many periods from the late Palaeozoic to the early Mesozoic, as is the case with the main dam site. They are interbedded with thin bands of shales, and pose a weak anisotrophy or fissility.

Terrace deposits and river bed deposits are widely distributed in the slopes of both banks, and are comprised of silty, unconsolidated sands and granules. These cover beds are estimated to be some 10 m in thickness on the right bank, and several meters on the left bank.

4.13.3. Civil Engineering Geology

The gist of the results of boring investigation carried out at the proposed re-regulating pondage site is as shown in Table 4-26.

Based on the distribution of exposed rocks, hard rock masses may appear at a depth of several meters in the river beds, but on the skirts of both banks, heavily weathered rocks or river bed deposits may continue to the level of the river beds.

Being low in height, the dam could be mounted on these deposits or heavily weathered rock masses, but the Lugeon permeability tests carried out at these sections reveal the pressure to increase only to 4 kgf/cm^2, and the Lugeon value to be as high as 30 Lu, thus indicating it is not easy to improve the permeability.

Table -4-1 . Characteristics of major tectonic lines in Peninsular Malaysia

Name, locality, fault trend	Fault length Fault width	Sense and length of displacement	References
Bok Bak fault zone Kedah — Perak N 30 W	80 km long 10 km wide	55 km aggregate left lateral	Burton, 1965
Kelau-Karak fault zone Pahang Approx. north-south	180 km long	5 to 10 km right lateal	Jaafar Ahmad, 1965; Khoo, 1968; Tjia, 1972
Lebir fault zone Trengganu — Pahang N 10-20 W	320 km long 2 km wide near Kuala Kerai	20-40 km (?) left lateral	Tjia, 1969, 1972; Aw, 1969;
Bukit Tinggi fault zone Selangor — Pahang N 35 — 40 W	70 km long	6 km left lateral	Shu, 1969, 1970
Kuala Lumpur fault zone N 60 — 70 W Selangor-Negeri Sembilan	100 km long 15 km wide	20 km left lateral	Gobbett, 1964; Stuaffer, 1968, 1969, Tjia, 1975
Mersing fault zone Johro N 70 W	130 km long 30 km wide	20 km left lateral	Stauffer, 1968; Suntharalingam, 1969; Chong and others, 1970
Ma' Okil fault Johor N 35 E	A few tens of kilometers	20 km right lateral?	Burton, 1965
Lepar fault zone Pahang N 45 W	At least 40 km long and at least 160 m wide	left lateral	Lee, 1974; Tjia, 1976

Table 4-2 Topographical Survey Work Done for the 1987 Investigation

No.	Kinds	SSK (m) N	SSK (m) E	Elevation (MSL-m)	Location
C1	Datum pt.	581,532.675	476,601.079	65.172	Left bank of dam site
C2	- ditto -	581,154.920	476,496.894	108.478	- ditto -
C3	- ditto -	581,181.428	477,203.041	108,972	Right bank of dam site
C4	- ditto -	581,565.152	477,345.303	115.970	Spillway site
P3	- ditto -	582,527.036	478,360.480	100.837	Right bank of saddle dam II site
P4	- ditto -	582,145.077	477,799.128	169.758	Left bank of saddle dam I site
B16	- ditto -	582,301.857	478,302.413	135.319	Right bank of saddle dam I site
QS6	- ditto -	582,772.404	477,013.672	206.285	Quarry site
TBM 2	Prov.bench mark	581,646.875	476,963.980	43.263	Right bank downstream of dam site
TBM 2-1	- ditto -	581,661.963	476,959.664	43.202	- ditto -
TBM 19	- ditto -	581,376.328	476,200.181	136.688	Left bank of dam site
TMB 22	- ditto -	581,676.687	477,193.365	42.617	Right bank downstream of dam site
P2	Boundary post	582,575.000	478,288.548	95.369	Right bank of saddle dam II site
QS2	- ditto -	582,764.899	477,435.123	68.635	Quarry site

Table 4-3(1) Borings for the Preliminary Investigation (1979)

Location	No.	Length (m)	Boring Head Elevation (m)	Lugeon Tests
Tualang Site:				
Main dam	A - 1	19	70	-
	A - 2	20	56	-
	A - 3	30	75	-
	A - 4	30	101	-
Saddle dam	A - 5	15	41	-
	A - 6	23	68	-
Jeram Panjang Site:				
Main dam	B - 1	20	87	-
	B - 2	26	42	-
	B - 3	18	43	-
	B - 4	20	62	-
Spillway	B - 5	25	112	-
Saddle dam	B - 6	20	42	-
	B - 7	30	80	-
Kiak Site:				
Main dam	C - 1	30	56	
Total	14	326		

Table 4-3(2) Boring for the investigation conducted this time (1987)

Location			No.	Length (m)	Boring Head Elevation (m)	Lugeon Tests (times)	Date (1987)	Coordinates **
Jeram Panjang Site	Main dam	Dam axis	D-1	60	107.889	10	8/9-8/27	N : 581152.320 E : 476511.710
			D-2	70	28.423	12	9/7-9/30	N : 581147.817 E : 476845.944
			D-3	60	115.109	10	9/7-10/12	N : 581156.440 E : 477246.595
		Powerhouse	D-4	20	42.113	2	7/26-7/31	N : 581363.726 E : 476682.812
		Spillway	D-5	40	60.073	6	8/23-9/8	N : 581507.743 E : 477186.195
			D-6	60	107.552	10	8/22-9/19	N : 581476.228 E : 477259.727
			D-7	30	61.392	* 4	9/3-9/12	N : 581435.288 E : 477387.775
	Saddle Dam I		S-1	40	103.306	6	7/11-7/30	N : 582123.006 E : 477912.774
			S-2	40	73.026	6	7/12-7/29	N : 582132.118 E : 478014.334
			S-3	40	43.108	6	7/6-7/18	N : 582171.246 E : 478192.132
			S-4	40	108.789	6	7/13-7/31	N : 582187.504 E : 478323.159
	Saddle Dam II		S-5	30	84.838	* 4	7/12-7/21	N : 582364.236 E : 478399.249
			S-6	25	70.532	3	7/14-7/26	N : 582500.487 E : 478538.094
	Quarry Site		Q-1	40	204.201	-	8/10-8/23	-
			Q-2	40	222.495	-	8/10-8/21	-
			Q-3	40	145.203	-	8/10-9/5	-
			Q-4	40	186.548	-	8/10-8/22	-
	Borrow Site		Bp-1	20	About 185	S.P.T 10	7/27-7/30	-
			BP-2	20	About 280	S.P.T 10	7/31-8/11	-
	Re-regulating Pondage		R-1	10	41.438	1	9/29	-
			R-2	10	27.570	1	9/23-9/26	-
			R-3	10	35.495	1	9/20-9/21	-
Total			22	785	-	88 (B.P.T20)		-

Note : * Lugeon permeability tests at D-7 (15m-20m) and S-5 (20-25m) were suspended when the injection pressure came up to 6 kgf/cm^2, causing seepage between bore hole wall and casings.

** Coordinates on the topographical plan with a scale of 1/500:

Table 4-4 Quantitative Data of Seismic Prospecting Tests

Site	Traverse Line No.	Traverse Line Length (m) Slope	Horizontal	Number of Spreads
Main Dam Site	SD - 1	770	747	7
	SD - 2	665	650	6
	SD - 3	225	224	2
Saddle Dam Site	SS - 1	555	506	5
Quarry Site	SQ - 1	775	742	7
	SQ - 2	995	957	9
	SQ - 3	445	410	4
Total	7 lines	4,430	4,236	40

Note: Geophones installed at 5-meter intervals.

Table 4-5 Results of Seismic Prospecting Tests at the Main Dam Site

Velocity Layer	Velocity (m/s)	Layer Thickness (m)	Rock Masses
No. 1	260 - 500	1 - 3 (5-6 along No. 7 Spread)	Topsoils - deposits
No. 2*	700 - 1,700	7 - 10 (2-3 along No. 1, 2 and 15 Spreads, and 15-17 along No. 7 Spread	Deposits - weathered rocks
No. 3	4,000 - 5,500	10 - 15 in depth from the surface (10 or less along Nos 12, 13 and 15 Spreads, and 20 or more along No. 7 Spread.	Almost fresh rock surfaces

Note : * There may exist blind layers with velocities of 2,300 - 2,500 m/s along Nos. 1 and 2 Spread, and 2,500 - 3,000 m/s along Nos. 6 and 7 Spread.

Table 4-6 Results of Seismic Prospecting Tests at the Saddle Dam I Site

Velocity Layer	Velocity (m/s)	Layer Thickness (m)	Rock Masses
No. 1	300 - 400	1-3 (6-8 along Nos. 4 and 5 spread)	Top soils - deposits
No. 2	700 - 1,100	20-30 on the ridge and 3-10 on the river beds	Deposits - heavily weathered rocks
No. 3	1,400 - 1,600	20 - 30	Weathered rocks
No. 4*	3,100 - 4,300	In depth from the surface; 10-15 on the river beds 40-50 on the right bank 30-35 on the left bank	Slightly weathered rocks - almost fresh rocks

Note: Fresh rock masses are assumed to be distributed at greater depths, but no confirmation has yet to be made by analysis.

Table 4-7 Results of Seismic Prospecting Tests at the Quarry Site

Velocity Layer	Velocity (m/s)	Layer Thickness (m)	Rock Masses
No. 1	300 - 400	1 - 3	Top soils - deposits
No. 2	800 - 900	5 - 6 along No. 12-14 Spreads on SQ-1; No. 1, 4 and 19 Spreads on SQ-2; and No. 15 and 16 Spreads on SQ-3. 10-15 (along spreads other than above)	Weathered rocks
No. 3	3,600 - 6,000	5 - 10 (Portions where No. 2 velocity layer is thick) 15 - 20 (Other portions above mentioned)	Slightly weathered rocks - almost fresh rocks

Note : There is a good possibility that the blind layers with intermediate velocities of 2,000-2,500 m/s may exist between the weathered rocks and slightly weathered rocks.

Table 4-8 Summary of Rock Samples

Test Items	Taken from Quarry Site		Taken from Main Dam Site	
	Boring No.	Depth(m)	Boring No.	Depth(m)
Physical & Uniaxial Compression Test	Q-1	16.0, 20.3 31.0, 36.7 37.2	D-1	56.0
	Q-2	18.4, 23.1 26.6, 30.0	D-2	41.0, 44.3
	Q-3	20.4, 32.0	D-5	16.8, 20.8
	Q-4	27.85, 35.7		
Sodium Sulphate Tests	Q-1	38.7-39.75		
	Q-3	27.4-27.95		
	Q-4	36.6-37.45		

Table 4-9 Summary of Soil Samples

Taken From *	Depth (m)	Sample No.	Location	Rock Species
Test Pit 1	2	TP-1-2	Eastern end of survey zone, and nearby Boring BP-1	Disintegrated granite
	3.5	TP-1-3.5		
	5	TP-1-5		
	5.4	TP-1-5.4		
Test Pit 2	2	TP-2-2	Slow slope of the low elevation sections on the left bank some 900 m upstream of the proposed main dam site	River terrace (flood plain) deposits
	3.5	TP-2-3.5		
	5	TP-2-5		
Test Pit 3	2	TP-3-2	Ridge of the right bank of the proposed saddle dam I, and nearby Boring S-4	Heavily weathered tuffaceous conglomerate
	3.5	TP-3-3.5		
	5	TP-3-5		
Road Slopes	Cut by 50cm below the surface	SP-1	Excavated slope of a forestry road leading to the borrow site A-4, 4 km away from the proposed main dam site	Disintegrated granite
		SP-2	- do -	Granite
		SP-3	Excavated slope of a forestry road to leading the barite quarry site, 3.6 km away from the proposed main dam site	Granite
		SP-4	Excavated slope of a road on the right bank of the proposed main dam site, 700 m south of Kg Pedak	Heavily weathered tuffaceous conglomerate

Taken From *	Depth (m)	Sample No.	Location	Rock Species
		SP-5	Excavated slope of a forestry road on the left bank of the proposed main dam site, 2.5 km away from the proposed main dam site	Heavily weathered tuff (purple)
		SP-6	- do -	Heavily weathered tuffaceous sandstone (dark red)
Total		16		

Note : * Refer to Fig. 4-7

Table 4-10 Rock Groups Underlying the Main Dam Site

Rock Group	Characteristics
Green Tuff	The largest distribution of all rock groups underlying the proposed main dam site. The grain size varies widely from fine to coarse. Joints are systematic and sharp. Fresh tuff is hard and stiff in nature. Cracks have films of calcite, serpentine or chlorite, and often pose slickenside on the surface.
Purple Tuff	The second largest distribution. It is finer in grain size than green tuff. Joints are generally tight and sharp. Bedding planes with green tuff are often transitional.
Tuffaceous Sandstone	Widely distributed on the right bank of the proposed dam site. Medium grained arkose with gray and gray green in color. Rock facies akin to coarse grained green tuff, and tend to change transitionally. It is observed in D-3 Boring.
Shale	Alternated with tuffaceous sandstone, but comparatively smaller in volume. Some several meters at the thickest. Black in color. Consisting of incompetent layers against sandstone, it is cracked finely and often found to be fractured. Cracks pose slickenside on the surface. It is observed in D-3 Boring.
Tuffaceous conglomerate	It exists in tuffs as an interbedding layer with a thickness of several meters. Containing granules, rounded or sub-rounded gravels in cobble size, consisting of green tuff, purple tuff, chert, shale and dark green lava, in a content ratio of less than 50%.

Table 4-11 Gist of the Results of Boring Investigations at the Main Dam Site

Boring No.	Location (Dam Axis)	Boring Head Elevation (EL m)	Depth of Weathered Zone (m)	High Permeability Zone (m)	Water Level of Bore Holes (m)
D-1	Left bank high elevation section	107.889	7	0-15	7.2
D-2	River Beds	28.423	5	0-20	0.9
D-3	Right bank high elevation section	115.109	21	0-25	28.5

Table 4-12 Classification of Rock Masses (*1)

Classification	Characteristics	Judgement from a Civil Engineering Point of View
A	Rock-forming minerals (*2) are fresh and not weathered nor altered. Joints and cracks are closely adhered to each other with no weathering visible along their planes. A clear sound is emitted when hammered.	Very good as rock-fill dam core material
B	Rock-forming minerals are weathered slightly or altered partially, but the rock is hard. Joints and cracks are closely adhered. A clear sound is emitted when hammered.	- ditto -
CH	Rock-forming minerals are weathered, but the rock is fairly hard. The bond between rock blocks is slightly reduced and each block is apt to exfoliate along joints and cracks by strong hammering. Some of the joints and cracks contain clay and other materials coloured by limonite. A slightly dull sound is emitted when hammered.	Fairly good as rockfill dam core foundation material
CM	Rock-forming minerals are weathered and the rock is slightly soft. Exfoliation of the rock occurs along joints and cracks by normal hammering. Some of the joints and cracks contain clay and other materials. A somewhat dull sound is emitted when hammered.	Almost durable as rockfill dam core foundation, material
CL	Rock-forming minerals are weathered and the rock is soft. Exfoliation of the rock occurs along joints and cracks by light hammering. Joints and cracks contain clay. A dull sound is emitted when hammered.	Not adequate for rockfill dam core foundation in general, but possible to be used as rock-transition foundation, material

Classification	*Characteristics*	Judgement from a Civil Engineering Point of View
D	Rock-forming minerals are weathered, and the rock is very soft. There is virtually no bond between rock blocks, and collapse occurs by slight hammering. *Joints and cracks contain clay.* A very dull sound is emitted when hammered.	Unsuitable for foundation

(*1) : Haruo Tanaka, 1968
(*2) : Except quarts

Table 4-13 Averages of RQD and Maximum Core Length of Borings at the Main Dam Site (CM or harder Class rocks)

Boring No.	RQD			Maximum Core Length		
	CM class rocks	CH class rocks	B class rocks	CM class rocks	CH class rocks	B class rocks
D-1	11	56	70	12	27	34
D-2	36	66	84	16	26	36
D-3	27	55	-	14	26	-

Table 4-14 Gist of the Results of Boring Investigations at the Spillway Site

Boring No.	Location	Boring Head Elevation (EL m)	Depth of Weathered Zone (m)	High Permea-bility Zone (m)	Water Level of Bore Holes (m)
D-5	On the northwest slope of the ridge	66.073	10	0-10	4.0
D-6	On the top of the ridge	107.552	23	0-25	13.7
D-7	On the southeast slope of the ridge	61.392	6.5	0-10	6.8

Table 4-15 Gist of the Results of Boring Investigation at the Powerhouse Site

Boring No.	Location	Boring Head Elevation (EL m)	Depth of Weathered Zone (m)	High Permea-bility Zone (m)	Water Level of Borehole (m)
D-4	Powerhouse	42.113	9	0-15	3.5

Table 4-16 Rock Groups Underlying the Saddle Dam I Site

Rock Group	Characteristics
Tuffaceous Conglomerate	Light purple or purple in colour as a whole. Contains subrounded gravels or breccias in pebble size consisting of purple tuffs and dark green tuffs, in the content ratio of more than 50%. The matrix is comprised of quartz and feldspars in granule size.
Tuffaceous Sandstone	Light purple in color as a whole. Medium and coarse grained arkose sandstones, consisting partially of what should be called rounded quartz grains or quartzites.
Tuff	Fine grained and purple in colour. Scalelike cracks increase as weathering goes on.

