.867	647,190.787	337.47
HV-59	7,008,889.947	643,709.119	268.49
HV-60	7,011,683.401	648,447.947	390.40
HV-61	7,007,453.566	645,345.999	179.13
HV-62	7,007,768.320	648,539.096	361.97
HV-63	7,006,026.775	647,287.362	221.65
HV-65	7,007,232.376	653,178.936	407.73
HV-66	7,003,095.904	650,527.095	166.83
HV-67	7,011,494.692	638,475.314	277.23
HV-68	7,006,948.615	636,262.999	253.52
HV-69	7,010,133.118	640,926.591	267.85
HV-70	7,006,746.719	639,119.135	441.81
HV-71	7,005,681.954	641,024.741	411.09
HV-72	7,002,841.947	643,382.489	329.98
HV-73	7,001,951.740	645,505.087	343.96
HV-74	7,004,892.588	650,073.629	383.45
HV-75	7,001,553.483	647,883.392	417.02
3. Benedito Novo Area			
VB-01	7,037,505.495	662,781.554	192.28
VB-02	7,036,799.322	660,850.145	256.95
VB-03	7,036,519.433	662,217.370	221.57
HV-76	7,039,424.987	658,435.787	347.22
HV-77	7,038,061.065	659,795.102	241.09
HV-78	7,039,486.936	661,144.841	207.07
HV-79	7,034,945.857	660,843.307	290.78
HV-80	7,039,760.729	663,383.047	212.34
HV-81	7,034,692.610	663,026.834	136.55

FIGURES

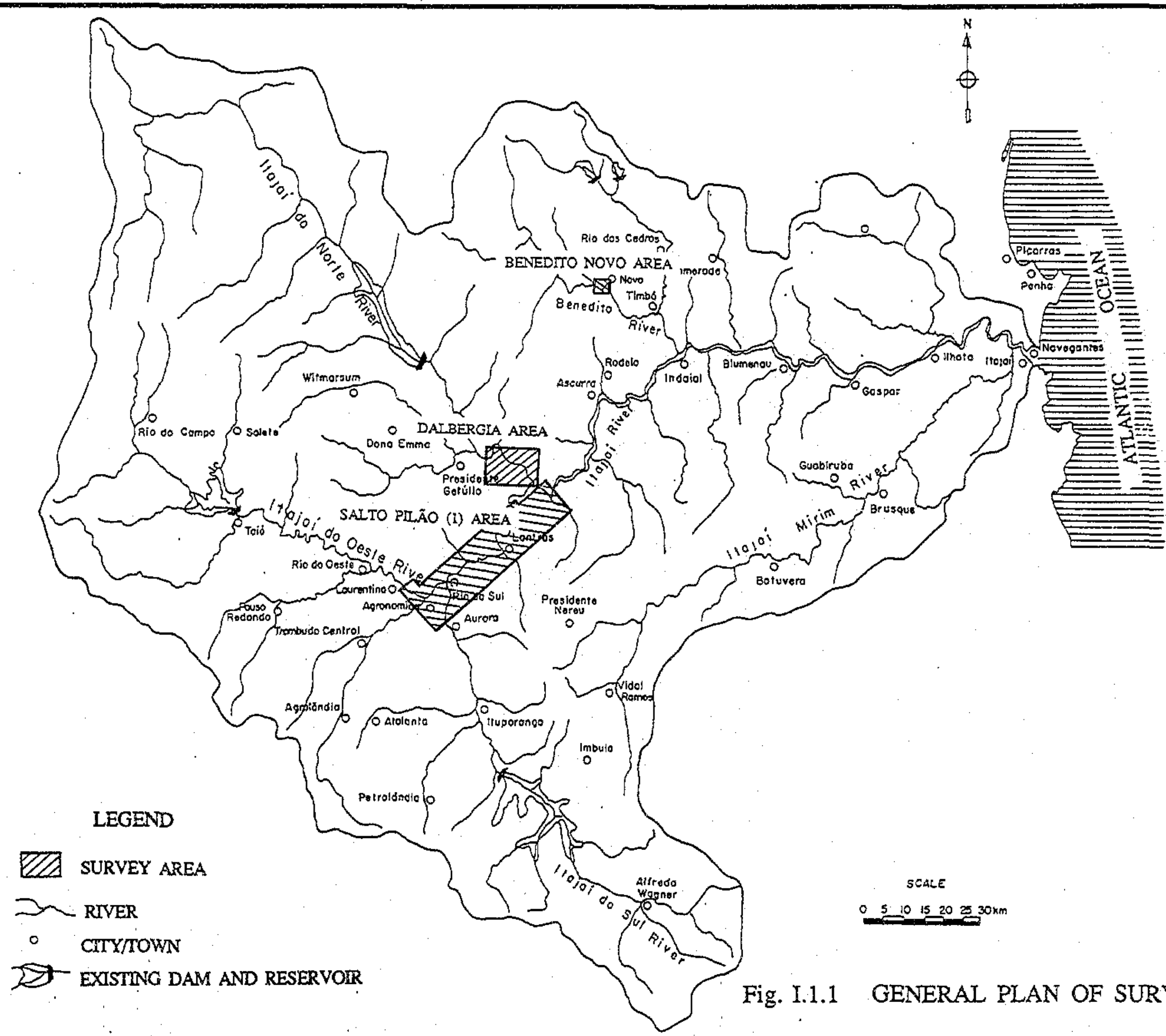
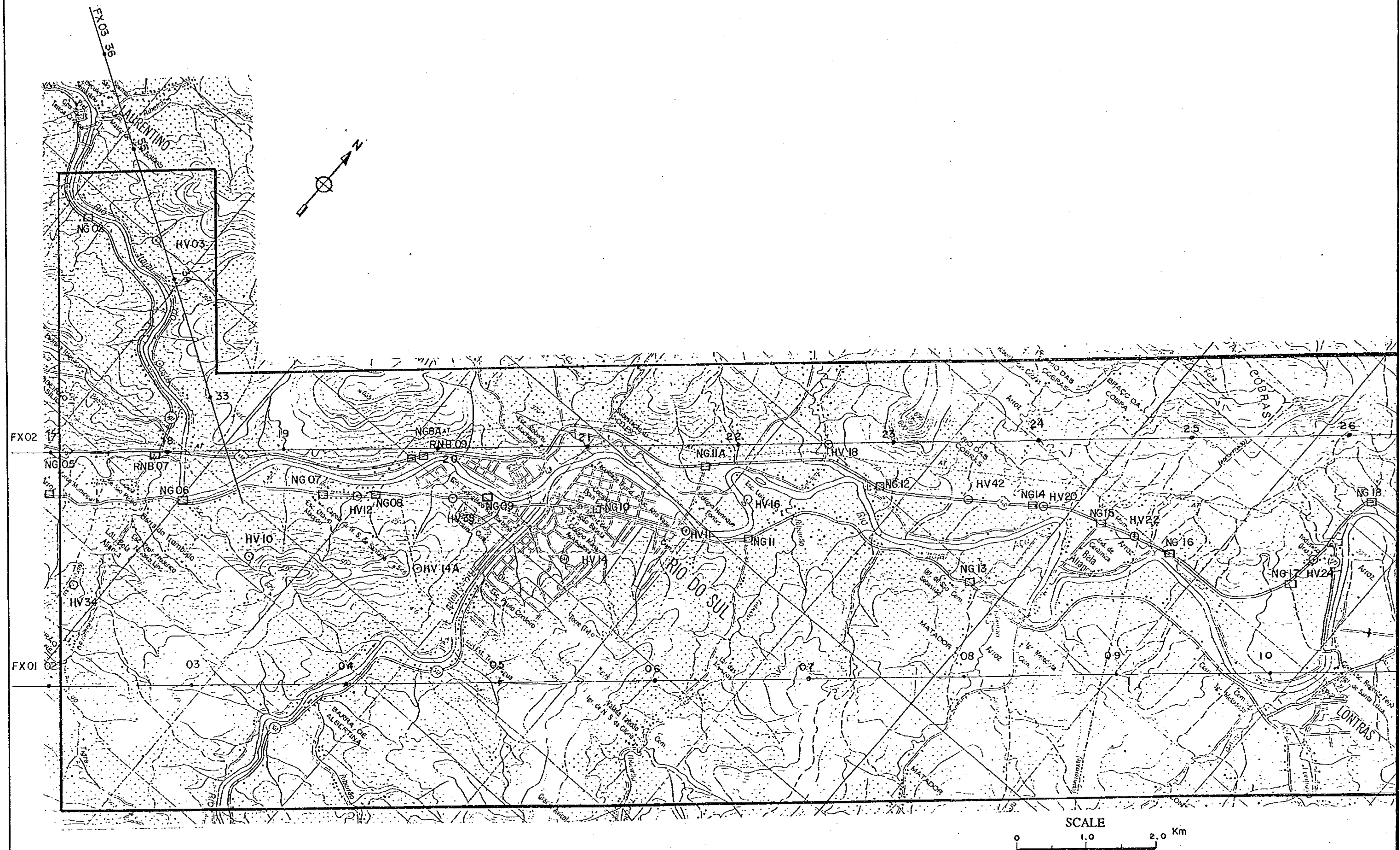


Fig. I.1.1 GENERAL PLAN OF SURVEY AREA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NG 02
HV 03
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FX02
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22
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RNB 07
NG 06
NG 07
HV12
NG08
NG8A
RNB 09
HV10
HV 14A
HV 14
NG 09
NG10
HV11
NG 11
NG11A
HV 18
HV 16
NG 12
HV42
NG14
HV20
NG15
HV22
NG 16
NG 17
HV24
NG 18
NG 13
HV 34
RIO DO SUL
COBRAS
FX01 02
03
04
05
06
07
08
09
10
BARRA DE ALBERTINA
MATADOR
LONTRAS
SCALE
0
1.0
2.0 Km

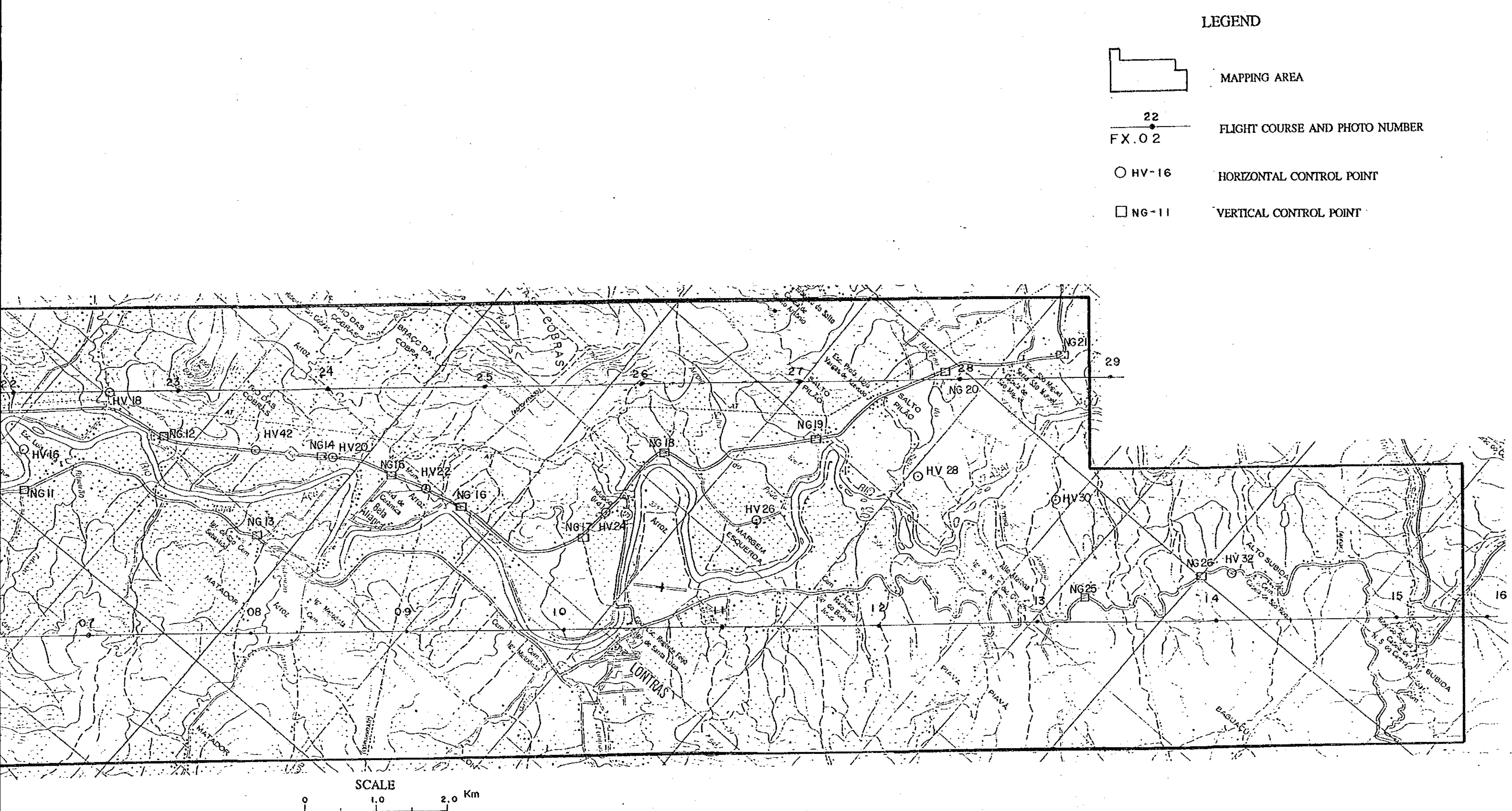
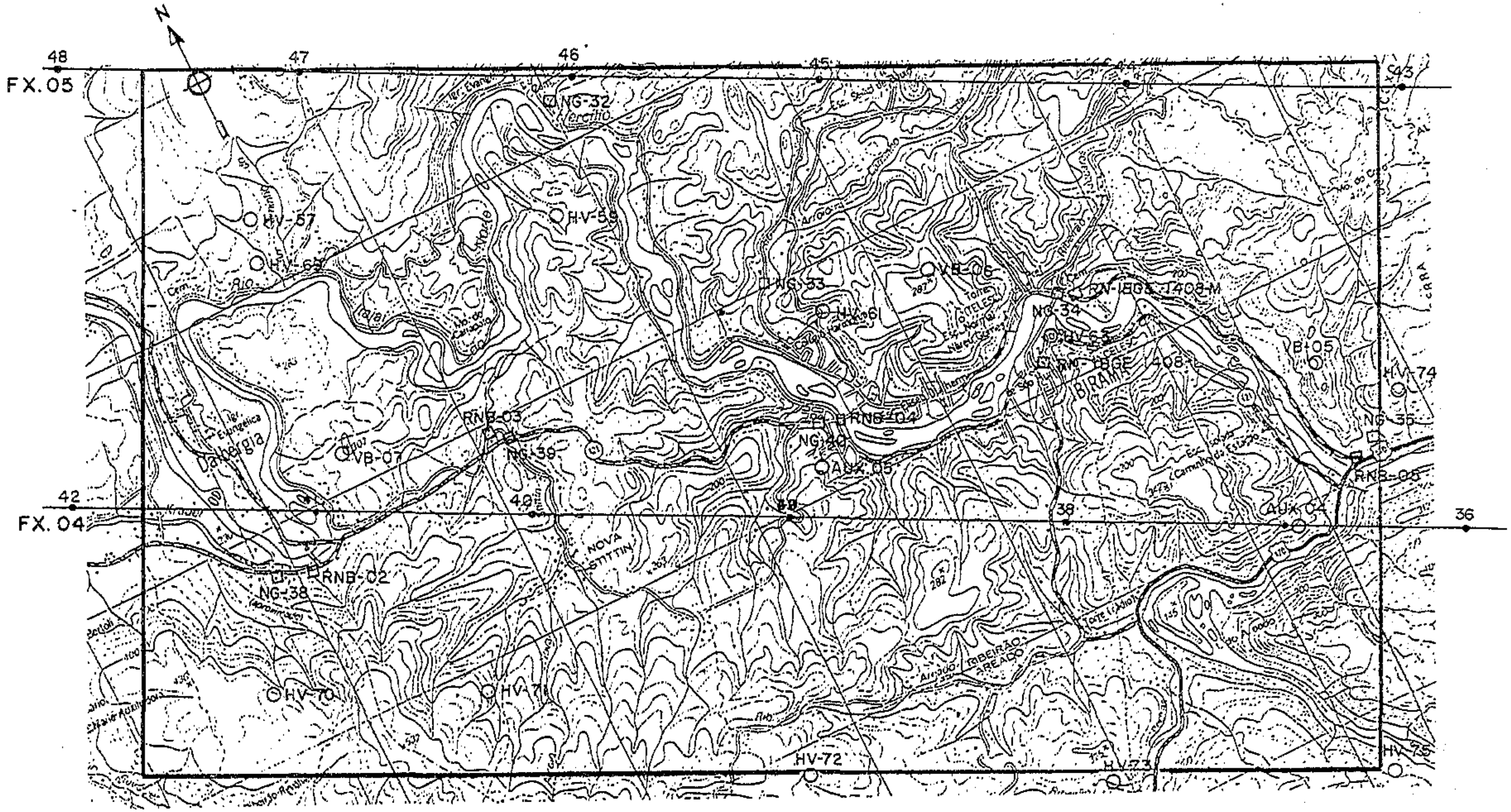


Fig. I.2.1 LOCALITY PLAN OF AERIAL PHOTOGRAPH AND GROUND CONTROL POINT FOR SALTO PILÃO (1) SCHEME



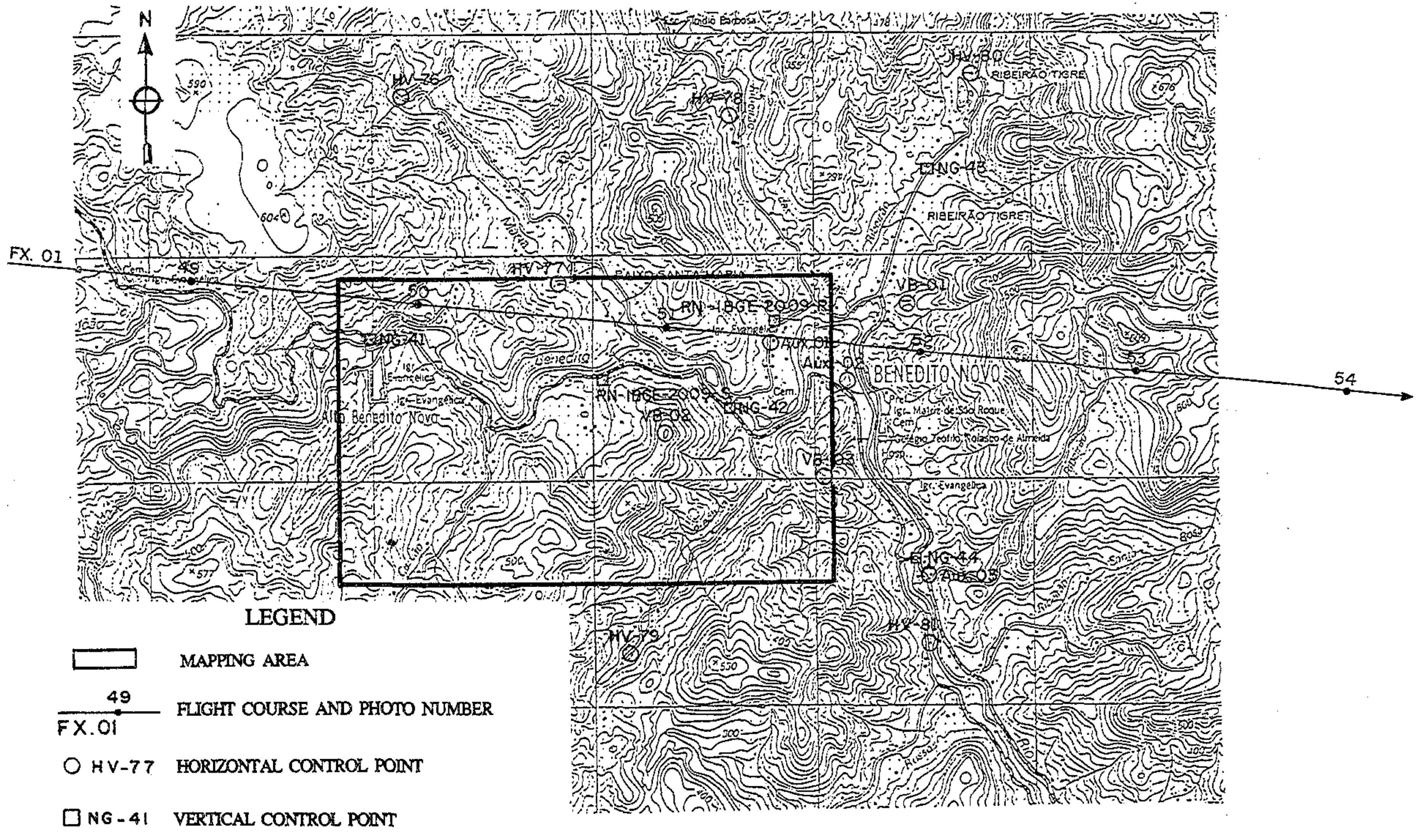
LEGEND

- MAPPING AREA
- 42 FX.04 — FLIGHT COURSE AND PHOTO NUMBER
- ○ HV-57 — HORIZONTAL CONTROL POINT
- □ NG-32 — VERTICAL CONTROL POINT

SCALE

0 1.0 2.0 Km

Fig. I.2.2 LOCALITY PLAN OF AERIAL PHOTOGRAPH AND GROUND CONTROL POINT FOR DALBERGIA SCHEME



SCALE

0 1.0 2.0 Km

Fig. I.2.3 LOCALITY PLAN OF AERIAL PHOTOGRAPH AND GROUND CONTROL POINT FOR BENEDITO NOVO SCHEME

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ANNEX II. GEOTECHNICAL INVESTIGATION

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1. INTRODUCTION

This report presents the results of the geotechnical investigation for pre-feasibility study on three hydropower schemes, i.e., Salto Pilão (1), Dalbergia and Benedito Novo schemes. This geotechnical investigation comprises core boring for the proposed damsites, headrace tunnel route and powerhouse site, permeability test (Lugeon test) at the damsites by use of bore hole and construction material survey for the three schemes.

