REPUBLIC OF INDONESIA

MINISTRY OF PUBLIC WORKS AND ELECTRIC POWER

FEASIBILITY REPORT
ON
ULAR RIVER FLOOD CONTROL
AND IMPROVEMENT OF IRRIGATION PROJECT

VOLUME III
SUPPORTING REPORT

JULY 1978

JAPAN INTERNATIONAL COOPERATION AGENCY

The Feasibility Report on Ular River Flood Control and Improvement of Irrigation Project is composed of three Volumes.

Volume I: Main Report

Volume II: Study Report

Volume III: Supporting Report

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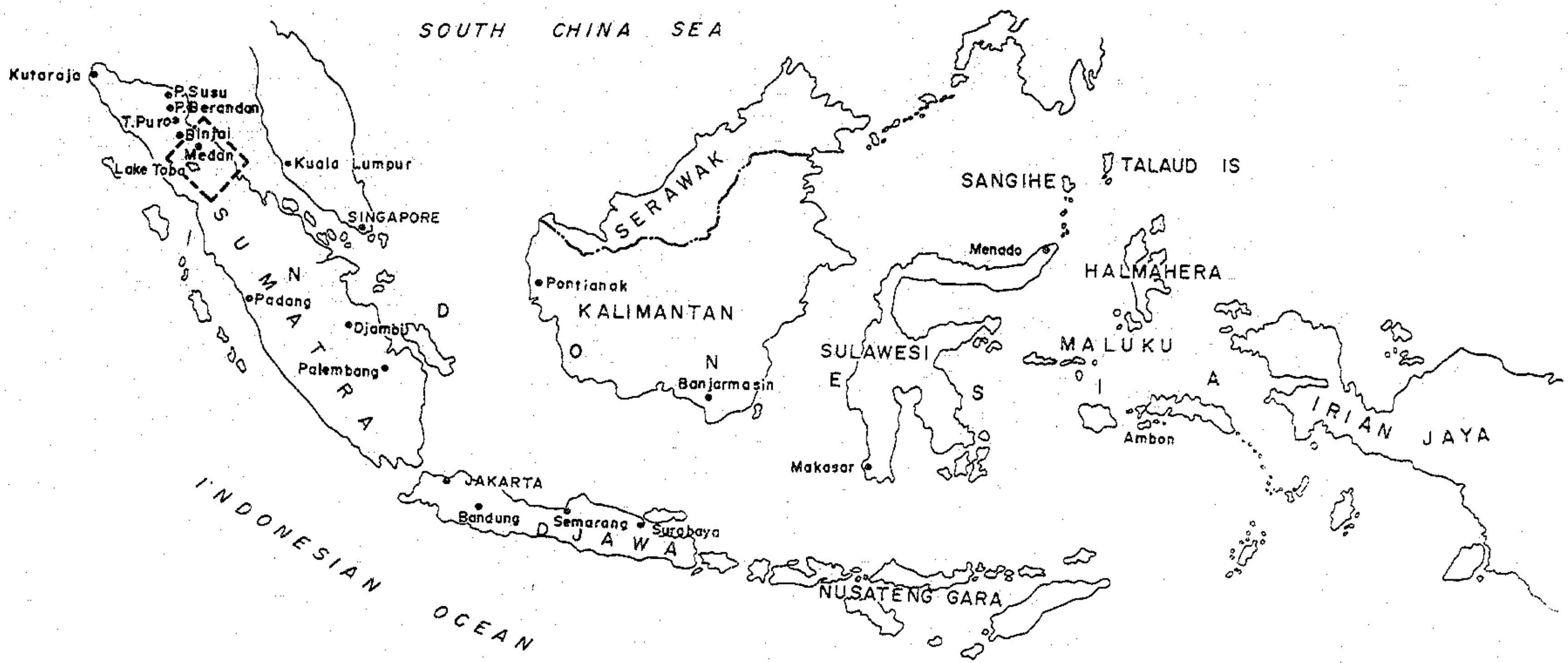
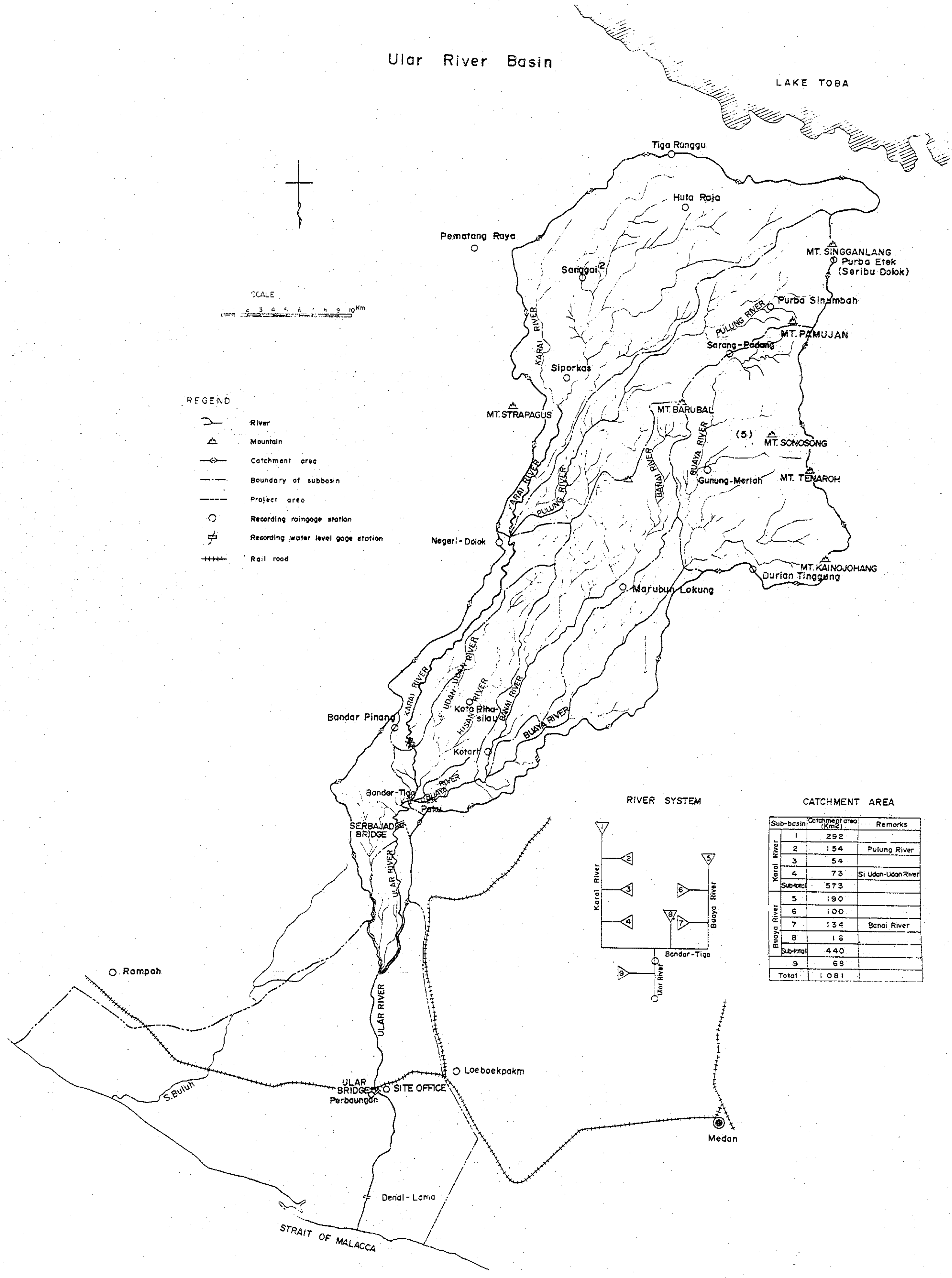
Location Map

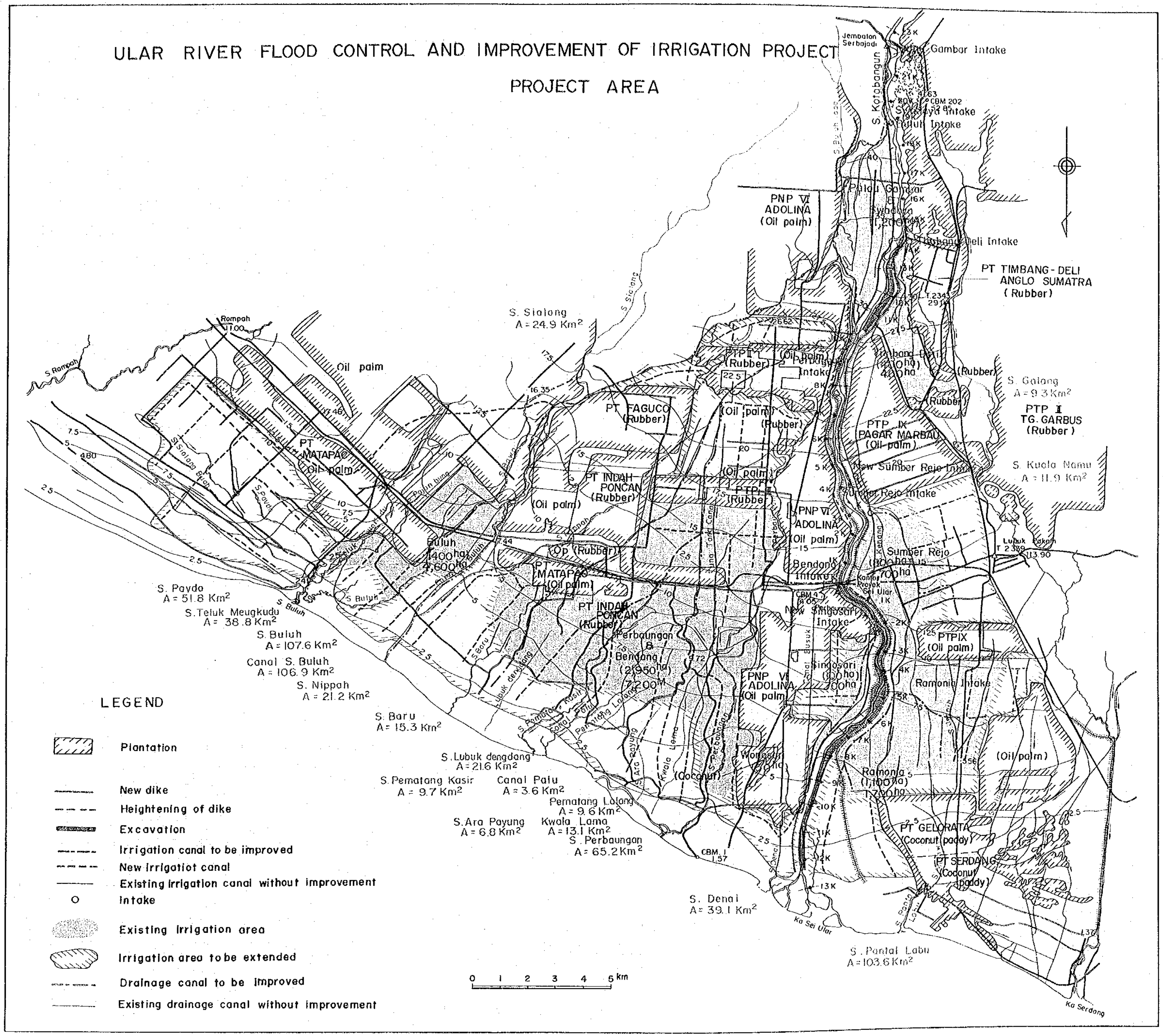


Ular River Basin



CATCHMENT AREA

	Sub-basin	Catchment area (Km2)	Remarks
Karai River	1	292	
	2	154	Pulung River
	3	54	
	4	73	Si Udan-Udan River
	Sub-total	573	
Buaya River	5	190	
	6	100	
	7	134	Banai River
	8	16	
	Sub-total	440	
	9	68	
	Total	1081	



ULAR RIVER FLOOD CONTROL AND IMPROVEMENT OF IRRIGATION PROJECT
PROJECT AREA
LEGEND
Plantation
New dike
Heightening of dike
Excavation
Irrigation canal to be improved
New irrigatiot canal
Existing irrigation canal without improvement
Intake
Existing irrigation area
Irrigation area to be extended
Drainage canal to be improved
Existing drainage canal without improvement
0 1 2 3 4 5 km
Gambar Intake
Deli Intake
PT TIMBANG-DELI
ANGLO SUMATRA
(Rubber)
PNP VI
ADOLINA
(Oil palm)
S. Sialang
A = 24.9 Km²
Oil palm
S. Galang
A = 9.3 Km²
PTP II
TG. GARBUS
(Rubber)
PTP IX
PAGAR MARBAU
(Oil palm)
S. Kuala Namu
A = 11.9 Km²
PT FAGUCO
(Rubber)
PT INDAH
PONCAN
(Rubber)
PT
MATAPAO
(Oil palm)
Sumber Rejo
Bendang Intake
New Singosari Intake
PTPIX
(Oil palm)
Ramonia Intake
Ramonia
PT GELORATA
(Coconut paddy)
PT SERDANG
(Coconut paddy)
(Coconut)
S. Payda
A = 51.8 Km²
S. Teluk Meugkudu
A = 38.8 Km²
S. Buluh
A = 107.6 Km²
Canal S. Buluh
A = 106.9 Km²
S. Nippah
A = 21.2 Km²
S. Baru
A = 15.3 Km²
S. Lubuk dengdang
A = 21.6 Km²
S. Pematang Kasir
A = 9.7 Km²
Canal Palu
A = 3.6 Km²
Pematang Lalang
A = 9.6 Km²
S. Ara Payung
A = 6.8 Km²
Kwala Lama
A = 13.1 Km²
S. Perbaungan
A = 65.2 Km²
S. Denai
A = 39.1 Km²
S. Pantai Labu
A = 103.6 Km²
Ka Sei Ular
Ka Serdang
Lubuk Pakam

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I. TOPOGRAPHIC SURVEY

1. Outline of the Works.

Surveyings were conducted by nine Indonesian surveying parties with the assistance of three Japanese Surveying Engineers in the period of the middle of August to the beginning of November, 1977. The items of surveying works were as follows.

a. Review of the existing bench marks in the project area.

b. Surveying works for preparing the plan of river channel improvement.

c. Surveying works for preparing the plan of irrigation and drainage canal lines.

d. Topographic surveyings for intake structures.

2. Review of the Existing Bench Marks in the Project Area.

(1) Leveling nets were established for making bench-marks leveling.

(2) Intemediate points were set at intervals ranging from 1 km to 1.3 km on the leveling routes. The intermediate points were used as the known points in other surveyings such as profile leveling, cross leveling and plane-table surveying.

(3) Closing errors of the leveling nets were adjusted by means of the least squares method.

3. Surveying Works for Preparing the Plan of River Channel Improvement.

(1) Proflie levelings and cross levelings were made at intervals of 250 m over a stretch from the river mouth to Serbajadi Bridge except a stretch of about 10 km included therein.