Table 4-17 Rock Groups Underlying the Saddle Dam II Site

Rock Group	Characteristics
Tuffaceous Conglomerate	Variant locally in colour,grain size, and grain composition. colour changes from grayish purple to red, and grains vary from silty clayey to sandy nature. Some sections are rich in quartz grains similar to quartzites and some in fine grained tuffs. Remarkable in variation of rock facies.
Meta-dacite	Greyish green to green in colour. Consists mainly of non-holocrystalline rocks including phenocrysts such as anhedral feldspars and pyroxenes of some 1 mm in grain diameter. In the area where these rocks are distributed, the valley forms overhanging cliffs explosed with hard rocks.

Table 4-18 Gist of the Results of Boring Investigation at the Saddle Dam I and II Site

Boring No.	Location	Boring Head Elevation (EL m)	Depth of Weathered Zone (m)	High Permeability Zone (m)	Water Level of Boreholes (m)
S-1	Dam I : left bank high elevation section	103.306	27	High to a depth of 35m 7.7Lu between 35-40m	31.7
S-2	Dam I : left bank medium elevation section	73.026	27	0-15	21.3
S-3	Dam I : riverbeds	43.108	10	0-30	4.5
S-4	Dam I : right bank high elevation section	108.789	23	0-15	28.35
S-5	Dam II: left bank	84.838	26	0-15	22.7
S-6	Dam II: right bank	70.532	7	0-10	-

Table 4-19 Rock Groups Underlying the Quarry Site

Rock Group	Characteristics
Green Fine-Grained Tuff	The same group as those lying at the main dam site. Massive and dense in nature and green or greenish blue in color. Cracks often clung with slickenside films of calcite, dark green chlorite and serpentine.
Purple Coarse-Grained Tuff	Purple in color as a whole. The rock groups that can be grouped into tuffaceous conglomerates or tuffaceous breccias are visible locally. Remarkable variations in lithofacies. Probably contains primary tuff and secondary tuff admixed with non-marine clastics during periods of unconsolidated ages. Generally, massive and dense in nature, but weakened locally due to the influence of hydrothermal alteration.
Purple Fine-Grained Tuff	Purple in color as a whole. There exist few admixtures of non-marine clastics. Sharp and systematic in joints. Locally anisotropic. Rock debris is hard and dense.
Purple Tuffaceous Breccia or Tuffaceous Conglomerate	There exist many admixtures of non-marine clastics, most of all rock groups. Contains different kinds of gravels in granule or cobble size in the content ratio of 50% or more. The same as the above rock group in physical properties.

Table 4-20 Gist of the Results of Boring Investigation at the Quarry Site

Boring No.	Location	Boring Head Elevation (EL m)	Depth of Weathered Zone (m)	High Permeability Zone (m)	Water Level of Bore Holes (m)
Q-1	Quarry Site	204.201	10	-	5.34
Q-2	ditto	222.495	9	-	20.43
Q-3	ditto	145.203	14	-	-
Q-4	ditto	186.548	10	-	8.95

Table 4-21 Averages of RQD and the Maximum Core Length of Borings at the Quarry Site

Boring No.	RQD		Maximum Core Length	
	CM class rocks	CH class rocks	CM class rocks	CH class rocks
Q-1	15	35	15	18
Q-2	47	40	21	19
Q-3	41	66	21	30
Q-4	22	43	16	20

Unconfined Compressive Strength, Absorption & Durability

Drilling No.	Depth of Sampling (m)	Rock Name	Rock Classification	Bulk Density (g/cm³)	Relative Density (g/cm³)	Relative Density on an oven dry basis (g/cm³)	Absorption (%)	Unconfined Compressive Strength (kgf/cm²)
Q－1	16.0	Coarse Tuff (Purple)	CH	1.59	2.69	2.71	0.5	686.5
	20.3	Coarse Tuff (Purple)	CH	1.61	2.92	2.70	0.8	192.0
	31.0	Coarse Tuff (Purple)	CH	――	――	2.70	0.9	Out
	36.7	Coarse Tuff (Purple)	CH	1.56	2.79	2.82	0.2	585.7
	37.2	Coarse Tuff (Purple)	CH	1.55	2.79	2.88	0.1	762.1
Q－2	18.4	Tuff Breccia (Purple)	CH	1.58	2.71	2.77	0.4	163.2
	23.1	Fine Tuff (Purple)	CH	1.59	2.75	2.75	0.1	177.6
	26.6	Fine Tuff (Greenish Blue)	CH	1.58	2.75	2.76	0.2	153.6
	30.0	Fine Tuff (Greenish Blue)	CH	1.59	2.71	2.78	0.5	211.2
Q－3	20.4	Coarse Tuff (Purple)	CH	1.56	2.76	2.82	0.7	456.1
	32.0	Fine Tuff (Greenish Blue)	CH	1.57	2.72	2.78	0.2	398.5
Q－4	27.85	Fine Tuff (Greenish Blue)	CH	1.58	2.77	2.81	0.1	1089.8
	35.7	Fine Tuff (Greenish Blue)	CH	1.56	2.82	2.81	0.2	Out
D－1	56.0	Fine Tuff (Purple)	B	1.57	2.74	2.80	0.6	691.3
D－2	41.0	Fine Tuff (Greenish Blue)	CH	1.61	2.74	2.71	0.4	1195.4
	44.3	Fine Tuff (Purple)	CH	1.58	2.74	2.86	0.9	489.7
D－5	16.8	Tuff Breccia (Purple)	B	1.56	2.77	2.82	0.1	969.8
	20.8	Tuff Breccia (Purple)	CH	1.75	2.74	2.80	0.1	984.2

Durability

Drilling No.	Depth of Sampling (m)	Rock Name	Durability (%)
Q－1	38.7 ～ 39.75	Fine Tuff (Purple)	4.6
Q－3	27.4 ～ 27.95	Coarse Tuff (Purple)	2.8
Q－4	36.6 ～ 37.45	Fine Tuff (greenish blue)	1.3

Table 4-22
The result of rock material test

Table 4-23 Gist of the Results of Boring Investigation at the Borrow Site A

Boring No.	Location	Boring Head Elevation (EL m)	Depth of Weathered Zone (m)	Depth of N Values Higher than 50 (m)	Water Level of Bore Holes (m)
BP-1	Borrow site	About 185	15	13 - 14	-
BP-2	ditto	About 280	16	16 - 17	-

Table 4-24 General Properties of Imprevious Materials
Used for Rockfill Dams in Japan

Name of Dam	Material	Specific Gravity	Max. Dry Density Yd. max	Water Content(%)			Atterberg Limits			Permeability cm/s	Testing Method for c φ	Design Value Yd.
				Natural	Optimum	Control Standards	LL	PL	Pl			
			t/m³		%	Wopt	%	%	%			
Tokachi	Talus deposits and silts	2.73	1.88	15.4	9.5	+0-2	30.2	18.4	11.8	8.7×10^{-6}	Triaxial	2.05
Takami	Talus deposits mixed with weathered shales (1:1-1:3)	2.75	2.19	9.2	8.3	Wopt +0-2.5	36 35	20 22	16 13	1.7×10^{-7}	ditto	1.90 2.00
Shirakawa	Weathered granites	2.65	2.03	10	9.3	9-12	27	15	12	5×10^{-6}	ditto	2.00
Gosho	Weathered tuffs mixed with river bed conglomerates (1:3)	2.536	1.766	17.5	16.4	17-20	47.9	28.3	19.6	5×10^{-7}	Triaxial CU	1.66
Kankako	Andesite deposits mixed with basalt clastics	2.75	1.89	20	14.9	Wopt -1-+3	52.9	25.5	27.4	1.2×10^{-7}	Triaxial	1.65
Shitoki	Weathered Schists	2.69	1.96	7-27	17-25	Wopt -1-+2.5	38	27	11	4×10^{-6}	ditto	1.75
Nanakitada	Weathered basalt resistates mixed with mudflow deposits	2.84	1.79	28	26	Wopt +0-3	80	45	35	1×10^{-6}	ditto	1.70
Tedorigawa	Weathered rocks and taluses	2.72	1.90	6-22	14.2	Wopt +1-3	42	21	21	3×10^{-6}	ditto	1.96
Iwaya	Quartzose porphyries and disintegrated talus deposits	2.63	1.75	15.3	17.2	Wopt +0-3	41.2	25.6	15.6	6.7×10^{-6}	ditto	1.762
Seto	Shales and weathered sandstone deposits	2.74	1.96	12.4	11.5	Wopt +0-3	34	20	14	1×10^{-6}	ditto	1.87
Kurokawa	Weathered rocks alternated with slates and cherts	2.68	1.83		15.5	Wopt +0-2	31	21	10	0.2 10 $\times 10^{-6}$	ditto	1.75
Inamura	Weathered green schist resistates (mixtures)	2.85	2.00	7-14	11.7	Wopt +0-3	34	22.4	11.6	3.1×10^{-7}	ditto	1.89
Shimogo	Tuffs mixed with cherts (1:1)	2.68	2.01	10.2	11.4	Wopt +0-2.5	38.6	20.1	18.3	1×10^{-6}	Triaxial CU	1.90

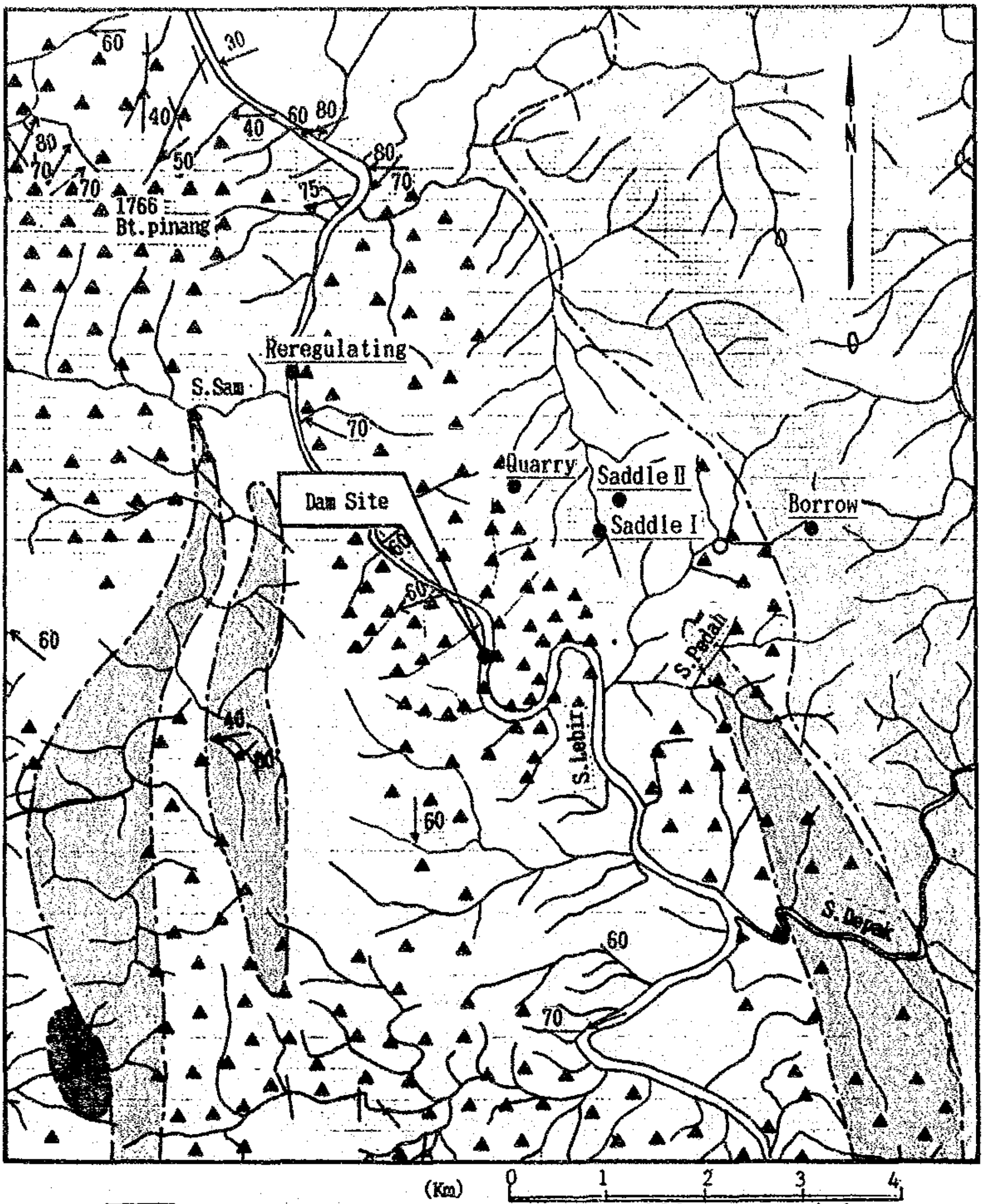
Surce: Geological Survey for Dams (Civil Engineering Society)

Location of sampling		Test Pit 1				Test Pit 2			Test Pit 3			Slope					
Sample No.		TP-1-2	TP-1-3.5	TP-1-5	TP-1-5.4	TP-2-2	TP-2-3.5	TP-2-5	TP-3-2	TP-3-3.5	TP-3-5	SP-1	SP-2	SP-3	SP-4	SP-5	SP-6
Sample depth (m)		2	3.5	5	5.4	2	3.5	5	2	3.5	5	—	—	—	—	—	—
Condition of sample		Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed	Disturbed
Natural water content(%)		28.5	29.9	(30.2) 30.1	27.5	(32.9) 35.2	(32.1) 33.4	(37.8) 34.0	(24.9) 16.5	13.1	15.6	20.8	14.9	17.2	9.4	13.6	23.3
Specific gravity		2.68	2.74	2.68	2.67	2.74	2.74	2.75	2.76	2.72	2.72	2.68	2.68	2.67	2.68	2.76	2.70
Wet density (g / cm³)		1.415	—	1.396	—	1.416	1.292	(1.286) 1.396	1.382	—	—	—	—	—	—	—	—
Dry density (g /cm³)		1.101	—	1.072	—	1.047	0.969	1.046	1.110	—	—	—	—	—	—	—	—
Natural void ratio		1.433	—	1.500	—	1.572	1.082	1.947	1.494	—	—	—	—	—	—	—	—
Degree of saturation		53.3	—	54.0	—	58.2	48.8	48.0	46.0	—	—	—	—	—	—	—	—
Atterberg limits	Liquid Limit (%)	78	61	42	39	82	62	43	34	28	37	46	40	34	27	29	44
	Plastic Limit (%)	36	35	34	34	37	36	32	23	11	23	37	31	29	21	23	27
	Plasticity index	42	26	8	5	45	26	11	11	7	14	9	9	5	6	6	17
Grain size analysis	(more than 2000 μm) Gravel (%)	31	23	23	28	15	22	24	42	50	37	35	24	33	44	29	27
	(74-2000 μm) Sand (%)	24	25	34	38	7	16	39	38	39	37	33	46	45	41	33	22
	(5-74μm) Silt (%)	8	18	23	17	12	16	16	8	2	5	17	19	13	8	22	19
	(less than 5 μm) Clay & colloid (%)	37	34	20	17	66	46	21	12	9	21	15	12	9	7	16	32
	Max.diameter(mm)	4.75	4.75	4.75	9.5	4.75	4.75	4.75	9.5	9.5	4.8	4.75	4.75	4.75	19.0	4.75	4.75
	Diam.at 60 (%)	0.80	2.0	0.40	1.2	0.32	0.075	0.060	0.025	3.4	1.7	1.4	0.85	1.2	2.3	2.4	0.22
	Diam.at 10 (%)	—	—	—	0.0014	—	—	—		—	—	0.003	0.004	0.010	0.011	0.002	0.004
Visual soil description		C'H	C'H	CL	CL	C'H	C'H	C'H	CL	ML	CL	CL	CL	CL	CL-ML	CL	CL
Unified soil classification AASHO method		A-7-5	A-7	A-4	A-4	A-7-5	A-7	A-7	A-6	A-4	A-6	A-4	A-4	A-4	A-4	A-4	A-7-6
Compaction test	Optmoisture content (%)	22.3	23.0	22.5	21.5	35.0	31.5	28.0	16.7	15.0	16.3	22.7	18.8	18.2	11.3	17.4	24.2
	Dry density (g/ cm³)	1.562	1.555	1.575	1.562	1.335	1.378	1.435	1.795	1.844	1.792	1.549	1.642	1.738	1.998	1.738	1.540
Permiability test	Falling head (cm/min)	—	—	—	—	—	—	—	0.364 $\times 10^{-6}$	—	—	—	—	1.596 $\times 10^{-6}$	—	—	—
	Constant head	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Triaxial compression test	Angle of internal friction(degree)	—	—	—	—	—	—	—	32.8	—	—	—	—	34.8	—	—	—
	Cohesion (kgf/cm²)	—	—	—	—	—	—	—	0.27	—	—	—	—	0.12	—	—	—
	Condition of drainage	—	—	—	—	—	—	—	CU	—	—	—	—	CU	—	—	—
Consolidation test	Preconsolidation Pressure (kgf/cm²)	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Compression index	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Rock Name		Granite	Granite	Granite	Granite	Sediment	Sediment	Sediment	Tuffaceous Sand stone	Tuffaceous Sand stone	Tuffaceous Sand stone	Granite	Granite	Granite	Tuffaceous Conglomera -te	Tuff	Tuffaceous Sand stone
remark:		* The measurement of L.L & P.L is done only for the material less than 0.42mm diameter. ** The value of ( ) at the column of natural water content is calculated based on the sample for the in-situ density measurement.															