The core boring investigation of 285 m in total was performed for the proposed damsite, headrace tunnel route and powerhouse site for the three schemes as shown in Table II.1.1. The bore hole location is shown in Figs. II.1.1, II.1.2, II.1.3, II.1.4, II.1.5 and II.1.6. Lugeon tests, 15 times in total were performed in these boreholes at the damsites. The construction material survey for dam in particular was carried out by means of geo-surface inspection without exploratory borings and also of geological map study.

2. REGIONAL GEOLOGY

2.1 Previous Geological Investigation

General geology for 16 hydro-potential schemes was investigated by means of field reconnaissance and geological maps without exploratory borings in the first stage. The results of the investigation were summarized in Table II.2.1.

2.2 Regional Topography

The Itajai river stretch along Rio do Sul city has a remarkably gentle river bed slope. The Itajai river changes its river bed slope from about 1:6000 to about 1:60 at about 25 km downstream of Rio do Sul. The project area of Salto Pilão (1) scheme is located in this river stretch with a steep river bed slopes (herein called rapid). The topography in the surrounding area of the project area shows the series of hilly land and low mountain area. The proposed damsite is located at about 28 km downstream of Rio do Sul city, about 3 km downstream of the upstream end of the rapid. The Itajai river flows northwards meandering this rapid and joins the Itajai do Norte river at about 13 km downstream of the proposed damsite. This rapid lasts up to the downstream of the junction of the Itajai do Norte river, at which the proposed powerhouse is located. Many small tributaries from southeast direction joins the Itajai river stretch between the proposed damsite and powerhouse site. The proposed headrace tunnel route is planned to be aligned crossing these tributaries.

The Itajai do Norte river in the upstream of the junction of the Itajai river also forms the rapid with river bed slope of about 1:160. The proposed damsite of Dalbergia scheme is located at the upstream end of this rapid, at about 4 km downstream of Dalbergia town. The both banks of the damsite are formed by steep mountain slopes. The proposed headrace tunnel is planned to be aligned along high ridges to the junction of the Itajai river, at which the proposed powerhouse is situated.

The Benedito river in the upstream of Benedito Novo town forms the rapid with river bed slope of about 1:60. The proposed damsite of Benedito Novo scheme is located at the upstream end of this rapid about 1 km downstream of Alto Benedito Novo town. The both banks of the damsite are formed by steep mountain slopes. The proposed headrace tunnel is planned to be aligned along high ridges to the downstream end of the rapid, at about 2 km upstream of Benedito Novo town, where the proposed powerhouse is located.

2.3 Regional Geology

Main geological layer in the three project areas consist of Santa Catarina complex, Gaspar formation, Campo formation and Subida Intrusive Bodies of precambrian era in geological time, which are associated locally with Rio do Sul and/or Itaraje formation of carboniferous time.

Lithologically those layers are composed of such rock types as Santa Catarina complex - gneiss and granite partly with diabase of Gaspar formation - slate, hornfels, Campo formation - rhyolite, and Subida Intrusive Bodies - granite. Rio do Sul or Itaraje formation is composed of shale. Geomorphological feature in the project areas is a series of outcrop of rocks in the river bed. In addition, meandering of river with sharp angle, which occurs along decomposed zone in the fault is another geological feature.

In the project area of Salto Pilão (1) scheme, Subida Intrusive Bodies - granite mainly distributes in the areas from the damsite to the headrace tunnel route. On the other hand, Campo formation - rhyolite and Gaspar formation - slate, hornfels spread in the sites of the proposed surge tank, penstock and powerhouse. In the surrounding area of damsite recent river deposit and/or terrace deposit distribute.

In the project area of Dalbergia scheme, geological layer is composed of gneiss of Santa Catarina complex through the whole area, although shale of Rio-do Sul or Itaraje formation appears locally in the ground surface of the upstream of the headrace tunnel route. Deep weathering along the fault is a remarkable geologic feature in this project area, which is especially conspicuous in the left bank of the damsite.

The geological layer in the project area of Benedito Novo scheme is also composed of gneiss associated with diabase of Santa Catarina complex. Relatively thick talus deposit distributes in the proposed intake site and along the penstock line route.

3. GEOLOGY FOR SALTO PILÃO (1) SITE

3.1 Damsite

In the first stage, only a damsite which was considered to be the most suitable based on the result of field reconnaissance was chosen. In this study, two alternative damsites were contemplated in addition to the selected damsite to determine the appropriate damsite from the viewpoint of construction cost of waterway facilities and power output. Location of 3 alternative damsites is shown in Fig. II.1.1. Geological plan and profile of the damsites are shown in Figs. II.1.1 and II.3.1 and results of boring and permeability test are summarized in Figs. II.3.2 and II.3.3.

3.1.1 Damsite - A

(1) Geological condition

Geological condition at the damsite - A was interpreted referring to the result of boring and water pressure test (Hole No. B1-1), which was carried out at the right side of the river on the damsite - B. About 30 m wide river deposit exists in the middle of the river cross section. Granite outcrops sporadically in the river bed portion. While granite in the both banks is weathered rather thickly. Granite is massive as a whole and any large open crack or fractured zone does not appear, though fault crosses dam axis in the left bank side. The permeability condition in granite is shown in Fig. II.3.3. It shows that quantity of water leakage in case of water head of 20 m is less than 0.1 l/min./m below the depth of hard rock line.

(2) Engineering geology

Granite has very hard and massive characteristics. It corresponds to A to B class of rock classification which has been defined by Standard of Central Research Institute of Electric Power Industry of Japan as shown in Table II.3.1. The excavation depth of foundation rock was estimated at about 10 m for dam abutment portions and about 2 m for river bed portion.

The rock properties of dam foundation were estimated as follows:

Rock classification	:	A, B
Compressive strength	:	more than 800 kg/cm^2
Static modulus of elasticity	:	more than 80,000 kg/cm^2
Cohesion	:	more than 40 kg/cm^2
Internal friction angle	:	55 to 65 degree
Static poisson's ratio	:	less than 0.2

As a foundation treatment, consolidation and curtain groutings will be needed only for left bank side to remedy the fault. Consolidation grouting with an interval of 4 m and depth of 5 m and curtain grouting with an interval of 2 m and depth of 20 m are proposed.

3.1.2 Damsite - B

(1) Geological condition

Granite outcrops in the whole river bed, Granite in both banks is covered with weathered layer. The result of the boring shows that this granite is in very hard and massive condition. Although vertical joints appears at an interval of 2 to 5 m, these are closed joints and any large open crack and fractured zone are not found.

(2) Engineering geology

Granite will be graded into excellent rock which corresponds to A to B class of rock classification. The excavation depth of foundation rock was estimated at about 2 m for river bed portion and about 10 m for the both abutment portions.

The rock properties of dam foundation were estimated as follows:

Rock classification	:	A to B
Compressive strength	:	more than 800 kg/cm^2
Static modulus of elasticity	:	more than 80,000 kg/cm^2
Cohesion	:	more than 40 kg/cm^2
Internal friction angle	:	55 to 65 degree
Static poisson's ratio	:	less than 0.2

The result of permeability test shows that quantity of water leakage in case of water head of 20 m is less than 0.1 l/min./m. Then, consolidation and curtain groutings may be omitted.

3.1.3 Damsite - C

(1) Geological condition

Geological condition in the damsite - C was interpreted based on geo-surface inspection and referring the result of boring and permeability test performed for the damsite - B. Outcrop of granite appears in the right side of the river bed and about 70 m wide river deposit is found in the left side of the river bed. Decomposed granite covers rather thick in the both banks. Although granite has hard and massive characteristics, open cracks are found in several places crossing the dam axis.

(2) Engineering geology

The rock properties of the dam foundation were estimated as follows:

Rock classification	:	B to CH
Compressive strength	:	more than 800 kg/cm^2
Static modulus of elasticity	:	80,000 to 40,000 kg/cm^2
Cohesion	:	40 to 20 kg/cm^2
Internal friction angle	:	40 to 50 degree
Static poisson's ratio	:	0.2 to 0.3

The excavation depth of foundation rock was estimated at about 2 m for the river bed portion and about 15 m for the abutment portions. According to the result of boring B1-1 and the observation of outcrop of granite, foundation rock for dam in this site is fairly hard, which can be graded into B to CH class of rock classification. However, since open cracks are found in some places in outcrop, it was judged that permeability is high (Condition is different from boring B1-1). Occurrence of loosed condition along the open crack by excavation will be anticipated. Consequently, consolidation and curtain groutings will be required. The consolidation grouting with an interval of 4 m and depth of 5 m, and curtain grouting with an interval of 2 m and depth of 40 m are proposed.

3.2 Headrace Tunnel

3.2.1 Geological condition

The geological plan and profile of the headrace tunnel route are shown in Fig. II.1.2 and result of boring is summarized in Fig. II.3.2. Granite distributes almost all parts of the tunnel route. The result of geo-surface inspection clarified that existence of rhyolite is found near the proposed surge tank portion. Thin diabase vein intrudes locally into granite. Several faults which involve small scale fractured zone were detected by geo-surface inspection and analysis of aerophotograph. Number of faults was 11 in total through the tunnel route.

At the proposed intake site for damsites - A and B, granite distributes and it is covered with thin soil and/or weathered layer. While since the proposed intake site for damsite - C is covered with rather thick soil and weathered layer, hard rock is not found in the tunnel foundation level.

3.2.2 Engineering geology

Granite will be graded into the excellent rock which corresponds to A class of rock classification. Rhyolite also shows sufficient hard and impermeable characteristics judging from geological inspection. Thus, there are no technical problems for tunnel excavation but supporting system would be required for about 110 m long fault zones. Consolidation grouting with an interval of about 3 m and depth of about 3 m will be required for the fault zones. Since both granite and rhyolite are so hard and massive that they are stable enough against the internal pressure of 2 kg/cm^2 of tunnel. It was therefore judged that consolidation grouting will be omitted except for the fault zones. Excavation rock material will be used for concrete aggregate.

For damsite - C plan, supporting system will be required for about 500 m long tunnel route at its beginning part, since over burden has no sufficient thickness for tunnel construction and tunnel route passes the tributary.

3.3 Surge Tank and Penstock Line

Geological condition of the proposed surge tank and penstock line was interpreted based on the geo-surface inspection. Geological plan and profile of these sites are shown in Fig. II.1.2.

3.3.1 Surge tank

Rhyolite outcrops in the form of small dome at the proposed surge tank site. The ground surface of the surge tank site is covered with decomposed rhyolite and soil with thickness of about 10 m. It is presumed that the surge tank is provided in about 10 m thick rhyolite zone. The result of geo-surface inspection shows that this rhyolite is very hard rock which corresponds to B class of rock classification. It was therefore judged that there are no technical problems for excavation work but consolidation grouting with an interval of 3 m and depth of 3 m will be needed.

3.3.2 Penstock line

The proposed penstock line route will pass rhyolite zone. Geo-surface inspection for outcrop of this rhyolite clarified that it has hard and massive characteristics which are graded into B class of rock classification, besides, any open crack and fractured zone are not found. It was therefore judged that there are no technical problems for excavation work but consolidation grouting with an interval of 3 m and depth of 3 m will be required.

Open air steel conduit type penstock line is conceivable as an alternative plan. However, about 10 m thick heavily weathered layer of decomposed soil layer overlies along the penstock line route. Since this layer is apt to be easily collapsed nature, it is judged that the open air steel conduit type plan is not economically suitable.

3.4 Powerhouse and Tailrace

Geological condition in these sites was interpreted based on the result of boring performed at the proposed powerhouse site as well as geo-surface inspection. Geological profile of the sites is shown in Fig. II.3.4.

3.4.1 Powerhouse

The result of boring shows that weathered rhyolite with fractured zone, which is graded as CL class and fractured rhyolite graded as CL to CM class, which is influenced by fault distribute up to depth of 11.6 m from ground surface. These fractured rhyolite varies to fresh one below the depth of 11.6 m. Although rock itself of these fractured zone is fairly hard, it is desirable that foundation for the powerhouse should be set at 11.6 m in depth.

3.4.2 Tailrace

The proposed tailrace route is located at the flat space of river deposit with thickness of about 2 m. About 5 m thick weathered and fractured rhyolite distributes below this river deposit. This fractured rhyolite varies to fresh one as the depth increases. This fractured rhyolite is fairly hard and bearable for foundation of the tailrace. It is desirable to set the foundation of the tailrace on this hard rock layer.

3.5 Construction Materials in the Project Area

The survey for construction materials was carried out by means of geo-surface inspection in the vicinity of the project area.

3.5.1 Coarse aggregate

Quarry site for coarse aggregate was investigated and a hilly mountain at about 1 km upstream from the right bank of the damsite - B was selected. Location of the proposed quarry site is shown in Fig. II.1.1. In this quarry site, outcrop of very hard and massive granite distribute in about 20 m in height, 200 m in length and 100 m in width. Granite zone is covered with weathered soil layer of about 10 m in the hill top. The estimated volume of fresh granite rock is about 400,000 m^3.

3.5.2 Fine aggregate

Borrow area for fine aggregate was investigated along the river course of the Itajai. Although river deposit and terrace deposit are scattered near the project area, these compositions consist of thin layer of silty soil. It will be obliged to produce the fine aggregate by crushing the rock materials in the proposed quarry site.

4. GEOLOGY FOR DALBERGIA SITE

4.1 Damsite

In order to select the appropriate damsite, two alternative damsites were contemplated in addition to the damsite which was proposed in the first stage. Location of 3 alternative damsites is shown in Fig. II.1.3. Geological plan and profile of the damsites are given in Figs. II.1.3 and II.4.1 and results of boring and permeability test are summarized in Figs. II.4.2 and II.3.3.

4.1.1 Damsite - A

(1) Geological condition

Geological condition of the damsite - A was interpreted referring to the boring and permeability test (Hole No. B7-1) which were performed at the damsite - C. Gneiss outcrops in the river bed forming small rapid. About 50 m wide river terrace deposit spreads in the left bank. Abutment of both banks is heavily weathered and decomposed into soil. Fault crosses the dam axis in the left side of the river bed.

(2) Engineering geology

Gneiss is relatively hard rock which corresponds to CH class of rock classification. The excavation depth of foundation rock was estimated at 2 m in the river bed, 15 m in the left bank and 10 m in the right bank. Since vertical joints associated with open cracks develop in several places in the river bed, high permeability was foreseen. The result of permeability test shows that quantity of water leakage is more than 3 l/min./m in case that water head is 20 m.

The rock properties were estimated as follows:

Rock classification	:	CH
Compressive strength	:	800 to 200 kg/cm^2
Static modulus of elasticity	:	80,000 to 40,000 kg/cm^2
Cohesion	:	40 to 20 kg/cm^2
Internal friction angle	:	40 to 55 degree
Static poisson's ratio	:	0.2 to 0.3

As a foundation treatment, consolidation and curtain groutings will be needed. Consolidation grouting with an interval of 4 m and depth of 10 m, and curtain grouting with an interval of 2 m and depth of 30 m are proposed. To remedy the fault portions in the left side of river, cutoff trench and filling work of concrete in it will be required.

4.1.2 Damsite - B

(1) Geological condition

Geological condition in the damsite - B was interpreted referring to the boring result and permeability test performed for the damsite - C.

Along the dam axis, gneiss distributes almost all part of the river bed. Right river bank in the upstream of the damsite is scored and river deposit is overlaid in the river bed. Right river bank of the damsite is covered with talus deposit. Left river bank is heavily weathered. Vertically and seriously dipped faults which may cause open crack and deep weathering intersect the dam axis in both the left and right sides of the river bed.

(2) Engineering geology

The rock properties were estimated as follows:

Rock classification	:	CH
Compressive strength	:	800 to 200 kg/cm^2
Static modulus of elasticity	:	80,000 to 40,000 kg/cm^2
Cohesion	:	40 to 20 kg/cm^2
Internal friction angle	:	40 to 55 degree
Static poisson's ratio	:	0.2 to 0.3

The excavation depth of the foundation was estimated at 2 m in the river bed, 15 m in the left bank side and 10 m in the right bank side. It is presumed from the result of the permeability test that quantity of water leakage in case of water head of 20 m is more than 3 l/min./m and its permeable zone continues up to depth of 30 m.

As the foundation treatment, both the consolidation and curtain groutings will be required. The consolidation grouting with an interval of 4 m and depth of 10 m and curtain grouting with an interval of 2 m and depth of 30 m are proposed. Besides, to remedy the fault position cut off excavation and filling work by concrete will be needed in the left side of damsite.

4.1.3 Damsite - C

(1) Geological condition

Gneiss develops along the dam axis and it outcrops in the right half part of the river bed. River deposit exists in the left half part of the river bed. In both banks, gneiss is weathered and it is decomposed into soil. Open cracks with 80 to 90 degrees and 10 to 20 degrees in dip, which are influenced by fault develops at an interval of 20 to 30 cm in the river bed.

(2) Engineering geology

The excavation depth of the foundation rock was estimated at 2 m in the river bed, 8 m in the left bank and 5 m in the right bank. The rock properties of the dam foundation were estimated as follows:

Rock classification	:	CH to CM
Compressive strength	:	800 to 200 kg/cm^2
Static modulus of elasticity	:	40,000 to 15,000 kg/cm^2
Cohesion	:	40 to 10 kg/cm^2
Internal friction angle	:	30 to 45 degree
Static poisson's ratio	:	0.2 to 0.3

The result of permeability test shows that quantity of water leakage in case of water head of 20 m is more than 10 l/min./m up to depth of 20 m and 3 to 5 l/min./m below 20 m in depth. Considering crackly and remarkably permeable conditions, both the consolidation and curtain groutings will be required. The consolidation grouting with an interval of 4 m and depth of 10 m and curtain grouting with an interval of 1 m and depth of 30 m are proposed. Besides, cutoff trench work and filling by concrete in it will be needed to cope with the fault in the left side of the river bed.