(2) Cross levelings were made over a river stretch between the distance-marks No.10 km and No.13 km including a paddy field area located between these two distance-marks in the lower part of Pulau Gambar. The average width of the cross-levelings was about 450 m. The scales of drawings are as follows.

V = 1/100, H = 1/1,000 for cross levelings.
V = 1/100, H = 1/20,000 for profile levelings.

(3) Profile and cross levelings on the Pulau Gambar canal were carried out at intervals of 200 m. The scales of drawings are as follows.

V = 1/200, H = 1/200 for cross levelings.
V = 1/100, H = 1/20,000 for profile levelings.

4. Surveying Works for Preparing the Plan of Irrigation and Drainage Lines.

Profile and cross levelings were carried out at intervals of 400 m. The scales of drawings are as follows.

V = 1/200, H = 1/200 for cross levelings.
V = 1/100, H = 1/20,000 for profile levelings.

5. Topographic Surveyings for Planning Intakes and Sluices.

Topographic surveyings for planning intakes and drainage sluices were carried out by means of plane-table-survey method on a scale of 1/500.

6. Results of Surveyings.

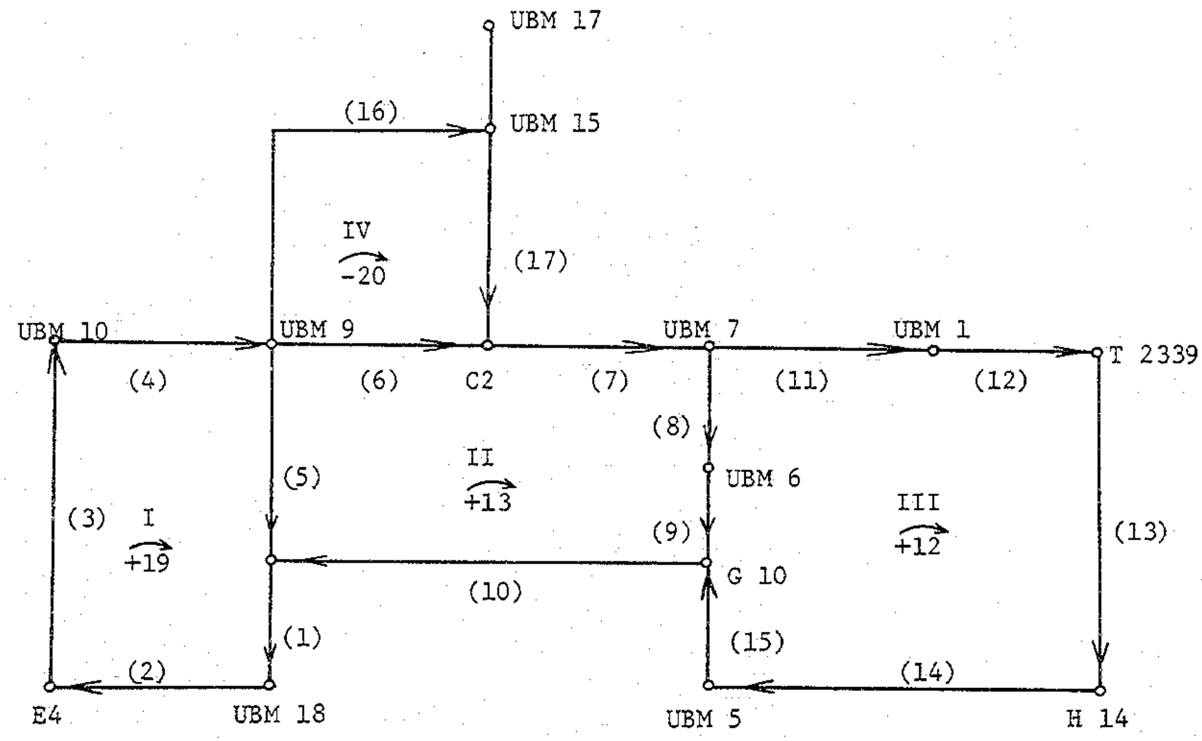
The results of surveyings are as follows.

Bench mark leveling	:	calculation sheets of leveling net and table of elevation of bench mark.	
Ular river	:	cross leveling profile leveling	103 sections
Pulau Gambar canal	:	cross leveling	11 sections (PC: 3 sheets)
		profile leveling	1 sheet
		table of elevation of survey stake	
Irrigation and drainage	:	cross leveling	612 sectios

MM	7 sheets	28 sections
PS	5 "	22 "
SB	15 "	57 "
PG	20 "	79 "
SN	33 "	128 "
SJ	13 "	47 "
KM	21 "	89 "
ST	13 "	48 "

	SL 1 "		3 "	
	BD 4 "		15 "	
	SS 6 "		24 "	
	BB 6 "		23 "	
	SR 6 "		24 "	
	RI 7 "		25 "	
	profile leveling		24 sheets	
	MM		1	
	PS		1	
	SB		2	
	PG		3	
	SN		5	
	SJ		2	
	KM		2	
	ST		2	
	SL		1	
	BD		1	
	SS		1	
	BB		1	
	SR		1	
	RI		1	
	table of elevation of survey stake			
Plane table survey :	intake		9 places	
	drainage sluice		1 place	

Adjustment of Leveling Net



(1) Observation Equations.

$$\left.\begin{array}{ll} L_1 = \ell_1 + \delta_1 & p_1 \\ L_2 = \ell_2 + \delta_2 & p_2 \\ \text{-----------} & \text{----} \\ L_{17} = \ell_{17} + \delta_{17} & p_{17} \end{array}\right\}$$

where, Li : Probable value.
ℓi : Observed value.
δi : Value to be corrected.
pi : Weight of leveling route.

(2) Condition Equations.

$$L_1 + L_2 + L_3 + L_4 + L_5 = 0 = \delta_1 + \delta_2 + \delta_3 + \delta_4 + \delta_5 + (\ell_1 + \ell_2 + \ell_3 + \ell_4 + \ell_5)$$
$$= \delta_1 + \delta_2 + \delta_3 + \delta_4 + \delta_5 + \omega_1$$

$$-L_5 + L_6 + L_7 + L_8 + L_9 + L_{10}$$
$$= 0 = -\delta_5 + \delta_6 + \delta_7 + \delta_8 + \delta_9 + \delta_{10} + \omega_2$$
$$-L_8 - L_9 + L_{11} + L_{12} + L_{13} + L_{14} + L_{15}$$
$$= -\delta_8 - \delta_9 + \delta_{11} + \delta_{12} + \delta_{13} + \delta_{14} + \delta_{15} + \omega_3$$
$$-L_6 + L_{16} + L_{17} = 0 = -\delta_6 + \delta_{16} + \delta_{17} + \omega_4$$

where, ωi : Clsoing error of each leveling mesh.

(3) Application of the Method of Lease Squares.

$$F \equiv \begin{cases} -2K_1 \ (\delta_1 + \delta_2 + \delta_3 + \delta_4 + \delta_5 + \omega_1) = 0 \\ -2K_2 \ (-\delta_5 + \delta_6 + \delta_7 + \delta_8 + \delta_9 + \delta_{10} + \omega_2) = 0 \\ -2K_3 \ (-\delta_8 - \delta_9 + \delta_{10} + \delta_{11} + \delta_{12} + \delta_{13} + \delta_{14} + \delta_{15}) = 0 \\ -2K_4 \ (-\delta_6 + \delta_{16} + \delta_{17} + \omega_4 \ ) = 0 \end{cases}$$

$$p_1\delta_1^2 \pm p_2\delta_2^2 + \dots\dots\dots + p_{16}\delta_{16}^2 + p_{17}\delta_{17}^2 = \text{minimum}$$

$$\frac{\partial F}{\partial \delta_1} = 0 , \quad \frac{\partial F}{\partial \delta_2} = 0, \ \dots\dots\dots \ \frac{\partial F}{\partial \delta_{16}} = 0, \quad \frac{\partial F}{\partial \delta_{17}} = 0$$

$\therefore$

$$\delta_1 = \frac{1}{p_1} = S_1K_1$$
$$\delta_2 = \frac{1}{p_2} = S_2K_1$$
$$\delta_3 = \frac{1}{p_3} = S_3K_1'$$
$$\delta_4 = \frac{11}{p_4} = S_4K_1$$

$$\delta_5 = \frac{1}{p_5}(K_1 - K_2) = S_5(K_1 - K_2)$$

$$\delta_6 = \frac{1}{p_6}(K_2 - K_4) = S_6(K_2 - K_4)$$

$$\delta_7 = \frac{1}{p_7}K_3 = S_7K_2$$

$$\delta_8 = \frac{1}{p_8}(K_2 - K_3) = S_8(K_2 - K_3)$$

$$\delta_9 = \frac{1}{p_9}(K_2 - K_3) = S_9(K_2 - K_3)$$

$$\delta_{10} = \frac{1}{p_{10}}K_2 = S_{10}K_2$$

$$\delta_{11} = \frac{1}{p_{11}}K_3 = S_{11}K_3$$

$$\delta_{12} = \frac{1}{p_{12}}K_3 = S_{12}K_3$$

$$\delta_{13} = \frac{1}{p_{13}}K_3 = S_{13}K_3$$

$$\delta_{14} = \frac{1}{p_{14}}K_3 = S_{14}K_3$$

$$\delta_{15} = \frac{1}{p_{15}}K_3 = S_{15}K_3$$

$$\delta_{16} = \frac{1}{p_{16}}K_4 = S_{16}K_4$$

$$\delta_{17} = \frac{1}{p_{17}}K_4 = S_{17}K_4$$

where, Si : Length of leveling route between conjunction points.

(4) Normal Equations.

Each Ki, was obtained by applying the method of the least squares, is substituted into the condition equations.

$$(S_1 + S_2 + S_3 + S_4 + S_5)K_1 - S_5K_2 + W_1 = 0$$

$$-S_5K_1 + (S_5 + S_6 + S_7 + S_8 + S_9 + S_{10})K_2 - (S_8 + S_9)K_3 - S_6K_4 + W_2 = 0$$

$$-(S_8 + S_9)K_2 + (S_8 + S_9 + S_{11} + S_{12} + S_{13} + S_{14} + S_{15})K_3 + W_3 = 0$$

$$-S_6K_2 + (S_6 + S_{16} + S_{17})K_4 + W_4 = 0$$

(5) Solution of Normal Equations.

Route No.	Length(s)	Observed Difference
1	1.04	-0.918
2	6.54	-0.702
3	4.65	+4.236
4	4.83	+1.466
5	3.76	-4.063
6	5.10	+5.855
7	2.76	-0.024
8	3.80	-3.403
9	7.95	-7.364
10	3.70	+0.886
11	7.22	+2.722
12	5.70	-1.314
13	13.52	-12.761
14	12.15	-0.015
15	2.28	+0.613
16	11.45	+11.922
17	6.28	-6.087

$$
\begin{array}{rrrrrl}
20.82K_1 & -\ 3.76K_2 & & & +\ 19 & = 0 \\
-3.76K_1 & +\ 27.07K_2 & -\ 11.75K_3 & -\ 5.10K_4 & +\ 13 & = 0 \\
 & -\ 11.75K_2 & +\ 52.62K_3 & & +\ 12 & = 0 \\
 & -\ 5.10K_2 & & +\ 22.83K_4 & -\ 20 & = 0
\end{array}
$$

	K_1	K_2	K_3	K_4	ω	α
	20.82	-3.76	0	0	19.00	36.06
		0.181	0	0	-0.913	-1.732
	-0.913	27.07	-11.75	-5.10	13	19.46
	0	-0.681	0	0	3.439	6.527
	0	26.389	-11.75	-5.10	16.439	25.987
	-0.117		0.445	0.193	-0.623	-0.985
K_1 =	-1.030	-0.623	52.62	0	12	52.87
		0.141	0	0	0	0
		-0.166	-5.229	-2.270	7.315	11.564
	K_2 =	-0.648	47.391	-2.270	19.315	64.434
				0.048	-0.408	-1.360
			-0.408	22.83	-20	-2.27
			0.035	0	0	0
		K_3 =	-0.373	-0.984	3.173	5.015
				-0.109	0.927	3.093
				21.737	-15.900	5.838
			K_4 =	0.732	0.732	-0.269

$\delta_1 = -1.07$	$\delta_7 = -1.79$	$\delta_{13} = -5.04$
$\delta_2 = -6.74$	$\delta_8 = -1.04$	$\delta_{14} = -4.53$
$\delta_3 = -4.79$	$\delta_9 = -2.19$	$\delta_{15} = -0.85$
$\delta_4 = -4.97$	$\delta_{10} = -2.40$	$\delta_{16} = 8.38$
$\delta_5 = -1.44$	$\delta_{11} = -2.69$	$\delta_{17} = 4.60$
$\delta_6 = -7.04$	$\delta_{12} = -2.13$	

Calculation of Height

Station No.	Observed Difference	Observed Height	Correction	Height	Remarks
T2339		13.900		13.900	
I 12	-0.384	13.516	0	13.516	
I 11	-0.646	12.870	-1	12.869	
I 10	+0.414	13.284	-1	13.283	
I 9	-2.652	10.632	-2	10.630	
I 8	-0.851	9.781	-2	9.799	
I 7	-0.902	8.879	-2	8.877	
I 6	-2.172	6.707	-3	6.704	
I 5	-1.152	5.555	-3	5.552	
I 4	-1.085	4.470	-3	4.467	
I 3	-1.623	2.847	-4	2.843	
I 2	-0.608	2.239	-4	2.235	
I 1	-0.398	1.841	-5	1.836	
H 14	-0.702	1.139	-5	1.134	
H 14		1.134		1.134	
H 13	+0.112	1.246	0	1.246	
H 12	+0.410	1.656	-1	1.655	
H 11	+0.034	1.690	-1	1.689	
H 10	+0.299	1.989	-1	1.988	
H 9	-0.028	1.961	-1	1.960	
H 8	+0.942	2.903	-2	2.901	
H 7	+0.028	2.931	-2	2.929	
H 6	-1.346	1.585	-2	1.583	
H 5	-0.063	1.522	-3	1.519	
H 4	-0.473	1.049	-3	1.046	
H 3	+1.191	2.240	-3	2.237	
H 2	+0.514	2.754	-3	2.751	
H 1	-1.354	1.400	-4	1.396	
UBM 5	-0.281	1.119	-4	1.115	
UBM 5		1.115		1.115	
G 12	+0.229	1.344	0	1.344	
G 11	-0.740	0.604	-1	0.603	
G 10	+1.124	1.728	-1	1.727	