Table 4-25
The result of soil material test

Table 4-26 Gist of the Results of Boring Investigation at the Re-Regulating Pondage Site

Boring No.	Location	Boring Head Elevation (EL m)	Depth of Base Rocks (m)	High Permeability Zone (m)	Water Level of Bore Holes (m)
R-1	Left bank	41.348	3.9	10 or higher	9.7
R-2	Right bank Riverside	27.570	8.3	ditto	3.3
R-3	Right bank	35.495	10 or deeper	ditto	7.1



Segimentary Rocks, mainly argillaceous with interbedded Limestone, Quartzite and Volcanic rocks. Brown dots indicate distribution of Tuff and Lava.

Tuffaceous conglomerate and sandstone including quartzite.

Granite, Granodiorite and related rocks.

Massive crystlline Limestones.

(a)Geological Boundary accurate to within 100 yards.
(b)More doubtful boundary.

Arrow indicating dip of bedding on schistosity.

Fig 4-1 Geological Map Of Damsite Region
(S=1/63.360)

1958 edition published by director of Geological Survey.

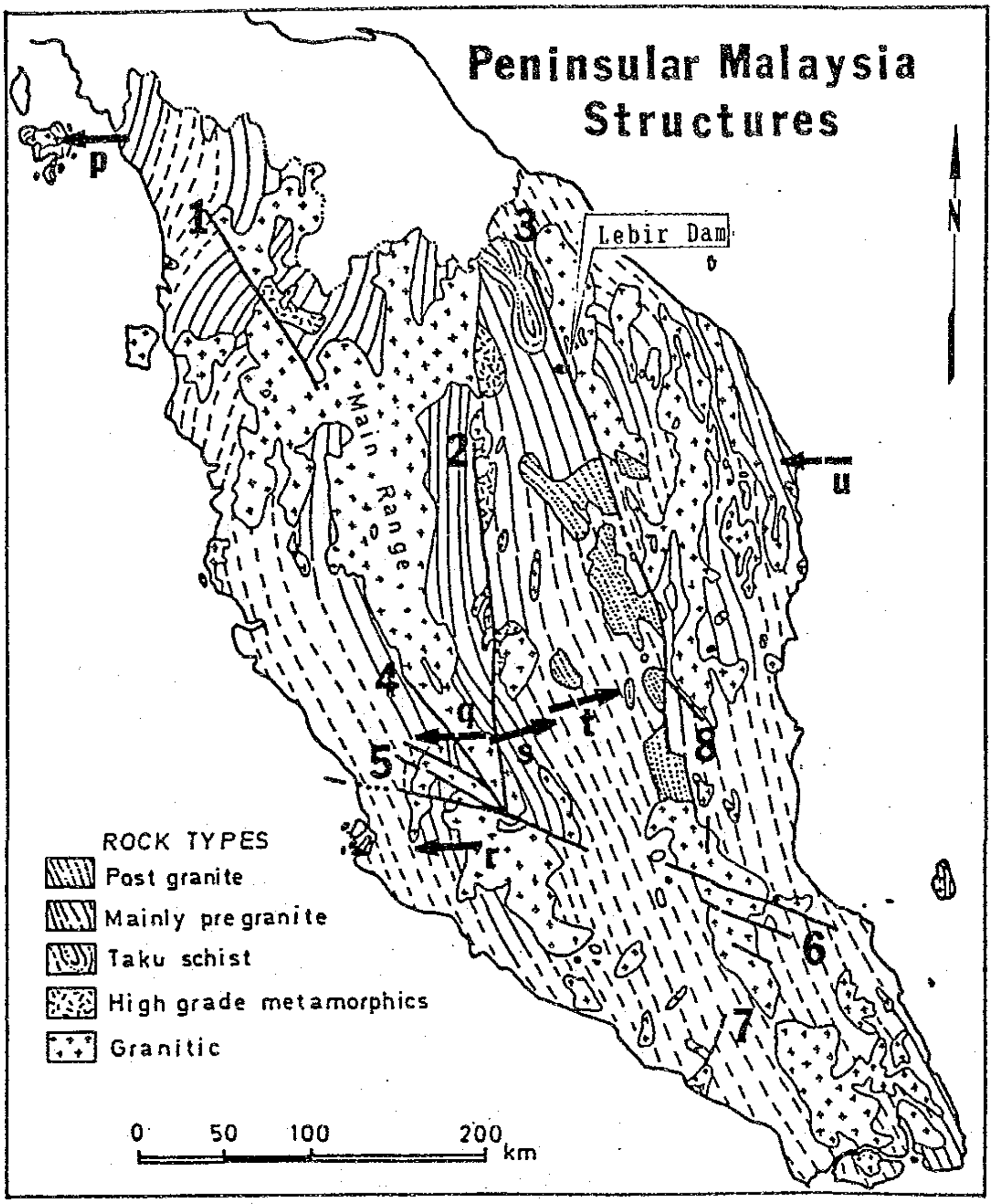


Figure 5. Structural trends in post-granitic rocks (post Early Jurassic), pre-granitic rocks (pre Late Triassic) and problematical Taku Schist, and the distribution of granitic and high-grade metamorphic rocks. Solid lines (dashed where uncertain, dotted where concealed) indicate major strike-slip faults: 1 = Bok Bak; 2 = Kelau-Karak; 3 = Lebir; 4 = Bukit Tinggi; 5 = Kuala Lumpur; 6 = Mersing; 7 = Ma Okil; 8 = Lepar zones. Arrows indicate the dominant directions of tectonic transport by the main and youngest period of intense deformation during Late Triassic-Early Jurassic time.

Fig 4-2 Major tectonic lines in Peninsular Malaysia (S=1/400,000)

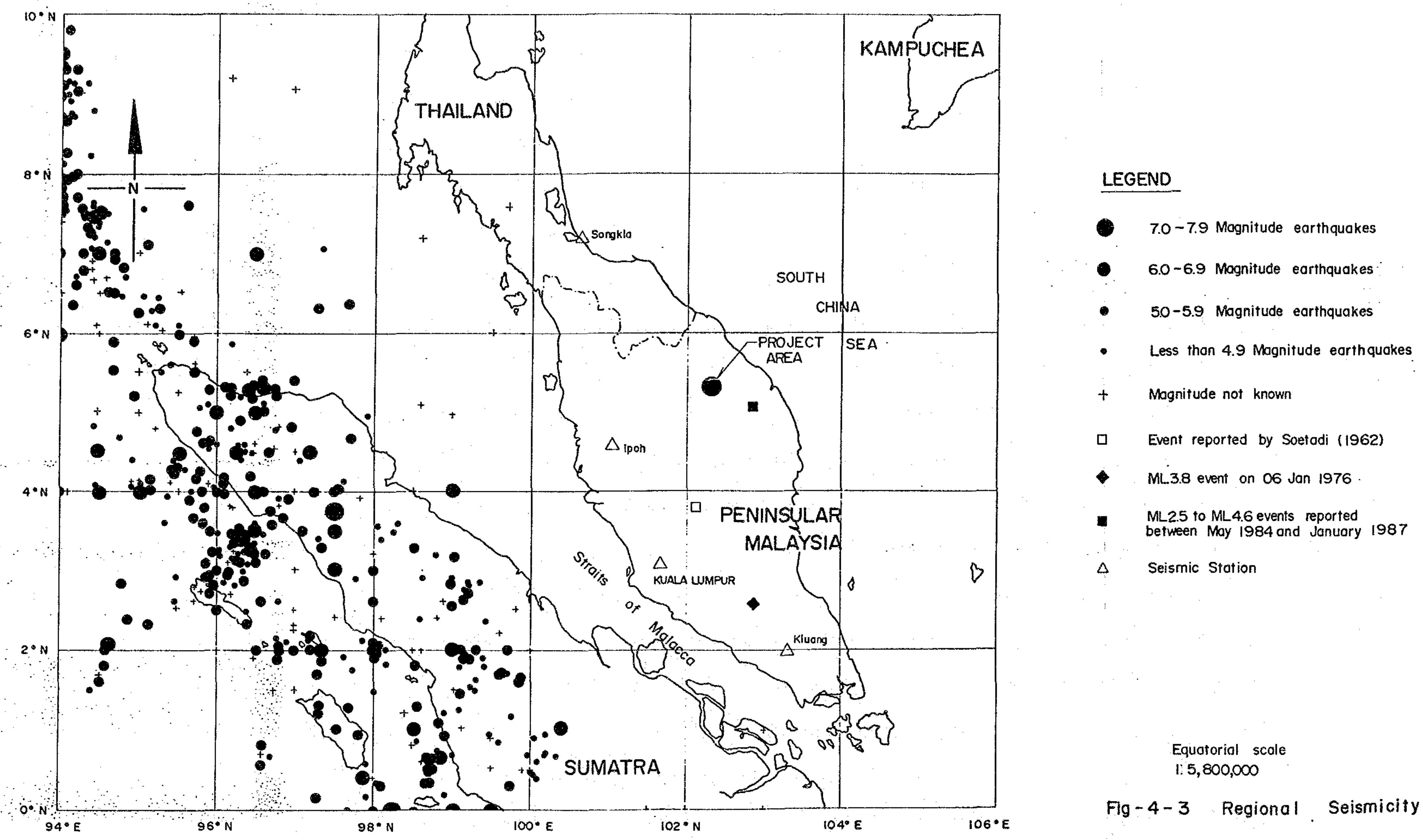
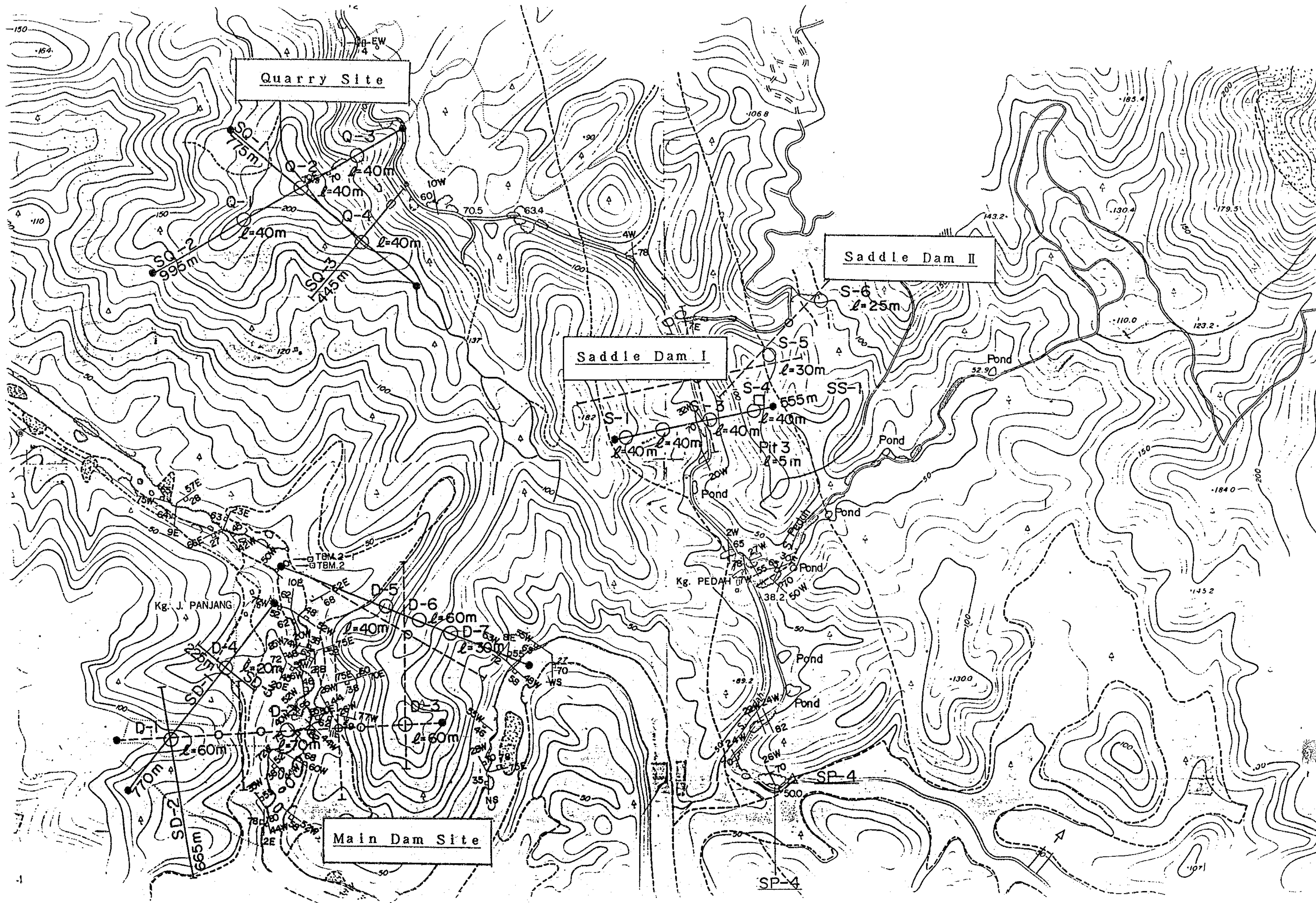


Fig - 4 - 3 Regional Seismicity


Quarry Site
Saddle Dam I
Saddle Dam II
Main Dam Site
SQ-1
775m
SQ-2
995m
SQ-3
445m
Q-1
Q-2
Q-3
Q-4
ℓ=40m
S-1
S-3
S-4
S-5
S-6
ℓ=25m
ℓ=30m
SS-1
555m
Pit 3
ℓ=5m
Pond
Kg. PEDAH
Kg. J. PANJANG
TBM.2
D-1
D-2
D-3
D-4
D-5
D-6
D-7
ℓ=60m
ℓ=20m
ℓ=70m
ℓ=30m
SD-1
225m
SD-2
665m
770m
SP-4

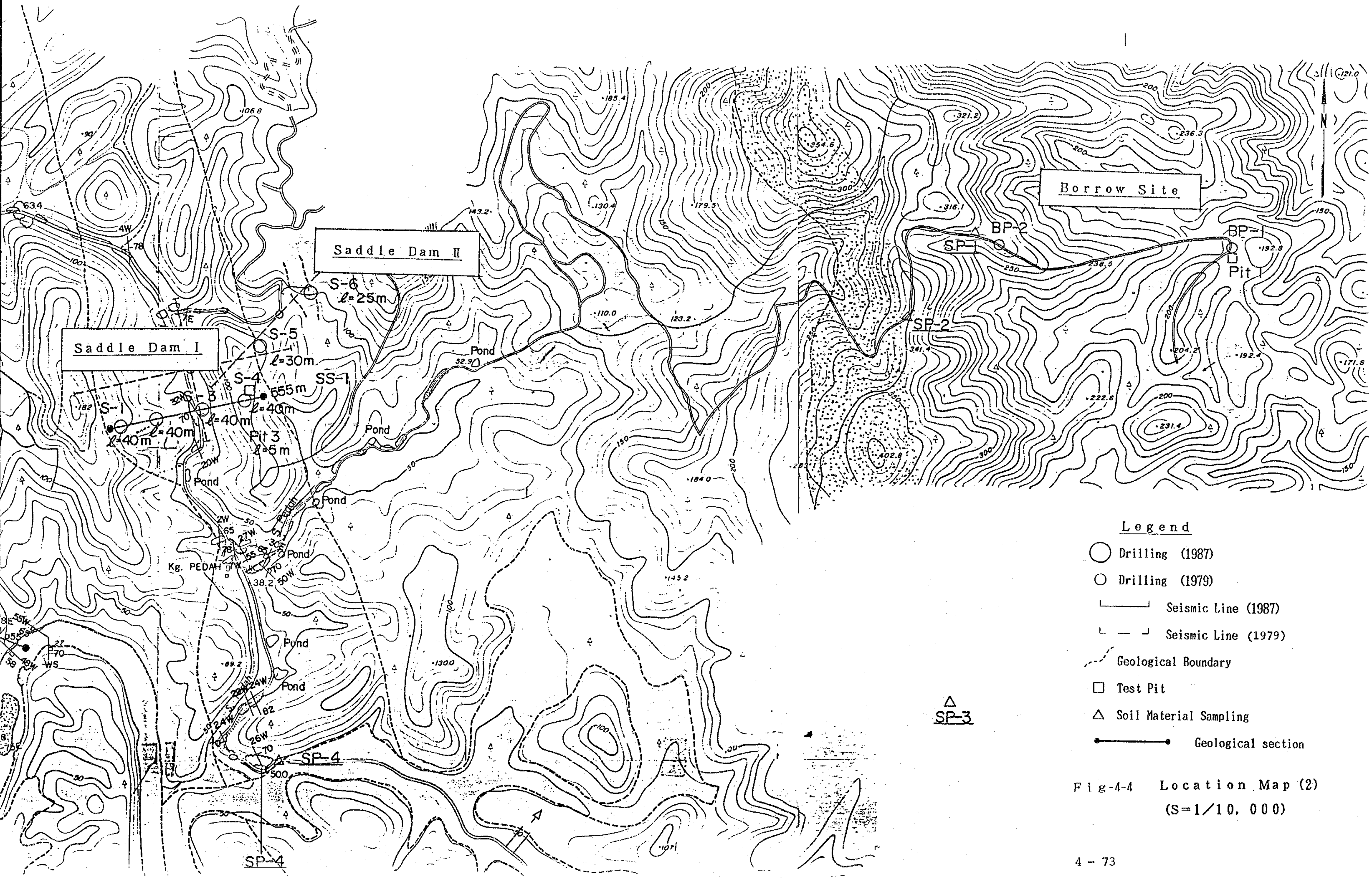


Fig-4-4 Location Map (2)
(S=1/10, 000)