4.2 Headrace Tunnel

4.2.1 Geological condition

Geological plan and profile of the headrace tunnel are shown in Fig. II.1.4. Result of boring is summarized in Fig. II.4.2.

Gneiss distributes in whole route of the headrace tunnel though shale is found and gneiss is weathered and loosened at the beginning part of the tunnel. According to geo-surface inspection and analysis of aerophotograph, faults were ascertained at the crossings of tributary and tunnel route.

At the proposed intake site for the damsite - A, weathered thin gneiss outcrops. At the intake site for the damsite - B, talus deposit covers the ground surface. Gneiss is found in wide area for the intake site for the damsite - C.

4.2.2 Engineering geology

Majority of the proposed tunnel route will pass a layer of very hard and tight gneiss which corresponds to A to B class of rock classification. It was therefore judged that there are no technical problems for tunnel excavation but supporting system will be needed for about 110 m long fault portions which exist sporadically. Consolidation grouting with an interval of 3 m and depth of 3 m will be needed for the fault portions. Excavated rock will be used for concrete aggregate.

Supporting system and consolidation grouting with an interval of 3 m and depth of 3 m will be needed for about 300 m long stretch near intake site of the damsite - B.

4.3 Surge Tank and Penstock Line

Geological condition of the proposed surge tank and penstock line was interpreted based on geo-surface inspection. Geological plan and profile of these sites are shown in Fig. II.1.4.

4.3.1 Surge tank

Gneiss distributes at the proposed surge tank site and it is covered with about 10 m thick weathered layer. It is presumed that this gneiss is hard and massive rock which corresponds to B class of rock classification, and there are no fractured zones. It was therefore judged that there are no technical problems for construction works for the surge tank but consolidation grouting with an interval of 3 m and depth of 3 m will be needed.

4.3.2 Penstock line

The proposed penstock line will pass gneiss layer. The result of geo-surface inspection clarified that this gneiss is hard and massive rock which corresponds to B class of rock classification and there are no fractured zones. It was therefore judged that there are no technical problems for tunnel excavation but consolidation grouting with an interval of 3 m and depth of 3 m will be required for whole tunnel stretch.

Open air steel conduit type penstock line is conceivable as an alternative plan. But about 10 m thick rather weathered and decomposed loose soil overlies along the penstock line route. Since this layer has to be removed in case of open air conduit type penstock line due to its soil characteristics with sliding, it is judged that this alternative plan is not appropriate.

4.4 Powerhouse and Tailrace

Geological condition at the proposed powerhouse and tailrace sites was interpreted based on the result of boring performed at the proposed powerhouse site and geo-surface inspection. Geological profile of these sites is shown in Fig. II.4.3.

4.4.1 Powerhouse

The proposed powerhouse site is covered with about 50 m wide and 9 m thick terrace deposit and about 2 m thick weathered gneiss. Fresh gneiss with hard and massive properties distributes below 11 m from the ground surface. It was therefore judged that foundation for generator is set on the hard gneiss zone and foundation of building is set on the terrace deposit layer because it is fairly consolidated.

4.4.2 Tailrace

The proposed tailrace site is located at the terrace deposit layer. Since this layer consists of clayey sand associated with gravels and boulders, it is considered that this layer is well consolidated and bearable for foundation of the tailrace.

4.5 Construction Materials in the Project Area

The construction materials survey was carried out by geo-surface inspection in the vicinity of the project area.

4.5.1 Coarse aggregate

Quarry site for coarse aggregate was investigated in the vicinity of the project area and hilly mountain at about 0.5 km upstream from right bank of the damsite - C was selected. Location of the proposed quarry site is shown in Fig. II.1.3. The quarry site with 50 m high and steep slope faces the river and gneiss outcrops for whole of the slope. Top of the quarry site is covered with about 5 m thick weathered layer. The estimated volume of fresh gneiss was estimated at about 500,000 m^3.

4.5.2 Fine aggregate

Borrow area for fine aggregate was investigated along the river course of the Itajai do Norte river. Although the river and terrace deposits are scattered near the project area, these consist of thin layer of silty soil and small amount of sand fraction. It will be therefore obliged to produce the fine aggregate by crushing the rock materials in the proposed quarry site.

5. GEOLOGY FOR BENEDITO NOVO SITE

5.1 Damsite

Three alternative damsites were contemplated to select the most appropriate site among them. Location of these sites is shown in Fig. II.1.5. Geological plan and profile of the damsites are shown in Figs. II.1.5 and II.5.1 and the result of boring and permeability test are summarized in Figs. II.5.2 and II.3.3. Geological condition of these 3 damsites was interpreted based on the boring result performed for the damsite - C and geo-surface inspection.

5.1.1 Damsite - A

(1) Geological condition

Outcrop of gneiss is found at downstream of the dam axis and about 3 m thick river deposit exists in upstream of the dam axis. Heavily weathered gneiss which is decomposed into soil distributes in the left bank. In the right bank, talus deposit covers the ground surface forming a gentle slope.

(2) Engineering geology

A rapid forms in the downstream of the damsite and gneiss outcrops. it was therefore judged that hard rock zone for dam foundation exists at 3 m below the river bed. The excavation depth of foundation rock was estimated at 5 m in the river bed and 10 m in both banks. The foundation rock consists of gneiss which is fairly hard and corresponds to CH to B class of rock classification. It is presumed that vertically dipped joints exist in an interval of 2 to 5 m.

The rock properties of dam foundation was estimated as follows:

Rock classification	:	CH to B
Compressive strength	:	more than 800 kg/cm^2
Static modulus of elasticity	:	80,000 to 40,000 kg/cm^2
Cohesion	:	40 to 20 kg/cm^2
Internal friction angle	:	40 to 55 degree
Static poisson's ratio	:	0.2 to 0.3

Result of permeability test shows that water leakage in case of 20 m of water head is as large as 23 l/min./m. It was therefore judged that both the consolidation and curtain groutings are needed as foundation treatment. The consolidation grouting with an interval of 4 m and depth of 5 m and curtain grouting with an interval of 2 m and depth of 15 m will be proposed. In addition about 30 m long and 50 m wide slope protection will be needed for talus deposit in the right bank.

5.1.2 Damsite - B

(1) Geological condition

This damsite is located at the intermediate portion of about 25 m high rapid. Outcrop of gneiss appears from place to place forming small step-like falls. Gneiss is heavily weathered and decomposed into soil in the left bank. In the right bank, about 100 m wide terrace deposit and talus deposit distribute. Vertically dipped joints develop in an interval of about 2 m crossing the dam axis.

(2) Engineering geology

The excavation depth of rock foundation was estimated at 2 m in the river bed, 10 m in the left bank and 5 to 10 m in the right bank. The foundation rock consist of gneiss which is fairly hard and corresponds to CH to B class of rock classification.

The rock properties of dam foundation was estimated as follows:

Rock classification	:	CH to B
Compressive strength	:	more than 800 kg/cm^2
Static modulus of elasticity	:	80,000 to 40,000 kg/cm^2
Cohesion	:	40 to 20 kg/cm^2
Internal friction angle	:	40 to 55 degree
Static poisson's ratio	:	0.2 to 0.3

Result of permeability test shows that water leakage in case of 20 m of water head is 23 l/min./m up to 8 m in depth. Judging from this result and occurrence of joints in the outcrop, it is considered that both consolidation and curtain groutings are required. The consolidation grouting with an interval of 4 m and depth of 5 m and curtain grouting with an interval of 2 m and depth of 15 m will be proposed. In addition, protection work against slope fall will be needed for talus deposit in the right bank. The extent of area to be protected will be 30 m in length and 50 m in width.

5.1.3 Damsite - C

(1) Geological condition

This damsite is located at just downstream of about 25 m high rapid. Gneiss associated partly with granite distributes in the river bed. The left bank is covered with weathered gneiss. In the right bank, talus deposit covers the ground surface. Seriously dipped joints cross the dam axis at an interval of 2 to 3 m. Fault with the same direction with river stretch intersects in the right side of the dam axis, and some open cracks occur due to influence of fault.

(2) Engineering geology

It was judged based on the result of boring performed at the right side of river that the excavation depth of rock foundation is about 2 m in the river bed, 5 m in the left bank and 3 to 5 m in the right bank. The foundation rock is composed of gneiss associated partly with granite, which is hard and corresponds to CH to B class of rock classification.

The rock properties of dam foundation was estimated as follows:

Rock classification	:	CH to B
Compressive strength	:	more than 800 kg/cm^2
Static modulus of elasticity	:	80,000 to 40,000 kg/cm^2
Cohesion	:	40 to 20 kg/cm^2
Internal friction angle	:	40 to 55 degree
Static poisson's ratio	:	0.2 to 0.3

Result of permeability test shows that quantity of water leakage is as large as 23 l/min./m in case of 20 m of water head and at the depth of 8 m. It was therefore judged that both the consolidation and curtain groutings are needed to remedy foundation rock. The consolidation grouting with an interval of 4 m and depth of 5 m and curtain grouting with an interval of 2 m and depth of 15 m will be proposed. In addition, about 50 m long and 30 m wide protection work against slope fall in the talus layer will be needed in the right bank.

5.2 Headrace Tunnel

5.2.1 Geological condition

Geological plan and profile along the tunnel route are shown in Fig. II.1.6.

Gneiss distributes through almost all the tunnel route except for its beginning part where talus deposit exists. Result of geo-surface inspection and analysis of aerophotograph clarifies that three fault sites are found at the crossing parts of tributary and tunnel route. Talus deposit covers the ground surface of the intake portion for 3 alternative damsites.

5.2.2 Engineering geology

Majority of the headrace tunnel route will pass hard and massive gneiss which corresponds to CH to B class of rock classification. It was judged that any fractured zone and water spring do not encounter for tunnel works. However, supporting system will be required for the fault and talus deposit zones in the inlet portion and several portions in the tunnel route. It was estimated that extent of occurrence of fault is 30 m through the tunnel route and that extent of occurrence of talus deposit is 300 m for inlet portion of damsites- A and B and 200 m for damsite - C. Consolidation grouting with an interval of 3 m and depth of 3 m will be needed for fault zones in the tunnel route. The consolidation grouting with an interval of 3 m and depth of 3 m will be also required for fault and talus deposit zone in the inlet portion. The excavated rock can be used for concrete aggregate.

5.3 Surge Tank and Penstock Line

Geological condition of the proposed surge tank and penstock line was interpreted based on geo-surface inspection. Geological plan and profile of these sites are shown in Fig. II.1.6.

5.3.1 Surge tank

The proposed surge tank site is covered with about 10 m thick weathered zone of gneiss and fresh gneiss distributes beneath its zone. It is presumed that this gneiss is hard and massive, which corresponds to B class of rock classification and any fractured zone does not exist. It was therefore judged that there are no technical problems for construction of surge tank shaft, but consolidation grouting with an interval of 3 m and depth of 3 m will be needed.

5.3.2 Penstock line

The proposed penstock line route will pass gneiss layer which is hard and massive and corresponds to B class of rock classification.

It is presumed that any fractured zone does not exist and water spring is not foreseen. It was therefore judged that there are no technical problems for tunnel excavation but consolidation grouting with an interval of 3 m and depth of 3 m will be needed for whole tunnel stretch.

Open air steel conduit type penstock line is conceivable as an alternative plan. However, the ground surface along the penstock line route is covered with about 10 m thick weathered layer and talus deposit consisting of boulder and they have a tendency of sliding. It was therefore judged that open air steel conduit type penstock line is unsuitable.

5.4 Powerhouse and Tailrace

Geological condition in these sites was interpreted based on the result of boring performed at the proposed powerhouse site and geo-surface inspection. Geological plan and profile of the sites are given in Fig. II.5.3.

5.4.1 Powerhouse

The proposed powerhouse site is located in a flat space of river deposit with thickness of about 5 m. Gneiss is underlaid this river deposit. This gneiss is hard rock which corresponds to CH to B class of rock classification in the zone from 5 m to 14 m in depth and B to A class in the zone below 14 m from ground surface. Horizontally closed joints occur at an interval of about 1 m, but any open crack and fractured zone are not found. It is desirable to set foundation of powerhouse at 5 m deep from the ground surface.

5.4.2 Tailrace

The proposed tailrace site is located at a river deposit layer. Since hard gneiss develops at 5 m from ground surface, it is desirable to set the foundation of the tailrace on this hard rock layer.

5.5 Construction Materials in the Project Area

The construction materials survey was carried out by geo-surface inspection in the vicinity of the project area.

5.5.1 Coarse aggregate

A hilly mountain at about 3 km upstream from the left bank of the damsite-C was selected as quarry site for coarse aggregate. Location of the proposed quarry site is shown in Fig. II.1.6.

Outcrop of gneiss appears in the shape of small cliff in this quarry site, and remaining area is covered with about 5 m thick weathered layer. The estimated volume of fresh gneiss was about 200,000 m^3.

5.5.2 Fine aggregate

Borrow area for fine aggregate was investigated along the river course of the Benedito river. Although river and terrace deposits are found in several river stretches, these consist of fine soil (clay and silt are dominant). It will be therefore obliged to produce the fine aggregate by crushing the rock materials in the proposed quarry site.

TABLES

Table II.1.1 WORK QUANTITY OF CORE BORING AND PERMEABILITY TEST

Scheme	Location	Core Boring		Permeability Test
		Hole No.	Depth (m)	(Nos.)
1. Salto Pilão (1)				
	Damsite	B1-1	20	4
	Waterway	B1-2	60	
	Powerhouse	B1-3	30	
2. Dalbergia				
	Damsite	B7-1	30	7
	Waterway	B7-2	75	
	Powerhouse	B7-3	25	
3. Benedito Novo				
	Damsite	B11-1	20	4
	Powerhouse	B11-2	25	
Total		8 holes	285 m	15 nos.

Table II.2.1 GEOLOGICAL STRATIGRAPHY IN THE ITAJAI RIVER BASIN

Geological Age	Name of layer	Lithology (Rock Type)
Quaternary	Alluvial and Colluvial sediment	Clay, Sand, Gravel
Cretaceous Jurassic	Serra Geral formation	Basalt , Diabase (Intrusive Basalt)
Carboniferous	Rio do Sul Itaraje formation	Sandstone, Mudstone, Shale
Precambrian (Protezoic)	Subida Intrusive Bodies	Granite, Diorite
	Campo Aleare formation	Sandstone, Mudstone, Associated with Intrusive Rhyolite
	Gaspar formation	Sandstone, Slate, Hornfels
	Brusque Metamorphic Complex	Phyllite, Schist, Associated with Gneiss, Granite
Precambrian (Archeozoic)	Taboleiro Complex	Gneiss, Granite
	Santa Catarina Complex	Gneiss, Granite

Table II.3.1 ROCK CLASSIFICATION FOR ENGINEERING GEOLOGY

(1) Rock classification

Rock class	Characteristics
A	Hard and fresh rocks. Rock-forming minerals are fresh and not weathered or altered. Joints and cracks are closed tightly, no weathering on their planes. Clear sound is emitted when hammered.
B	Hard and fresh rocks. Rock-forming minerals are weathered slightly or partially altered. Joints and cracks are closed tightly, without weathering. Clear sound is emitted when hammered.
CH	Fairly hard and slightly weathered rocks. Rock-forming minerals, except quartz, are weathered or altered. Tightness of joints and cracks is slightly reduced and each block is apt to be exfoliated along joints and cracks which sometimes contain clay and other materials, stained by limonites. Slightly dull sound is emitted when hammered.
CM	Slightly soft and moderately weathered rock. Rock-forming minerals, except quartz, are weathered or altered. Exfoliation occurs along joint and cracks by hammering. Joints and cracks sometimes contain clay and other materials. Slightly dull sound is emitted when hammered.
CL	Soft and weathered rocks. Rock minerals are weathered. Exfoliation occurs easily along joints and cracks by hammering. Joints and cracks contain clay and other materials. Dull sound is emitted when hammered.
D	Very soft, highly weathered, fractured and/or altered rocks. Rock-forming minerals are highly weathered. Joints and cracks are very loose, easily collapse by weak hammering, which contain clay and other materials. Very dull sound is emitted when hammered.

(2) Engineering properties

Rock class	Compressive strength (qu kg/cm^2)	Modulus of elasticity (Es kg/cm^2)	Modulus of deformation (Ed kg/cm^2)	Seismic velocity (km/sec)	Poisson's ratio
A&B	more than 800	more than 80,000	more than 50,000	more than 3.7	less than 0.2
CH	more than 800 or 800 to 200 or (less than 200)	80,000 to 40,000	50,000 to 20,000	3.7 to 3	0.2 to 0.3
CM	800 to 200 or (less than 200)	40,000 to 15,000	20,000 to 5,000	3 to 1.5	0.2 to 0.3
CL	400 to 200 or (less than 200)	less than 15,000	less than 5,000	less than 1.5	more than 0.3
D	less than 200	less than 15,000	less than 5,000	less than 1.5	more than 0.3

Rock class	Cohesion (kg/cm^2)	Internal friction angle (degree)	Borehole test	
			Modulus of deformation (kg/cm^2)	Modulus of elasticity Es (kg/cm^2)
A&B	more than 40	55 to 65	more than 50,000	more than 100,000
CH	40 to 20	40 to 55	60,000 to 15,000	150,000 to 60,000
CM	20 to 10	30 to 45	20,000 to 3,000	60,000 to 10,000
CL&D	less than 10	15 to 38	less than 6,000	less than 15,000

Notes ;
(1) Compressive strength shows the result of rock piece test.
(2) Figures in bracket show the compressive strength for soft rocks.
(3) Modulus of elasticity and deformation show the results of in situ plate loading tests.
(4) Es means secantial elasticity.