Station No.	Observed Difference	Observed Height	Correction	Height	Remarks
G 10		1.727		1.727	
L 7	+0.488	2.215	0	2.215	
L 6	+1.232	3.447	0	3.447	
L 5	+0.914	4.361	+1	4.362	
L 4	+0.106	4.467	+1	4.468	
L 3	+1.366	5.833	+1	5.834	
L 2	+0.126	5.959	+2	5.961	
L 1	+2.380	8.339	+2	8.341	
UBM 6	+0.752	9.091	+2	9.093	
UBM 6		9.091		9.093	
K 3	+0.954	10.047	0	10.047	
K 2	+1.288	11.335	0	11.335	
K 1	+0.940	12.275	+1	12.276	
UBM 7	+0.221	12.496	+1	12.497	
UBM 7		12.497		12.497	
B 7	+0.146	12.643	0	12.643	
B 6	-0.562	12.081	-1	12.080	
B 5	+0.566	12.647	-1	12.646	
B 4	+1.506	14.153	-2	14.151	
B 3	-0.100	14.053	-2	14.051	
B 2	+1.398	15.451	-2	15.449	
B 1	+2.266	17.717	-3	17.714	
UBM 1	-2.498	15.219	-3	15.216	
UBM 1		15.216		15.216	
A 4	-0.048	15.168	0	15.168	
A 3	+0.018	15.186	-1	15.185	
A 1	+0.432	15.618	-2	15.616	
T2339	-1.116	13.902	-2	13.900	
G 10		1.727		1.727	
G 9	+1.070	2.797	0	2.727	
G 8	+0.622	3.419	0	3.419	
G 7	+1.746	5.165	0	5.615	
G 6	+0.934	6.099	-1	6.098	
G 5	-1.682	4.417	-1	4.416	
G 4	+0.703	5.120	-1	5.119	
G 3	+0.164	5.284	-1	5.283	
G 2	-1.764	3.520	-1	3.519	
G 1	-1.438	2.082	-1	2.081	
G 0	+0.370	2.452	-1	2.451	
F 11	+0.292	2.744	-2	2.742	
F 10	+0.637	3.381	-2	3.379	
F 9	-1.492	1.889	-2	1.887	
F 8	+0.724	2.613	-2	2.611	
F 8		2.611		2.611	
UBM 8	-0.918	1.693	-1	1.692	

Station No.	Observed Difference	Observed Height	Correction	Height	Remarks
UBM 8		1.692		1.692	
F 5	-0.514	1.178	-1	1.177	
F 4	+0.316	1.494	-3	1.491	
F 3	-0.166	1.328	-4	1.324	
F 2	-0.588	0.740	-6	0.734	
E 4	+0.250	0.990	-7	0.983	
F 8		2.611		2.611	
J 3	+1.667	4.278	0	4.278	
J 2	+2.502	6.780	0	6.780	
J 1	+1.244	8.024	+1	8.025	
UBM 9	-1.350	6.674	+1	6.675	
UBM 9		6.675		6.675	
C 7	+1.492	8.167	-1	8.167	
C 6	+1.354	9.521	-3	9.518	
C 5	+0.666	10.187	-4	10.183	
C 3	+2.103	12.290	-6	12.284	
C 2	+0.240	12.530	-7	12.523	
C 2		12.523		12.523	
C 1	-0.820	11.703	-1	11.702	
UBM 7	+0.796	12.499	-2	12.497	
E 4		0.983		0.983	
E 3	+0.980	1.963	-1	1.962	
E 2	+0.360	2.323	-2	2.321	
E 1	+3.108	5.431	-4	5.427	
UBM10	-0.212	5.219	-5	5.214	
UBM10		5.214		5.214	
D 3	-1.106	4.108	-1	4.107	
D 2	+0.670	4.778	-2	4.776	
D 1	+0.282	5.060	-4	5.056	
UBM 9	+1.620	6.680	-5	6.675	
UBM 9		6.675		6.675	
N 1	+2.195	8.870	+1	8.871	
N 2	+2.862	11.732	+2	11.734	
N 3	-2.514	9.218	+2	9.220	
N 4	+2.242	11.460	+3	11.463	
N 5	-1.062	10.398	+4	10.402	
N 6	+4.021	14.419	+5	14.424	
N 7	+0.334	14.753	+6	14.759	
N 8	+1.424	16.177	+6	16.183	
N 9	+0.368	16.545	+7	16.552	
UBM15	+2.052	18.597	+8	18.605	

Station No.	Observed Difference	Observed Height	Correction	Height	Remarks
UBM15		18.605		18.605	
P 5	-1.100	17.505	+1	17.506	
P 4	+0.268	17.773	+2	17.775	
P 3	-2.003	15.770	+2	15.772	
P 2	-1.372	14.398	+3	14.401	
P 1	-1.046	13.352	+4	13.356	
C 2	-0.834	12.518	+5	12.523	
E 4		0.983		0.983	
UBM11	-0.104	0.879		0.879	
H 14		1.134		1.134	
UBM 4	-0.422	0.712		0.712	
CBM401		48.427		48.427	
UBM17	+2.996	51.423		51.423	
UBM15		18.605		18.605	
Q 1	-0.107	18.498	-2	18.496	
Q 2	+2.573	21.071	-4	21.067	
Q 3	+0.553	21.624	-6	21.618	
Q 4	+1.863	23.487	-7	23.480	
Q 5	+0.510	23.997	-9	23.988	
Q 6	+1.266	25.263	-11	25.252	
Q 7	+0.190	25.453	-13	25.440	
Q 8	2.833	28.286	-15	28.271	
Q 9	+1.172	29.458	-17	29.441	
Q 10	+2.616	32.074	-19	32.055	
Q 11	+7.324	39.398	-21	39.377	
Q 12	-1.812	37.586	-22	37.564	
Q 13	+3.431	41.017	-24	40.993	
Q 14	-4.562	36.455	-26	36.429	
UBM16	+0.901	37.356	-28	37.328	
R 1	+2.140	39.496	-30	9.466	
R 2	+0.824	40.320	-32	40.288	
A	+1.161	41.481	-34	41.447	
R 3	+1.316	42.797	-35	42.762	
R 4	+0.602	43.399	-37	43.362	
R 5	+2.387	46.236	-39	46.117	
UBM17	+5.228	51.464	-41	51.423	

Result of Beck Mark Leveling

Station	Elevation	Remarks	Station	Elevation	Remarks
	m			m	
T2339	13.900		UBM 8	1.692	
A 1	15.616		F 8	2.611	
A 3	15.185		F 9	1.887	
A 4	15.168		F 10	3.379	
UBM 1	15.216		F 11	2.742	
B 1	17.714		G 0	2.451	
B 2	15.449		G 1	2.081	
B 3	14.051		G 2	3.519	
B 4	14.151		G 3	5.283	
B 5	12.646		G 4	5.119	
B 6	12.080		G 5	4.416	
B 7	12.643		G 6	6.098	
UBM 7	12.497		G 7	5.615	
C 1	11.702		G 8	3.419	
C 2	12.523		G 9	2.797	
C 3	12.284		G 10	1.727	
C 5	10.183		G 11	0.623	
C 6	9.518		G 12	1.344	
C 7	8.167		UBM 5	1.115	
UBM 9	6.675		H 1	1.396	
D 1	5.056		H 2	2.751	
D 2	4.776		H 3	2.237	
D 3	4.107		H 4	1.046	
UBM10	5.214		H 5	1.519	
E 1	5.427		H 6	1.583	
E 2	2.321		H 7	2.929	
E 3	1.962		H 8	2.901	
E 4	0.983		H 9	1.960	
UBM11	0.879		H 10	1.988	
F 2	0.734		H 11	1.689	
F 3	1.324		H 12	1.655	
F 4	1.491		H 13	1.246	
F 5	1.177		H 14	1.134	

Station	Elevation	Remarks	Station	Elevation	Remarks
	m			m	
UBM 4	0.712		N 7	14.759	
I 1	1.836		N 8	16.183	
I 2	2.235		N 9	16.552	
I 3	2.843		UBM15	18.605	
I 4	4.467		P 1	13.356	
I 5	5.552		P 2	14.401	
I 6	6.704		P 3	15.772	
I 7	8.877		P 4	17.775	
I 8	9.799		P 5	17.506	
I 9	10.630		Q 1	18.496	
I 10	13.283		Q 2	21.067	
I 11	12.869		Q 3	21.618	
I 12	13.516		Q 4	23.480	
J 1	8.025		Q 5	23.988	
J 2	6.780		Q 6	25.252	
J 3	4.278		Q 7	25.440	
K 1	12.276		Q 8	28.271	
K 2	11.335		Q 9	29.441	
K 3	10.047		Q 10	32.055	
UBM 6	9.093		Q 11	39.377	
L 1	8.341		Q 12	37.564	
L 2	5.961		Q 13	40.993	
L 3	5.834		Q 14	36.429	
L 4	4.468		UBM16	37.328	
L 5	4.362		R 1	39.466	
L 6	3.447		R 2	40.288	
L 7	2.215		A	41.447	
N 1	8.871		R 3	42.762	
N 2	11.734		R 4	43.362	
N 3	9.220		R 5	46.197	
N 4	11.463		UBM17	51.423	
N 5	10.402				
N 6	14.424				

II. SOIL SURVEY AND STUDY

1. General.

Soil surveys were carried out for the purpose of:

study of foundation for embankment,

study of foundation for structure,

study of borrow pit,

study of river-bed materials,

study of construction equipment and

study of construction method.

For the purpose mentioned above, the following tests were conducted in the period from August 15, 1977 to November 2, 1977.

Cone penetration test.

Swedish sounding test.

Auger boring test.

Vane test (in-situ test).

N value test (machine boring).

Soil test such as measurement of natural moisture content, grain-size analysis, specific gravity test, liquid limit test, plastic limit test, wet density test and unconfined compression test.

2. Sites and Kinds of Tests.

The sites and kinds of tests which were conducted during this study are shown in Fig.2-1-1 to Fig.2-2-11. The former group of the figures is mainly for the study of foundation of embankment and the latter group is mainly for the study of foundation of structures.

3. Data Obtained.

Soil profiles were drawn based on the results of cone penetration tests, Swedish sounding tests, Auger boring tests and machine borings. These are shown in Fig.2-3-1 to Fig.2-3-35, of which Fig.2-3-1 shows legend for soil profiles, Figs.2-3-2 to 2-3-25 give soil profiles which were used mainly for study of embankment and Figs.2-3-26 to 2-3-35 give soil profiles which were used mainly for study of structures.

By use of the results of cone penetration tests and Swedish sounding tests, q_u and ϕ were obtained by the following relations.

N value -- in standard penetration test, driving number required for penetration sampler by 30 cm is called N value,

q_u -- unconfined compressive strength, and

ϕ -- angle of internal friction.

$$q_u = 0.05 + \frac{q_c}{30} \quad /1$$

$$N = \frac{q_c}{4} \quad /1$$

$$q_u = 0.0045\ Wsw + 0.0075\ Nsw \quad /2$$

$$N = 2 + 0.067\ Nsw \text{ ---- for gravel, sand or sandy soil} \quad /2$$

$$N = 3 + 0.05\ Nsw \text{ ---- for clay or cohesive soil} \quad /2$$

$$\phi = \sqrt{12\ N} + 20 \quad /3$$

where :

q_u	: unconfined compressive strength	: Kg/cm^2
q_c	: Cone index	: Kg/cm^2
N	: N value	
Wsw	: Load of Swedish sounding test	: Kg
Nsw	: Number of half-turns in Swedish sounding test	: times/m(depth)
ϕ	: Angle of internal friction	: degree

/1 Soil Survey Report for the Ular River Urgent Flood Control Project, August 1973.

/2 Inada's formula; from Soil Survey Method published by the Japan Society for Soil and Foundation Engineering.

/3 Dunham's formula, ditto.

A machine boring was conducted near the confluence of Pulau Canal (S. Kotabangun) and the Ular river at Stake 10.3 km on the right side bank of the Ular. The site of the machine boring is shown in Fig.2-2-11. Fig.2-3-35 shows the boring log.

The results of soil tests are shown in Tables 2-1-1 to 2-1-6.

Vane tests in situ were conducted at the sites shown in Figs.2-1-1 to 2-1-6 and the results are shown in Tables 2-2-1 to 2-2-3.

4. Study of Section of Embankment.

It was assumed from the soil profiles that the foundation on the stretch from 0.0 km to -13.5 km is composed of alternations of strata of cohesive soil and loosen sand and the surface is generally covered by cohesive soil. The foundation around -12 km is comparatively soft and the depth of soft layer is estimated to be about 10 m. Therefore, this site was chosen for the study of section of embankment to be constructed on a poor foundation. The study was made under the conception that internal friction is null. In other area, the soil condition is almost the same as those on the stretch from 0.0 km to 10.0 km where the emergent works for improvement have been executed. Therefore, the standard cross section in case of the Urgent Project can be applied to the other area.

(1) Soil condition for study of section of embankment.

The following soil conditions were assumed for the study of section of embankment.

(a) γ_{tE} : Wet density of soil for embankment.

This value was assumed at 1.8 t/m^3 because sands from the river bed will be used as materials for embankment.

(b) γ_t : Wet density of soil in foundation.

Based on the results of soil tests shown in Tables 2-1-1 to 2-1-6, natural moisture content and specific gravity were assumed as follows.

W_n = 75 % : natural moisture content.

G_s = 2.65 : specific gravity.

In case the values of degree of saturation and unit weight of pore water are taken as follows,

S_r = 100 : degree of saturation

$\gamma_w = 1$ t/m : unit weight of pore water,

wet density of soil, γ_t, is calculated as follows.

$$\gamma_t = 1 + \frac{\gamma_w(G_s - 1) + \frac{G_s W_n}{100}(\gamma_w - 1)}{\gamma_w + \frac{G_s W_n}{100}} = 1 + \frac{1.65}{1 + 0.0265\ W_n} \quad /1$$

$$= 1.55\ t/m^3$$

(c) e_o : Initial void ratio.

The value of initial void ratio is calculated as follows by use of the values of W_n and G_s mentioned above.

$$e_o = \frac{W_n \times G_s}{100} = 1.99$$

(d) c : Cohesion of foundation.

The following values were obtained as the results of field tests.

$W_{sw} = 25$ kg : Load in Swedish sounding test.

$q_c = 2$ to $3\ kg/cm^2$: Cone index.

/1 The following equation holds theoretically.

$$\gamma_t = G_s \frac{1 + W_n/100}{1 + e_n}$$

If we apply saturated state to the above equation

$$\frac{e_n}{W_n} \times 100 = \frac{G_s}{w}$$

to the above equation, we get

$$\gamma_t = 1 + \frac{\gamma_w(G_s - 1) + \frac{G_s W_n}{100}(\gamma_w - 1)}{\gamma_w + \frac{G_s W_n}{100}}$$

where γ_t is wet density, G_s is specific gravity, e_n is natural void ratio, γ_w is density of pore water and W_n is natural moisture content.

Applying these values to the emperical formula shown in the previous paragraph 3, we get

$$q_u = 0.0045W_{sw} + 0.0075N_{sw} = 0.0045 \times 25 + 0 = 0.11 \text{ kg/cm2}$$

and employing the following relation

$$c = \frac{q_u}{2}$$

we get,

$$c = \frac{0.11}{2} = 0.055 \text{ kg/cm}^2 = 0.55 \text{ t/m}^2.$$

Using the value $q_c = 2 \text{ kg/cm}^2$,

$$q_u = 0.05 + \frac{q_c}{30} = 0.12 \text{ kg/cm}^2$$

$$c = \frac{q_u}{2} = 0.06 \text{ kg/cm}^2 = 0.6 \text{ g/m}^2.$$

If we use the value $q_c = 3 \text{ kg/cm}^2$,

$$q_u = 0.15 \text{ kg/cm}^2$$

$$c = 0.08 \text{ kg/cm}^2 = 0.8 \text{ g/m}^2.$$

On the other hand, it is judged from the results of Vane Tests shown in Tables 2-2-1 to 2-2-3 that cohesion, c, can be estimated at 0.7 to 0.9 t/m^2. In view of this fact, 0.8 t/m^2 was taken as the value of cohesion of foundation, and angle of internal friction was fixed to be zero.