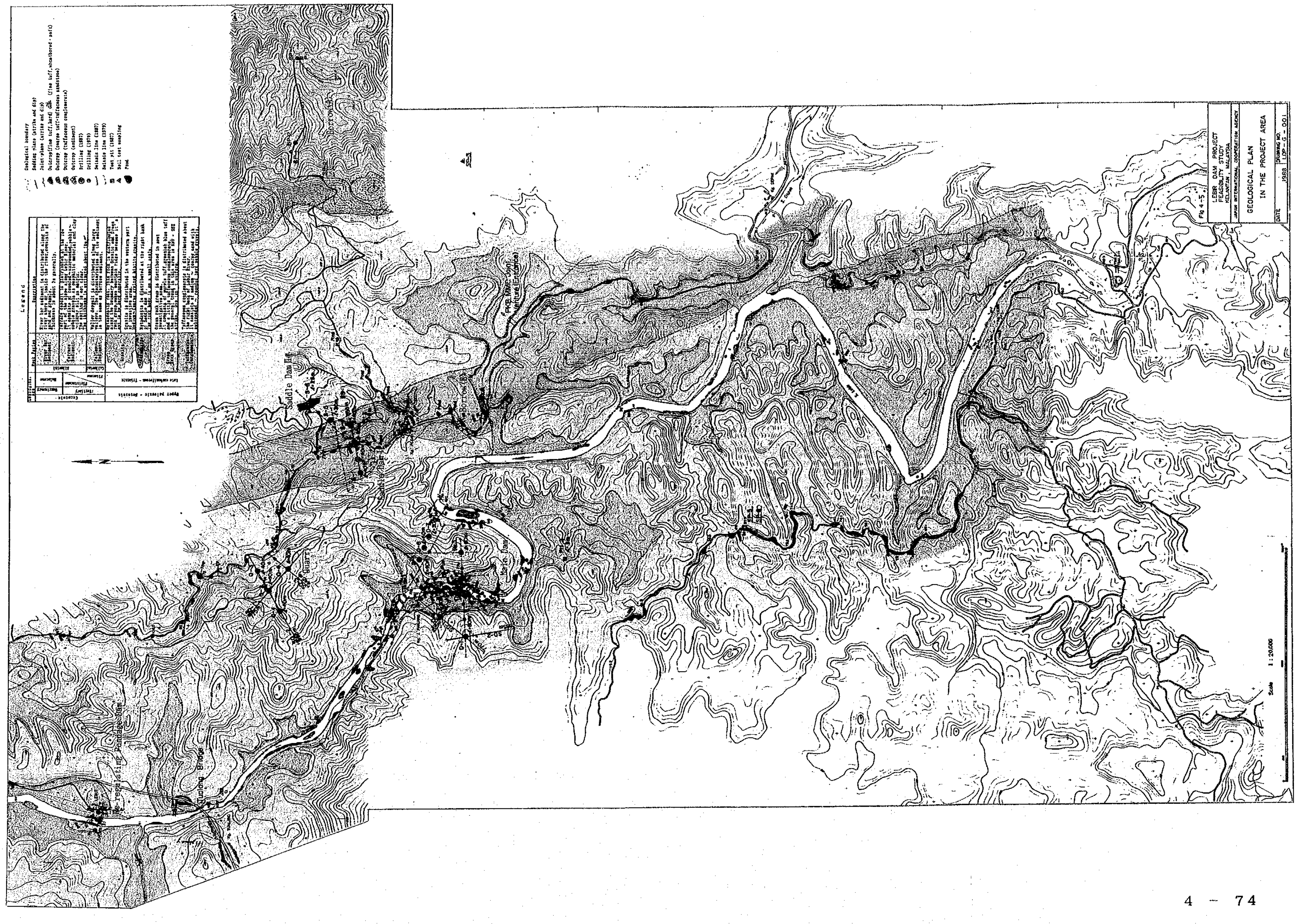
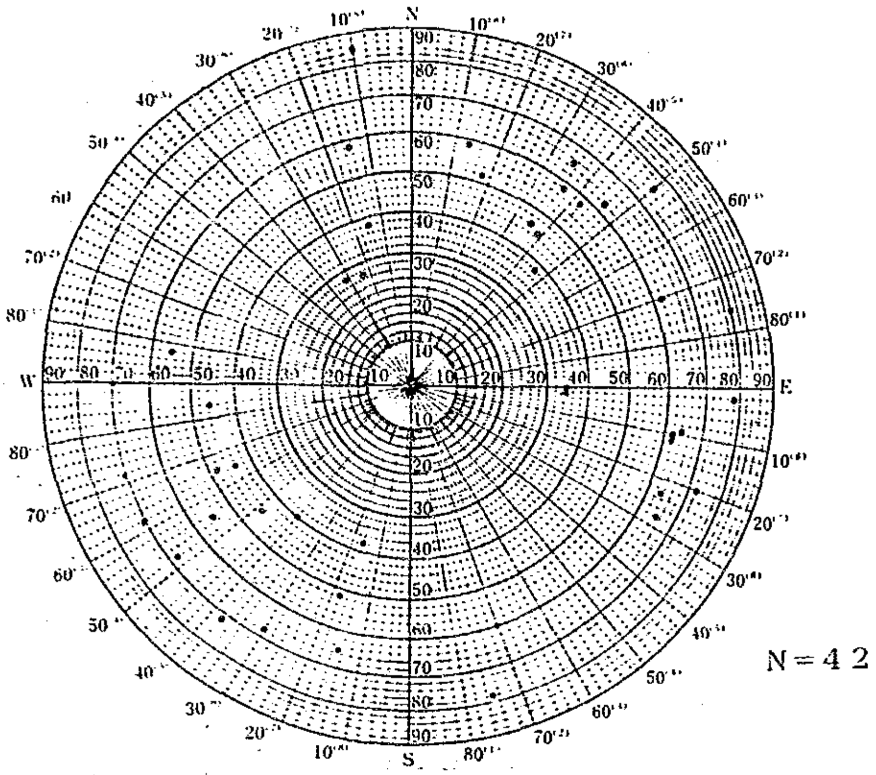


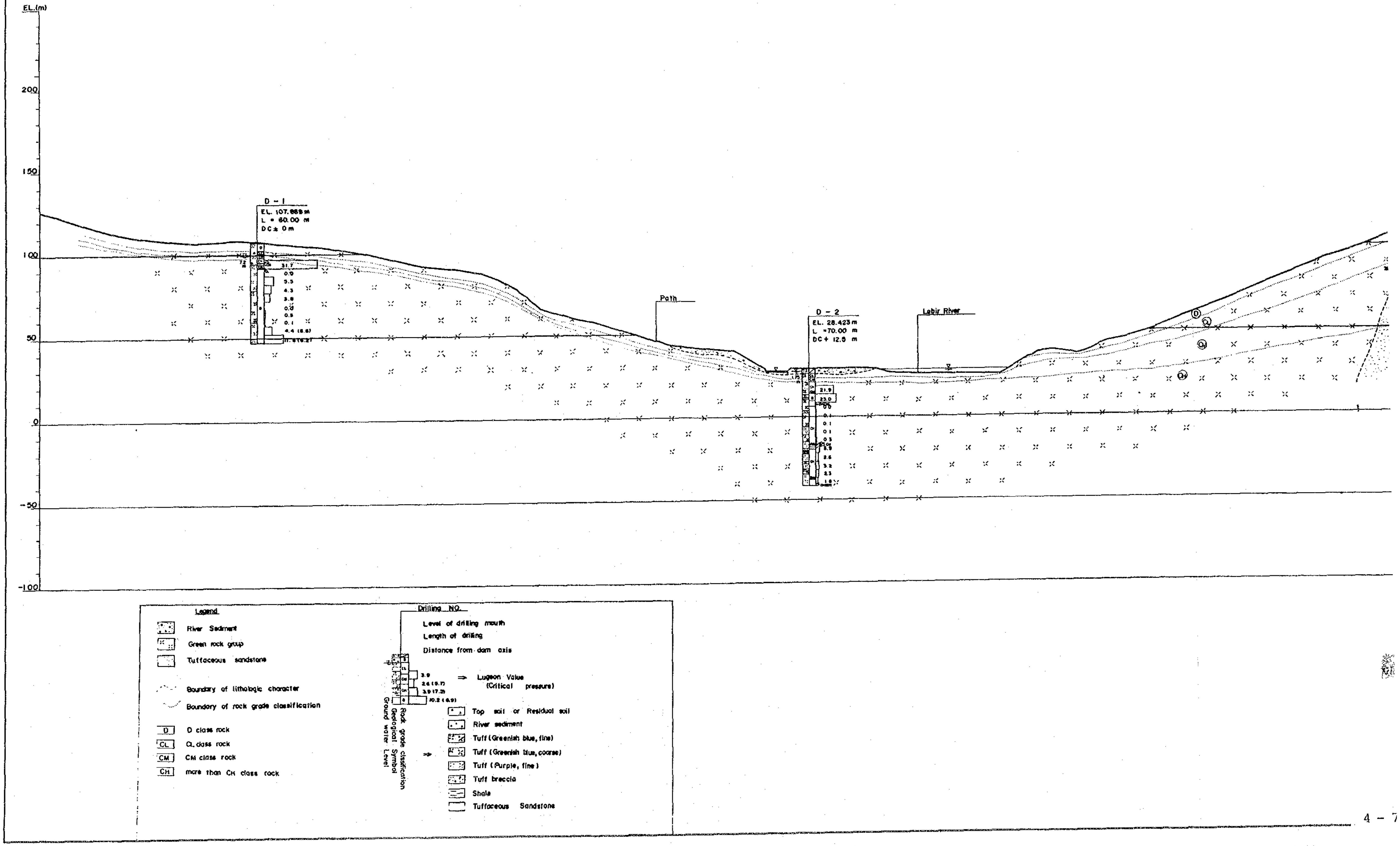
Fig 4-5
LEBIR DAM PROJECT
FEASIBILITY STUDY
KELANTAN, MALAYSIA
JAPAN INTERNATIONAL COOPERATION AGENCY
GEOLOGICAL PLAN
IN THE PROJECT AREA
DATE 1988
DRAWING NO. LDP - G - 001
Legend
Saddle Dam
Main Dam
Quarry
Tualang Bridge
Scale 1:20,000

Fig. 4-6 Joint Directions at the Proposed Dam Site
(By Schmidt Net Projection)



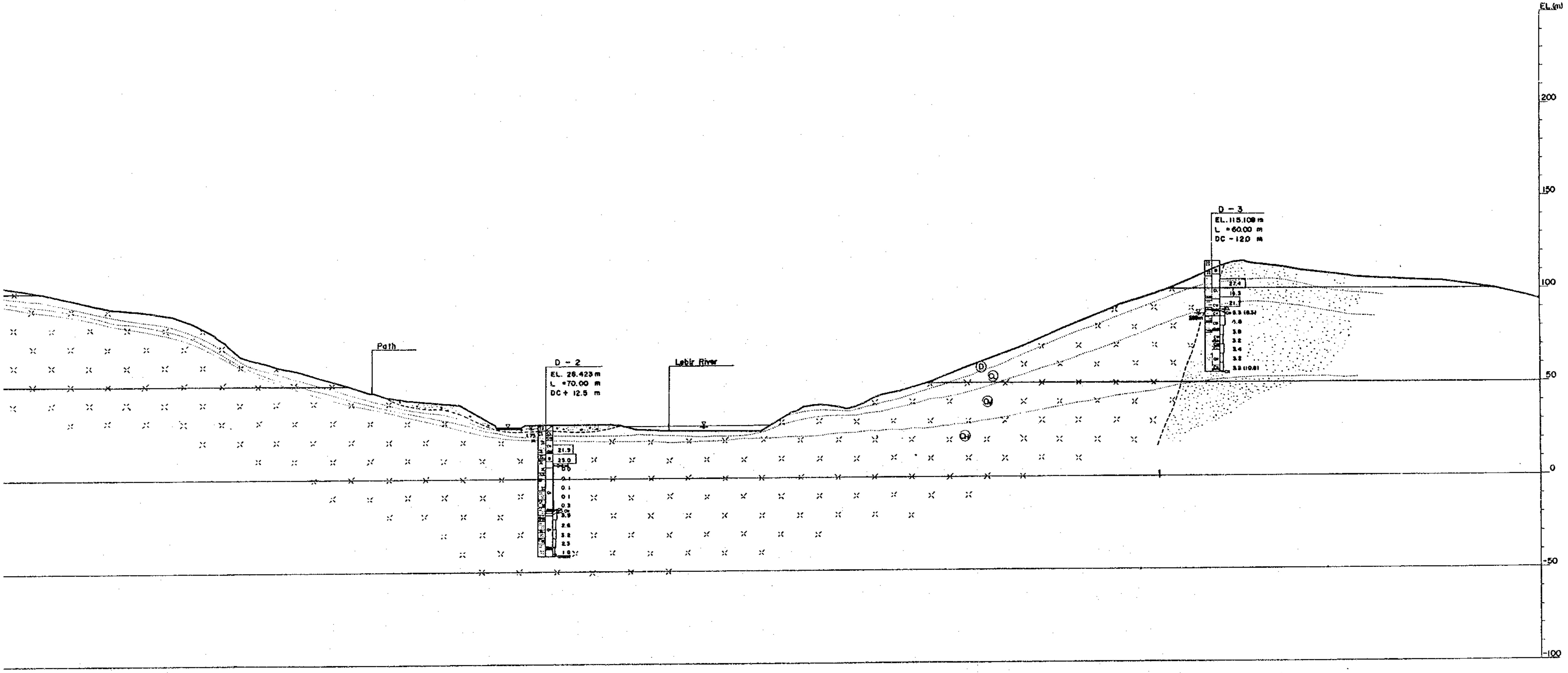
MAIN DAM AXIS

S = 1 : 2,000



MAIN DAM AXIS

S = 1 : 2,000



Drilling NO.
Level of drilling mouth
Length of drilling
Distance from dam axis

3.8 ⇒ Lugeon Value (Critical pressure)
2.6 (5.7)
3.9 (7.2)
10.2 (6.9)

Rock grade classification
Geological Symbol
Ground water Level

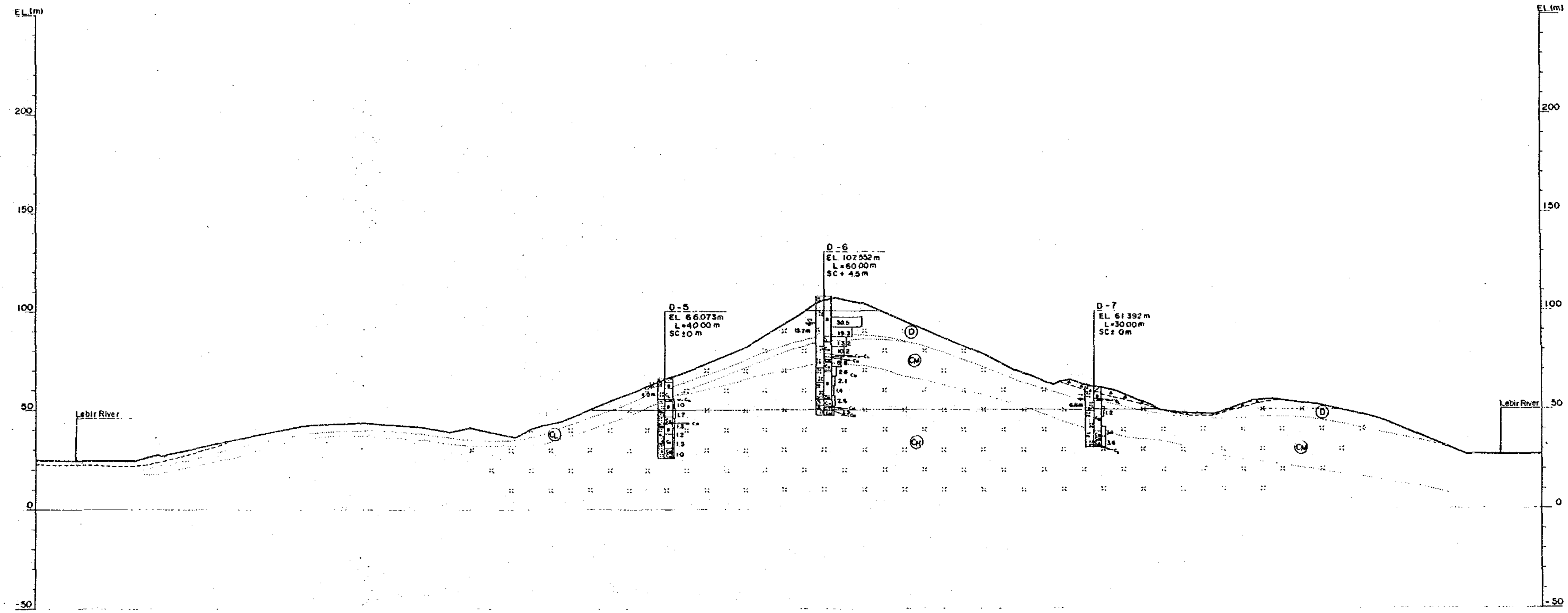
- Top soil or Residual soil
- River sediment
- Tuff (Greenish blue, fine)
- Tuff (Greenish blue, coarse)
- Tuff (Purple, fine)
- Tuff breccia
- Shale
- Tuffaceous Sandstone