Source ; Standard of Central Research Institute of Electric Power Industry

FIGURES

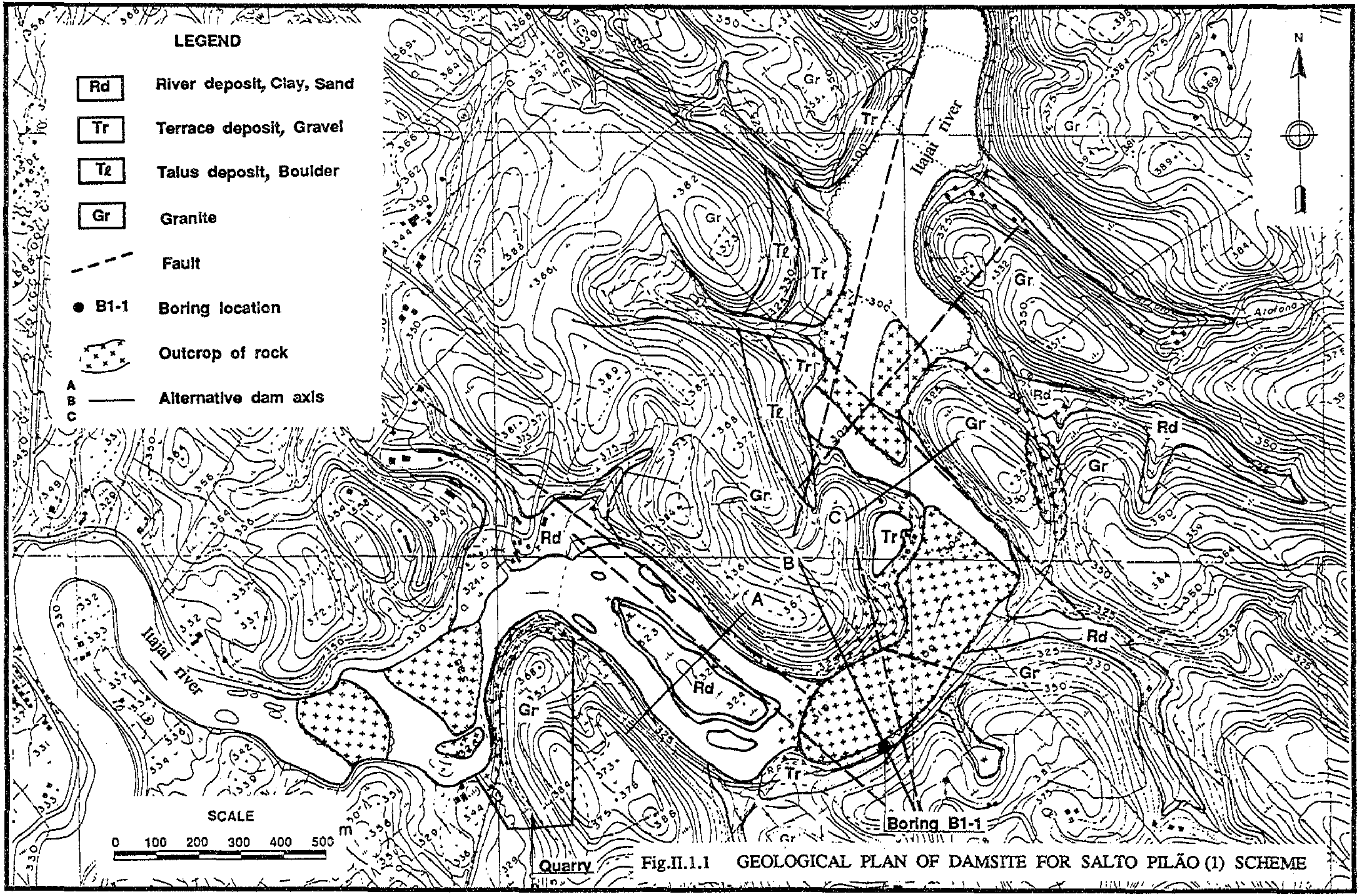
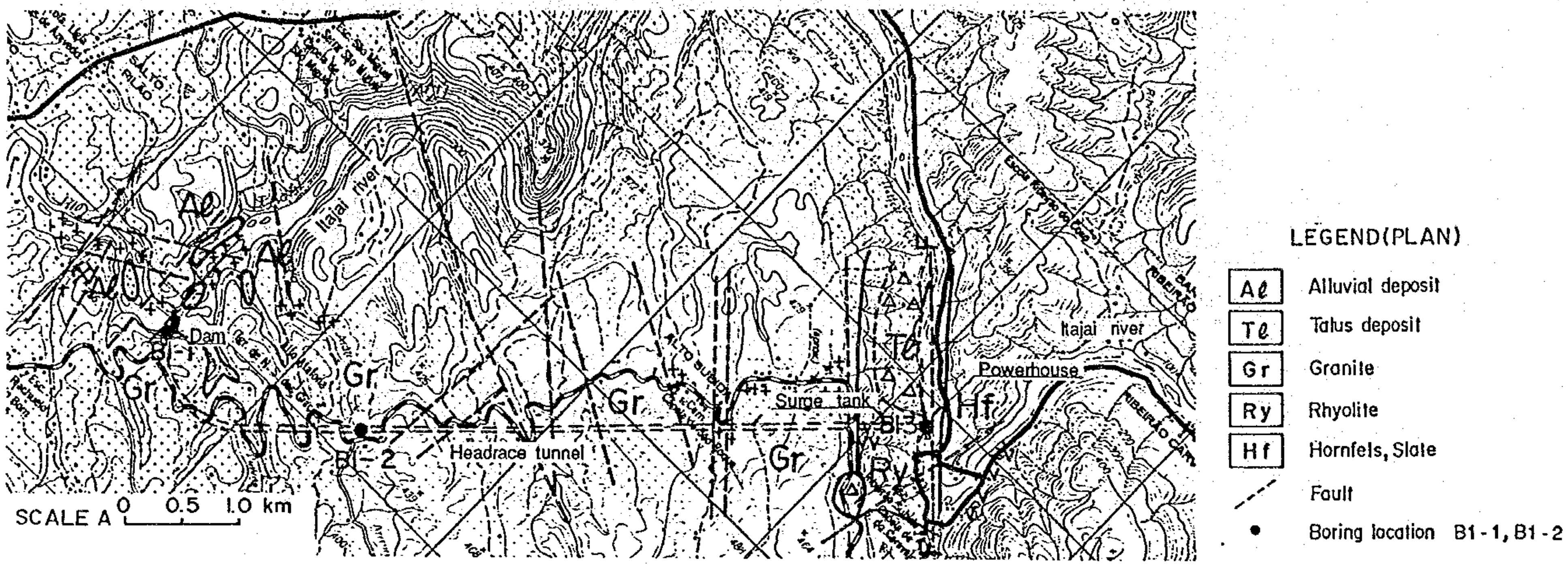
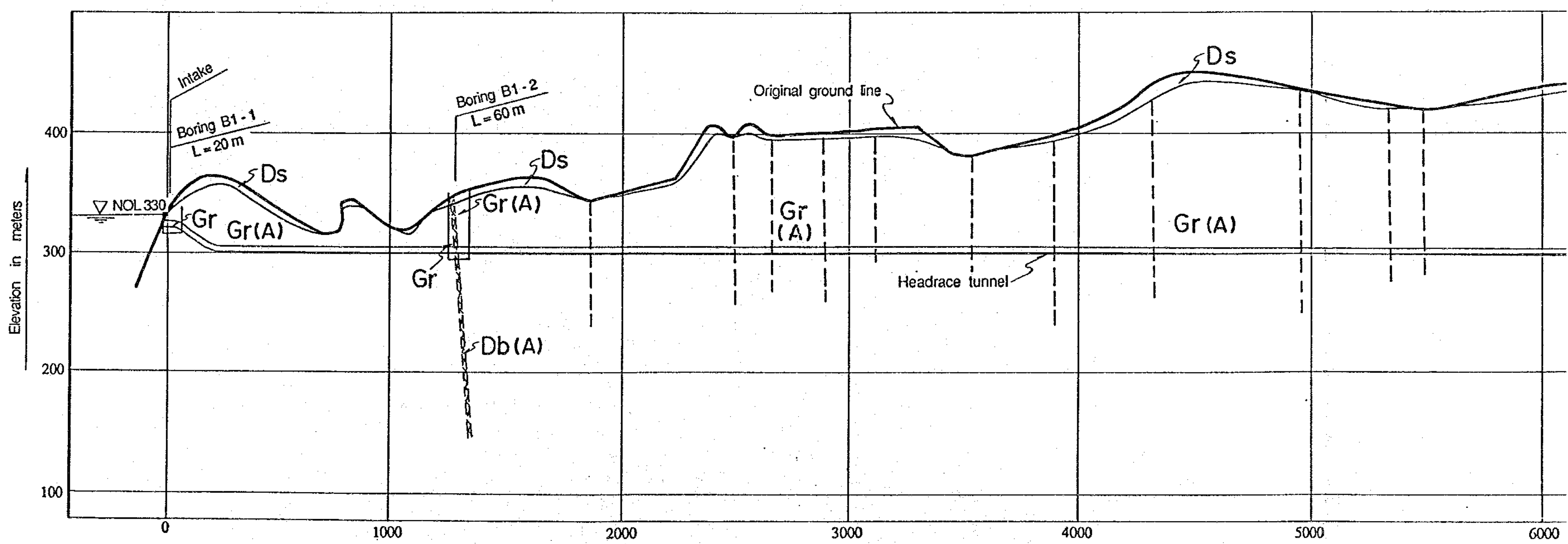


Fig.II.1.1 GEOLOGICAL PLAN OF DAMSITE FOR SALTO PILÃO (1) SCHEME

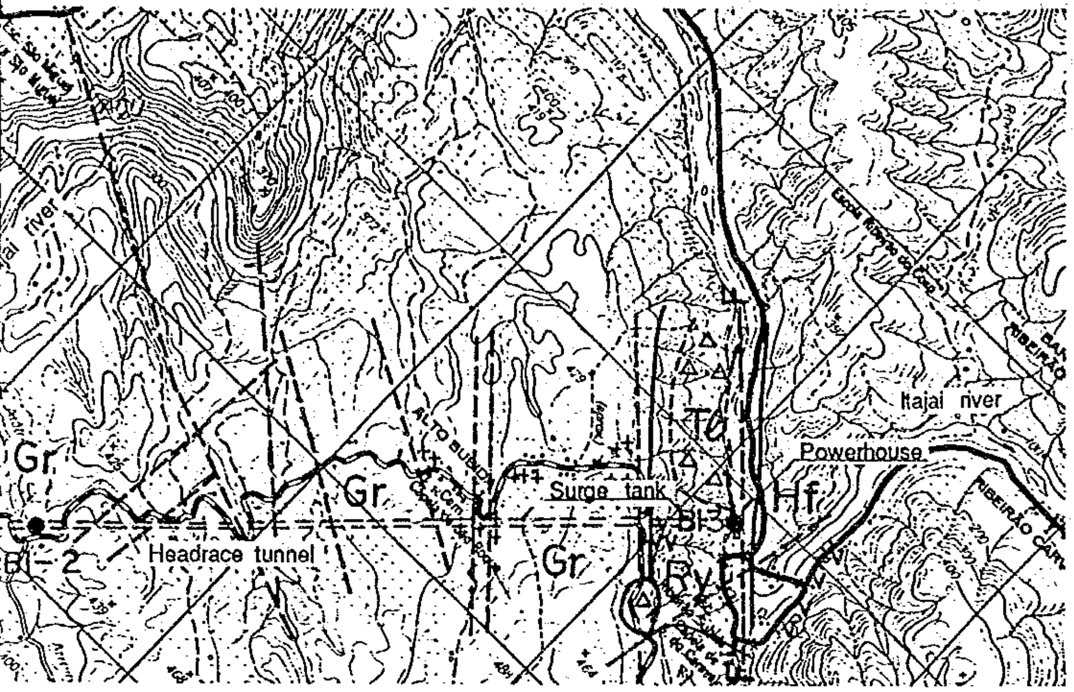


Geological Plan
SCALE A



Geological Profile

Fig II.1.2



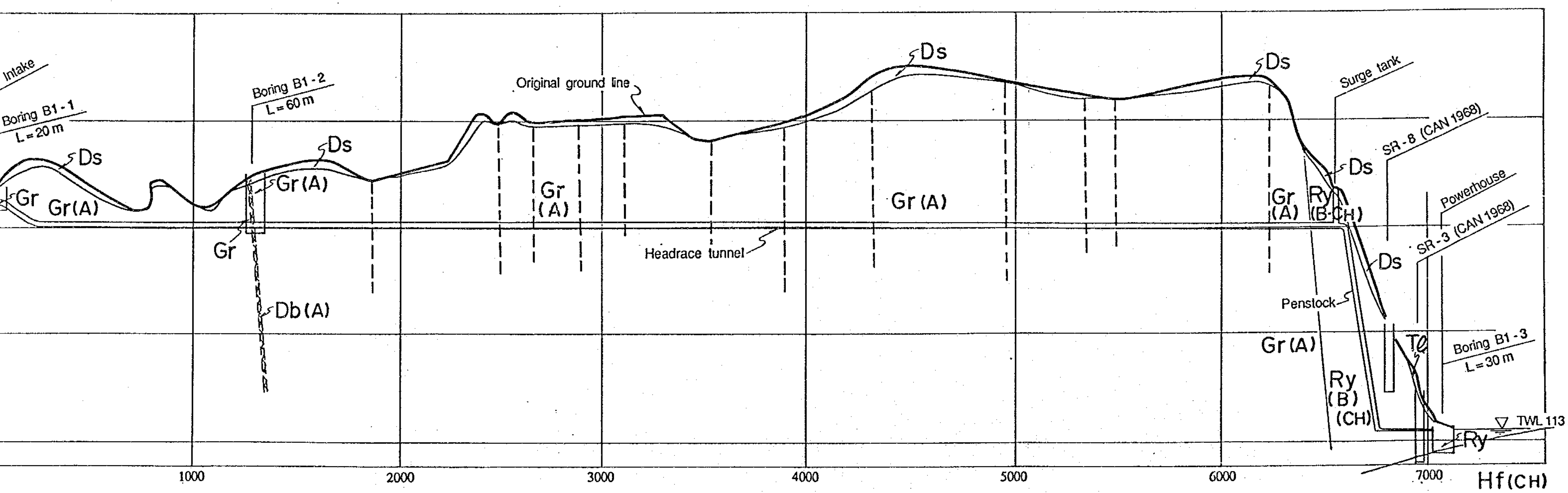
Geological Plan
SCALE A

LEGEND(PLAN)

- Aℓ — Alluvial deposit
- Tℓ — Talus deposit
- Gr — Granite
- Ry — Rhyolite
- Hf — Hornfels, Slate
- Fault
- Boring location B1-1, B1-2

LEGEND(PROFILE)

- Rd — River deposit
- Tℓ — Talus deposit
- Ds — Decomposed rock into soil
- Gr — Granite
- Ry — Rhyolite
- Hf — Hornfels, Slate
- Db — Diabase(Intrusion)
- Fault
- CH,B,A Rock class



Geological Profile

Fig II.1.2 GEOLOGICAL PLAN AND PROFILE OF WATERWAY FOR SALTO PILÃO (1) SCHEME

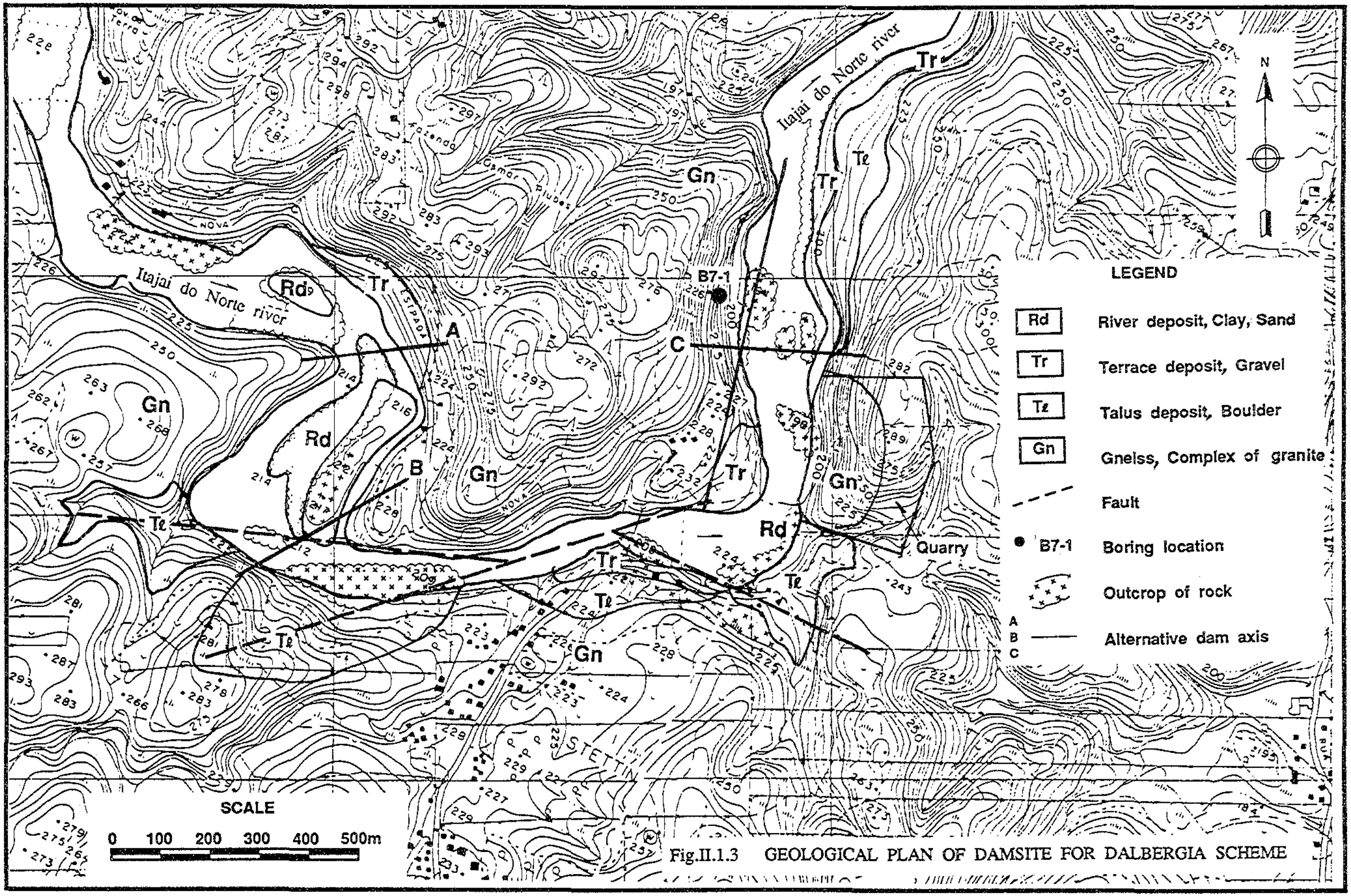
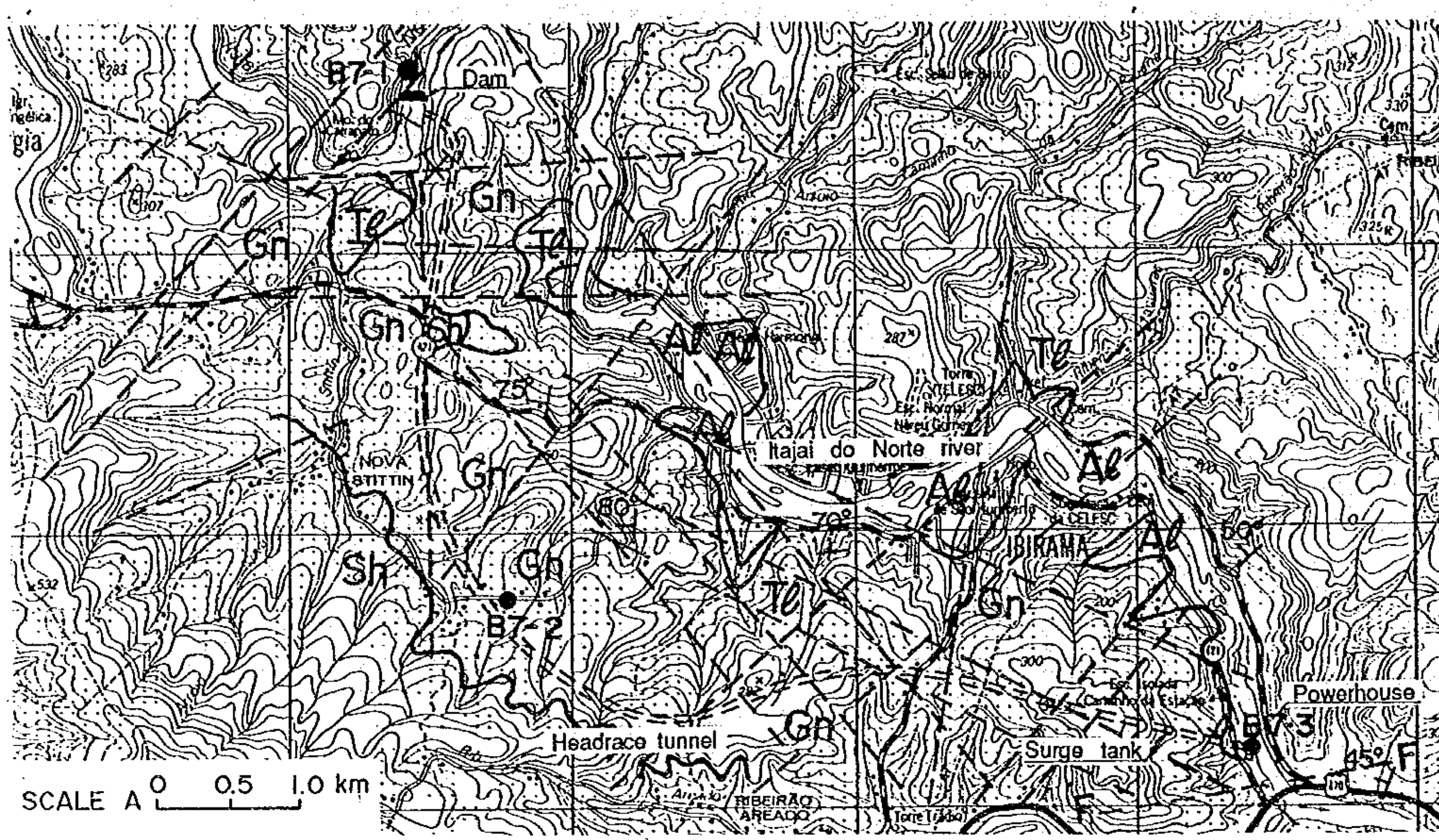