(e) P_o : Precompression load.

Soil of foundation is assumed to be normally consolidated clay. Therefore, the following formula was employed for calculation of precompression load.

$$P_o = (\gamma_t - \gamma_w)Z$$

where Z is a half of thickness of soft layer. If we use a value,

$$Z = \frac{10.5}{2} = 5.25 \text{ m}$$

based on the condition shown in Fig.2-4-1,

$$P_o = (1.55 - 1.0) \times 5.25 = 2.89 \text{ t/m}^2.$$

(f) C_c : Compression index.

The Skempton formula

$$C_c = 0.009(W_L - 10)$$

was adopted, where W_L is liquid limit. W_L is estimated at 60 % from the results of soil tests.

$$C_c = 0.009\ (60 - 10) = 0.45.$$

(g) C_v : Coefficient of consolidation.

It is generally reported that the value of C_v is 5×10^{-2} to 5×10^{-4} for alluvial clay. In this case, C_v was assumed at 1×10^{-3} cm^2/s.

(h) $\frac{\Delta c}{\Delta p}$: Coefficient of increase in strength by consolidation of clay.

Δc : Increased cohesion due to consolidation of clay.

Δp : Increased compressive stress in foundation due to embankment.

The value of $\frac{\Delta c}{\Delta p}$ is usually fixed by means of consolidated-undrained triaxial compression test. However, since the value of $\Delta c/\Delta p$ is generally 1/4 to 1/3, 1/4 was taken in this study.

(2) Stability of slope.

Study was made on stability of slope by use of the Tayler method shown in Fig.2-4-2. First, a trapezoid section with a height of 3.5 m and slope of 1:2 was taken up for the study (see Fig.2-4-1).

$$F_s = No \frac{c}{\gamma_{tE} H_E} \quad \text{(see Fig.2-4-2)}$$

where F_s is safety factor, No is stability number, γ_{tE} is wet density of soil for embankment, H_E is height of embankment and c is cohesion.

If we use the values

$c = 0.8\ t/m^2$ (see 4.(1)(d))

$\gamma_{tE} = 1.8\ t/m^2$ (see 4.(1)(a))

$H_E = 3.5\ m$ (see Fig.2-4-1)

$H = 10.5\ m$ (see Fig.2-4-1)

$d = H/H_E = 3$,

and we assume that the slope is 1 : 2 or $\beta = 26.6°$, the value of No is obtained by Fig.2-4-2.

$$No = 5.6$$

Therefore, F_s is calculated as follows.

$$F_s = 5.6 \times \frac{0.8}{1.8 \times 3.5} = 0.71$$

This value means that the embankment is unstable. Therefore, continuous work of banking up to 3.5 m should not be allowed, but two-stages construction should be considered.

It was planned that a trapezoid section with a height less than 3.5 m and a slope of 1:2 is constructed at the first stage and then another trapezoid section with a slope of 1:2 is constructed at the second stage up to the design height 3.5 m on the first trapezoid section. For this purpose, banking height at the first stage was calculated assuming $F_s = 1.2$. This value of H_E was found as follows.

$$H_E = \frac{No \times c}{\gamma_{tE} \times F_s} = \frac{5.6 \times 0.8}{1.8 \times 1.2} = 2.1 \text{ m.}$$

Therefore, 2.0 m is recommendable as the first-stage banking height.

At the second stage, the embankment must be constructed up to the height of 3.5 m in accordance with the design of river. If we assume a double section with a bottom width of 40 m as shown by thick line in Fig.2-4-1 and assume that a single trapezoid section with a bottom width of 40 m and a height of 3.5 m can substitute for the double section for examining the stability, the value of β is estimated at 11°; accordingly No = 6.0 is obtained from Fig.2-4-2. Next, two cases must be considered; one is a case that no increase in cohesion is considered and the other is a case that increase in cohesion by the first-stage banking is considered.

(a) A case without consideration of increase in cohesion.

$$F_s = 6.0 \frac{0.8}{1.8 \times 3.5} = 0.76$$

This means that the embankment is unstable.

(b) A case with consideration of increase in cohesion by the first-stage banking.

$$\frac{\Delta c}{\Delta p} = 0.25$$

$$\Delta p = \gamma_{tE} H_E = 1.8 \times 2.0 = 3.6 \text{ t/m}^2$$

Therefore $\Delta c = 0.25 \times 3.6 = 0.9 \text{ t/m}^2$

This $\Delta c = 0.9 \text{ t/m}^2$ is a value under the condition that the degree of consolidation, U, is 100 % and should be decreased in accordance with the value of U.

Time required for consolidation is directly proportional to the square of drain path D in consolidating layer. Therefore, the presence of sand layer which will be a drain-layer for the consolidating strata must have an influence upon the consolidating time.

Fir.2-4-5 given in the next paragraph shows the relation between settlement and consolidation time with a parameter of D = 5.25, 2.50, 2.00 and 1.5 m. On the other hand, the foundation in this area is composed of alternations of cohesive-soil strata and loosen-sand strata as mentioned in the previous section 3. Therefore, considering the soil condition shown in soil profiles, we assumed D = 2.00 m. If the first-stage embankment is left as it is for 3 months (90 days), the degree of consolidation U will become about 50 % as read in Fig.2-4-5. Accordingly the value of cohesion c will be increased by $\Delta c \times 50\% = 0.9 \times 0.5 = 0.45$ t/m^2. Namely the cohesion c will become,

$$c = 0.8 + 0.45 = 1.25 \ t/m^2$$

in 3 months after the completion of the first-stage embankment.

F_s is now calculated as follows by use of increased cohesion.

$$F_s = \frac{6.0 \times 1.25}{1.8 \times 3.5} = 1.19$$

From this result, it may be said that the embankment with a height of 3.5 m is safe.

(3) Consolidation.

Settlement is estimated by the following formula,

$$S = \frac{C_c}{1 + e_o} H \log \frac{P_o + \Delta P}{P_o} \qquad /1$$

where S is settlement in m, C_c is compression index, e_o is initial void ratio, H is thickness of layer to be consolidated in m, P_o is precompression load in t/m^2 and Δp is increased compressive stress in foundation due to embankment in t/m^2.

Consolidation time is estimated by the following formula.

$$t = \frac{D^2}{C_v} T$$

where t is time for consolidation, D is length of drain path, C_v

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/1 Soil-mechanics Handbook published by Japan Society of Soil-mechanics and Foundation Engineering.

is coefficient of consolidation and T is time factor in consolidation. There is the following theoretical relation between T and U.

U (%)	10	20	30	40	50	60	70	80	90	100
T	0.008	0.031	0.071	0.126	0.196	0.287	0.403	0.567	0.848	∞

(a) Settlement.

The following values were obtained in the previous paragraph 4(1).

$C_c = 0.45$

$e_o = 1.99$

$H = 10.5$ m

$P_o = 2.89$ t/m^2

Therefore, settlement S can be calculated if Δp is known. Δp can be estimated by use of Osterberg's figure named "Influence Value for Vertical Stress Under Embankment Load of Infinite Length" which is shown as Fig.2-4-3. In using this figure, it must be noted that,

A, B, C, D : point A, point B, point C, point D at the depth of $Z = H/2$ as shown in Fig.2-4-1,

Δp_{A1} : Δp at point A at the first stage of embankment,

Δp_{A2} : Δp at point A at the second stage of embankment,

z : must be replaced by Z,

P : must be replaced by $\gamma_{tE} H_E$,

σ_z : must be replaced by Δp.

Settlement of the section A at the first-stage embankment, S_{A1}, can be calculated by the above settlement formula by use of the above-mentioned values. The calculated Δp's and S's are listed as below.

$\Delta p_{A1} = 3.56$ t/m^2	$S_{A1} = 55.1$ cm	
$\Delta p_{A1} = 1.76$ "	$S_{A2} = 32.6$ cm	total 87.7 cm
$\Delta p_{B1} = 3.54$ "	$S_{B1} = 54.9$ "	
$\Delta p_{B2} = 0.83$ "	$S_{B2} = 17.3$ "	total 72.2 cm

$\Delta p_{C1} = 2.53\ t/m^2$ $S_{C1} = 43.1$ cm

$\Delta p_{C2} = 0.02$ " $S_{C2} = 0.5$ " total 43.6 cm

$\Delta p_{D1} = 1.05$ " $S_{D1} = 21.3$ "

$\Delta p_{D2} = 0.01$ " $S_{D2} = 0.2$ " total 21.5 cm

These results are shown in Fig.2-4-4.

(b) Consolidation time.

For the purpose of estimating the commencement time of the second-stage embankment, consolidation time t at section A was calculated by the above-mentioned formula giving the values of U ranging from 10 % to 100 % and the values of D ranging from 5.25 m to 1.50 m, where the value of C_v was assumed at 1×10^{-3} cm^2/s as mentioned previously. The results are shown in the following table.

Consolidation Time in Days

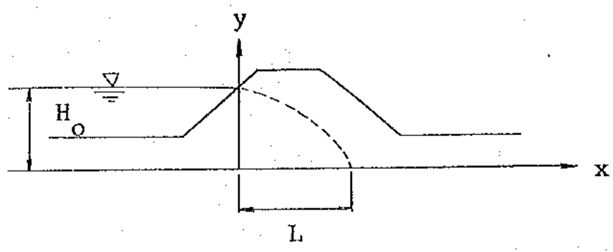
U	T	t				Settlement
%		D=5.25m	D=2.50m	D=2.00m	D=1.50m	S_{A1} cm
10	0.008	26	6	4	2	5.5
20	0.031	99	22	14	8	11.0
30	0.071	226	51	33	18	16.5
40	0.126	402	91	58	33	22.0
50	0.196	625	142	91	51	27.6
60	0.287	916	208	133	75	33.1
70	0.403	1,286	292	187	105	38.6
80	0.567	1,809	410	262	148	44.1
90	0.848	2,705	613	393	221	49.6
100		∞	∞	∞	∞	55.1

(4) Seepage.

On the occasion of the Urgent Flood Control Project, study was made of seepage line based on the condition that the coefficient of permeability of the body of levee, K, is 1×10^{-2} cm/s. In the present project, it was assumed that almost all the levees on the stretch from 0 km to about -12 km would be constructed with dredged soil. Therefore, it was assumed that the coefficient of permeability of the body of levee would be 1×10^{-1} cm/s considering grain size of river-bed materials. Accordingly it is necessary to coat the surface of embankment with cohesive soil.

Thickness of cohesive soil which shall coat the levee was studied by use of the following formula proposed by Dr. Mononobe (see Nagaho Mononobe: Hydraulics)

$$L = C\sqrt{\frac{K}{\lambda}H_o t} \qquad y = H_o(1 - \frac{x^2}{L^2})$$



In this formula,

L : creeping distance of seepage line in time t and on condition that H_o is constant.

C : Coefficient that is nearly equal to 2 m/hr.

K : Coefficient of permeability

1×10^{-1} cm/s = 3.6×10^{0} m/hr for sand

1×10^{-5} cm/s = 3.6×10^{-4} m/hr for cohesive soil.

λ : Porosity

0.4 for sand

0.6 for cohesive soil.

H_o : Mean water depth during time t. This depth was assumed at 3 m considering allowance to 2.7 m that is the difference between the mean height of the landside ground and the design high water level.

t : Duration of the design high water level. This value was assumed at 24 hr. The past record shows that duration of flood stages nearly equal to this design high water level does not exceed 12 hr; but 24 hr was taken considering allowance.

Fig.12 shows the seepage line in case the levee was coated with cohesive soil of 50 cm in thickness. The process of calculations are given in the Data Book.

The seepage line given in Fig.12 indicates that state in saturation in the levee body. But it is presumed to take much time before reaching this state partly because quantity of seepage through the cohesive soil for coating will be little and partly because the porosity of levee body is comparatively large. Therefore, the seepage line after 24 hr should appear in a location lower than the above-mentioned, probably near ground water table.

Based on the quantity of seepage through the cohesive soil and the porosity of the levee body, the height of saturation in the levee body, h, was calculated as follows.

Following the Darcy Law, quantity of seepage is expressed by the following equation.

$$q = kiA$$

where q is quantity of seepage, i is hydraulic gradient (= difference between two water heads/thickness of cohesive soil = 3 m/0.5 m = 6.0) and A is area for seepage (=wetted perimeter per unit width = 6.7 m × 1 m; see Fig.12).

$$q = KiA = 3.6 \times 10^{-4} \times 6.0 \times 6.7 = 1.45 \times 10^{-2}\ m^3/hr.$$

Quantity of seepage for 24 hr is,

$$Q = 1.45 \times 10^{-2} \times 24 = 3.5 \times 10^{-1}\ m^3.$$

On the other hand, void of levee body is expressed by,

$$V_a = \lambda(1 - \frac{S_r}{100})V = \lambda(1 - \frac{S_r}{100}) \times B \times h$$

where λ is porosity (= 0.4 for sand), S_r is degree of saturation (80 % was taken assuming a wet state), V is volume of levee body saturated with water (= B × h), B is width of levee (= 16 m referring to Fig.12) and h is height of saturated sand of the levee body (see Fig.12).

$$V_a = 0.4 \times (1 - 0.8) \times 16 \times h$$

$$h = \frac{V_a}{0.4 \times 0.2 \times 16}$$

Putting $V_a = Q$, we get,

$$h = \frac{3.5 \times 10^{-1}}{1.28} = 2.7 \times 10^{-1} = 0.3\ m.$$

This result is shown in Fig.12 with broken line. This study shows that the thickness of cohesive soil is enough if about 50 cm is taken.

5. River-bed Materials.

Sounding with a stick from on a boat and examination with walk in shoals proved that river-bed materials are sands on the stretch from 0 km to -13.5 km. Samplings were made at -13 km, -10 km, -6 km, and -4 km and grain-size analysis and measurement of specific gravity were conducted. The results are shown in Table 2-1-5 and Fig.2-4-6. The mean grain size is 1.15 mm.

6. Some Comments on Construction Equipment and Earth Work.

As shown in Figs.2-3-1 to 2-3-35, the area projected for embankment is covered with soft cohesive soil, loosen sand or sandy soil; therefore, construction equipment to be used for this area should be of small contact pressure.

Cone-index is generally used as a yardstick which gives the relation between contact pressure of equipment and strength of soil; therefore, this is shown in the following table.

Minimum Cone-Index for Construction Equipment

Equipment	Cone-index q_c
Swampy bulldozer	Workable under 4
Bulldozer (middle-size)	5 to 7
Bulldozer (large-size), scraper	7 to 10
Auto-scraper	10 to 13
Truck (6 to 7.5 ton)	15 or more

As this river has many shoals and frequent evacuations will be needed during the flood season.