Fig4-7

LEBIR DAM PROJECT FEASIBILITY STUDY KELANTAN, MALAYSIA	
JAPAN INTERNATIONAL COOPERATION AGENCY	
GEOLOGICAL SECTION (1) MAIN DAM AXIS	
DATE Mar. 1989	DRAWING NO. LDP-G-002

SPILLWAY

S = 1:2000



Legend

- Top soil or residual soil
- River bed sediment
- Green rock group
- Boundary of lithologic character
- Boundary of rock grade classification
- D — D class rock
- CL — CL class rock
- CM — CM class rock
- CH — more than CH class rock

Drilling NO.
Level of drilling mouth
Length of drilling
Distance from center line

Lugeon Value (Critical pressure)

Ground water Level
Geological Symbol
Rock grade classification

- Top soil or residual soil
- Tuff (greenish blue, fine)
- Tuff (greenish blue, coarse)
- Tuff (purple, fine)
- Tuff breccia

Fig4-8

LEBIR DAM PROJECT
FEASIBILITY STUDY
KELANTAN, MALAYSIA

JAPAN INTERNATIONAL COOPERATION AGENCY

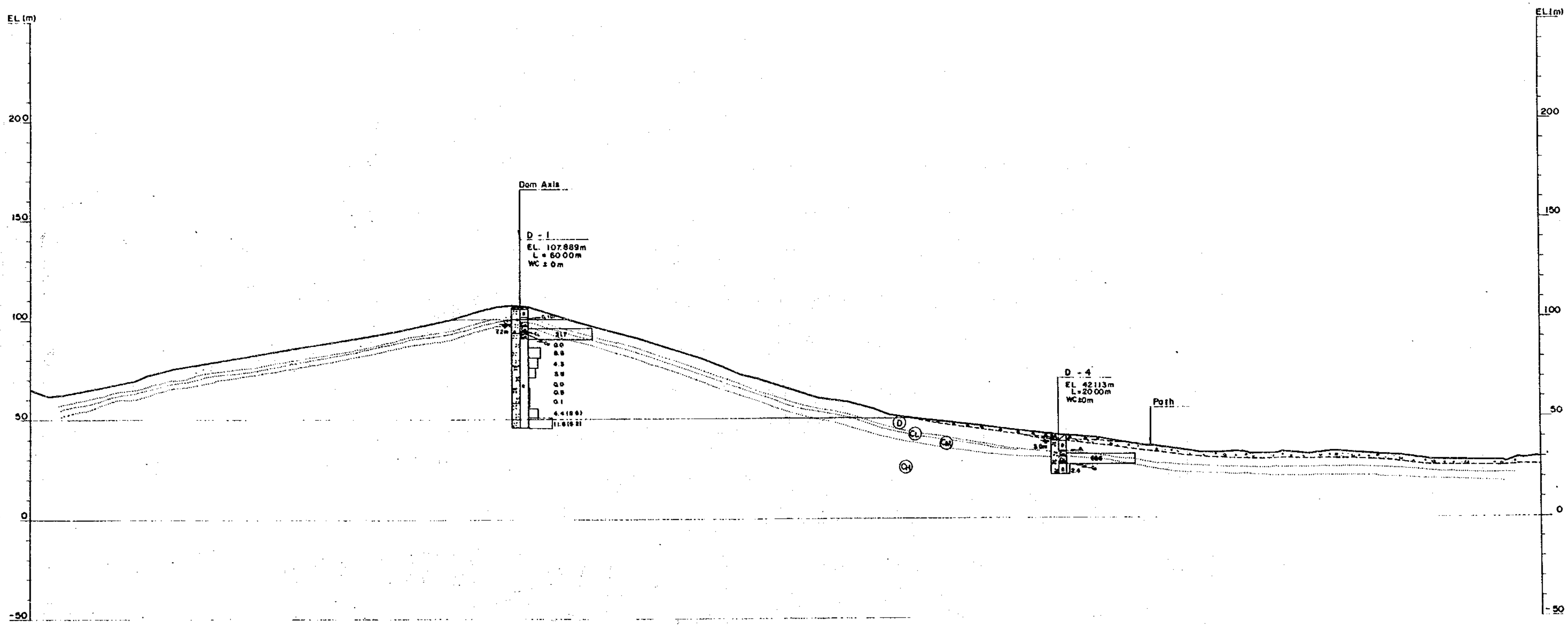
GEOLOGICAL SECTION (2)

SPILLWAY

DATE	DRAWING NO.
Mar. 1989	LOP-G-003

PENSTOCK - POWERHOUSE - OUTLET

S=1 : 2000



Legend

- Top soil or residual soil
- River Sediment or Terrace sediment
- Green rock group
- Boundary of lithologic character
- Boundary of rock grade classification
- [D] D class rock
- [CL] CL class rock
- [CM] CM class rock
- [CH] more than CH class rock

Drilling NO.
Level of drilling mouth
Length of drilling
Distance from center line

Ground water Level
Geological Symbol
Rock grade classification
2.7
Lugeon Value (Critical pressure)

- Top soil or residual soil
- River sediment or Terrace sediment
- Tuff (purple, fine)
- Tuff (greenish, fine)
- Tuff breccia

LEBIR DAM PROJECT
FEASIBILITY STUDY
KELANTAN, MALAYSIA
JAPAN INTERNATIONAL COOPERATION AGENCY

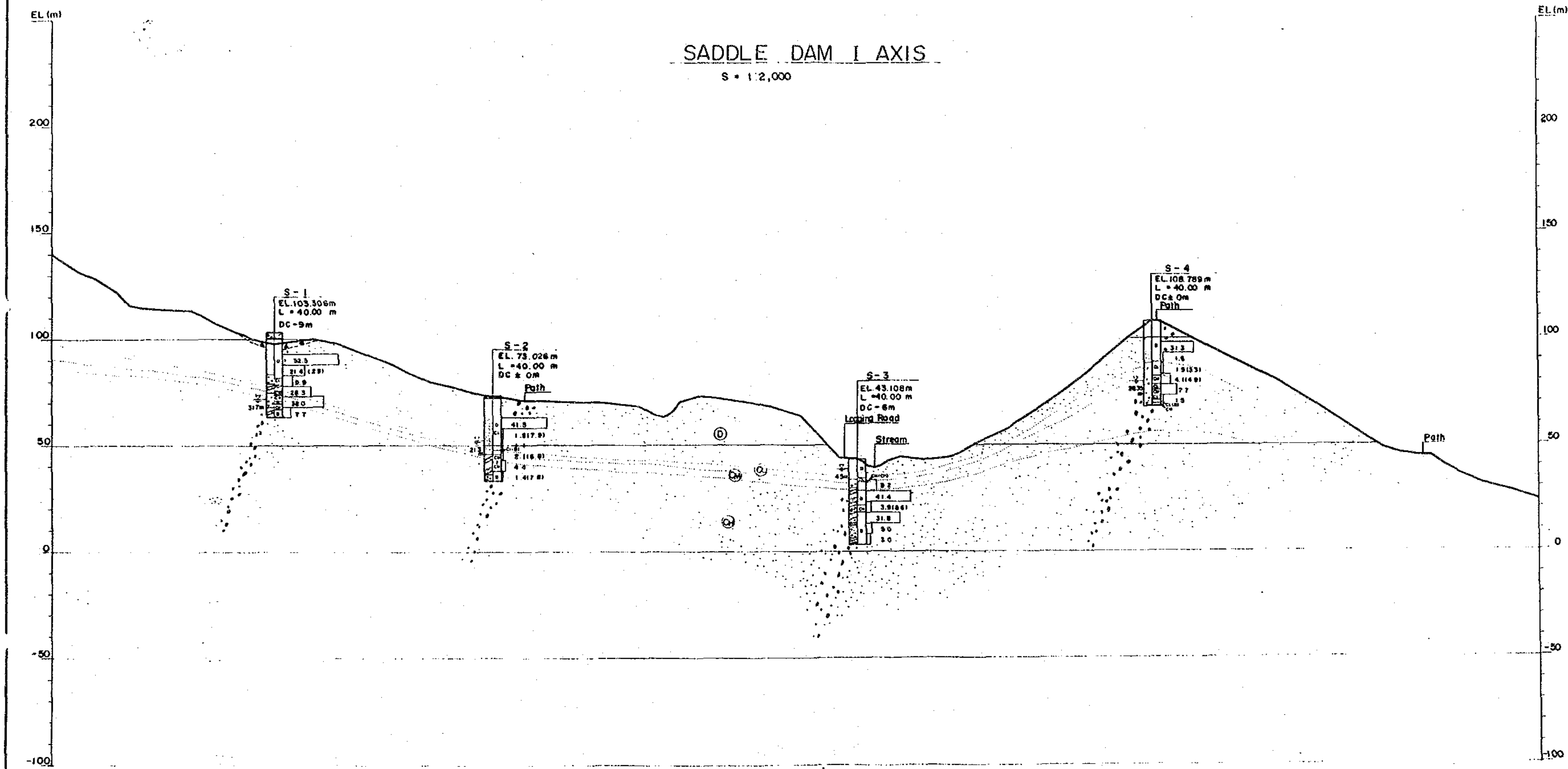
GEOLOGICAL SECTION (3)
PENSTOCK - POWERHOUSE - OUTLET

DATE	DRAWING NO.
Mar. 1989	LOP - G - 004

Fig4-9

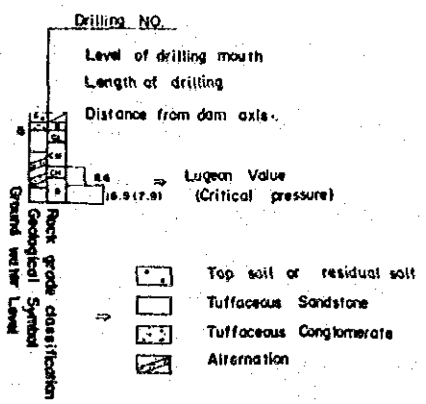
SADDLE DAM I AXIS

S = 1:2,000



Legend

- Top soil or residual soil
- Tuffaceous Sandstone
- Tuffaceous Conglomerate
- Boundary of lithologic character
- Boundary of rock grade classification
- Boundary of seismic velocity
- D : D class rock
- CL : CL class rock
- CM : CM class rock
- CH : more than CH class rock

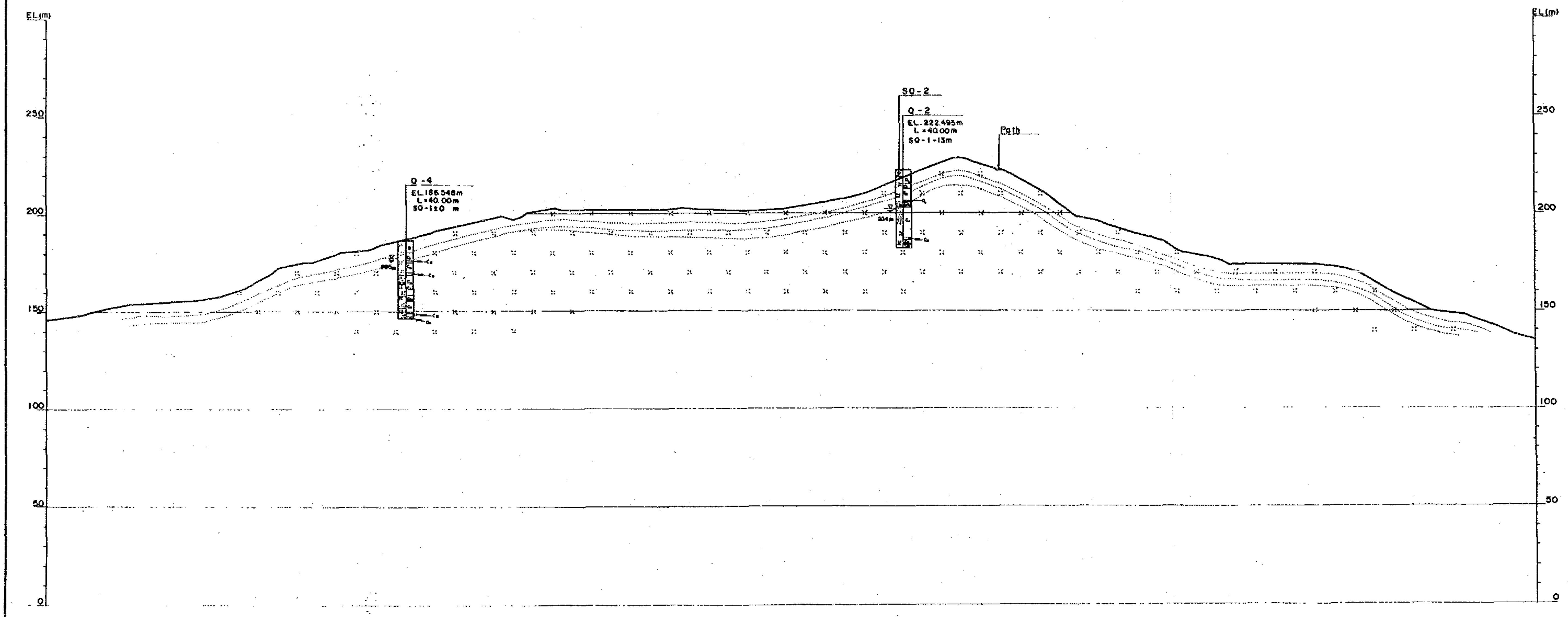


LEBIR DAM PROJECT FEASIBILITY STUDY KELANTAN, MALAYSIA	
JAPAN INTERNATIONAL COOPERATION AGENCY	
GEOLOGICAL SECTION (4) SADDLE DAM I AXIS	
DATE Mar. 1989	DRAWING NO. LDP-G-005

Fig4-10

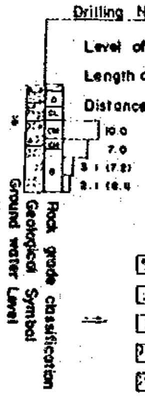
QUARRY(SQ - I)

S = 1:2000



Legend

	Green rock group
	Boundary of lithologic character
	Boundary of rock grade classification
	Boundary of seismic velocity
D	D class rock
CL	CL class rock
CM	CM class rock
CH	more than C_H class rock



	Top soil or residual soil
	Tuff (purple, fine)
	Tuff (purple, coarse)
	Tuff (greenish blue, fine)
	Tuff breccia

LEBIR DAM PROJECT
FEASIBILITY STUDY
KELANTAN, MALAYSIA

JAPAN INTERNATIONAL COOPERATION AGENCY

GEOLOGICAL SECTION (5)

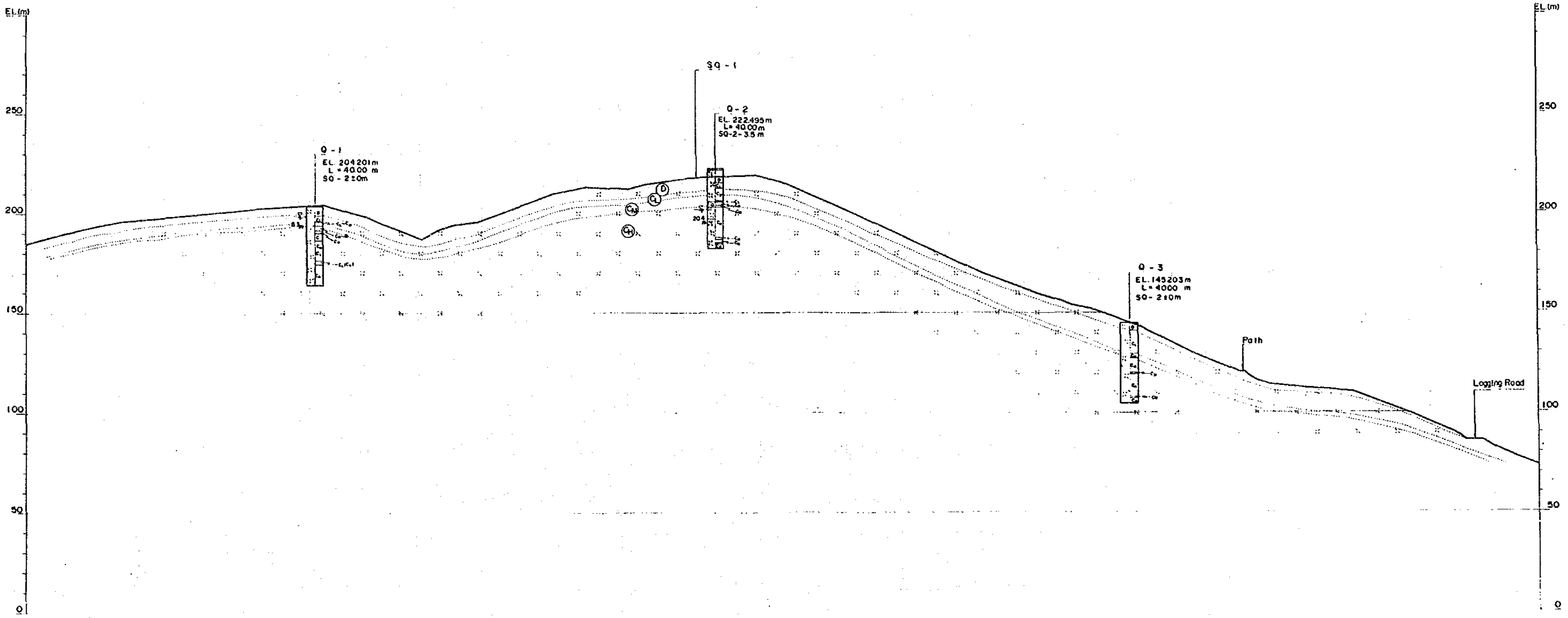
QUARRY (SQ - I)

DATE Mar. 1989 | DRAWING NO. LDP - G - 006

Fig4-11

QUARRY (SQ-2)

S = 1:2000



Legend

- Green rock group
- Boundary of lithologic character
- Boundary of rock grade classification
- Boundary of seismic velocity
- [D] D class rock
- [CL] CL class rock
- [CM] CM class rock
- [CH] more than CH class rock

Drilling NO.