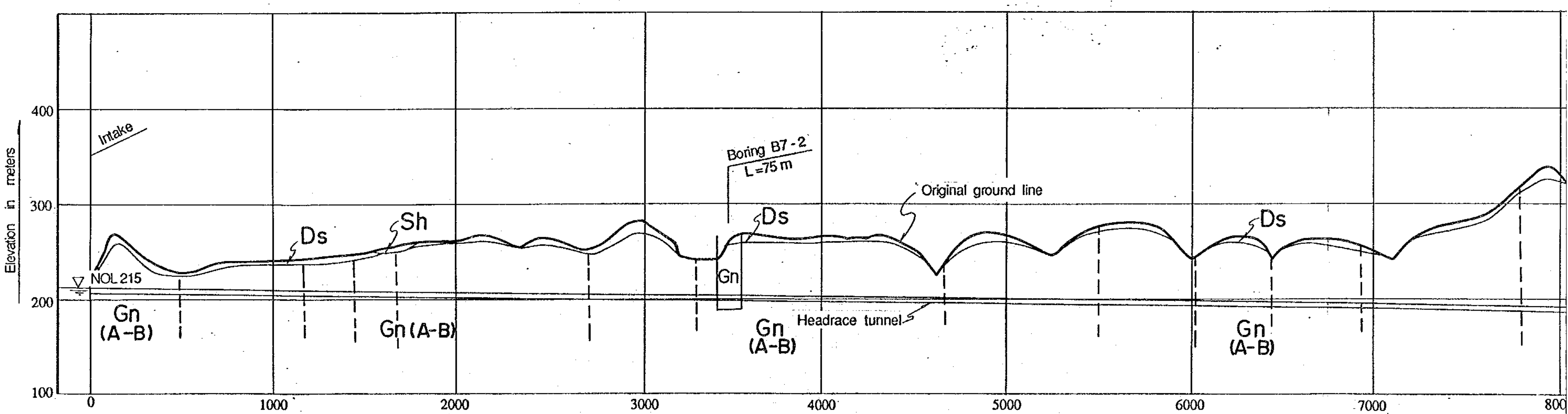
Fig.II.1.3 GEOLOGICAL PLAN OF DAMSITE FOR DALBERGIA SCHEME



LEGEND(PLAN)

Al	Alluvial deposit
Tl	Talus deposit
Sh	Shale
Gn	Gneiss
Hf	Hornfels, Slate
	Fault
•	Boring location B7-1, B7-2, B7-3

Geological Plan
SCALE A



Geological Profile

Fig II.1.4 GEOLOGIC
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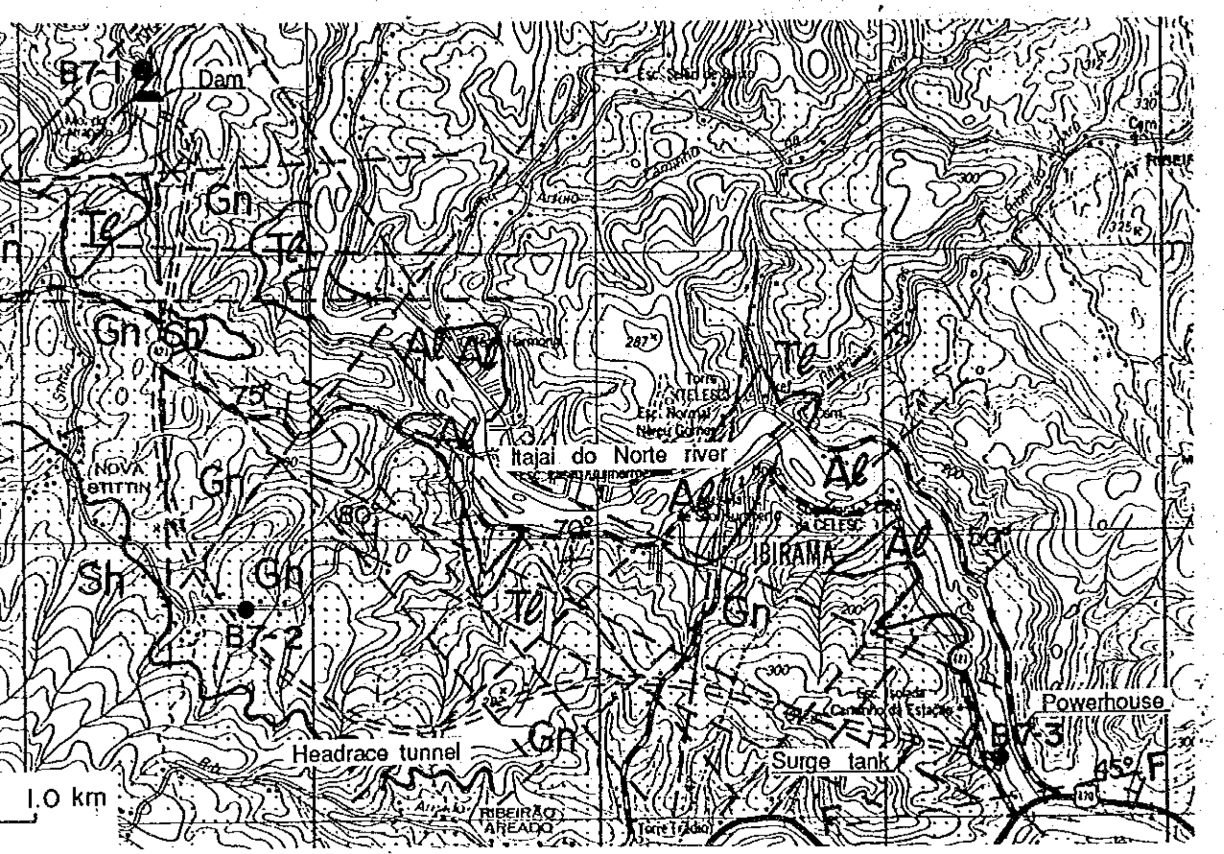


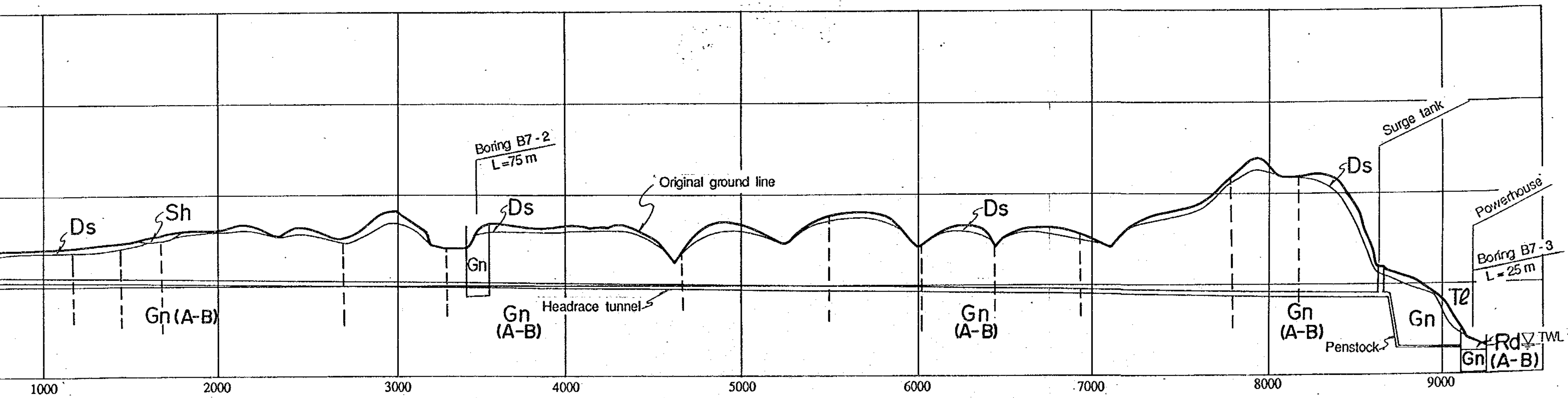
Geological Plan
SCALE A

LEGEND(PLAN)

Aℓ	Alluvial deposit
Tℓ	Talus deposit
Sh	Shale
Gn	Gneiss
Hf	Hornfels, Slate
	Fault
•	Boring location B7-1, B7-2, B7-3

LEGEND(PROFILE)

Rd	River deposit
Tℓ	Talus deposit
Ds	Decomposed rock into soil
Sh	Shale
Gn	Gneiss
A - B	Rock class



Geological Profile

Fig II.1.4 GEOLOGICAL PLAN AND PROFILE OF WATERWAY FOR DALBERGIA SCHEME

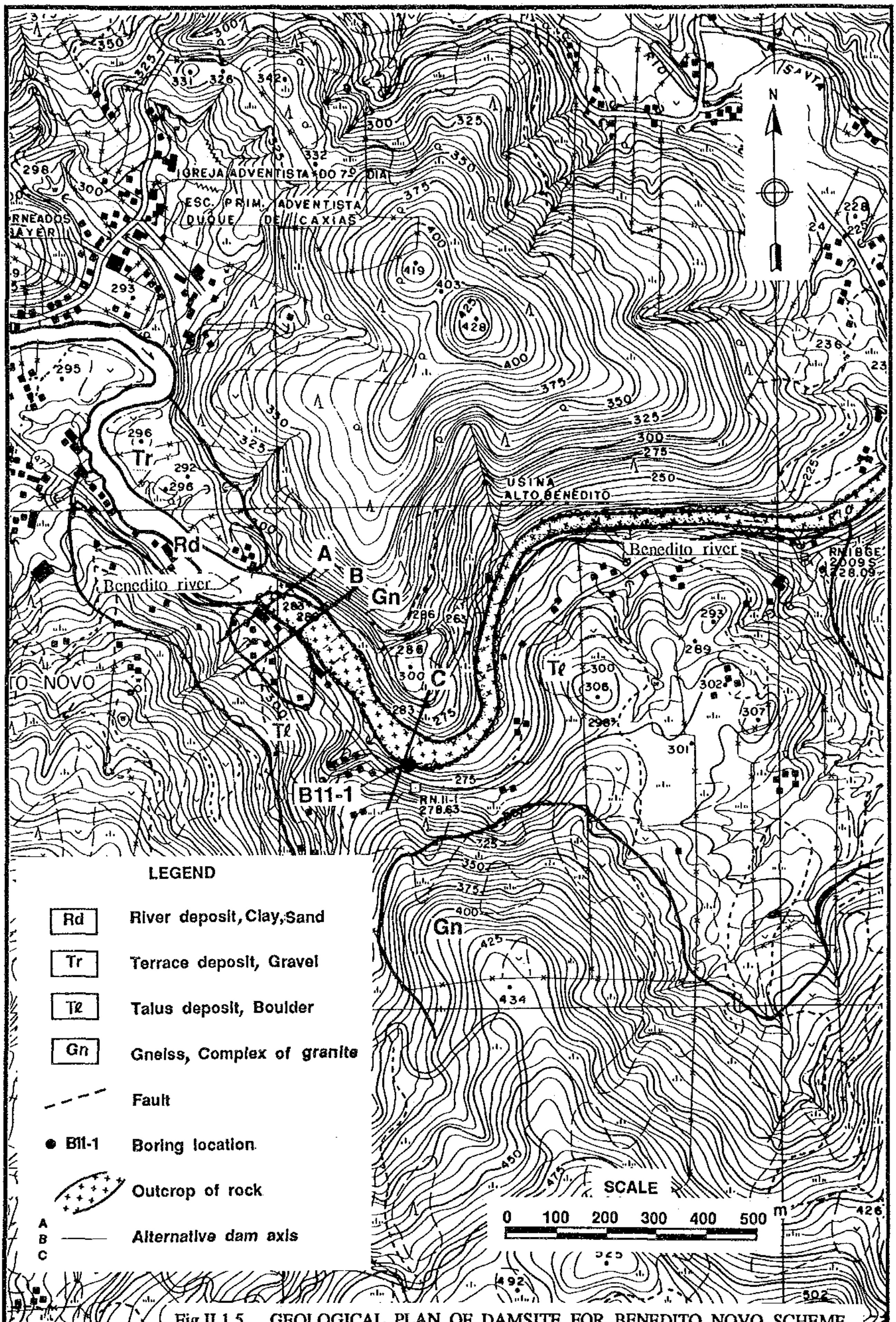
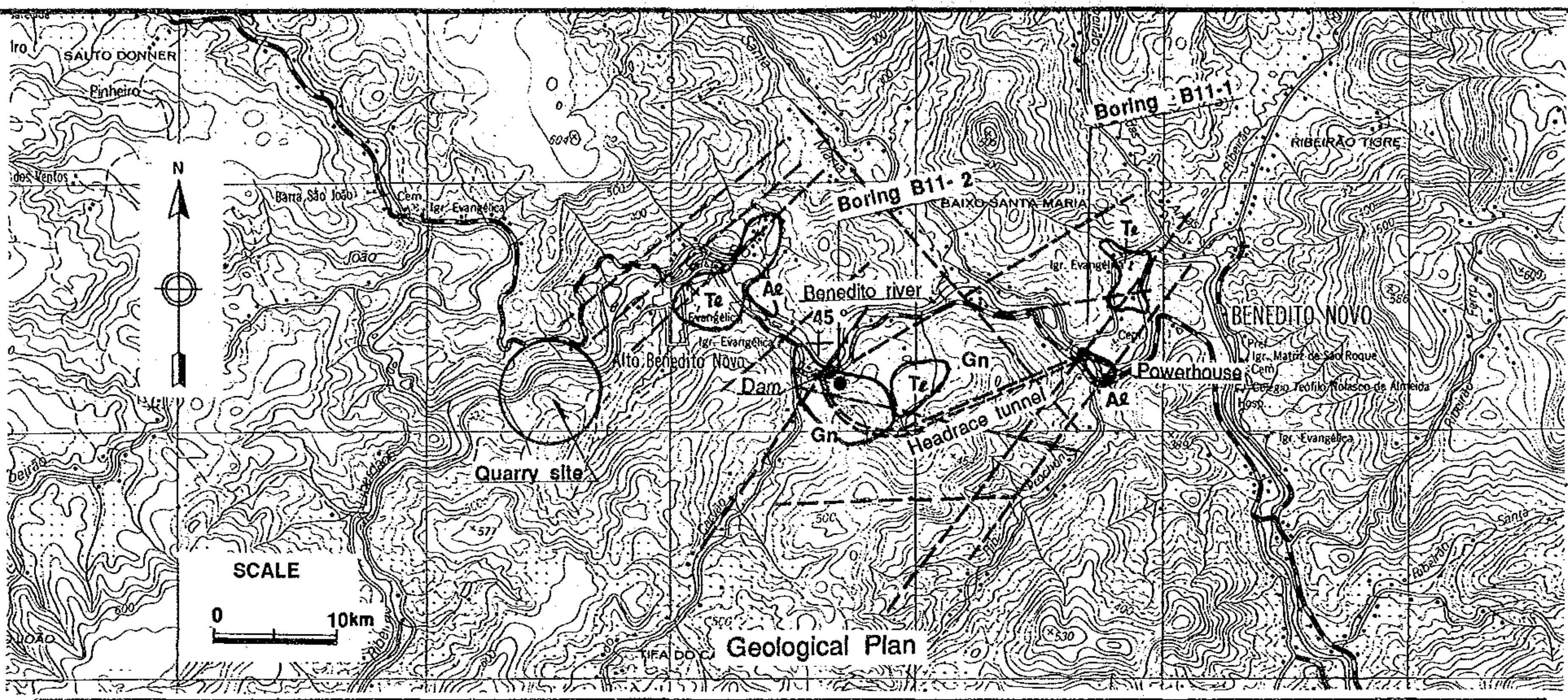
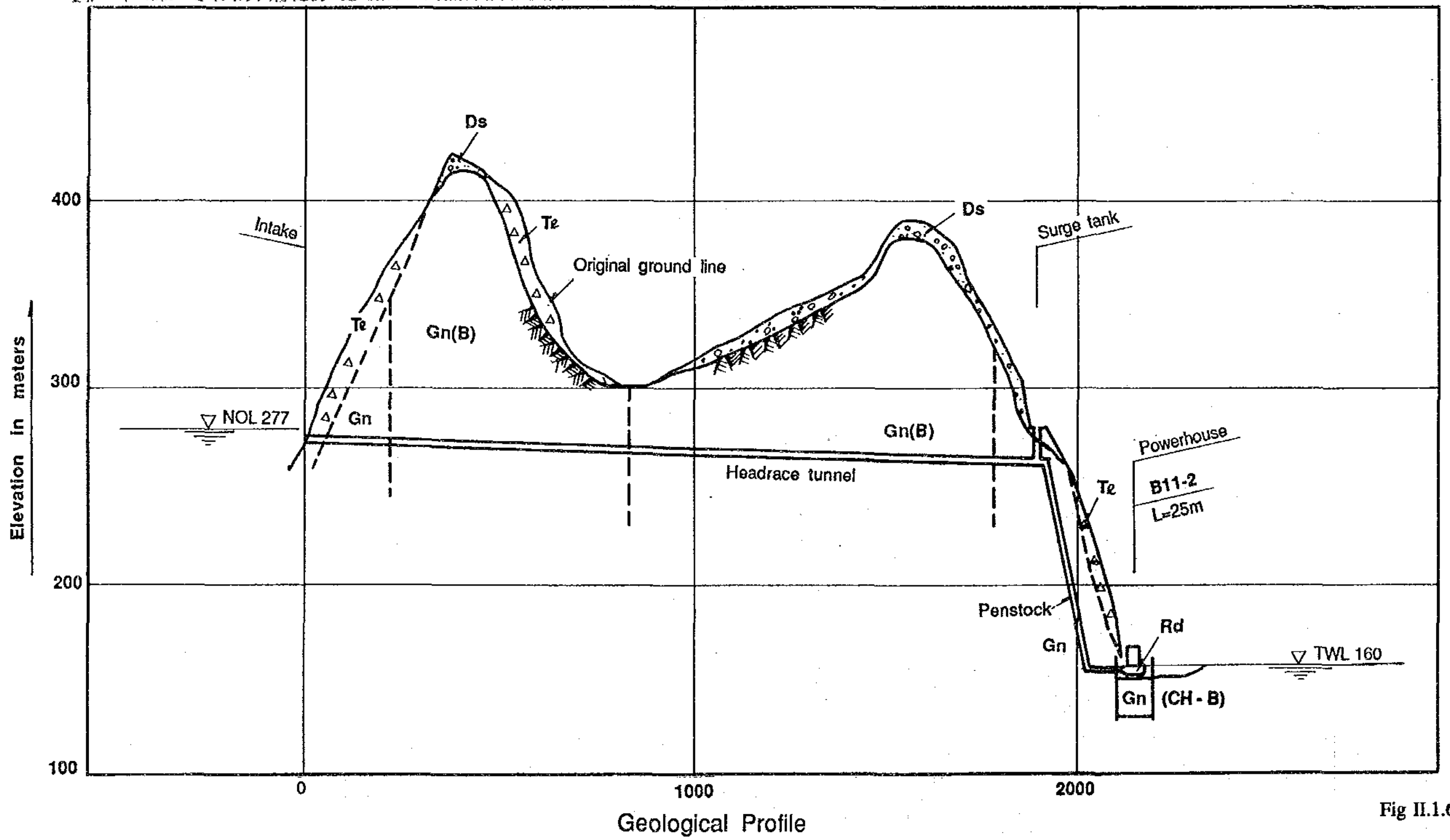