Ground water table is so high that it is necessary to try to improve the traficability as much as possible by means of lowering the ground water table. For this purpose, it should be taken into consideration

a. to execute the construction during the dry season,

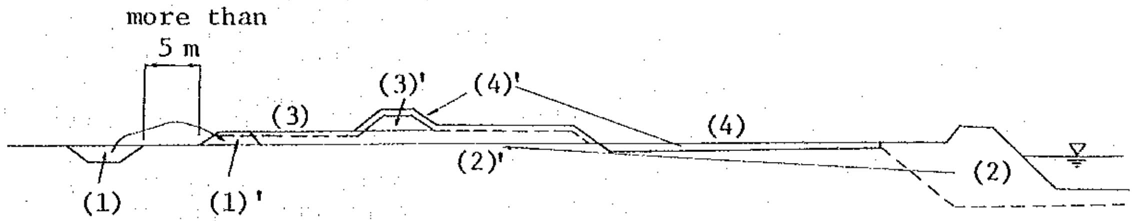
b. to provide with drainage channels or to consider drainage with pumps if natural drainage is difficult, and

c. to pay careful attention so as to drain excess water on the construction surface and the access roads.

The following two alternatives are recommendable as the order of construction of levees near the distance-mark -12 km.

Alternative 1.

Construction order is described below referring to the following figure.



(a) Excavate drainage channel shown as (1) by man power and bank with excavated soil at (1)'.

(b) Dredge soil at (2) and dump the dredged soil at (2)' as the first stage of embankment.

(c) Bank at (3)' with the soil from the surface of (2)' in 3 months after the execution of (2)'.

(d) Cover the surface of the dike with cohesive soil taken from high-water channel at (4).

It must be noted that, in banking, consideration must be made on a banking height which includes the height of settlement due to consolidation.

Alternative 2.

This alternative is a plan in case the execution of construction will extend over a period of two years.

(a) Execute the first-stage banking at (2)' with the soil dredged at (2) and leave it for one year.

(b) Employ (2)' as transportation passage for construction equipment and excavate drain at (1). In this case, excavation must be conducted from the river mouth if possible.

(c) The following works will be done by the same way as Alternative 1.

III. NETWORK OF HYDROLOGICAL OBSERVATION STATIONS

It was planned to additionally provide with the following stations for reinforcing the observation of hydrological quantities and the communication system for information of them.

2 recording rain-gage stations at Tiga Juhar and Serbajadi.

5 recording water-level-gage stations at Balapulung, Sipinggan, Mabar, Negeri Dolok and Esperance.

3 VHF-radio fixed stations at Bandar Tiga, Serbajadi and the Site Office at Perbaungan.

3 VHF-radio mobile stations (cars).

The costs of them are as follows.

1. Recording rain-gage stations.

Cost per station.

Construction work	Rp 7,000,000
Instrument	$2,500
Recording paper	$83 for 5 years
1 spare timer	$580
Miscellaneous	$40 for 5 years
Others	Rp 140,000 & $750
Total: local currency	Rp 840,000
foreign currency	$3,950
Cost for two stations	Rp 1,680,000 & $7,900

2. Recording water-level-gage stations.

Cost per station.

Construction work	Rp 1,900,000
Instrument	$2,500
Recording paper	$250 for 5 years

1 spare timer	$580
Miscellaneous	$200
Others	Rp 380,000 & $700
Total:	Rp 2,280,000 & $4,230
Cost for five stations	Rp 11,400,000 & $21,160

3. Three VHF-radio fixed stations and three VHF-radio mobile stations.

Construction work	Rp 2,000,000 x 3	= Rp 6,000,000
Instruments	$4,980 x 3	= $14,940
Spare parts	$500 x 3	= $1,500
Mobile stations with instruments	$18,460 x 3	= $55,390
Supervision (2 engineers for 30 days each)		$22,400
Others	Rp 2,000,000 & $5,350	
Total	Rp 8,000,000 & $99,500	

Grand total of the above	Rp 21,080,000 & $128,600

Table 2-1-1 SOIL TEST DATA (FOR FOUNDATION)

LOCATION : – 12 Km TESTED BY :

SAMPLE NO			R1	R1	R6	R7	R7	R7
DEPTH		m	0.0 ~ 0.3	0.9 ~ 1.2	0.5 ~ 0.8	0.3 ~ 0.7	0.7 ~ 0.9	0.9 ~ 1.2
GRAIN SIZE DISTRIBUTION	GRAVEL (2000μ <)	%	0	0	0	0	0	6
	SAND (74 ~ 2000μ)	%	67	46	41	69	75	76
	SILT (5 ~ 74μ)	%	} 33	} 54	} 59	} 31	} 25	} 18
	CLAY (< 5μ)	%						
	MAX. GRAIN SIZE	mm						
	UNIFORMITY COEFF. U_c							
	CURVATURE COEFF. U_c'							
CONSISTENCY INDEX	LIQUID LIMIT w_L	%		62	60	100	58	60
	PLASTIC LIMIT w_P	%			45	48	43	
	PLASTICITY INDEX I_P				15	52	15	
CLASSIFICATION			Sandy fine grained soil	Fine grained soil	Fine grained soil	Sandy fine grained soil	Sandy fine grained soil	Sandy fine grained soil
SPECIFIC GRAVITY G_s			2.64 2.75			2.58	2.58	
NATURAL CONDITION	MOISTURE CONTENT w	%	61.5	70.8	75.0	73.0	111.6	93.8
	WET DENSITY γ_t	g/cm³						
	VOID RATIO e							
	DEGREE OF SATURATION S_r	%						
MECHANICAL CHARACTERISTICS – UNCONFINED COMPRESSION TEST	STRENGTH q_u	kg/cm²						
	DEFORMATION COEFF E_{50}	kg/cm²						
	SENSITIVITY RATIO S_t							
DIRECT SHEAR TEST	※CONDITION							
	COHISION c	kg/cm²						
	ANGLE OF INTERNAL FRICTION ϕ							
TRIXIAL COMPRESSION TEST	※CONDITION							
	COHESION c	kg/cm²						
	ANGLE OF INTERNAL FRICTION ϕ							
CONSOLIDATION TEST	YIELD STRESS p_y	kg/cm²						
	COMPRESSION INDEX C_c							

REMARKS :

UU : Unconsolidated – undrained shear test
CU : Consolidated – undrained shear test
CD : Consolidated – drained shear test
Vinculums (‾) shall be marked above the signs like $\overline{UU}$ $\overline{CU}$ and $\overline{CD}$ when pore pressure measured

Table 2-1-2 SOIL TEST DATA (FOR FOUNDATION)

LOCATION : −11.25 Km TESTED BY :

SAMPLE NO		R_1	R_6	R_6	R_6	R_6	
DEPTH	m	0.0 ~ 0.3	0.0 ~ 0.3	0.9 ~ 1.2	1.2 ~ 1.5	2.7 ~ 3.0	~
GRAIN SIZE DISTRIBUTION: GRAVEL (2000μ <)	%	0	8	0	0	8	
GRAIN SIZE DISTRIBUTION: SAND (74~2000μ)	%	65	73	63	63	64	
GRAIN SIZE DISTRIBUTION: SILT (5~74μ)	%	} 35	} 19	} 37	} 37	} 28	
GRAIN SIZE DISTRIBUTION: CLAY (< 5μ)	%						
GRAIN SIZE DISTRIBUTION: MAX. GRAIN SIZE	mm						
GRAIN SIZE DISTRIBUTION: UNIFORMITY COEFF. U_c							
GRAIN SIZE DISTRIBUTION: CURVATURE COEFF. U_c'							
CONSISTENCY INDEX: LIQUID LIMLT w_L	%	64	62	68			
CONSISTENCY INDEX: PLASTIC LIMIT w_p	%	48		55	36	34 — 44	
CONSISTENCY INDEX: PLASTICITY INDEX I_p		16		13			
CONSISTENCY INDEX:							
CLASSIFICATION		Sandy fine grained soil	Sandy fine grained soil	Sandy fine grained soil	Sandy fine grained soil	Sandy fine grained soil	
CLASSIFICATION							
SPECIFIC GRAVITY G_s						2.68 2.74	
NATURAL CONDITION: MOISTURE CONTENT w	%	99.4	56.9	111.7	39.4	70.1	
NATURAL CONDITION: WET DENSITY γ_t	g/cm³						
NATURAL CONDITION: VOID RATIO e							
NATURAL CONDITION: DEGREE OF SATURATION S_r	%						
MECHANICAL CHARACTERISTICS: UNCONFINED COMPRESSION TEST: STRENGTH q_u	kg/cm²						
MECHANICAL CHARACTERISTICS: UNCONFINED COMPRESSION TEST: DEFORMATION COEFF. E_{50}	kg/cm²						
MECHANICAL CHARACTERISTICS: UNCONFINED COMPRESSION TEST: SENSITIVITY RATIO S_t							
MECHANICAL CHARACTERISTICS: DIRECT SHEAR TEST: ※CONDITION							
MECHANICAL CHARACTERISTICS: DIRECT SHEAR TEST: COHISION c	kg/cm²						
MECHANICAL CHARACTERISTICS: DIRECT SHEAR TEST: ANGLE OF INTERNAL FRICTION ϕ							
MECHANICAL CHARACTERISTICS: TRIXIAL COMPRESSON TEST: ※CONDITION							
MECHANICAL CHARACTERISTICS: TRIXIAL COMPRESSON TEST: COHESION c	kg/cm²						
MECHANICAL CHARACTERISTICS: TRIXIAL COMPRESSON TEST: ANGLE OF INTERNAL FRICTION ϕ							
MECHANICAL CHARACTERISTICS: CONSOLIDTON TEST: YIELD STRESS p_y	kg/cm²						
MECHANICAL CHARACTERISTICS: CONSOLIDTON TEST: COMPRESSION INDEX C_c							
MECHANICAL CHARACTERISTICS: CONSOLIDTON TEST:							
MECHANICAL CHARACTERISTICS: CONSOLIDTON TEST:							

REMARKS :

UU : Unconsolidated − undrained shear test
CU : Consolidated − unarained shear test
CD : Consolidated − drained shear test
Vinculums (−) shall be marked above the signs like UU, CU and CD when pore pressure measured

Table 2-1-3 SOIL TEST DATA (FOR FOUNDATION)

LOCATION : ~ 11 Km TESTED BY :

SAMPLE NO		L4	L4	L4	L4	L4	L4
DEPTH m		0.0 ~ 0.3	0.5 ~ 1.0	1.0 ~ 1.25	1.25 ~ 1.9	1.9 ~ 2.4	2.4 ~ 3.0
GRAIN SIZE DISTRIBUTION	GRAVEL (2000μ <) %	0	8	5	0	0	8
	SAND (74 ~ 2000μ) %	65	67	88	85	79	81
	SILT (5 ~ 74μ) %	35	25	7	15	21	11
	CLAY (< 5μ) %						
	MAX. GRAIN SIZE mm						
	UNIFORMITY COEFF. U_c			9.1	26.4		
	CURVATURE COEFF. U_c'			1.3	2.2		
CONSISTENCY INDEX	LIQUID LIMLT w_L %	51	77				73
	PLASTIC LIMIT w_p %	30	34		31		34
	PLASTICITY INDEX I_p	21	43				39
CLASSIFICATION		Sandy fine grained soil	Sandy fine grained soil	Sand, Mixed fine grained soil	Sand, Mixed fine grained soil	Sandy fine grained soil	Sandy fine grained soil
SPECIFIC GRAVITY G_s		2.50 2.56	2.61 2.68	2.77 2.65			
NATURAL CONDITION	MOISTURE CONTENT w %	66.5	46.8	22.0		49.6	28.1
	WET DENSITY γ_t g/cm³						
	VOID RATIO e						
	DEGREE OF SATURATION S_r %						
MECHNICAL CHARACTERISTICS — UNCONFINED COMPRESSION TEST	STRENGTH q_u kg/cm²						
	DEFORMATION COEFF E_{50} kg/cm²						
	SENSITIVITY RATIO S_t						
DIRECT SHEAR TEST	※ CONDITION						
	COHISION c kg/cm²						
	ANGLE OF INTERNAL FRICTION ϕ						
TRIXIAL COMPRESSION TEST	※ CONDITION						
	COHESION c kg/cm²						
	ANGLE OF INTERNAL FRICTION ϕ						
CONSOLIDTION TEST	YIELD STRESS p_y kg/cm²						
	COMPRESSION INDEX C_c						

REMARKS :

UU : Unconsolidated - undrained shear test
CU : Consolidated - undrained shear test
CD : Consolidated - drained shear test
Vinculums (‾) shall be marked above the signs like $\overline{UU}$, $\overline{CU}$ and $\overline{CD}$ when pore pressure measured

Table 2-1-4 SOIL TEST DATA (FOR FOUNDATION)

LOCATION : −10 Km TESTED BY :

		Unit						
SAMPLE NO			L_4	L_4	L_4	R_2	R_2	R_2
DEPTH		m	0.3 ~ 0.6	1.2 ~ 1.5	1.8 ~ 2.4	0.3 ~ 0.6	0.6 ~ 0.9	1.2 ~ 1.5
GRAIN SIZE DISTRIBUTION	GRAVEL (2000μ <)	%	8	6	8	9	0	0
	SAND (74 ~ 2000μ)	%	85	70	64	52	53	88
	SILT (5 ~ 74μ)	%	7	24 }	28 }	39 }	47 }	12
	CLAY (< 5μ)	%						
	MAX. GRAIN SIZE	mm						
	UNIFORMITY COEFF U_c							
	CURVATURE COEFF. U_c'							
CONSISTENCY INDEX	LIQUID LIMLT w_L	%		43	60	62	50	79
	PLASTIC LIMIT w_p	%		31	22	27	36	26 ~ 33
	PLASTICITY INDEX I_p			12	38	35	14	53 ~ 46
CLASSIFICATION			Sand, Mixed fine grained soil	Sandy fine grained soil	Sandy fine grained soil	Sandy fine grained soil	Sandy fine grained soil	Sand, Mixed fine grained soil
SPECIFIC GRAVITY G_s			2.56 2.74			2.68 2.61		2.63
NATURAL CONDITION	MOISTURE CONTENT w	%	24.5	57.3	71.2	48.0	73.2	34.1
	WET DENSITY γ_t	g/cm³						
	VOID RATIO e							
	DEGREE OF SATURATION S_r	%						
MECHNICAL CHARACTERISTICS — UNCONFINED COMPRESSION TEST	STRENGTH q_u	kg/cm²						
	DEFORMATION COEFF E_{50}	kg/cm²						
	SENSITIVITY RATIO S_t							
DIRECT SHEAR TEST	※CONDITION							
	COHISION c	kg/cm²						
	ANGLE OF INTERNAL FRICTION ϕ							
TRIXIAL COMPRESSION TEST	※ CONDITION							
	COHESION c	kg/cm²						
	ANGLE OF INTERNAL FRICTION ϕ							
CONSOLIDTION TEST	YIELD STRESS p_y	kg/cm²						
	COMPRESSION INDEX C_c							

REMARKS :