Level of drilling mouth

Length of drilling

Distance from seismic line

10.0

7.0

3.1 (7.2)

2.1 (8.1)

Lugeon Value (Critical pressure)

Rock grade classification

Geological Symbol

Ground water level

- Top soil or residual soil
- Tuff (purple, fine)
- Tuff (purple, coarse)
- Tuff (greenish blue, fine)
- Tuff breccia

LEBIR DAM PROJECT FEASIBILITY STUDY KELANTAN, MALAYSIA	
JAPAN INTERNATIONAL COOPERATION AGENCY	
GEOLOGICAL SECTION(6)	
QUARRY (SQ-2)	
DATE Mar. 1989	DRAWING NO. LDP - G - 007

Fig4-12

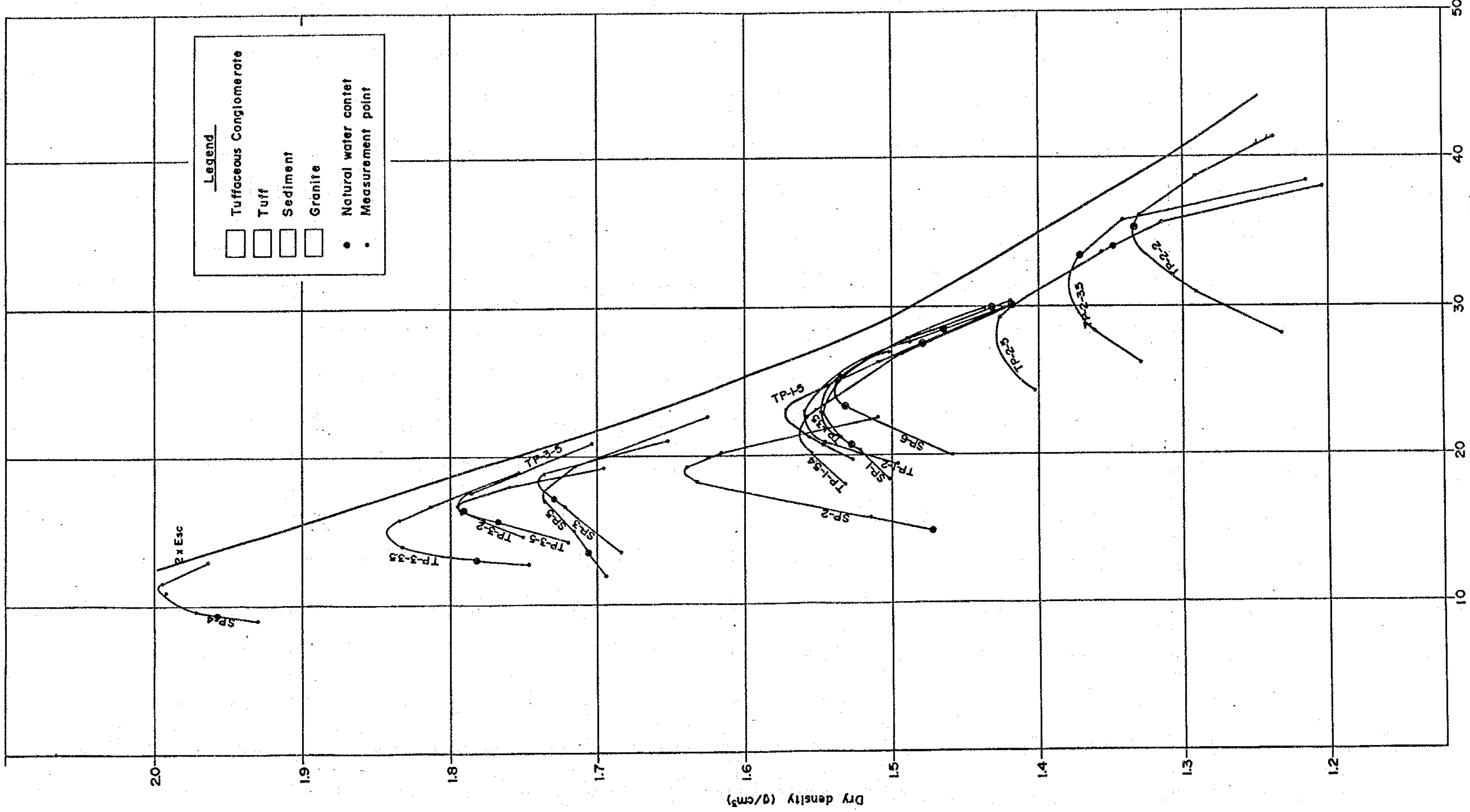


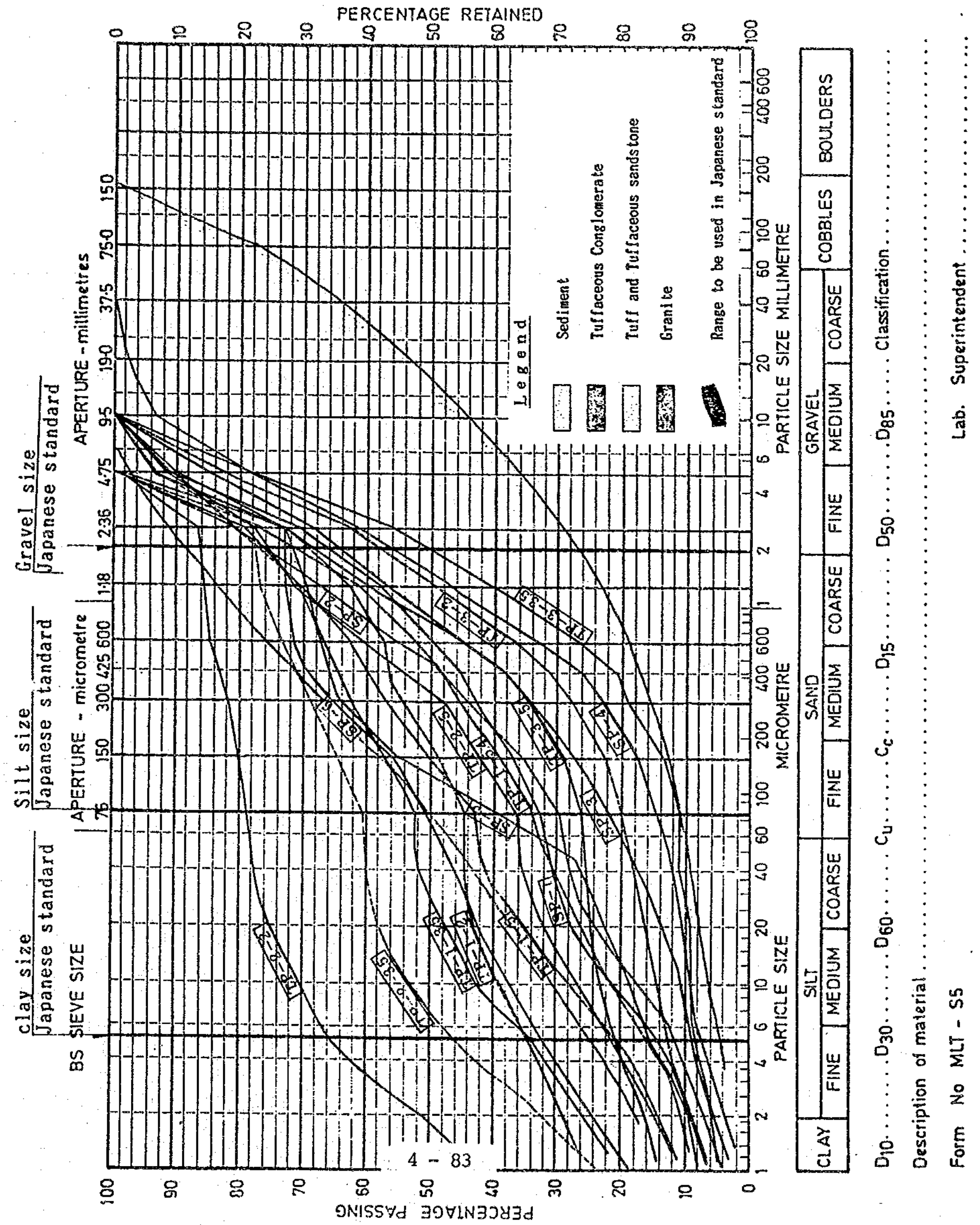
Fig 4-13
Compaction
Curve

Fig 4-14 PARTICLE SIZE DISTRIBUTION CHART

Structure:.................... Zone:..........

Tested by.........Plotted by ....Checked by.........SAMPLE NO.................

Date.............Date.............Date.............REGISTRATION...............



5. Hydrology

5. Hydrology

5.1. General Description of the Kelantan River Basin

The Kelantan River Basin extends approx. 60 km east to west and 130 km north to south, and is enclosed by the 1,000 m to 2,000 m high central mountain range, with the highest peak of G. Tahan (EL. 2,207 m) in the south. There is a north to south 1,000 m class mountain range in the east bounded by the Terengganu State, and a 500 m high mountain range in the west divided by the Golok River Basin. Only the north side of the basin is open toward the South China Sea.

Generally, the Kelantan River Basin is classified into two parts, a mountainous basin and a flat terrain basin. The basin in the mountainous area includes the watershed of the Galas River, the main stream, and those of three major tributaries, namely, the Pergau, Lebir and Nenggiri rivers. The above-mentioned Galas River and Lebir River join in the flat terrain area, and thereafter, the river is known as the Kelantan River. This river meanders gently in the vast downstream alluvial plain, and discharges its load into the South China Sea near Kota Bharu in the north.

The Lebir River is located on the east rim of the mountainous basin. It originates in the Taman Negara on the northern skirt of the central mountain range, and has a total catchment area of about 3,400 km^2 joining its major tributaries of the Aring, Relai, Chalil, Depak, and Sam rivers on the course down to the confluence with the Galas River. As mentioned above, the east side of the Lebir basin is the coastal mountain range and the west side is approx. 300 m high dome shaped small mountains, by which the basin is divided from the watershed of the Galas River.

The Lebir River is a kind of mountain river, but the course is very gentle, with a riverbed slope of about 1/2,600 in the downstream reach of 100 km out of the total 120 km.

The river valley is also broad and slopes very gently at both banks.

The stream flow gauging stations in the Kelantan river system are shown on the location map (see Fig. 5-1). The catchment area of each gauging station is given below:

Catchment Areas of Stream Gauging Stations

Station	River	Catchment Area km 2
Tualang	Lebir	2,480
Bertam	Nenggiri	3,950
Dabong	Galas	7,480
Guillemard	Kelantan	12,100

5.2. Hydrological Data

5.2.1. Low Flow Data

The following data in respect of the Kelantan River and the Lebir River have been collected;

	Daily Discharge	Monthly Average Discharge
Guillemard Discharge Gauging Station	from 1958 to 1984	from 1950 to 1984
Tualang Discharge Gauging Station	from 1976 to 1984 (much missing data involved)	from 1976 to 1984

5.2.2. Flood Data

(1) Flood measurement record

As stated in the previous section, four gauging stations have been installed over the Kelantan river system. Their locations are indicated on Fig. 5-1. Two types of flood data have been collected and classified in this Study. The first is the peak flood stage records, and the other is the time-duration flood stage records.

The peak flood stage records are as described hereunder: The largest flood so far found in the records at Guillemard Bridge was 16,000 m^3/sec. on January 6, 1967. (There is a record of 27,000 m^3/sec. in 1926, but the reliability is questionable).

The place where most flood records have been taken is at the Guillemard Bridge Site. The records cover every year from 1940 to 1986.

On the other hand, the records available at the Tualang site have been collected since 1926. However, many are missing. Only 16 flood records can be correlated with those at the Guillemard Bridge. All available data is shown on Tables 5-4 to 5 in Appendix. The values in the tables are flood discharges converted from the flood stages.

Also, collected data includes the records of hourly stages of floods covering the period from 1965 to 1986. These records are shown on Appendix Figs. 5-4 to 13. The floods obtained from the records are tabulated by the four gauging stations shown below:

Period of Flood \ Station	Tualang	Bertam	Dabong	Guillemard
	m^3/s	m^3/s	m^3/s	m^3/s
1965 Dec. 1 - 6	--	--	--	5,770
1967 Jan. 2 - 6	--	--	--	16,000
1969 Nov.29 - Dec.4	--	--	--	6,140
1973 Dec. 5 - 9	--	--	5,540	9,600
1974 Dec. 26 - 1975 Jan. 7	--	--	2,510	4,320
1981 Nov.30 - Dec.4	570	--	1,640	1,870
1982 Dec.12 - 16	2,810	1,080	3,630	7,120
1983 Dec. 3 - 7	3,900	--	5,950	12,010
1984 Dec.21 - 25	3,430	--	4,220	7,740
1986 Nov.25 - Dec.3	--	--	3,380	6,900

-- indicates that data is missing.

The values of flood discharges in the above table are converted by using the stage-discharge rating curves (Fig. 5-14 through Fig.5-17 in Appendix) modified in this Study on the basis of the stage discharge rating curves (Tables 5-6 to 9 in Appendix) prepared by DID, in order to facilitate the estimation of discharges at higher water stages.

For instance, the stage-discharge rating curve for Guillemard Bridge site was prepared by modifying the curve prepared by DID in the following manner:

The part of the curve relating to high water stages more than 20 m, was modified based on the result of uniform flow calculation using the river cross-sectional shape at Guillemard Bridge site, and the river slope of its upstream

and downstream course at the site, with application of the Manning roughness coefficient. (refer to Figs. 5-18 and 5-19 in Appendix)
(n = 0.033 and the river-bed gradient of 1/6,000.)

The stage-discharge rating curves for the foregoing four gauging stations thus modified are shown in Appendix Figs. 5-14 to 5-17. Attention is invited to the fact that the conversion of Appendix Figs. 5-4 to 5-13 has been made in use of the curves prepared by DID before modification.

(2) Rainfall Data

Collected rainfall data includes the records of daily rainfall and hourly rainfall. The daily rainfall records have been collected from a total of 72 gauging stations. All of these stations are located inside the Kelantan River Basin. The data period is from 1947 to 1987.

Conversely, hourly rainfall records were collected from four stations over the period since 1980. These are very small in number in relation to the large study area. Among those four stations located on the Lebir River are "Kampong Lebir" and "Jeram Panjang". Others such as "Kampong Kuala Betts (No.17)" and "Cegar Atas (No.19)" are located in the Nenggiri River basin.

The data periods of each gauging station are listed in Appendix Table 5-10.

(3) Records of River Cross Sections

19 cross sections were collected for the Kelantan River, covering 86 km of the river course downstream of the confluence with the Lebir River. The survey was done at 5 km intervals in 1976.