Fig.II.1.5 GEOLOGICAL PLAN OF DAMSITE FOR BENEDITO NOVO SCHEME



LEGEND (PLAN)

Aℓ	Alluvial deposit
Tℓ	Talus deposit
Gn	Gneiss
- - -	Fault
●B11-1	Boring location



LEGEND (PROFILE)

Rd	River deposit
Tℓ	Talus deposit
Ds	Decomposed rock into soil
Gn	Gneiss
- - -	Fault
CH,B	Rock class

Fig II.1.6 GEOLOGICAL PLAN AND PROFILE OF WATERWAY FOR BENEDITO NOVO SCHEME

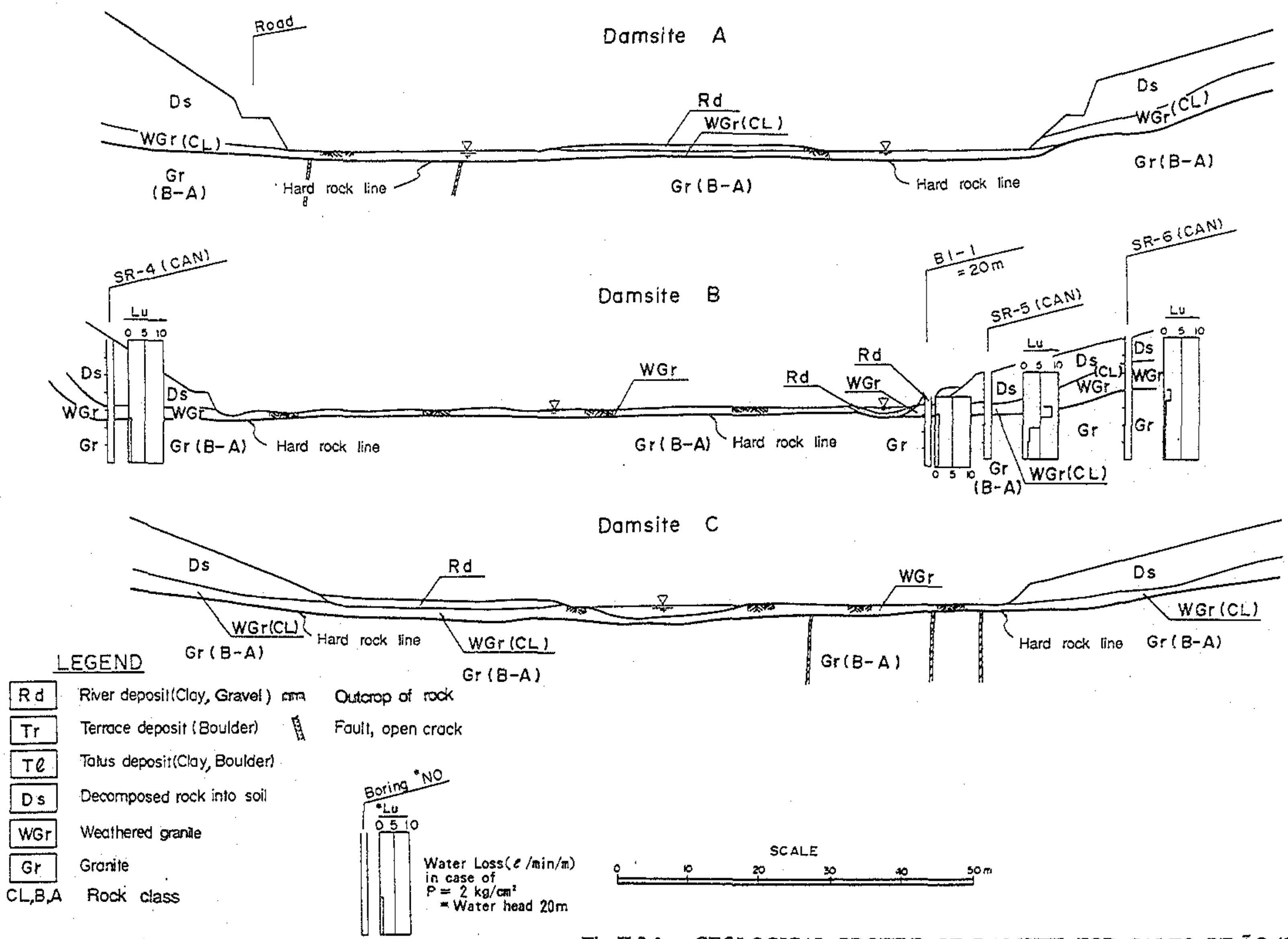


Fig II.3.1 GEOLOGICAL PROFILE OF DAMSITE FOR SALTO PILÃO (1) SCHEME

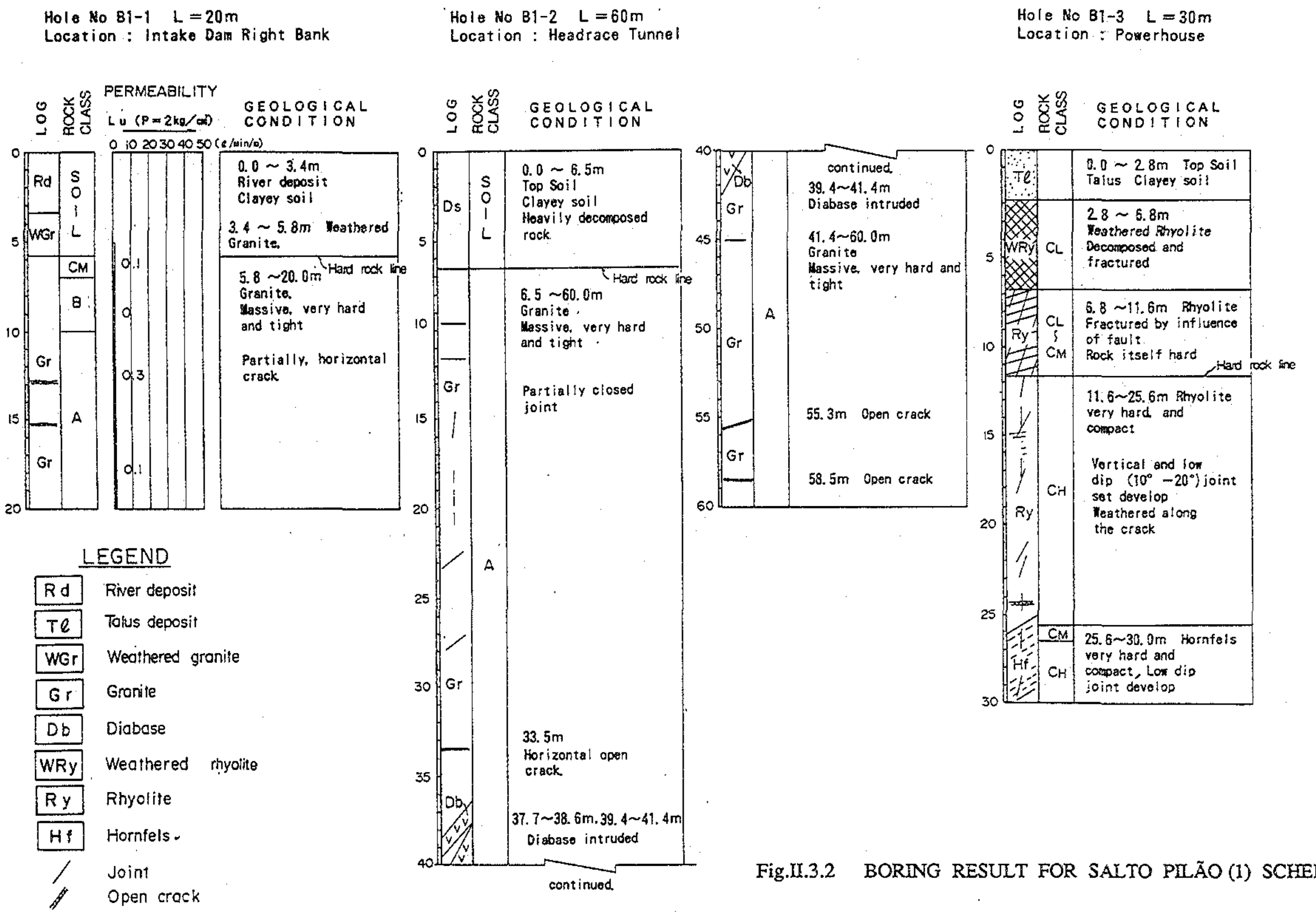


Fig.II.3.2 BORING RESULT FOR SALTO PILÃO (1) SCHEME

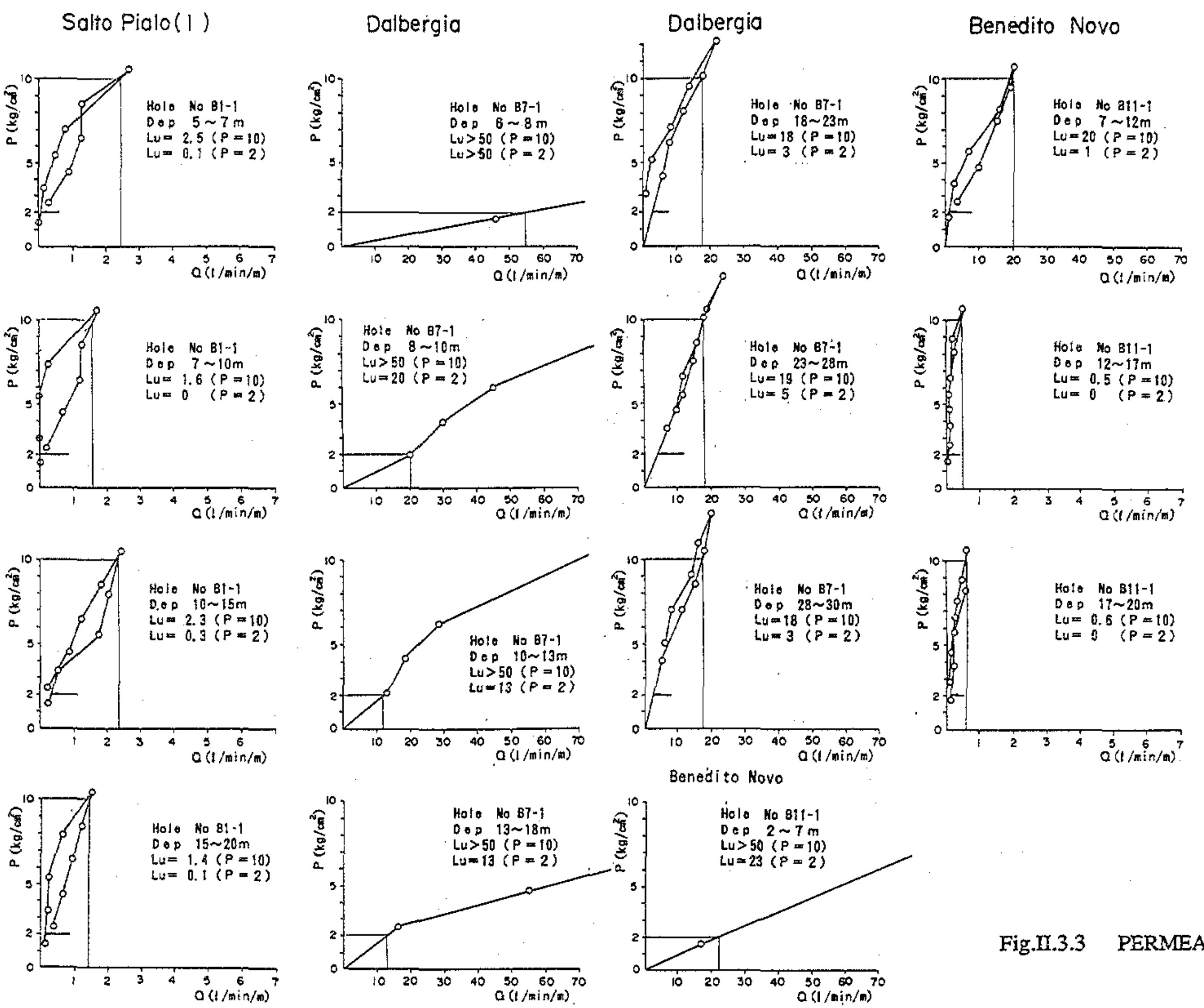


Fig.II.3.3 PERMEABILITY TEST RESULT

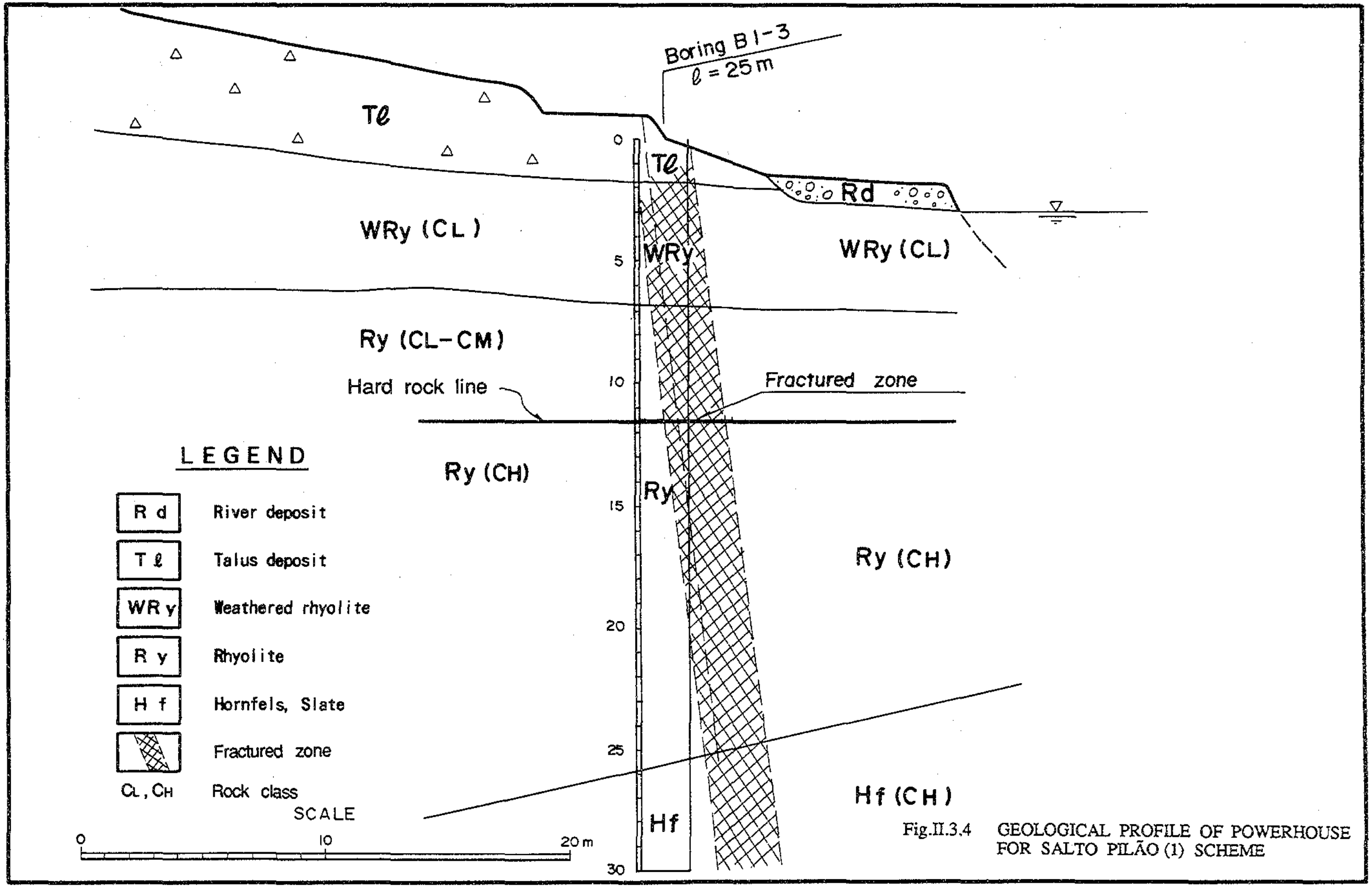


Fig.II.3.4 GEOLOGICAL PROFILE OF POWERHOUSE FOR SALTO PILÃO (1) SCHEME

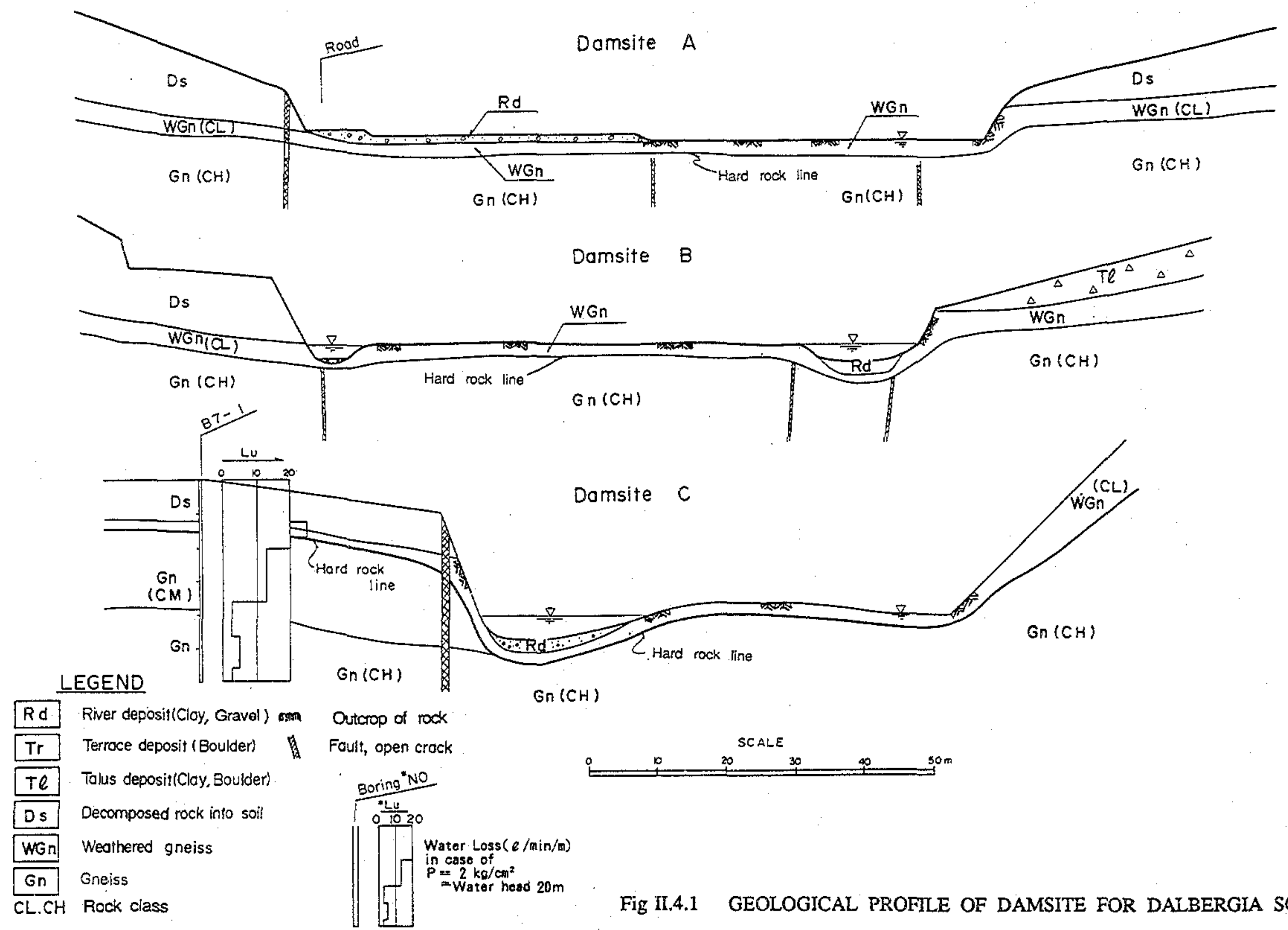


Fig II.4.1 GEOLOGICAL PROFILE OF DAMSITE FOR DALBERGIA SCHEME

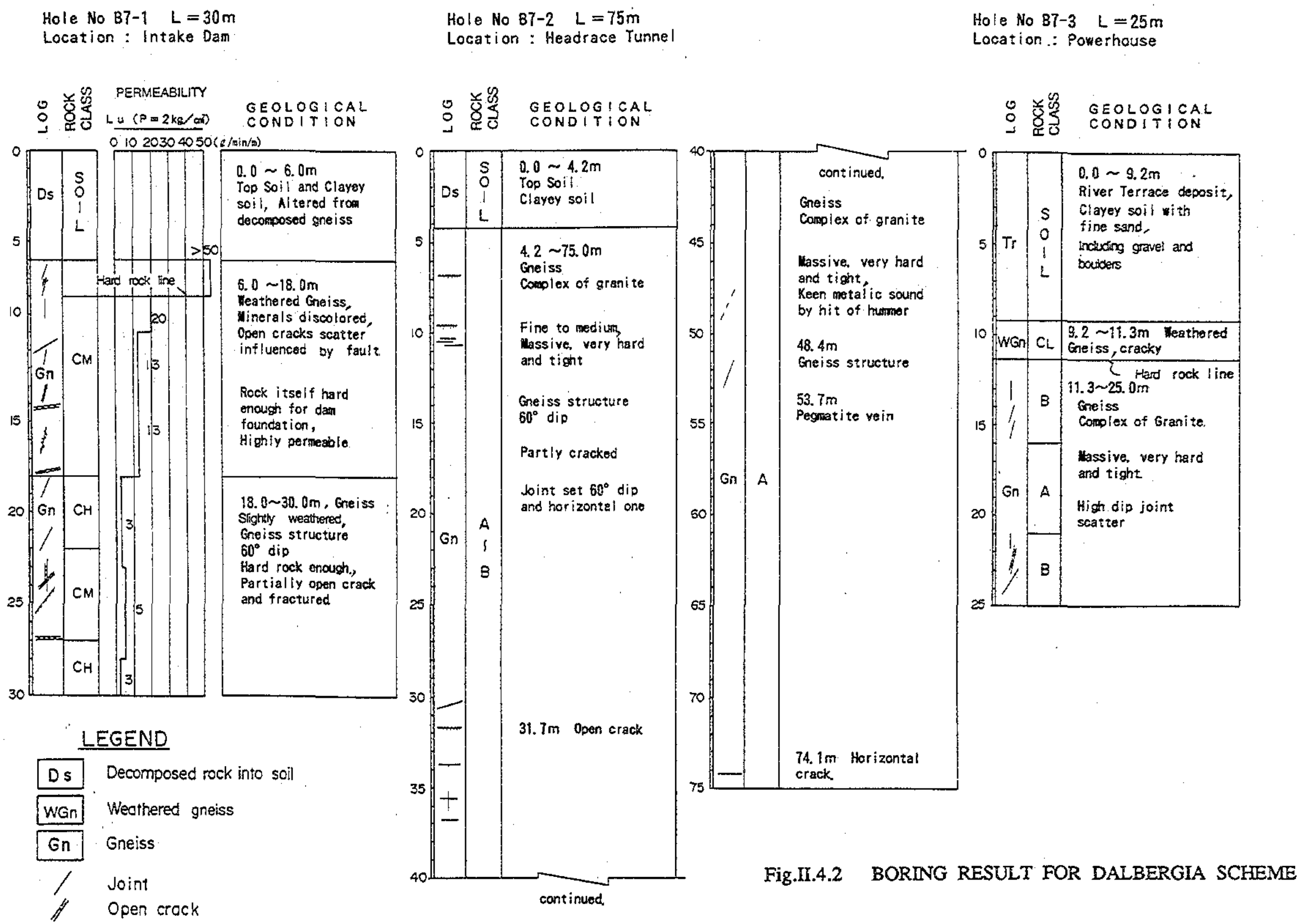


Fig.II.4.2 BORING RESULT FOR DALBERGIA SCHEME