UU : Unconsolidated − undrained shear test
CU : Consolidated − undrained shear test
CD : Consolidated − drained shear test
Vinculums (−) shall be marked above the signs like UU, CU and CD when pore pressure measured

Table 2-1-5 SOIL TEST DATA (FOR FOUNDATION)

LOCATION: River bed

TESTED BY:

SAMPLE NO		-13 Km	-10 Km	-6 Km	-4 Km		
DEPTH	m	0 ~	0 ~	0 ~	0 ~	~	~
GRAIN SIZE DISTRIBUTION	GRAVEL (2000μ <) %	4	9	15	14		
	SAND (74~2000μ) %	96	91	85	86		
	SILT (5~74μ) %						
	CLAY (< 5μ) %						
	MAX. GRAIN SIZE mm						
	UNIFORMITY COEFF U_c	2.3	2.2	2.3	2.5		
	CURVATURE COEFF U'_c	1.0	1.0	1.1	1.2		
CONSISTENCY INDEX	LIQUID LIMIT w_L %						
	PLASTIC LIMIT w_p %						
	PLASTICITY INDEX I_p						
CLASSIFICATION		Sand	Sand	Sand	Sand		
SPECIFIC GRAVITY G_s		2.72	2.60	2.78	2.64		
NATURAL CONDITION	MOISTURE CONTENT w %						
	WET DENSITY γ_t g/cm³						
	VOID RATIO e						
	DEGREE OF SATURATION S_r %						
MECHANICAL CHARACTERISTICS — UNCONFINED COMPRESSION TEST	STRENGTH q_u kg/cm²						
	DEFORMATION COEFF E_{50} kg/cm²						
	SENSITIVITY RATIO S_t						
DIRECT SHEAR TEST	*CONDITION						
	COHISION c kg/cm²						
	ANGLE OF INTERNAL FRICTION ϕ						
TRIXIAL COMPRESSION TEST	*CONDITION						
	COHESION c kg/cm²						
	ANGLE OF INTERNAL FRICTION ϕ						
CONSOLIDTION TEST	YIELD STRESS P_y kg/cm²						
	COMPRESSION INDEX C_c						
Mean grain size dm mm		1.11	1.18	1.55	1.53		

REMARKS:

$$dm = \sum_{P=0\%}^{P=100\%} d \cdot \Delta P \Big/ \sum_{P=0\%}^{P=100\%} \Delta P$$

Where,
dm: mean grain size
d : mesh size of sieve
p : percent finer
ΔP: percent retained

UU: Unconsolidated – undrained shear test
CU: Consolidated – undrained shear test
CD: Consolidated – drained shear test
Vinculums (‾) shall be marked above the signs like $\overline{UU}$, $\overline{CU}$ and $\overline{CD}$ when pore pressure measured

Table 2-1-6 SOIL TEST DATA (FOR FOUNDATION)

LOCATION: 10.3 Km Right Boring Site NO. 4 TESTED BY:

		SAMPLE NO	1	2				
		DEPTH m	0.2 ~	0.2 ~	~	~	~	~
GRAIN SIZE DISTRIBUTION		GRAVEL (2000μ <) %						
		SAND (74~2000μ) %						
		SILT (5~74μ) %						
		CLAY (< 5μ) %						
		MAX. GRAIN SIZE mm						
		UNIFORMITY COEFF. U_c						
		CURVATURE COEFF. U_c'						
CONSISTENCY INDEX		LIQUID LIMIT w_L %						
		PLASTIC LIMIT w_P %						
		PLASTICITY INDEX I_P						
CLASSIFICATION								
SPECIFIC GRAVITY G_s								
NATURAL CONDITION		MOISTURE CONTENT w %						
		WET DENSITY γ_t g/cm³	1.55	1.65				
		VOID RATIO e						
		DEGREE OF SATURATION S_r %						
MECHNICAL CHARACTERISTICS	UNCONFINED COMPRESSION TEST	STRENGTH q_u kg/cm²	0.38	0.44				
		DEFORMATION COEFF. E_{50} kg/cm²	13.6	20.0				
		SENSITIVITY RATIO S_t						
	DIRECT SHEAR TEST	※CONDITION						
		COHISION c kg/cm²						
		ANGLE OF INTERNAL FRICTION ϕ						
	TRIXIAL COMPRESSION TEST	※ CONDITION						
		COHESION c kg/cm²						
		ANGLE OF INTERNAL FRICTION ϕ						
	CONSOLIDTION TEST	YIELD STRESS p_y kg/cm²						
		COMPRESSION INDEX Cc						
			Old dike	Old dike				

REMARKS:

UU: Unconsolidated – undrained shear test
CU: Consolidated – undrained shear test
CD: Consolidated – drained shear test
Vinculums (‾) shall be marked above the signs like $\overline{UU}$, $\overline{CU}$ and $\overline{CD}$ when pore pressure measured

Table 2-2-1 Result of Shear Stress Vane Test (τ Kg / cm²)

Location	-12 Km L 1		-12 Km L 2		-12 Km L 3		-12 Km L 8		-12 Km R 1		-12 Km R6	
Depth m \ Size of Vane	Small 5X10	Large 7.5X15	Small 5X10	Large 7.5X15	Small 5X10	Large 7.5X15	Small 5X10	Large 7.5X15	Small 5X10	Large 7.5X15	Small 5X10	Large 7.5X15
0.0							dike					
0.5	0.11 (0.09)	0.16 (0.15)	0.04 (0.03)	0.08 (0.08)	0.14 (0.12)	0.10 (0.10)	0.26 (0.25)		0.44 (0.42)	0.23 (0.22)	0.09 (0.08)	0.10 (0.09)
1.0	0.24 (0.21)	0.16 (0.15)	0.07 (0.05)	0.08 (0.07)	0.35 (0.33)	0.06 (0.06)			0.18 (0.15)	0.19 (0.19)	0.13 (0.11)	0.23 (0.22)
1.5	0.26 (0.22)		0.16 (0.13)		0.27 (0.24)				0.10 (0.06)	0.12 (0.11)	0.17 (0.14)	
2.0	0.16 (0.12)	0.17 (0.16)	0.16 (0.13)	0.16 (0.15)	0.19 (0.15)	0.12 (0.11)			0.15 (0.10)	0.23 (0.22)	0.13 (0.09)	0.10 (0.09)
2.5	0.13 (0.08)	0.13 (0.11)	0.16 (0.11)	0.13 (0.12)	0.15 (0.11)	0.14 (0.13)			0.44 (0.37)	0.27 (0.26)	0.14 (0.08)	0.12 (0.10)
3.0	0.13 (0.07)	0.11 (0.09)	0.17 (0.12)		0.09 (0.05)				0.22 (0.14)		0.13 (0.06)	
3.5	0.12 (0.05)	0.16 (0.14)	0.17 (0.12)	0.13 (0.11)	0.17 (0.12)	0.12 (0.10)			0.17 (0.08)		0.13 (0.05)	0.13 (0.11)
4.0	0.20 (0.12)	0.16 (0.14)	0.14 (0.08)	0.13 (0.11)	0.20 (0.14)	0.14 (0.12)			0.13 (0.02)		0.25 (0.16)	0.12 (0.09)
4.5	0.14 (0.06)		0.16 (0.09)		0.25 (0.19)				0.13 (0.01)		0.19 (0.08)	
5.0	0.20 (0.11)	0.16 (0.14)	0.26 (0.20)	0.16 (0.14)	0.20 (0.14)						0.16 (0.05)	0.14 (0.10)

( ): - Friction of Rods

Table 2-2-2 Result of Shear stress Vane Test (τ Kg / cm²)

Location	-12 Km R7		-11.25 Km R4'		-11.25 Km R6		-11 Km L4		-11 Km L5		-10Km L1-1	-10Km L2-1
Depth m \ Size of Vane	Small 5 X 10	Large 7.5 X 15	Small 5 X 10	Large 7.5 X 15	Small 5 X 10	Large 7.5 X 15	Small 5 X 10	Large 7.5 X 15	Small 5 X 10	Large 7.5 X 15	Small 5 X 10	Smal 5 X 10
0.5	0.10 (0.08)	0.10 (0.10)	0.13	0.09	0.13	0.08	0.12	0.05	0.18	0.08	0.02	0.11
1.0		0.13 (0.12)	0.13		0.15		0.05		0.20		0.27	0.14
				0.10		0.13		0.27		0.12		
1.5	0.28 (0.21)	0.23 (0.21)	0.36		0.34		0.14		0.09			
2.0	0.10 (0.02)	0.13 (0.11)	0.44	0.29<	0.44	0.29<	0.13	0.16	0.19	0.29<		
2.5	0.16 (0.06)	0.13 (0.10)	0.19		0.34		0.36		0.29			
								0.29<				
3.0	0.17 (0.07)	0.10 (0.06)	0.21		0.32		0.24		0.28			
3.5	0.14 (0.03)	0.10 (0.07)	0.19		0.23				0.31			
4.0	0.17 (0.05)	0.13 (0.09)	0.19		0.21							
4.5	0.19 (0.06)	0.16 (0.13)	0.22		0.26							
5.0	0.13	0.16 (0.12)			0.54							

( ) : - Friction of Rods

Table 2-2-3 Result of Shear Stress Vane Test (τ Kg / cm²)

Location	-10KmL 2-2	-10Km L3	-10Km L4	-10Km L5	-10KmR4	-10KmR5	- 9 Km R 4		- 9 Km R 9		Boring site NO. 3	
Size of Vane / Depth m	Small 5X10	Small 5X10	Small 5X10	Small 5X10	Large 7.5X15	Small 5X10	Small 5X10	Large 7.5X15	Small 5X10	Large 7.5X15	Small 5X10	
0.0												
0.5	0.09	0.08	0.12	0.12	0.09	0.20	0.25	0.23	0.19	0.22	0.28	
1.0	0.34	0.09	0.13	0.12	0.08	0.13	0.26		0.34	0.17	0.24	
								0.25				
1.5				0.09	0.08	0.09	0.15		0.16			
2.0				0.09	0.10	0.21	0.19	0.29 <	0.35			
2.5				0.16			0.26		0.28			
3.0												
3.5												
4.0												
4.5												
5.0												

Fig. I-1 Surveying Route and Elevation of Bench Mark

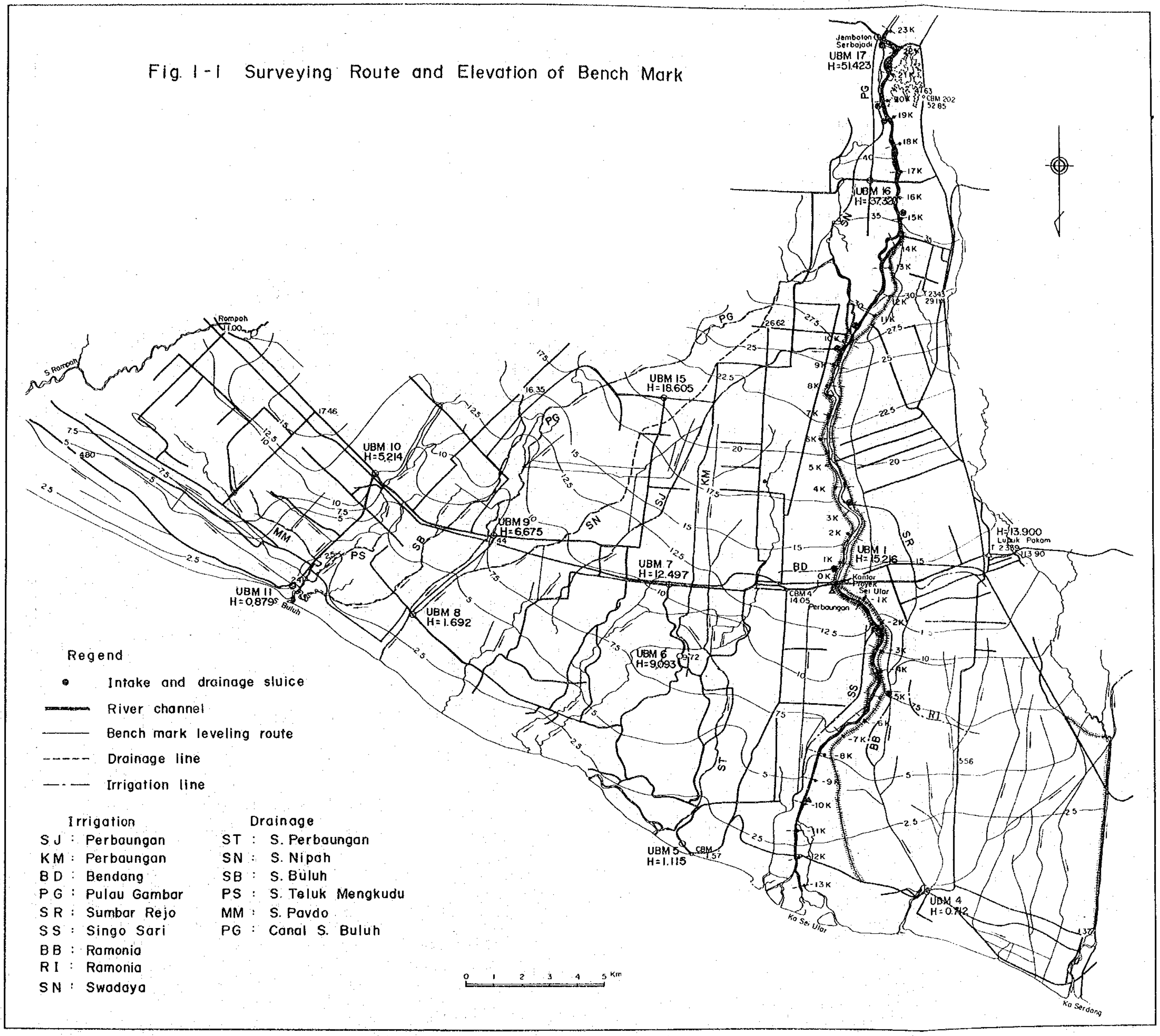
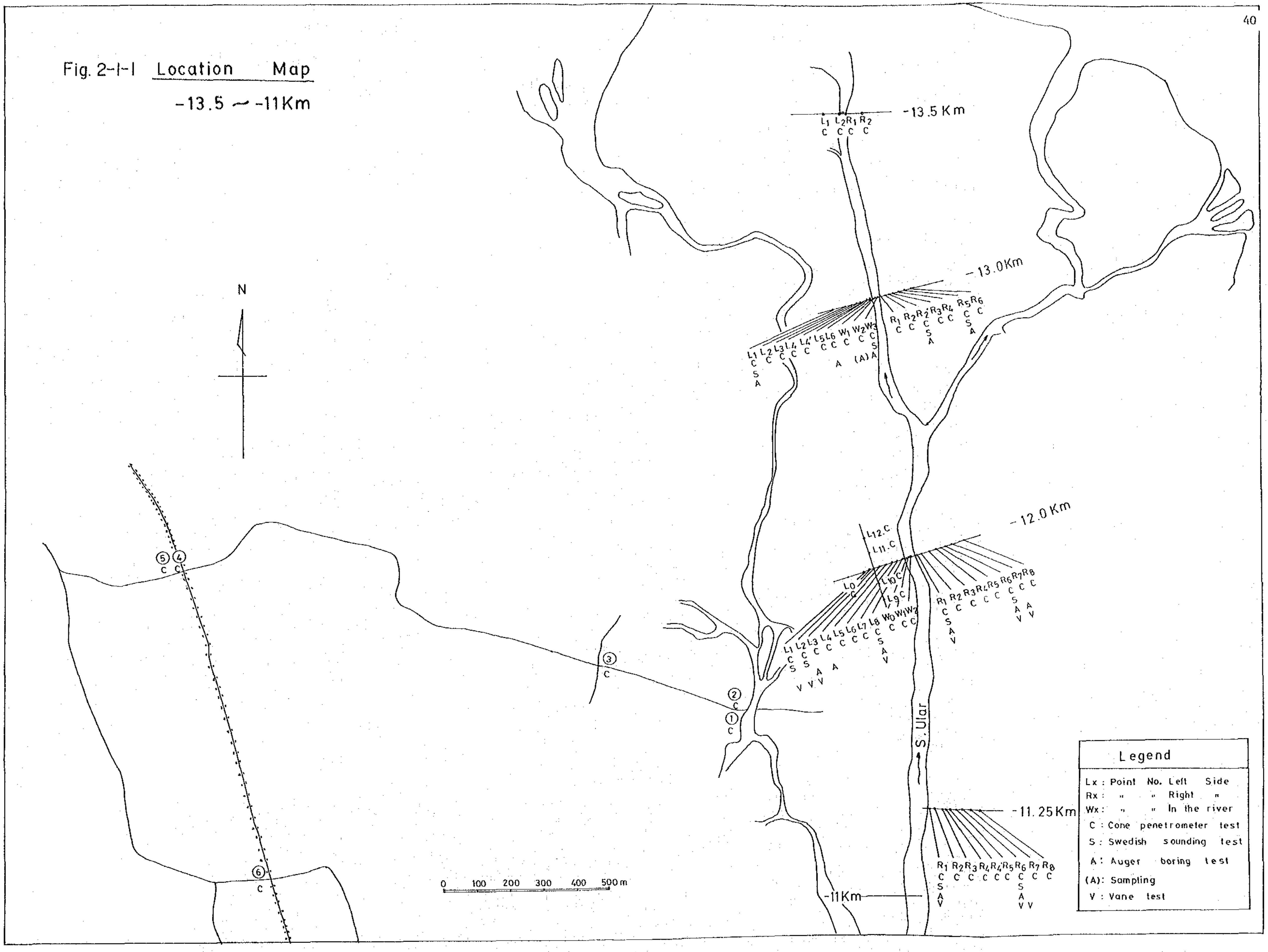