In addition to this, the cross sections of the Lebir River around the proposed dam site were surveyed at the initial stage of this study. This locations of these survey sections are shown in Appendix Fig.5-20.

5.3. Low Flow Analysis

5.3.1. Mean Monthly Flow at the Dam Site

To estimate energy generation of the Project, it is necessary to use long term river discharge data at the dam site. So far, however, no actual measurement records at the proposed dam site are available. The only available actual measurement data are the daily inflow records at the Tualang site for the period from 1976 to 1984, which is still not sufficient to estimate the long term river discharges. It is, therefore, desirable to estimate the correlation with actual measurement data available at other suitable neighbouring stations. Fortunately, at Guillemard Bridge, located about 30 km downstream of the confluence of the Lebir River and the Galas River, actual measurement records covering the period from 1950 to 1984 are available. The correlation of mean monthly discharges at the Tualang site and at Guillemard Bridge was consequently studied. As a fairly good correlation was found between them, mean monthly flow at the Tualang site could be obtained by a linear regression formula on the basis of actual measurement records at Guillemard Bridge. (Appendix Figs. 5-1 to 3, and Table 5-1)

The mean monthly runoff at the Jeram Panjang dam site is the value estimated by applying the ratio of the catchment area of the Jeram Panjang dam site to that of the Tualang site.

Any missing monthly flow data in the records at Guillemard Bridge was supplemented by the linear regression formula in use of the data available at the Iskandar Bridge on the Perak River, because of the good correlation between them (Appendix Fig. 5-21). Other missing monthly data at both the Guillemard and

Iskandar Bridges was supplemented by using the monthly flow in the previous Interim Report, which was estimated synthetically by using a tank model based on the daily flow data at the Lalok site.

Mean annual stream flow at the Jeram Panjang site thus derived is 112.57 m^3/s, and at Guillemard Bridge it is 567.13 m^3/s. (Refer to Tables 5-1 and 5-2).

5.3.2. Daily Flow at Tualang

The irrigation plan in the downstream area of the dam site requires a 10-day mean daily flow. The base data for use in the downstream irrigation plan is the daily flow at Guillemard Bridge. It is, however, necessary to assess the daily discharges downstream of the dam site (Tualang site) in order to estimate flow conditions after the completion of the dam.

Daily discharge records are available at Guillemard Bridge for the period from 1958 to 1984, but those at the Tualang site are only for the period from 1976 to 1984. (There is a lack of records during the period of 1982 to 1984.)

Similar to mean monthly flow, daily discharges at both locations also show good correlation. Therefore, daily flow at the Tualang site was also estimated by the linear regression method (Appendix Figs. 5-22 through 5-24).

No supplementation of the missing data at Guillemard Bridge was made at this stage of the Study.

Appendix Fig. 5-25 shows the present 10-day mean flow at Guillemard Bridge (Appendix Table 5-21), while Figs. 5-26 and 5-27 represent the estimated 10-day mean flow for two discharge cases from the dam; 70 m^3/s and 80 m^3/s. (Appendix Table 5-22)

5.3.3. Rating Curves (at Tualang and at Jeram Panjang Dam site)

Measurement of discharge river water level has been conducted at the Tualang site since September, 1979. The data does not include any record of more than 1,000 m^3/s except for a record of 3,900 m^3/s obtained by float measurement on November 29, 1979.

These actual measurement records were processed for extension by the least square method, and a rating curve up to 4,000 m^3/s was developed. The discharges of more than 4,000 m^3/s were estimated by uniform flow depth corresponding to each discharge, on the basis of the river cross-section surveyed at the Tualang site, where an automatic discharge gauge is installed.

The discharges at the Jeram Panjang Dam site were estimated by use of the rating curve for the Tualang site and by non-uniform flow calculation, based on the river cross-section surveyed at this time. (Refer to Appendix Fig. 5-28).

5.4. Flood Analysis

5.4.1. Analysis Procedures

The objective of this analysis is to obtain probable flood flow rates at the Lebir Dam site and at the Guillemard Bridge site. This is because the Lebir Dam site is the proposed dam site and Guillemard Bridge is the base point for the study of flood control on the Kelantan River.

One of the methods for deriving probable flood flow is the simulation of probable flood by use of a runoff model, prepared for the estimate of runoff from probable rainfall. This method will make it possible to simulate flood variations from the beginning to the recession, as time elapses. This method will, however, require sufficient precipitation data and flood discharge-duration measurement records. An adequate model for conversion of rainfall to runoff will also be critically important.

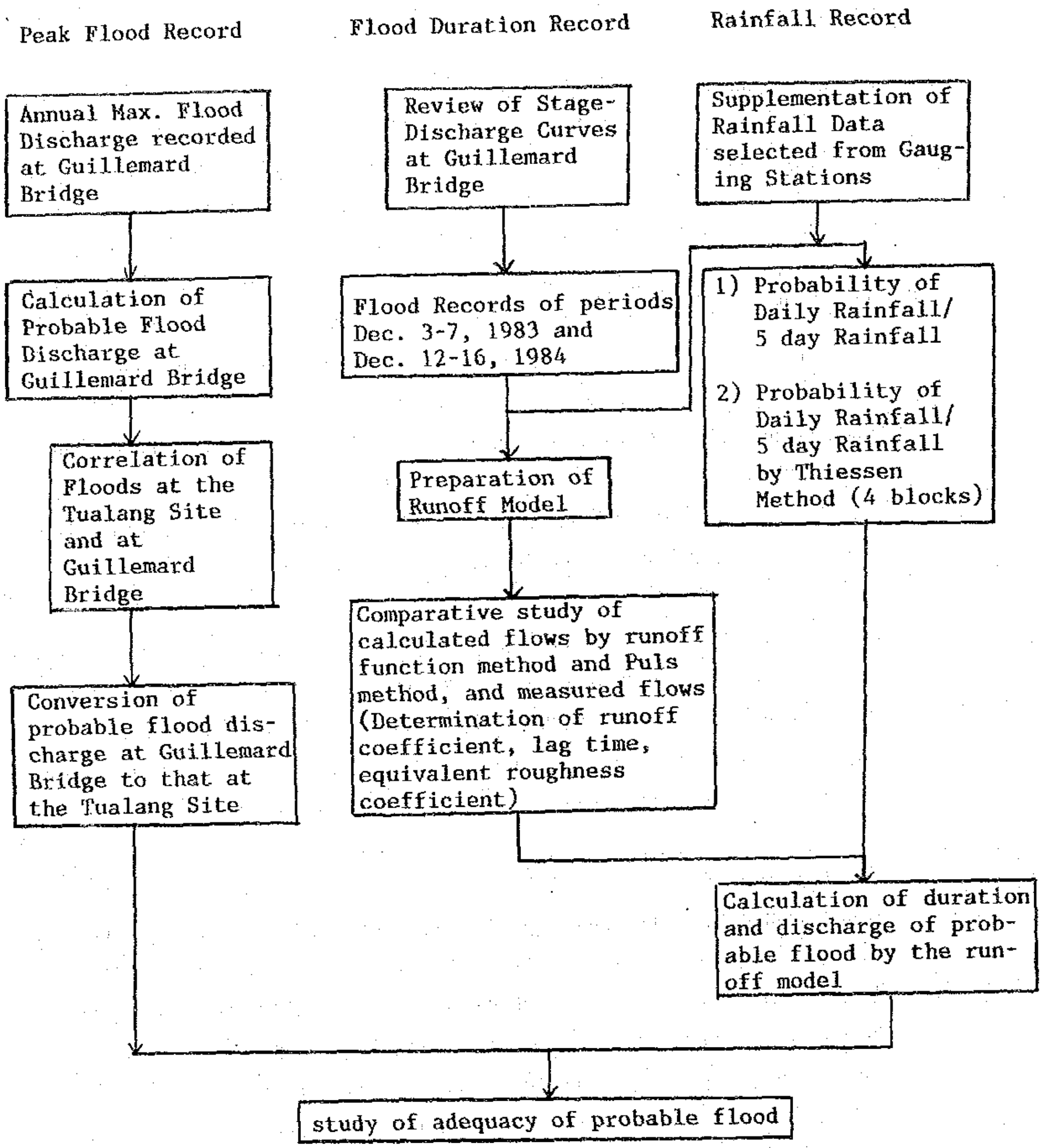
In the meantime, since a large amount of peak flood discharge data is available at the Guillemard Bridge Site, another consideration is to introduce this data directly into the probability calculations. Generally speaking, this method would give less accurate results when the subject basin's land utilization condition suddenly changes in the data period, or in the future if any change especially affecting retention functions over the broad area within the basin, is expected. On the other hand it will present an acceptable range of accuracy where there is less rainfall data, and the land utilization in the basin has changed without having greatly affected the water retention function of the basin.

Land utilization conditions in the drainage basins of the Lebir Dam site and of the Guillemard Bridge site have changed due to rubber and palm oil plantation development. However, these changes are not seen to have basically changed the original water retention function of the land in the basin.

It is, therefore, considered that the method of directly introducing the peak discharges into probability calculation is applicable for this basin. Although one deficiency of the method is that the time series variation of probable flood discharge cannot be estimated, the accuracy of a model for runoff analysis from rainfall can be assessed as valid, if the values of probable peak discharges computed by the above method, and the peak discharges estimated in the runoff analysis based on the rainfall data are compared, and agreed.

Based on the above concept, the study was conducted according to the flow chart on the next page.

Flow Chart of Hydrological Study



5.4.2. Probable Flood Calculated from Peak Flood Records

Annual maximum discharges measured at Guillemard Bridge every year during the period, 1940 to 1986 shown on Appendix Table 5-4, were plotted on probability paper. (See Fig. 5-2). Probable floods calculated by the Gumbel-Chow method based on these figures are tabulated as follows:

Probable Floods
(Using the Gumbel-Chow Method)

Probable Return Period in years	Guillemard	Tualang
	m^3/s	m^3/s
10,000	31,320	15,660
1,000	24,503	12,252
200	19,730	9,865
100	17,671	8,834
20	12,845	6,423
10	10,714	5,357
5	8,493	4,247

As there are few measurement discharge records at the Tualang Site located downstream of the Lebir dam site, this data is not directly introduced into the probability calculation. The chart showing the relationship between the peak discharge of the same flood measured at Guillemard Bridge and at the Tualang Site is used. (See Fig. 5-3) On this chart, an envelope line is drawn to such an extent that the peak discharges at the Tualang Site are over-estimated as shown on Fig. 5-3. This envelope line shows that peak discharges at the Tualang Site are half of those at Guillemard Bridge.

5.4.3. Probable Flood Based on Runoff Analysis of Rainfall

5.4.3.1. Processing and Analysis of Rainfall Data

(1) Selection of gauging station

There are a total of 72 rainfall gauging stations within the Kelantan River Basin. The selection of the gauging stations subject to the study has been done twice. The first selection was based on the following:

- Locations should be upstream of Guillemard Bridge.
- Locations can be confirmed on the topographical map.
- The records must have been made over a long term.
- The distances between the respective stations must be almost uniform.

Twenty stations out of 72 were selected. A further second step selection was made by checking if the correlations of data at the respective stations were significant enough to interpolate the missing values in the records. As a result, 14 gauging stations of the 20 mentioned above were chosen. The locations of these finally selected stations are as shown in Fig. 5-4.

The significant levels of correction coefficients obtained are given in Appendix Fig. 5-29.

(2) Flood rainfall and interpolation of missing rainfall data

The continuity of rainfall at the time of large floods in the past was seen to be approximately within 5 days. Floods of relatively longer duration and with corresponding rainfall records were sought among the data at Guillemard Bridge where the rating curves were prepared. The flood selected for examination is the one which occurred in December, 1983. Rainfall at the time of this flood continued for 5 days. Because of this, peak daily rainfall records and 5-day rainfall records over the flood season months, i.e. October to January, in the years from 1947 to 1985 were examined.

As the occurrence of peak daily rainfall is not always on the same day at each gauging station, and there are on average two separate 5-day rainfall periods per month, a total of about 5,000 cases of daily rainfall amounts were examined. It was found that the correlation coefficient of peak daily rainfall among the respective gauging stations is poor, i.e. 0.709 (See Appendix Table 5-11).

However, 5-day rainfall volume involving peak daily rainfall (the period of 5-day was shifted to gather the maximum amount of rainfall in total) show somewhat better correlations (a coefficient of 0.908 as shown in Appendix Table 5-12). On the other hand, the peak daily rainfall and 5-day rainfall volume of any one station itself has a fairly good correlation with a coefficient of 0.921 (See Appendix Table 5-14). Therefore, the missing portion of the data is supplemented in the following procedure:

(a) To estimate the missing 5-day rainfall volume from the correlation at each station.

(b) To estimate the missing peak daily rainfall by substituting the 5-day rainfall obtained in (a) above into the correlation of 5-day rainfall and peak daily rainfall within the station.

In this procedure, the correlations of the 5-day rainfall volume and peak daily rainfall were analyzed by a mean curve.

When interpolating the missing values, the order of priority among the respective gauging stations is set (Appendix Table 5-16) depending on the correlation coefficients.

Thus, annual maximum 5-day rainfall and peak daily rainfall every year from 1947 to 1985 are obtained as per Appendix Table 5-17 to 19.

The following table shows the maximum 5-day rainfall volume and the highest peak daily rainfall at each gauging station in the month of December, 1983 when the above-mentioned flood occurred.

Maximum 5-day total rainfall and peak daily rainfall recorded at each gauging station at the time of December 1983 flood were compared with the records at the same station during the period from 1947 to 1985, and ranked in order from the largest to smallest value. The results of this comparison are tabulated as follows:

Maximum 5-day Total Rainfall & Peak Daily Rainfall at each Station in December, 1983

S. No.	Max. 5-day		Peak Daily		S. No.	Max. 5-day		Peak Daily	
	Rainfall (mm/5 days)	Rank	Rainfall (mm/day)	Rank		Rainfall (mm/5 days)	Rank	Rainfall (mm/day)	Rank
2	315.0	2	89.0	4	12	237.4	5	108.1	6
4	188.2	4	104.2	3	13	370.7	7	173.2	8
5	241.9	8	122.2	8	14	561.6	3	254.4	4
7	324.7	4	194.4	4	15	254.7	17	128.5	13
8	173.1	3	121.0	4	16	382.3	9	220.9	9
9	280.0	8	125.5	12	17	286.0	14	133.0	2
10	266.0	4	113.1	5	20	740.8	1	292.0	1
11	342.6	4	342.6	4					

The rank shows the order of magnitude among 39 records of the annual maximum values each year from 1947 to 1985.

(3) Thiessen Division

The basin area dominated by rainfall recorded at the respective gauging stations were obtained by Thiessen's method. Prior to such division by Thiessen's method, the watershed of the objective sites had been established on the topographical map, and the coverage area of each rainfall gauging station had also been drawn. (See Fig. 5-5).

The objective sites for which the river flow discharge is to be estimated are the locations of four discharge gauging stations, including three proposed dam sites on the Kelantan River Basin, namely, Lebir, Nenggiri and Dabong. However, after the study described in the following item d), the rainfall gauging station representing the watershed of the proposed Lebir dam was changed to Tualang (No. 20) in Fig. 5-4 from Kg. Aring (No. 5) in Fig. 5-5. The weight distribution to each of those sites by the Thiessen method is tabulated in Appendix Table 5-20.

(4) Probable 5-day rainfall and probable daily rainfall

Probable rainfall at No. 20 Gauging Station (at Tualang) was computed as representative rainfall for the watershed of the Lebir dam, and the obtained values are tabulated in the following table. As to rainfall for other dam sites, 5-day rainfall and daily rainfall in the records of each gauging station were converted into the Thiessen rainfall according to the weight distribution mentioned above, and probability calculations were made using the Gumbel-Chow method. The results are also shown in the following table.

Probable Thiessen 5-Day Rainfall
(by Gumbel - Chow method)

Basin / Probability in years	Lebir Dam	Nenggiri Dam	Nenggiri Dam to Dabong Dam	Lebir Dam, Dabong Dam to Guillemard
	mm	mm	mm	mm
10,000	1,624	498	786	865
1,000	1,257	400	625	695
200	1,000	333	513	576
100	890	304	464	525
50	778	274	416	473
20	630	235	351	404
10	515	205	301	351
5	396	174	248	296

Probable Thiessen Daily Rainfall

Basin / Probability in years	Lebir Dam	Nenggiri Dam	Nenggiri Dam to Dabong Dam	Lebir Dam, Dabong Dam to Guillemard
	mm	mm	mm	mm
10,000	743	277	394	409
1,000	575	222	314	329
200	458	184	258	273
100	402	167	234	250
50	356	150	210	226
20	288	128	178	194
10	236	111	153	169
5	181	93	127	143

(5) To calculate the relation between daily rainfall and hourly rainfall

Runoff calculation requires values of hourly rainfall. However, the rainfall records in the subject basin are not sufficient for use in the analysis. In such cases, the following conversion of daily rainfall to hourly rainfall is generally used in Japan:

$$r_T = R_{24}\ (T/24)^{1/3}$$

where, r_T = T-hour rainfall

R_{24} = Daily rainfall in mm.