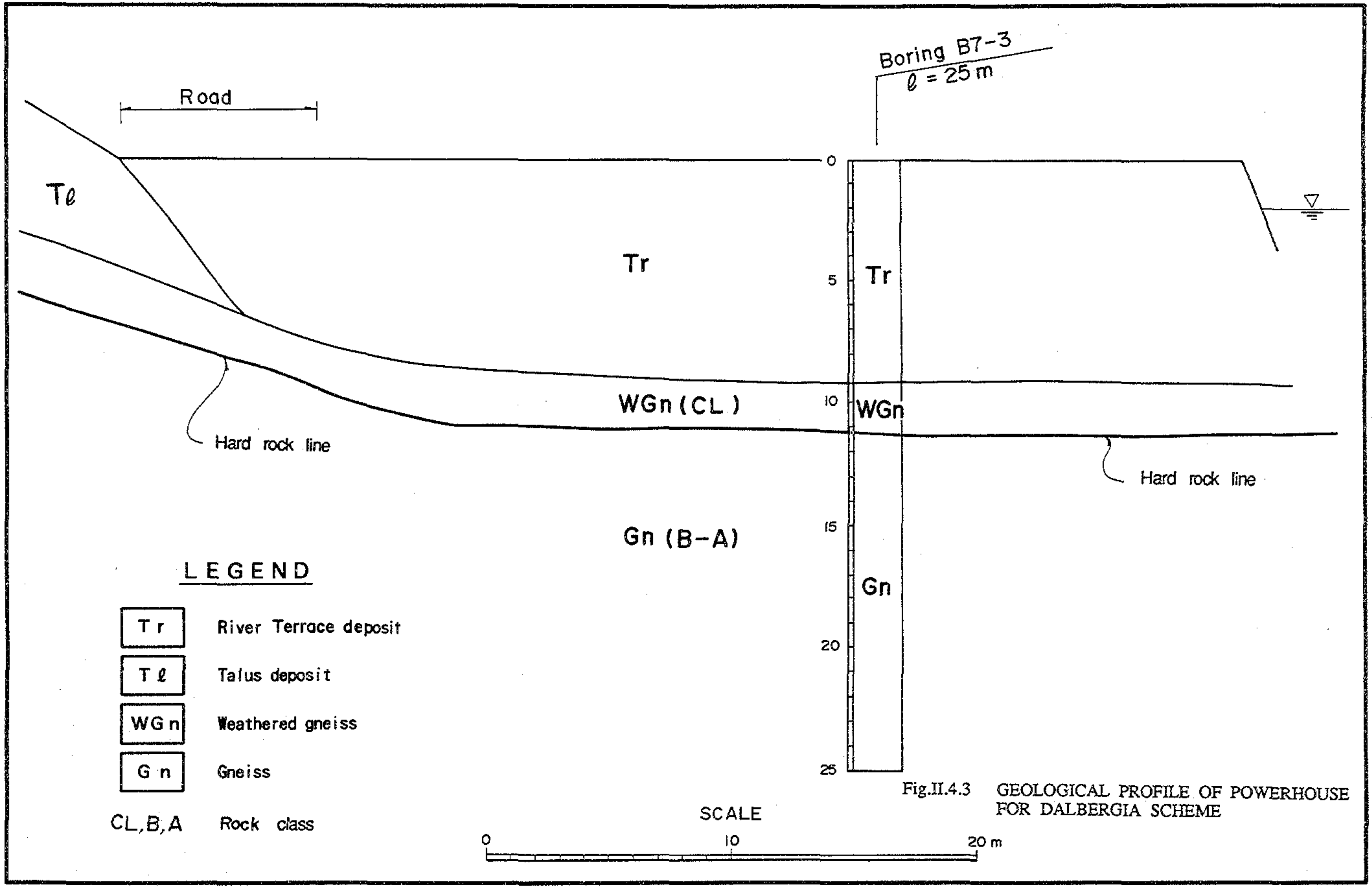


Fig.II.4.3 GEOLOGICAL PROFILE OF POWERHOUSE FOR DALBERGIA SCHEME

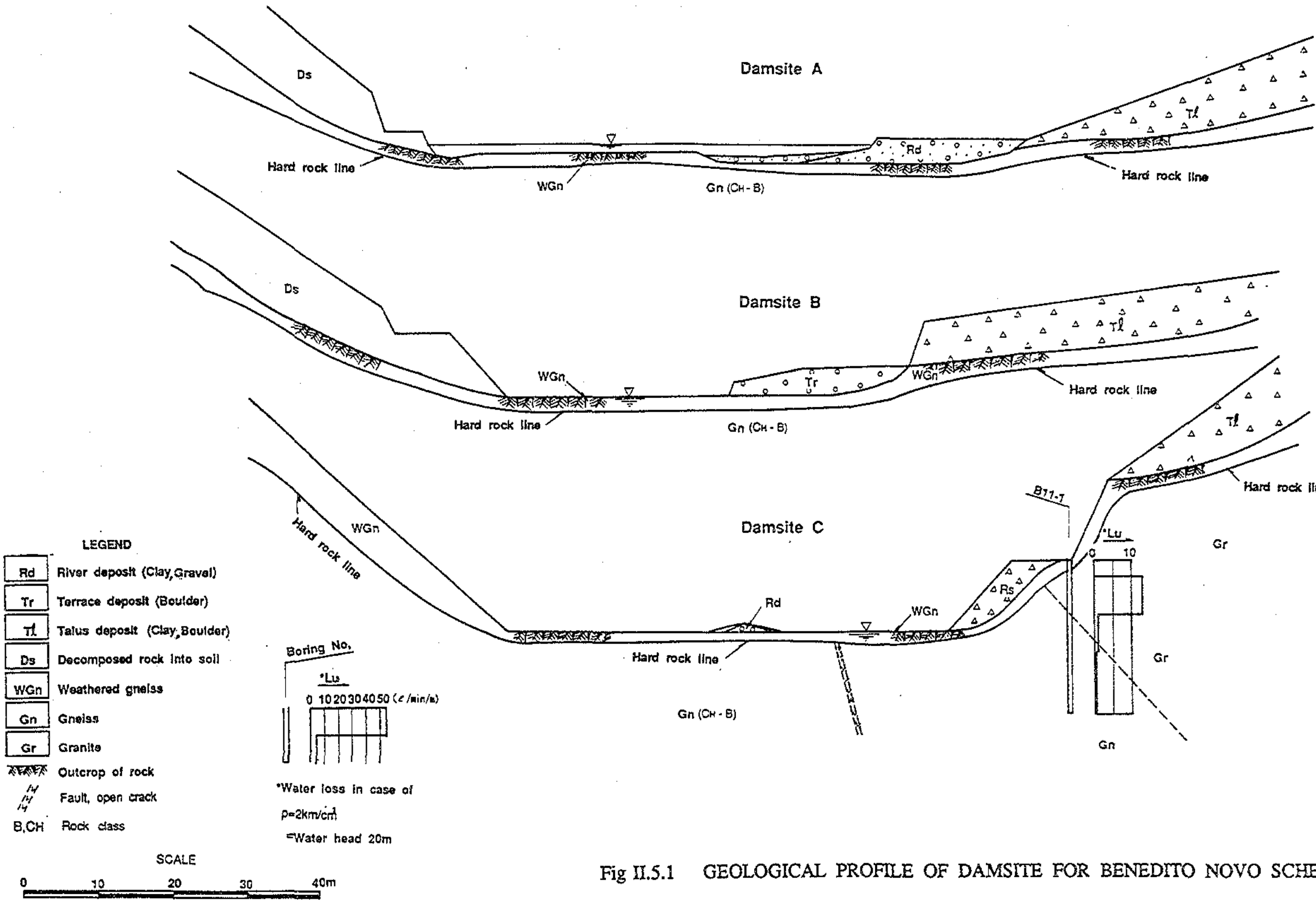


Fig II.5.1 GEOLOGICAL PROFILE OF DAMSITE FOR BENEDITO NOVO SCHEME

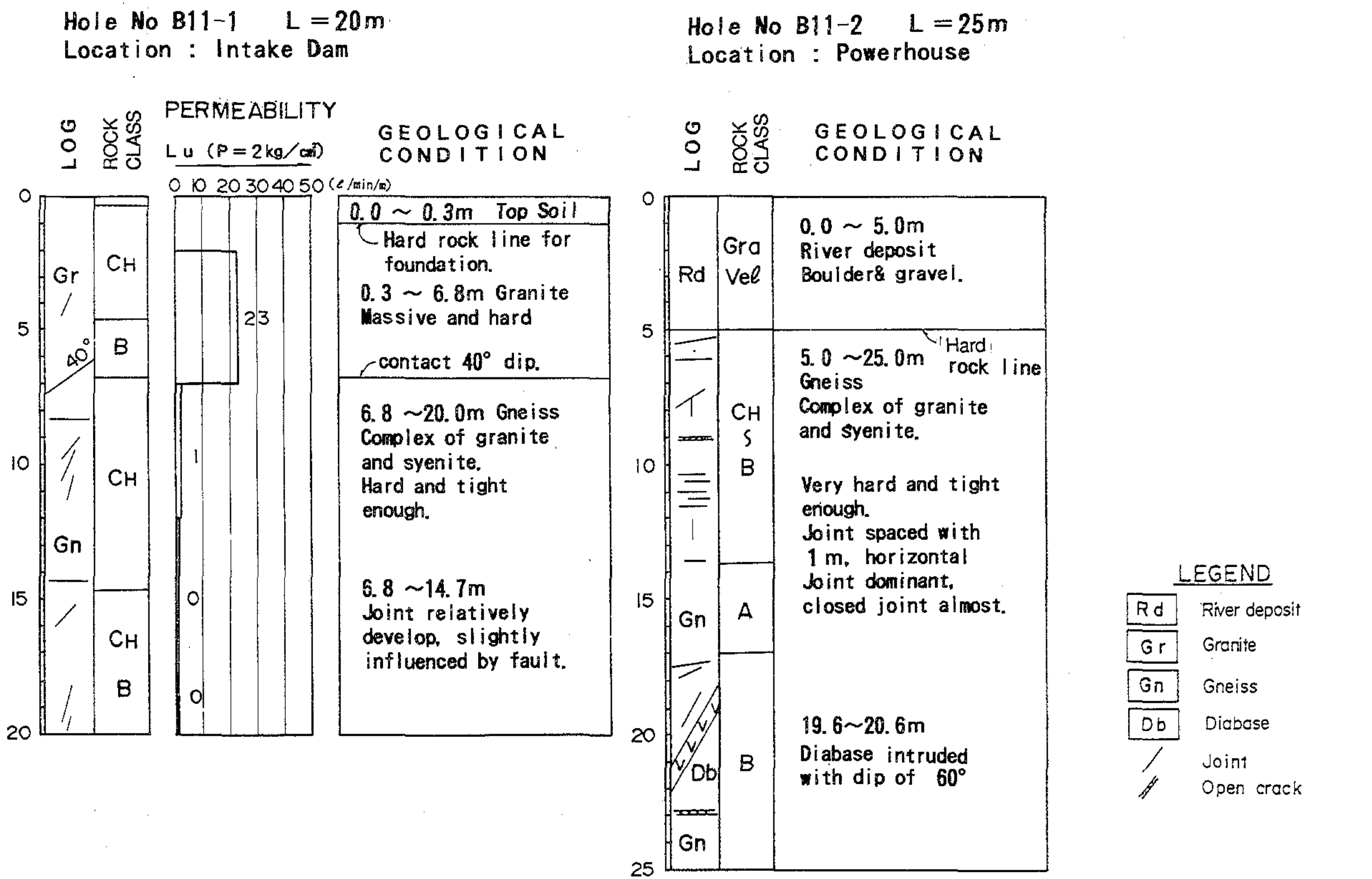


Fig.II.5.2 BORING RESULT FOR BENEDITO NOVO SCHEME

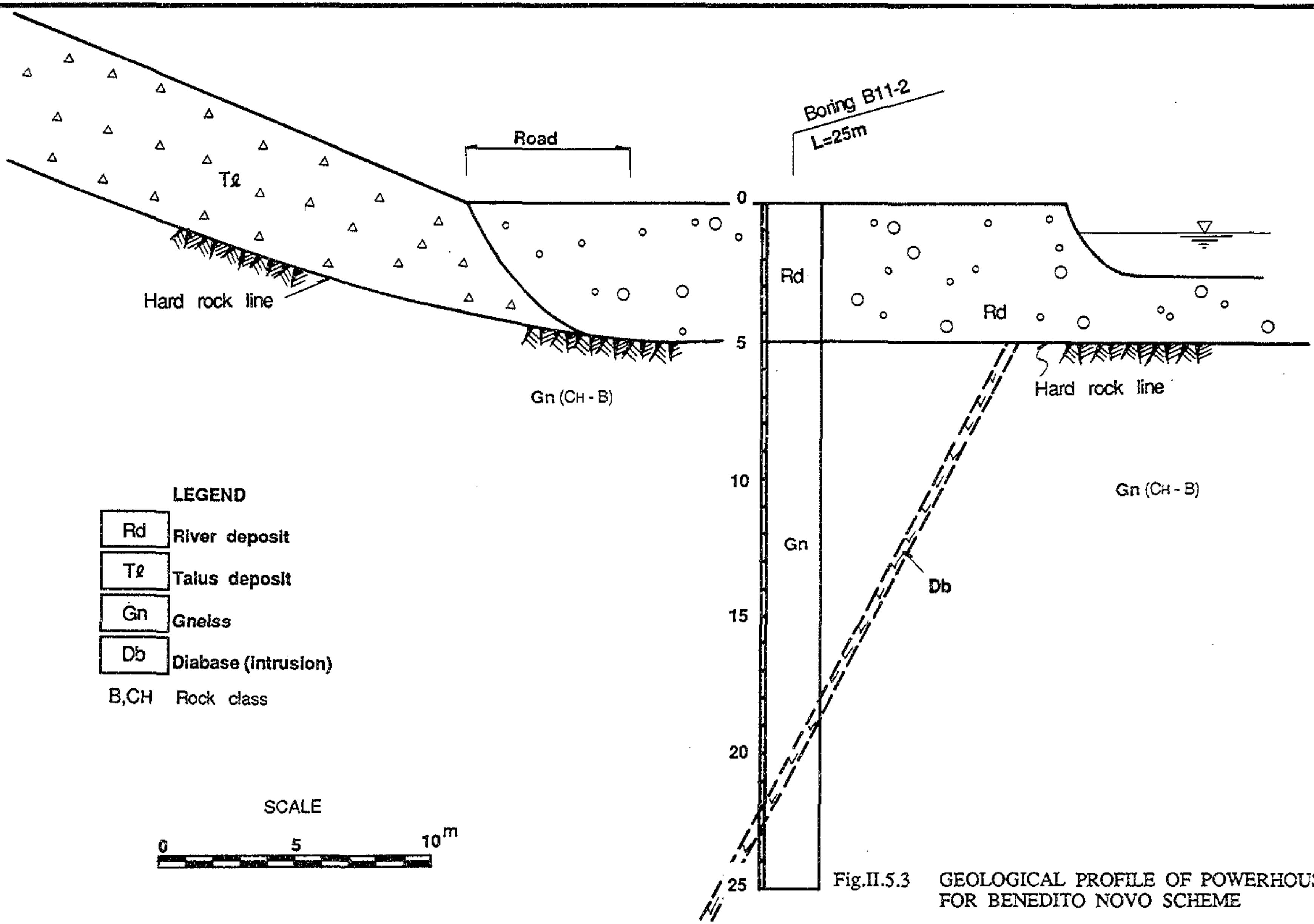


Fig.II.5.3 GEOLOGICAL PROFILE OF POWERHOUSE FOR BENEDITO NOVO SCHEME

ANNEX III

STUDY ON HYDROPOWER DEVELOPMENT

ANNEX III. STUDY ON HYDROPOWER DEVELOPMENT

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1. INTRODUCTION

In the master plan in the first stage, three hydropower schemes, Salto Pilão (1), Dalbergia and Benedito Novo schemes were selected from among 16 identified potential hydropower schemes. In the master plan study, several alternative hydropower schemes were conceived and the project components which were considered to be the most suitable were selected based on the result of geo-surface inspection.