Fig. 2-1-1 Location Map
-13.5 ~ -11Km



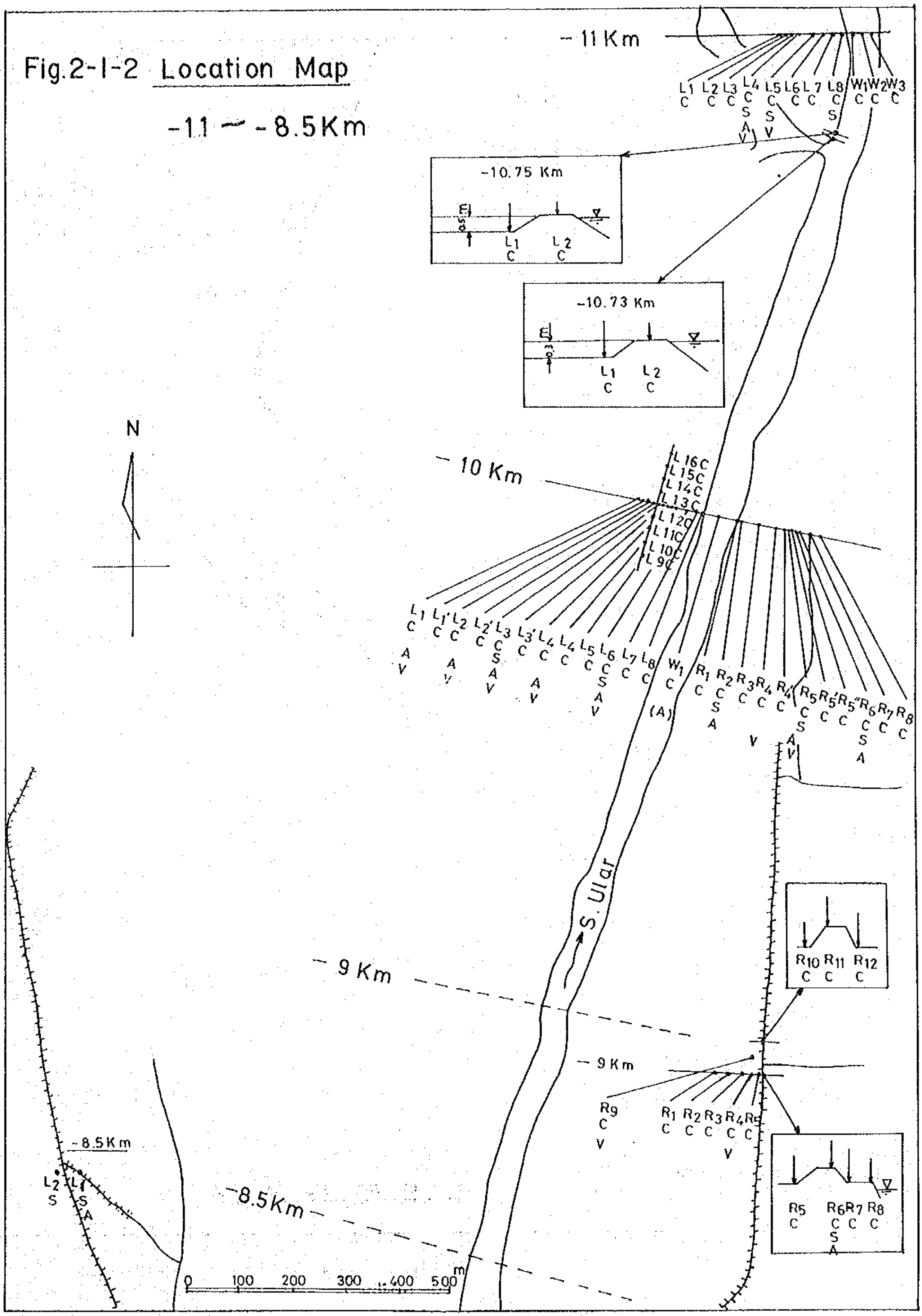


Fig.2-1-2 Location Map

-11 ~ -8.5Km

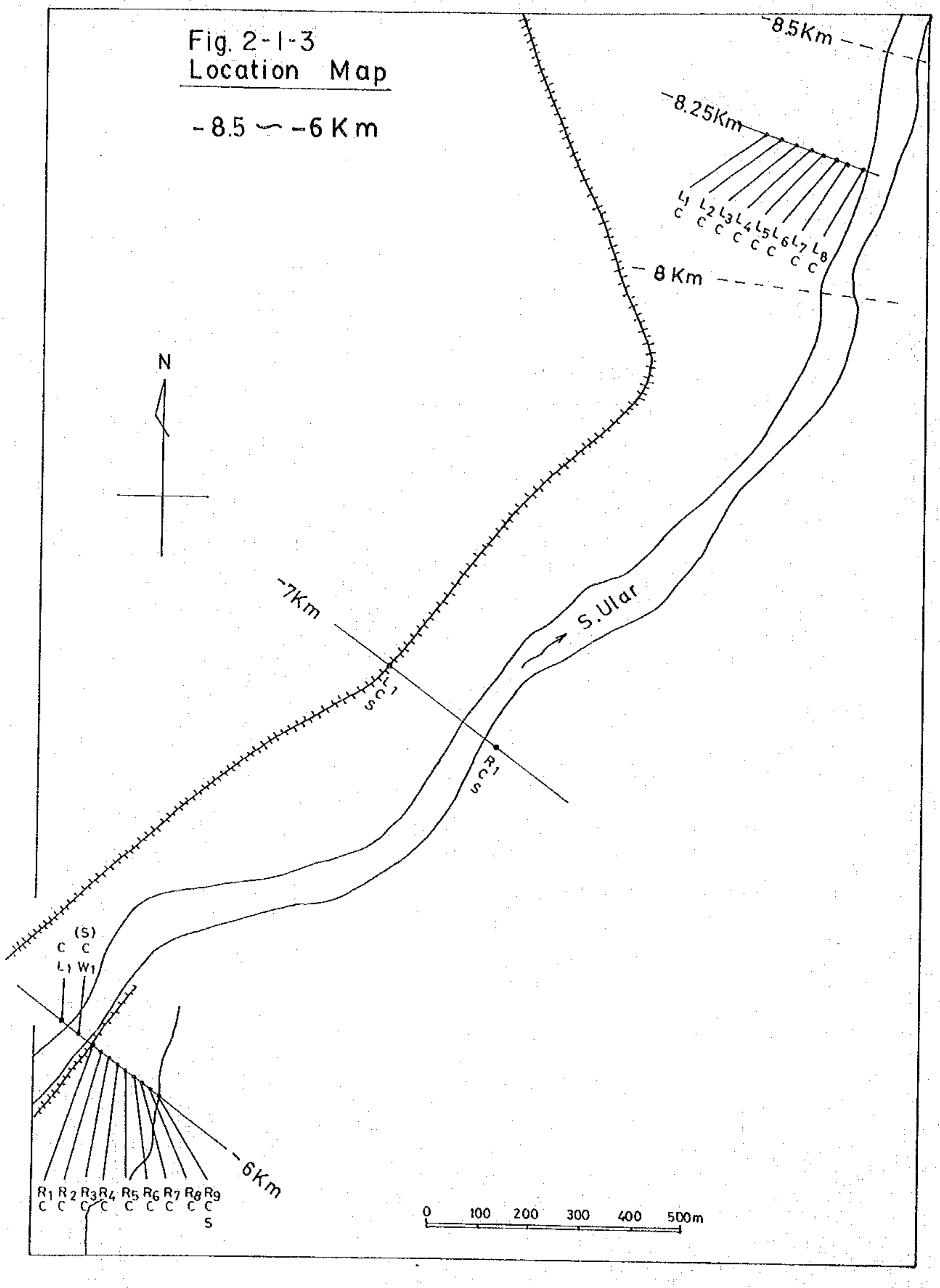


Fig. 2-1-3
Location Map
-8.5 ~ -6 Km
N
-8.5Km
-8.25Km
8 Km
-7Km
S. Ular
-6Km
0 100 200 300 400 500m

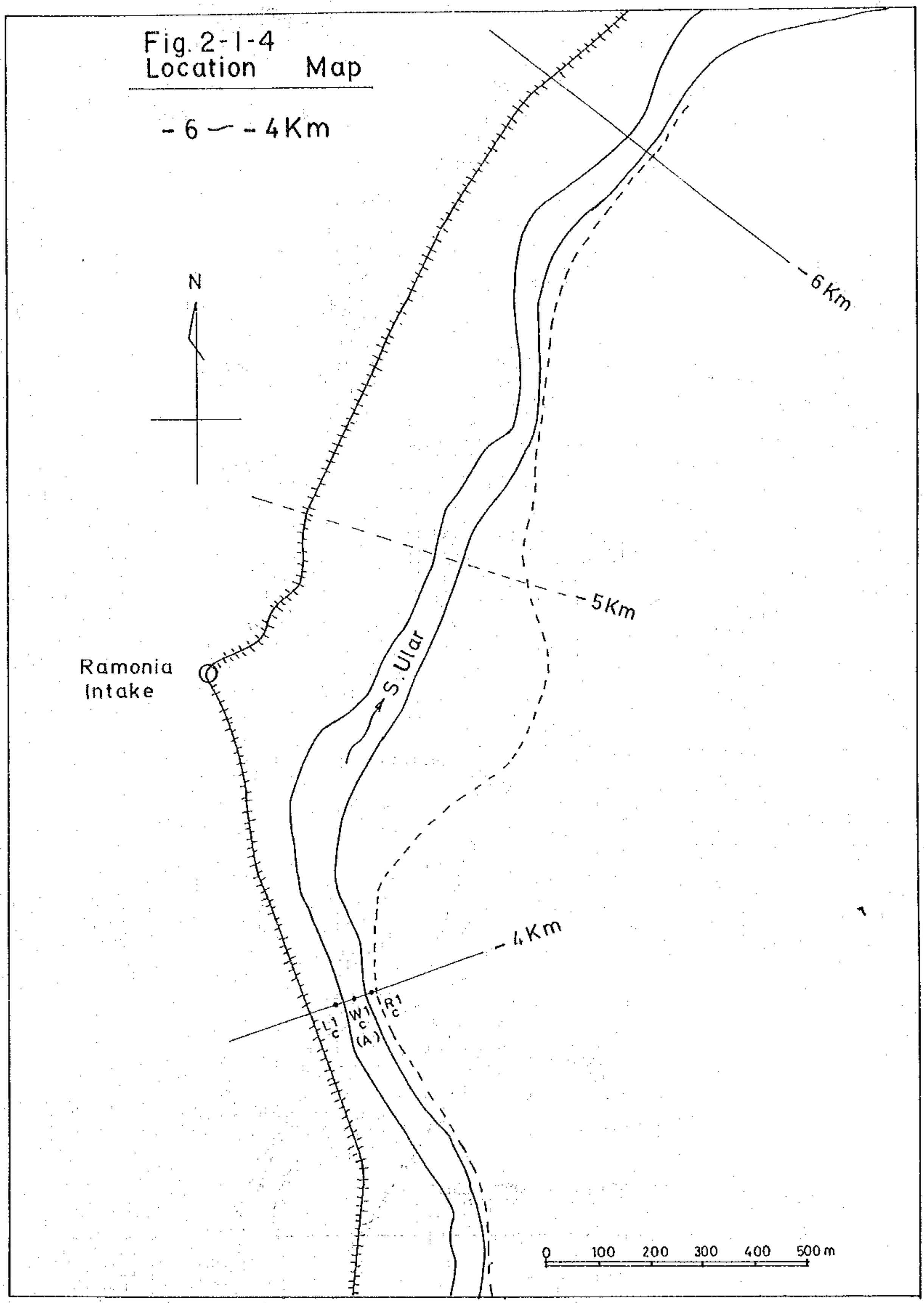


Fig. 2-1-4
Location Map
-6 ~ -4Km
N
-6Km
-5Km
-4Km
Ramonia Intake
S. Ular
L1c
W1c (A)
R1c
0 100 200 300 400 500m