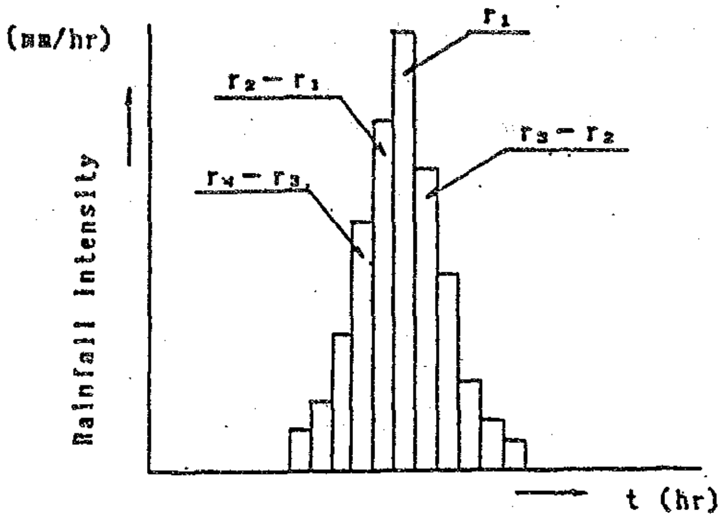
The Value K in the equation of $r_1 = R_{24}\ (T/24)^K$ is obtained using peak hourly rainfall and daily rainfall figures, and the result is presented in Appendix Fig. 5-30. If the value of K becomes smaller, the peak hourly rainfall becomes larger. From the above figure, the value K, in the range of the high daily rainfall, is read and K = 0.5 is obtained.

Applying this value to the equation;

$$r_T = R_{24}\ (T/24)^{1/2}$$

the amount of daily rainfall is converted to hourly rainfall.

If the hourly rainfall pattern is not known, rainfall per hour, T = 1,2,3, ... 24 Hr, is computed by the above equation and then r_1, $r_2 - r_1$, $r_3 - r_2$, $r_4 - r_3$ ... are obtained successively. These can be shown in a hyetograph as depicted below:



5.4.3.2. Preparation of Runoff Model

(1) Calculation Formula

Runoff model was prepared by classifying the runoff process from rainfall into two states; 1) a runoff state flowing to the river and 2) a runoff state along the river course.

At first, the amount of rainfall flowing to the river was computed by the runoff function method and next, the runoff volume along the river course was computed by Puls method which is one of the storage function methods.

The runoff function equation is expressed as:

$$Q = (r/3.6)\ Af\ \alpha^2\ t \cdot EXP\ (-\alpha t) \ \ldots\ldots\ (1)$$

Where, A : Catchment area (km^2)

f : Runoff coefficient

α : 1/ta,

r : Hourly rainfall

ta : Lag time from the beginning of the rainfall to the appearance of peak runoff

t : Time elapsed (hr)

Conversely, the characteristic of the Puls method is that storage in the river channel (S) is only regarded as the runoff function (O). The storage equation is expressed as:

$$S = KO \quad \ldots\ldots (2)$$

and the continuous flow equation is:

$$t\ (I_1 + I_2)/2 = t\ (O_1 + O_2)/2 + S_2 - S_1 \quad \ldots\ldots (3)$$

Where, I_1 : Inflow at the beginning of time t (m^3/sec)
I_2 : Inflow at the end of time t (m^3/sec)
O_1 : Outflow at the beginning of time t (m^3/sec)
O_2 : Outflow at the end of time t (m^3/sec)
$S_2 - S_1$: Variation in storage in the section
K : Puls' constant

The constant K and storage S are approximated respectively in the following equations:

$$S = BL_n\ (h_{n-1} + h_n)/2 \quad \ldots\ldots\ldots (4)$$

Where, B = River width
L_n = Distance between the sections n-1 and n
h = Depth

From the equations (2) and (4)

$$K = L_n B_h / O$$
$$= L_n / V \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Where, V = Velocity of flow

Assuming uniform river flow at V in the equation (5),

$$V = i^{3/10}\ Q^{2/5}/B^{2/5}\ n^{3/5} \ \ldots\ldots\ldots\ldots \ (6)$$

Where, i = Gradient of the river

Q = River flow (m^3/sec.)

n = Manning's roughness coefficient

Substituting the above into the equation (5),

$$k = i^{3/10}\ B^{2/5}\ n^{3/5} \ \ldots\ldots\ldots\ldots\ldots\ldots \ (7)$$

As explained above, the Puls method makes it possible to simulate the flow conditions over a certain duration of time by setting the cross-section of the river and the coefficient.

(2) Selection of Flood Records compared for the preparation of a model.

For the preparation of a runoff model of the compared floods, among other measurement data, the following is required:

- The measurement must be recorded at the objective site.
- The peak discharge should be large enough.
- Hydrograph must be prepared for the record.
- The data at the rainfall gauging station located upstream must be available for examination.

As mentioned in Section 5.2.2, hydrographs for 10 floods at Guillemard Bridge were measured and kept as records.

At the Tualang gauging station located just downstream of the Lebir dam site, four floods were measured at the same time as the records at Guillemard Bridge.

In addition to which the flood records at the Dabong Gauging station were checked in this respect. After all the data was reviewed, the flood in December 1983 was selected.

A peak discharge of 12,010 m^3/sec. at the Guillemard Bridge was recorded during the flood of December 1983. This is the second largest (next to the flood in January 1967) of all the floods with hydrographs. The flood continued for 8 days and ranks as having the longest recorded duration of a major flood.

In terms of the peak flood discharges recorded during the period from 1940 to 1986, it is ranked as the fourth largest.

Corresponding rainfall records are also available for the flood on December 1983.

Since Guillemard Bridge is the significant base point for planning downstream flood control, it is necessary to prepare a model relevant to small flood discharge cases. This is also one of the reasons why the December 1984 flood was selected.

The peak discharge of this flood at Guillemard Bridge was 7,740 m^3/sec. and at the Tualang site, it was 3,430 m^3/sec.

From the above study, it was decided to apply two floods, recorded respectively in December, 1983 and December, 1984 for the preparation of a runoff model.

For further reference, "Note on peak discharges in flood hydrographs as a basis of a flood control plan" described in the flood Control Plan of the "The Manual for River

Work in Japan" published by the Ministry of Construction, Japan is given hereinafter. According to this Note, the significance can be verified by comparing the flood in December, 1983.

The above mentioned Note explains that in determining a flood hydrograph as the basis of a development plan, it is necessary to evaluate at which point the peak discharge will rank within the group of previous floods arranged in order of magnitude.

This is generally referred to as the rate of coverage. It is desirable that the rates of coverage, in rivers of nearly the same conditions, are more or less in a balanced range on a nation-wide basis.

According to this method, the rate of coverage is normally greater than 50%. For most major rivers, the rates often range from 60% to 80%.

The rates of coverage were obtained based on peak discharge records for the period from 1940 to 1986 as follows:

Rate of Coverage	Discharge at Guillemard
80% (Ranked 9th from the top)	8,800 m^3/sec.
60% (Ranked 18th from the top)	6,050 m^3/sec.
50% (ranked 22nd from the top)	4,630 m^3/sec.

The flood discharge in December, 1983 is ranked as the 4th largest among all floods recorded during the period from 1940 to 1986 and is 91% in terms of rate of coverage. A total of 45 references from peak discharges in the same period were used.

(3) Models of the basis and river channel

The following four stations were determined as the objective points where the amount of runoff is to be obtained:

- Tualang Gauging Station on the Lebir River
- Bertam Gauging Station on the Nenggiri River
- Dabong Gauging Station on the Galas River
- Guillemard Gauging Station on the Kelantan River

Locations of these stations and rivers are shown on Fig. 5-1 (Appendix Fig. 5-31). The drainage basin is subsequently divided into the four areas mentioned below:

- Upstream basin of the Tualang Station (248 km^2)
- Upstream basin of the Bertam Station (3,950 km^2)
- The basin between the Bertam Station and the Dabong Station (3,530 km^2)
- The basin upstream of the Guillemard Station but below the Tualang and Dabong Stations (2,140 km^2)

In the simulation model, therefore, the catchment upstream of the Tualang station is analysed directly by the runoff function method. The same method is applied for the catchment area in the upstream basin of the Bertem Station. Conversely, for the basin from the Tualang and Bertam stations to the Guillemard station downstream, the runoff is calculated by the synthetic method of flow conditions in the river course, and the catchment area in the basin. In other words, the stream flow condition is calculated by the Puls method. For this, it is necessary to know the gradient and river width of each section of almost equal intervals along the river course. Furthermore, the joining point of the runoff from the basin between Bertam and Dabong (by runoff function method) is made at the Dabong Station.

The runoff from the basins between Tualang and Guillemard, and between Dabong and Guillemard are considered as the inflow at the confluence of the Lebir and Galas rivers.

17 river cross-sections are used for simulation in the 36 km length from Tualang to the confluence mentioned above, and 25 sections are in the 50 km from Bertam to Dabong. 18 sections are in the 36 km from Dabong to the confluence with the Lebir River, and 8 sections are in the 34 km from this confluence to Guillemard Bridge. These cross-sections, and the variations in the river width, are as shown in Appendix Fig. 5-32. The above-mentioned is the simulation model being used for a comparison with the recorded floods.

For the purpose of evaluating the flood control effect of the Project however, the proposed Lebir dam site instead of Tualang, is regarded as the upstream end of the Lebir River, and the proposed Nenggiri dam site is on the Nenggiri River instead of at Bertam. The Lebir dam site is located 2.6 km upstream from the Tualang station and the number of river cross-sections is consequently increased by one, while the Nenggiri dam site is 20 km upstream of the Bertam Station. The number of cross-sections is increased by 10.

The time period of flood concentration at the site after rainfall is affected by topographical features of the basin. The figure can be obtained by using Rziha's equation which is currently used in the Bayern region of West Germany. The equation is:

$$W = 20\ (h/L)^{0.6}$$

Where, W = Flow velocity of flood

h = Elevation difference between the upper end of the basin and the reservoir

L = Distance from the upper end of the basin to the reservoir

From the velocity obtained above, the time period of flood appearing at the site (Ta) is computed in the following equation.

$$Ta = L/W$$

The results of the computation for the above mentioned four divisions of the basin are enumerated below:

	h (m)	L (km)	W (m/s)	Ta (hr)
Tualang basin	1,350	115	1.4	23
Bertam basin	1,730	130	1.5	24
Bertam to Dabong	720	94	1.1	24
Dabong/Tualang to Guillemard	560	61	1.2	14

After a study of the runoff coefficient, they are determined as follows:

Tualang basin	0.8
Bertam basin	1.0
Bertam to Dabong	1.0
Dabong/Tualang to Guillemard	0.7

Furthermore, Manning's roughness coefficient of the river was determined at 0.04.

These values mentioned above are in a generally acceptable range.

(4) Comparison of simulation results and recorded flood discharges

The simulation results based on 5-day rainfall amounts with respect to the two floods which occurred in December 1983 and December 1984, were compared with actual recorded values on the same floods. This comparison is shown in Figs. 5-6 and 5-7. The comparative figures of their peak discharges are tabulated below:

Station	Dec. 1983 Flood		Dec. 1984 Flood	
	Recorded value	Computed value	Recorded value	Computed value
	m^3/s	m^3/s	m^3/s	m^3/s
Tualang	3,900	4,057 (54)	3,430	2,665 (41)
Bertam		2,553 (173)		2,292 (163)
Dabong	5,950	5,407 (328)	4,220	4,944 (309)
Guillemard	12,010	11,287 (476)	7,740	9,397 (438)

Note: Values in parenthesis are base flow (mean monthly flow in the preceding month)

As can be seen in the above table, peak discharges in the December 1983 flood records and those computed show a fairly good correlation at the respective sites of Tualang, Dabong and Guillemard. However, some difference is noted at the time of the occurrence of peak discharges. In the calculation, the peak discharge appears to be earlier by 6 hours at Tualang, 3.6 hours at Dabong and 13.2 hours at Guillemard.

In the December 1984 flood, the peak discharges both at Tualang and at Dabong are in a good agreement between the recorded values and calculated values. However, the peak discharges at Guillemard, as computed, were 21% larger than the recorded values. With respect to the time of the peak discharge, the computed time of occurrence is 1.2 hours and 7.8 hours earlier at Tualang and Guillemard respectively.

However, at Dabong, it was 8.1 hours later than the actual time in the record. It can therefore be said that the recorded values at Tualang and Dabong and those computed on the hydrograph, agree with each other in fairly good order.

At the Guillemard site, the peak discharge occurs earlier in the computation.

Values of 5-day rainfall used in the calculations that were derived by the Thiessen method, are given in Tables 5-3 and 5-4. The rainfall data relating to the flood in December 1983 is available at No.17 gauging station (hourly rainfall records) and No.20 gauging station (daily rainfall records). Since no data is available at other stations, it was obtained by interpolation. The hourly rainfall pattern at No.17 gauging station was adopted to represent the rainfall pattern over the whole basin.

Thiessen weighted rainfall in the Tualang upstream basin relating to the flood in December 1983, was derived from the daily rainfall records at the Tualang gauging station (No.20 gauging station). Rainfall records at No.5 gauging station, which is located in the middle of the Tualang upstream basin were not used. This is because the Thiessen weighted rainfalls obtained, using records at No.5 gauging station, were quite different from those at No.20 gauging station, as shown below:

Gauging Station	1st day	2nd day	3rd day	4th day	5th day	Total
No.5	123 mm	52	62	15	14	266
No.20	342	145	172	42	39	740

Amounts of rainfall at No.20 gauging station are remarkably large compared with those at other stations.

The following discharges at Tualang are the values obtained by simulation separately conducted using the above respective rainfalls.

1,824 m^3/s	rainfall at No.5
4,057 m^3/s	rainfall at No.20

Compared with the measured flood discharge of 3,900 m^3/s at the Tualang site, the flood discharge at No.20 gauging station is considered to be more representative of this flood.

Daily rainfall patterns related to the flood in December 1984 are available at No.2, 10, 13, 14, 17 and 20 stations. Others were interpolated as required.

For the rainfall pattern in the Tualang upstream basin, the hourly rainfall pattern of No.17 station is used, while the rainfall patterns for other blocks were prepared, by assuming a type of pattern of intensity rising to a peak in the middle, from the equation $r_t = R24\ (T/24)^{1/2}$. This is because there are no available hourly rainfall patterns.

5.4.3.3. Probable Flood Discharge Based on Runoff Model

In this section, probable rainfall obtained in Item (4) of Section 5.4.3.1. is applied over the catchment area upstream of Guillemard Bridge, and the runoff from this rainfall is calculated by use of a simulation model developed in the previous section. The hyetograph (5-day rainfall pattern) is prepared of the type having a peak on the 3rd day. Precipitations in other days were distributed, based on the conditions of the 5-day rainfall in December 1983, except for the peak rainfall. In other words, it could be said that the probable rainfall was estimated from the data of the flood which occurred in December 1983.

Under these conditions, peak flood discharge corresponding to each probable rainfall was simulated, the results of which are given in the following Table.

Probable Flood Discharges

(m^3/s)

Probability in years	Tualang	Bertam	Dabong	Guillemard
10,000	10,604	6,876	16,081	31,413
1,000	8,282	5,600	12,985	25,078
200	6,663	4,730	10,835	20,679
100	5,951	4,339	9,902	18,752
50	5,260	3,944	8,965	16,851
20	4,323	3,439	7,715	14,315
10	3,595			
5	2,846			

5.4.4. Comparison of Probable Flood Based on Peak Discharge Records with Probable Flood Based on Runoff Analysis

The following table gives comparative figures of probable floods of various return periods, calculated on the basis of probability analysis using two different types of data. The results are based on the recorded peak flood discharges (Section 5.4.2.) at the Guillemard Bridge site, and at the Tualang Site, and the runoff analysis of probable rainfall (Section 5.4.3.3.).

As seen in the table, probable floods at Guillemard in the recurrence of 100 years to 10,000 years estimated by both methods indicate a fairly good agreement, while at Tualang, peak flood discharges estimated based on the Guillemard Bridge values become larger than those estimated by runoff analysis. This is due to an excessively large envelope curve showing the relationship between peak discharges at Guillemard and at Tualang (Fig. 5-3). It is, therefore, considered safe enough even if the values based on the runoff analysis are used. Flow conditions of floods of probabilities of 10,000 years, 1,000 years and 50 years at each site are shown in Appendix Fig. 5-33 to 5-35.

P	Floods based on peak discharges measured		Floods based on rainfall runoff analysis		
	Guillemard	Tualang	Guillemard	Tualang	Tualang/Guillemard
	m^3/s	m^3/s	m^3/s	m^3/s	
10,000	31,320	15,660	31,413	10,604	0.338
1,000	24,503	12,252	25,078	8,282	0.330
200	19,730	9,865	20,679	6,663	0.322
100	17,671	8,834	18,752	5,951	0.317
50	15,604	7,801	16,851	5,260	0.312
20	12,845	6,423	14,315	4,323	0.302
10	10,714	5,357	12,340	3,595	0.291
5	8,493	4,247	10,294	2,846	0.276