In this Annex III several alternatives for power facilities for the selected 3 hydropower schemes were conceived and the most appropriate development scale was selected through the optimization study, and design of the optimized scale of the power facilities was made on pre-feasibility level. These studies were carried out based on the topographic maps at a scale of 1:10,000 and contour interval of 5 m, which were surveyed in this stage and also referring the results of geotechnical investigation by means of core boring and geo-surface inspection for major power facility sites.

This report presents the result of pre-feasibility study on 3 hydropower schemes and describes the following items;

- Procedure of study
- Study on Salto Pilão (1), Dalbergia and Benedito Novo hydropower schemes
- Construction plan and cost estimate for Salto Pilão (1), Dalbergia and Benedito Novo hydropower schemes

2. PROCEDURES OF STUDY

2.1 Study Procedures

The selected 3 hydropower schemes are all run-of-river type utilizing fully the head available in the rapid river stretch. In the master plan stage, the most appropriate damsite and powerhouse site were selected based on the topographic maps at a scale of 1:50,000 with a contour interval of 20 m and at a scale of 1:10,000 with contour interval of 10 m only for river stretch. According to the topographic maps at a scale of 1:10,000 with a contour interval of 5 m, which were prepared in the second stage, several alternative dam axes are conceivable though the powerhouse site is topographically limited. Then the study on the hydropower schemes was performed by two steps, namely, selection of dam axis and optimization study and pre-feasibility grade design for the project components.

The items of the hydropower schemes are listed as follows;

(i) Selection of dam axis
(ii) Optimization study, and pre-feasibility grade design for the project components
(iii) Assessment of power output and energy
(iv) Planning of construction works and cost analysis
(v) Estimate of work quantity and construction cost

These study procedures are illustrated in Fig. III.2.1. Details of the study procedures are present in the followings.

2.2 Concept for Hydropower Development Plan

The composition of power plants in whole Brazil power system in 1989 based on energy sources was 91 % for hydropower, 3 % for oil fired thermal, 2 % for coal thermal,1 % for nuclear and 3 % diesel and other power plants. The share of 91 % for the hydropower plants to the total installed capacity is remarkably higher than that of general power system in other countries and such high share will last for a long period at least up to 2010.

It is a general practice of power system to operate nuclear and coal thermal power plants for base power supply and hydropower plants and gas turbine and diesel power plants for peak power supply. While, in the south/southeast power system, run-of-river type hydropower plants are mainly operated for base power supply together with nuclear and coal fired thermal power plants and reservoir type hydro plants are operated for peak power supply.

The relationship curves between the power demand forecast and power supply under power expansion program of the south-southeast system for the power and energy show that the rate of reserve for energy is smaller than that for power. It implies that the power system should develop power plants for energy supply, namely base power supply.

The existing 12 hydropower plants owned by CELESC which are all the run-of-river type are operated to supply about 6 % of the total energy required by CELESC with constant load or scheduled constant loads corresponding to the daily river flow for base power supply in combination with the power received from ELETROSUL.

In this circumstances, it is considered that the hydropower plants to generate a cheaper electric energy and to supply base power to the CELESC power system together with the existing CELESC's hydropower plants should be planned in the Itajai river basin.

2.3 Assumption and Conditions for Study

2.3.1 Discharge for power generation

The development ratio is defined as the ratio of average turbine discharge to maximum plant discharge. The optimum scale of the power facilities in the first stage was determined based on this development ratio. The optimum scale of the 3 hydropower schemes was decided based on a development ratio of 0.7 for Salto Pilão (1) and Dalbergia schemes and 0.6 for Benedito Novo scheme.

It was considered by power study that the hydropower plants to be developed in the Itajai river basin should be those to generate cheaper electric power energy and to supply base power to CELESC power system. The hydropower schemes with facilities to supply the cheapest electric power energy were selected in the first stage based on the foregoing development ratio. Thus an optimization study on project components was carried out based on the discharge with the same development ratio as applied in the master plan in the first stage. The relationship among the average turbine discharge, maximum plant discharge and the development ratio is shown in Fig. III.2.2 and summarized as follows;

Name of scheme	Average turbine discharge (firm discharge) (m3/s)	Maximum plant discharge (m3/s)	Development ratio
Salto Pilão (1)	50.3	71.9	0.7
Dalbergia	19.3	27.6	0.7
Benedito Novo	8.4	13.9	0.6

2.3.2 Criteria for study

In order to formulate the hydropower development plan, the power supply system of ELETROSUL, which supplies more than 90 % of the power demand in Santa Catarina state through CELESC, should be taken into account. The power supply system in the south region is interconnected with the power supply system in the southeastern region. The power output and energy were therefore calculated based on the criteria specified by ELETROBRAS.

The south/southeastern power supply systems are mainly composed of hydropower plants (92 % of the total installed capacity). This implies that power generation depends largely on the hydrological conditions in the regions. Accordingly ELETROBRAS established the following criteria;

(1) The firm energy will be approximated to the average energy generated during hydrologically critical period in the interconnected system.

(2) The hydrologically critical period in the interconnected system is defined as the period from April 1949 to November 1956 as illustrated in Fig. III.2.3, in which the ordinate is the total monthly power output (MW) equivalent to reservoir storage for all the existing hydropower plants and promising hydropower projects in the interconnected system and the abscissa shows the period from 1930 to 1982.

(3) The guaranteed energy is defined as the mean energy generated in the plant during the critical period of the 1,000-year synthetic flow plus a proportional part of the power deficit in the system, and it is expressed as follows;

$$Eq = Gi + Gi \times Ds/Gs,$$

$$Ds = Os - Gs$$

where;

	Eq	;	Guaranteed energy (MW)
	Gi	;	Medium energy generated in the critical period (MW)
	Ds	;	Medium deficit of the system in the critical period (MW)
	Gs	;	Medium energy generated in the system in the critical period (MW)
	Os	;	Supply of energy in the system (MW)

According to the power calculation in the interconnected system of the south and southeastern regions, the relationship among firm energy based on the critical period, mean energy probably occurring less than 5 % based on 1,000-year synthetic analysis and estimated guaranteed energy is calculated as shown in Fig. III.2.4. Based on this figure, the guaranteed energy can be defined as 90 % of the firm energy.

(4) The secondary energy is defined as the energy producible in excess of the firm energy and it is usually calculated as the difference between the long term average energy and firm energy.

(5) The economic viability of a hydropower project in the interconnected system is analyzed by comparing the "unit cost of guaranteed energy" of the project with the "marginal cost of expanded energy".

The cost of the guaranteed energy is obtained by the following expression:

$$CEUG = \frac{CIA - 8{,}760 \cdot CRES \cdot ES - 1{,}000 \cdot CMP \cdot PG}{8{,}760\ EG}$$

where;	CEUG	;	Unit cost of guaranteed energy in US$/MWh
	CIA	;	Annual equivalent cost, in US$; corresponds to the total investment cost multiplied by capital recovery factor for a useful life of 50 years at 10 % per annum (0.1009)
	CRES	;	Reference cost of secondary energy, in US$/MWh; is considered to be fuel cost of 10 US$/MWh; which is estimated as the cost of weighted mean of fuel for coal, gas, oil and nuclear
	ES	;	Secondary energy, in MW
	CMP	;	Marginal cost of peak, in US$/MW
	PG	;	Guaranteed peak of power plant, in MW
	EG	;	Guaranteed energy in MW on an average

In this expression, the marginal cost of peak, CMP is regarded as null due to the following reason;

The power supply in the interconnected systems of the south and southeastern regions will be composed mainly of the majority of hydropower plants and several thermal plants. Power generation is, therefore, subject to hydrological conditions in the system area. According to the past power output recorded, the power energy does not always increase compared with extent of power installation, in other words, it may be said that there is at present excess power capacity. In view of these conditions, the marginal cost of peak is regarded as null.

The marginal cost of expanded energy of the system, which actually represents a composition of unit cost of the guaranteed energy for every five years was revised in May 1991. The revised marginal cost is as follows;

Five-Year Period	Marginal Cost of Expanded Energy (US$/MWh)
1991 - 1995	45
1996 - 2000	48
2001 - 2005	58
2006 - 2010	71
2011 onward	86

2.4 Selection of Dam Axis

In addition to the dam axis proposed in the master plan study in the first stage, two dam axes were contemplated in this study. In order to select the most appropriate dam axis among these three alternative dam axes, comparative study was made from two aspects, i.e. technical and economic aspects and environmental aspect. Economic evaluation was made by means of comparison of unit cost of the guaranteed energy, which is the criteria specified by ELETROBRAS.

2.5 Optimization Study and Pre-feasibility Grade Design

An optimization study on project components was carried out based on the same development ratio as applied for the master plan in the first stage. The items to be studied and designed on pre-feasibility basis are as follows;

(i) Method of river diversion and its scale
(ii) Type and scale of dam
(iii) Type and scale of spillway
(iv) Route of waterway and its dimension
(v) Type and dimension of surge tank
(vi) Type and dimension of penstock line
(vii) Type of powerhouse and tailrace and their dimensions
(viii) Type of generating equipment and number of unit

(ix) Power transmission method and its line route

2.6 Assessment of Power Output and Energy

Based on the dimension of the optimized project components, installed capacity and firm and secondary energy will be assessed.

2.7 Construction Plan and Cost Analysis

The construction plan of 3 hydropower schemes will be worked out based on the workable days throughout a year, construction method to be applied and referring to the meteorological and topographic conditions of the project areas.

The unit cost data obtained from similar projects which were implemented or are being executed by CELESC and ELETROSUL will be analyzed and unit prices to be used for cost estimate will be assessed.

2.8 Estimate of Work Quantity and Construction Cost

Based on the dimensions of the project components, which were determined through the optimization study, work quantities of the project components will be estimated. The construction cost will be estimated based on the work quantities and assessed unit prices. Annual disbursement schedule for construction fund will be estimated based on the construction time schedule.

3. STUDY ON SALTO PILÃO (1) HYDROPOWER SCHEME

3.1 Site and Type of Dam

In addition to the dam axis proposed in the master plan study in the first stage, two dam axes were contemplated in this study. Location of these dam axes is shown in Fig. III.3.1.

In order to select the most appropriate dam axis among these three alternative dam axes, comparative study was made assuming that the type of dam is of concrete gravity due to the reasons as stated in paragraph 3.2.2.

Comparative study to select the suitable dam axis was made from two aspects, i.e. technical and economic aspects and environmental aspect. Economic evaluation was made by means of comparison of unit cost of the guaranteed energy, which is the criteria specified by ELETROBRAS.

Cross sections of the three dam axes are shown in Fig. II.3.1 in Annex II. The dam axis-A is located at the upmost of the conceivable river stretch. The river width is about 315 m and about 30 m wide river deposit exists in the middle of the river cross section. It was confirmed by the result of boring that the dam foundation consists of hard and massive granite and it corresponds to A to B class of rock classification which has been defined by standard of Central Research Institute of Electric Power Industry of Japan. The excavation depth of foundation rock was estimated at about 10 m for dam abutment and about 2 m for river bed portion. As a foundation treatment, consolidation and curtain groutings are needed only for left bank side to remedy the fault crossing dam axis in the left bank side.

The dam axis-B is located at about 450 m downstream from the dam axis-A. The river width of the dam axis-B is about 265 m. The foundation of this dam axis consists of granite. The result of boring clarified that this granite is very hard and massive condition. The excavation depth of foundation rock was estimated to be about 2 m for river bed portion and about 10 m for both abutment portions. It was presumed that consolidation and curtain groutings may be omitted due to excellent rock characteristics.

The dam axis-C is situated at about 600 m downstream from dam axis-B. The river width is about 220 m. Outcrop of granite appears in the right side of the river bed and about 70 m wide river deposit is found in the left side of the river bed. Although the granite has hard and massive characteristics, open cracks are found in several place crossing the dam axis. The excavation depth of foundation rock was estimated to be about 2 m for the river bed portion and about 15 m for the abutment portions. To remedy the open cracks, consolidation and curtain groutings are required.

For respective dam axes, full supply water level (F.S.L) of reservoir was set as follows considering the daily regulation capacity required for power generation;

Dam axis	F.S.L (EL.m)
A	330
B	330
C	319

For concrete gravity type dam, following dam section was adapted;

- Upstream slope ; Vertical
- Downstream slope ; 1:1
- Crest width ; 4.5 m
- Freeboard above F.SL ; 2 m

Location of the headrace tunnel from the intake site to the surge tank site is shown in Fig. III.3.2. Total length of the headrace tunnel for the respective dam axes is as follows;

Dam axis	Length of headrace tunnel (m)
A	8,100
B	6,940
C	6,550

The result of core boring and geo-surface inspection clarified that granite distributes almost all parts of the headrace tunnel route, and several faults which involve small scale fractured zone were detected. This granite was graded into the excellent rock corresponding to A class of rock classification.

At the proposed intake site for dam axes-A and -B, granite distributes and it is covered with thin soil and/or weathered layer. Since the proposed intake site for the dam axis-C is covered with rather thick soil and weathered layer, hard rock is not found in the tunnel foundation level. Thus, for dam axis-C supporting system was required for about 500 m long tunnel route in its beginning part.

The inside diameter of the headrace tunnel was set at 5.2 m which is the same dimension as applied for the master plan, but tunnel lining was decided at 0.45 m considering the rock characteristics.

Location of the proposed surge tank site is shown in Fig. III.3.2. It was presumed that the surge tank is provided in rhyolite zone which is very hard rock corresponding to B class of rock classification. In this study, a simple type surge tank was assumed and its inside diameter was assumed to be 4 times that of the headrace tunnel.

The proposed penstock is of underground inclined pressure shaft type. Its route passes rhyolite zone which has hard and massive characteristics corresponding to B class of rock classification. Total length of the penstock line was estimated to be 610 m. The penstock line with one lane and its diameter of 3.8 m was adapted in this study. Open air conduit type penstock line is conceivable as an alternative plan. However, about 10 m thick heavily weathered layer of decomposed layer soil overlies along the penstock line route. Since this layer may be easily collapsed, it was judged that the open air steel conduit type plan is not economically suitable.

The result of core boring at the proposed powerhouse site showed that weathered rhyolite with fractured zone distributes up to depth of 11.6 m from ground surface and these fractured rhyolite varies to fresh one below the depth of 11.6 m.

An open-air type powerhouse of 27 m wide and 49 m long was adapted in this study and its foundation was set at 11.6 m in depth. A Francis type power generating equipment was applied in consideration of extent of effective head and installed capacity.

The transmission line was assumed to connect the existing transmission line located near the project site. About 7 km long and 138 kV transmission line was planned.

Based on the foregoing dimensions of the project components, work quantities for the cases of three dam axes were estimated. Using the same unit prices as applied for the master plan, the construction cost was estimated as shown in Tables III.3.1 to III.3.3.

The power energy to be generated for the cases of three dam axes was assessed assuming the following tail water level (T.W.L.);

Discharge	TWL (EL.m)
Firm discharge, 50 m3/sec	110.5
Max. plant discharge, 71.9 m3/sec	110.6

The assessed firm energy, guaranteed energy and secondary energy are as follows;

(Unit; GWh)

Dam axis	Firm energy	Guaranteed energy	Secondary energy
A	762.6	686.4	65.4
B	766.1	689.5	66.4
C	727.6	654.9	63.1

Based on the estimated construction cost and power energy, unit cost of the guaranteed energy was estimated as shown in Table III.3.4. This table shows that unit cost of the guaranteed energy for the case of dam axis-B is US$17.5/MWh which is the smallest value among three cases.

While according to the result of the environmental impact study, the number of household and acreage of land to be submerged by the scheme were estimated as follows;

Dam axis	Number of household	Acreage of land (km2)
A	87	2.59
B	87	2.88
C	8	0.33

It shows that large influence exerts to the riparian people if the dam axis-A or -B is adopted. Besides, majority of the resort complex which is located at the left bank of the Itajai river immediately upstream of the dam axis-A will be submerged if the dam axis-A or B is selected. It is important to reserve the forest for habitat of the important bird species. If the dam axis-A or B is selected, many forests will be submerged. The forest area to be submerged is the minimum for the dam axis-C.

While, impounded water level by reservoir reaches to upstream of Rio do Sul city for the case of dam axes-A and B. Extent of impounded water level for the case of dam axis-C is only about 0.8 km from the damsite. Although the impounded water level for dam axes-A and -B increases only by 1 to 2 m than the ordinary water level and it is confined within the river channel, it may exert psychological impact for menace of flood on the riparian people.

The second smallest value of the unit cost of the guaranteed energy is US$18.8/MWh for the case of dam axis-C. Difference of the unit cost of the guaranteed energy between the cases of dam axes-B and C is only US$0.7/MWh and decrease in the guaranteed energy in case of dam axis-C against that for the dam axis-B is only about 5 %.

Considering the result of the economic comparison and effect to the environmental aspects, dam axis-C was selected for further study.

3.2 Optimization Study and Pre-feasibility Design of Project Components

3.2.1 River diversion

The river diversion works consist of diversion tunnel and upstream and downstream cofferdams. Since the proposed damsite is situated at just downstream of the river channel bent sharply toward left side, the diversion tunnel was planned to be located in the left side bending in U-shape. In view of the dam type of concrete gravity as explained in the following paragraph, design peak flood for river diversion was decided to be 1,100 m^3/sec with 2-year probability. The design flood for the river diversion is not so large magnitude compared with that for case of fill type dam. There is a sufficient river bed slope between the upstream and downstream sites of the diversion tunnel. In order to lower the height of cofferdam, a free flow type diversion tunnel was applied in this study. The planned length of the diversion tunnel is 560 m. The result of hydraulic calculation showed that diameter of the diversion tunnel is 9.8 m and maximum water level to discharge the design peak flood of 1,100 m^3/sec is EL 319.3 m. A concrete gravity type cofferdam was planned for upstream cofferdam considering term of the construction period and unpredictable flood exceeding the design flood peak during the construction works. Crest elevation of the upstream cofferdam was decided at EL 320 m considering 0.7 m of freeboard. Maximum height of the upstream cofferdam is 17 m. General plan of the river diversion is shown in Fig. III.3.3.

3.2.2 Dam

The proposed damsite is located at rapid river stretch with V-shaped channel. Both river banks form hills with a relative height of about 50 m. The river width is about 220 m. Outcrop of granite appears in the right side of the river bed and about 70 m wide river deposit is found in the left side of the river bed.

Full supply level of the reservoir was set at EL 319 m to keep the storage capacity sufficient for daily flow regulation, to enable the plant to function at its installed capacity even with a inflow of average turbine flow in the dry season. The determined daily regulation capacity is 622,000 m^3. As a type of dam in consideration of the topographic, geological and planning aspects, concrete gravity was adopted due to the following reasons:

(i) Since the dam foundation consists of hard rock, it is suitable to apply the concrete gravity type dam.

(ii) The river valley at the proposed damsites forms V-shape. Since height of the dam is around 20 m, dam volume is relatively small even if the concrete gravity type dam is applied.

(iii) Since there is topographically no space to provide a spillway beside the damsite, it is obliged to provide the spillway structure in the dam body. If a fill type dam is applied, problem of water leakage may occur between the junction of the concrete structure for the spillway and zones of the fill type dam. Besides, safety of dam may not be guaranteed if the spillway gates are maloperated and flood flow overtops above the crest of the fill type dam.

Considering the geological properties of the dam foundation, the excavation depth of foundation rock was estimated at about 2 m in the river bed and 10 m in both river bank sides. The dam section was determined based on stability analysis against overturning. The analysis was made so as to satisfy the condition that the resultant acting force is within the middle-third of the base for the case of full reservoir condition with water surface of 319 m and horizontal coefficient of 0.1 for earthquake, and following dam section was adopted;

- Upstream slope ; Vertical
- Downstream slope ; 1:1
- Crest width ; 4.5 m
- Freeboard above full supply level ; 1.5 m

The planned dam is 260 m in crest length, EL 320.5 m in crest elevation and 20.5 m in maximum height. General plan and profile of the dam are shown in Figs. III.3.3. and III.3.4.