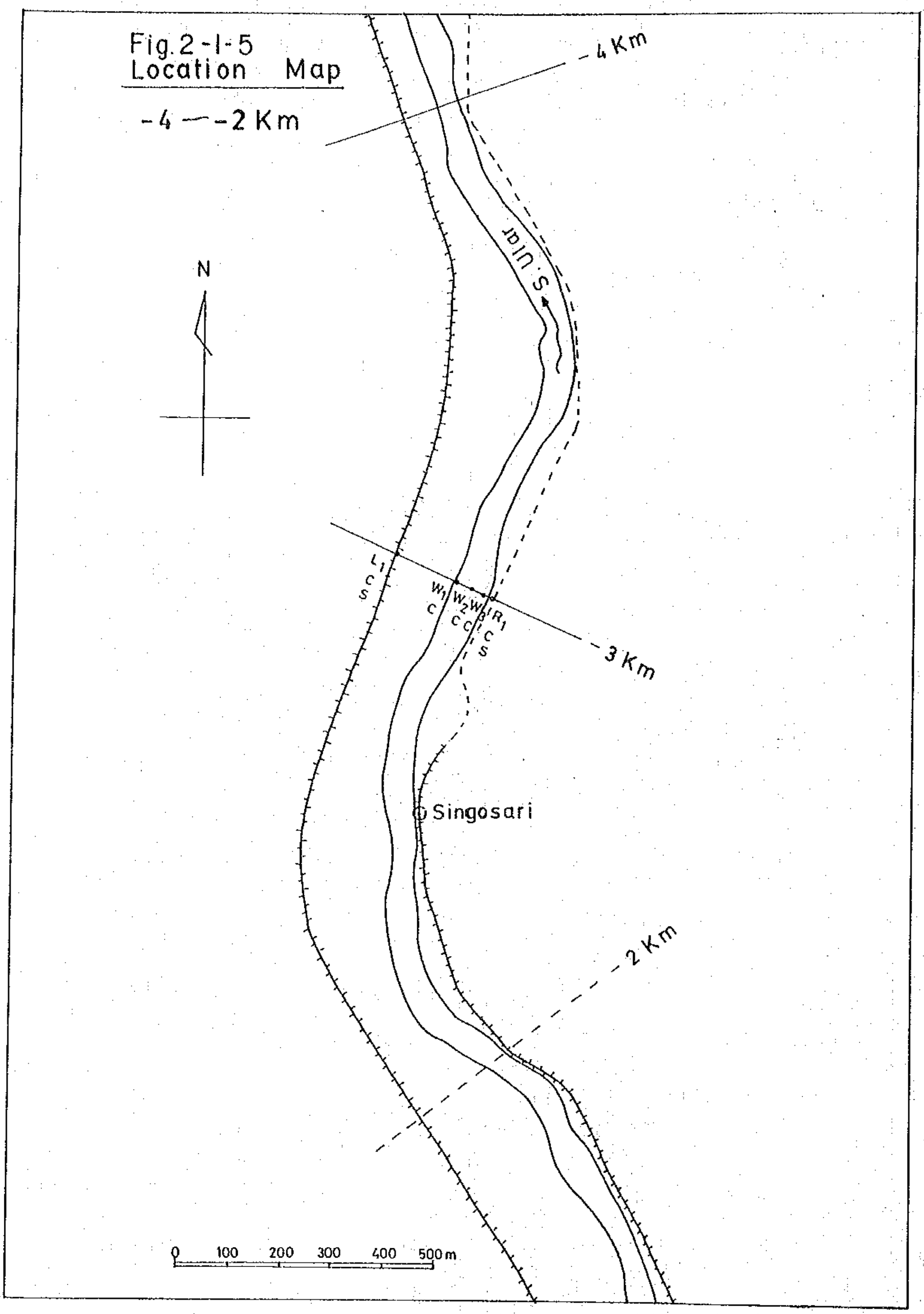


Fig. 2-1-5
Location Map
-4 — -2 Km
N
4 Km
S. Ular
L1
C
S
W1
C
W2
C
W3
C
R1
C
S
3 Km
Singosari
2 Km
0
100
200
300
400
500 m

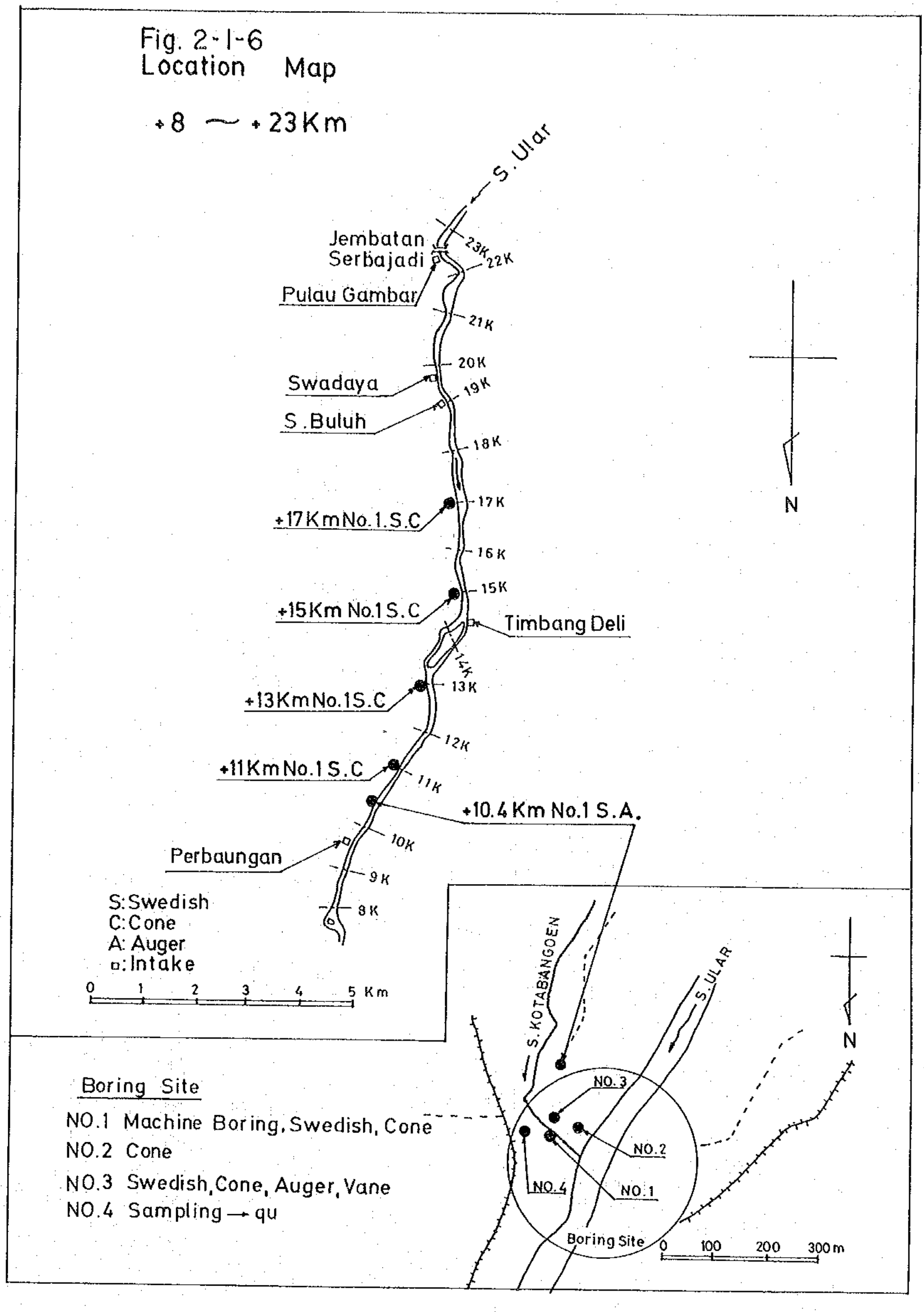


Fig. 2-1-6
Location Map
+8 ~ +23Km

Fig. 2-2-1
Location Map for Intake

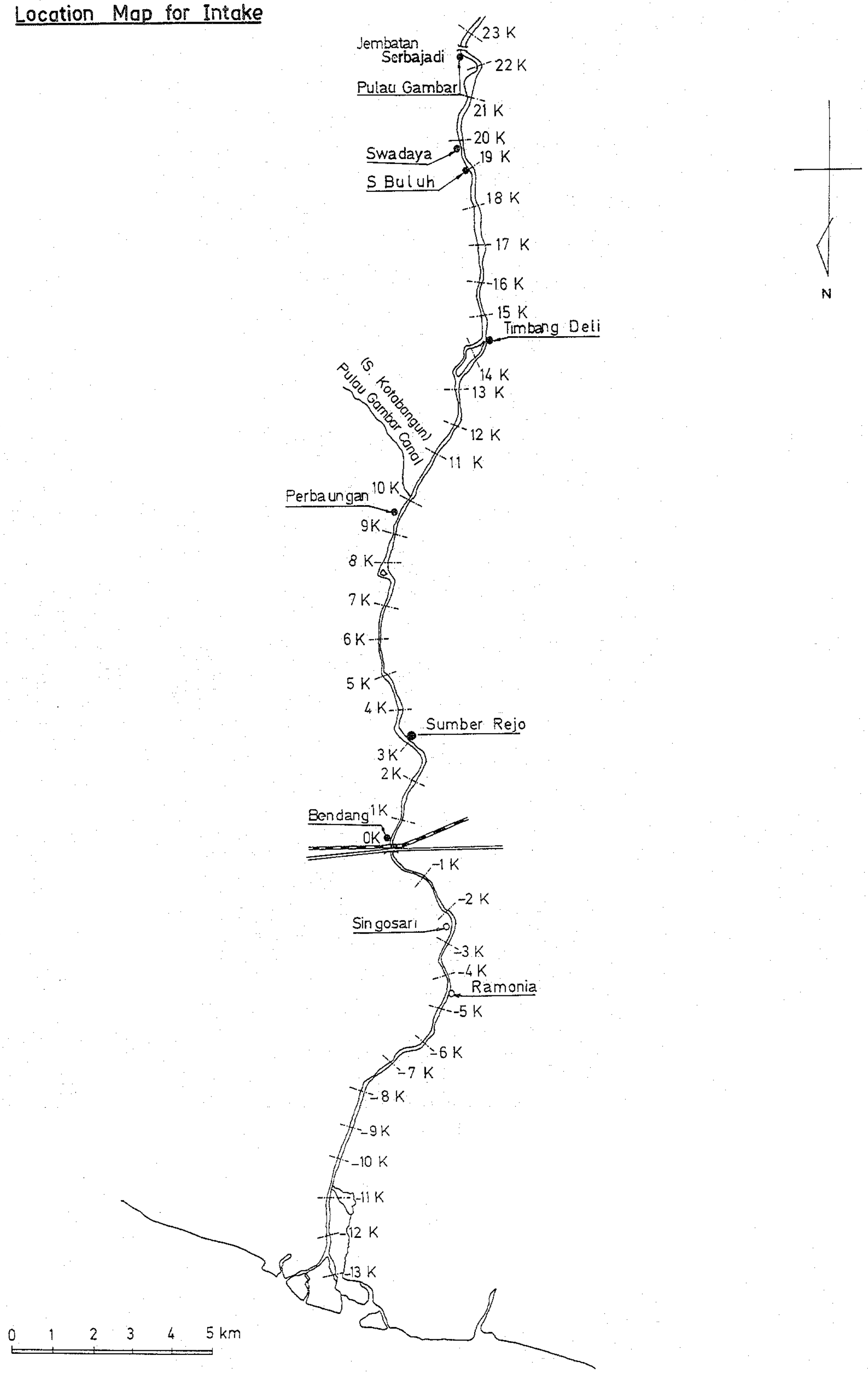


Fig. 2-2-2 Ramonia Intake

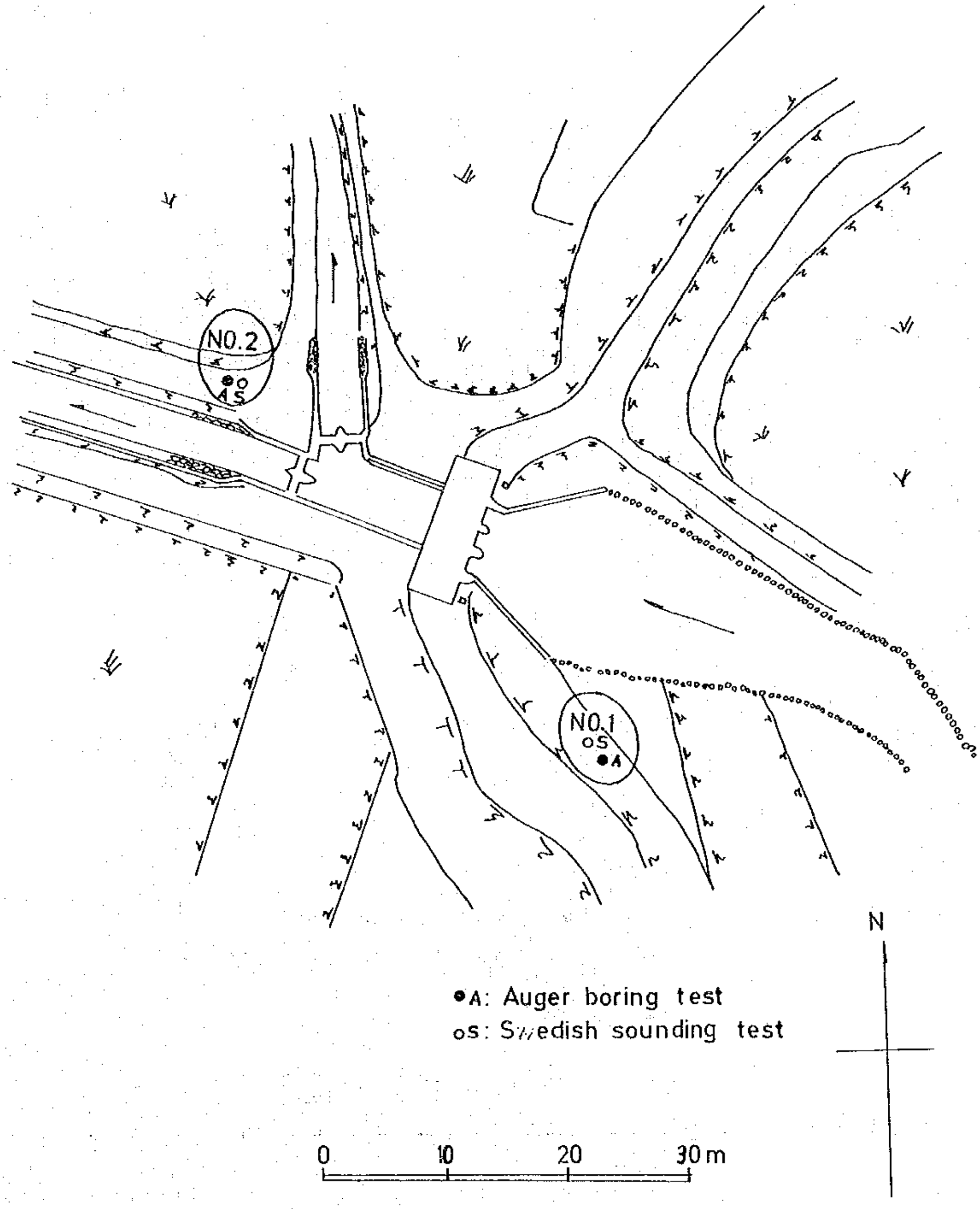


Fig. 2-2-3 Singosari Intake

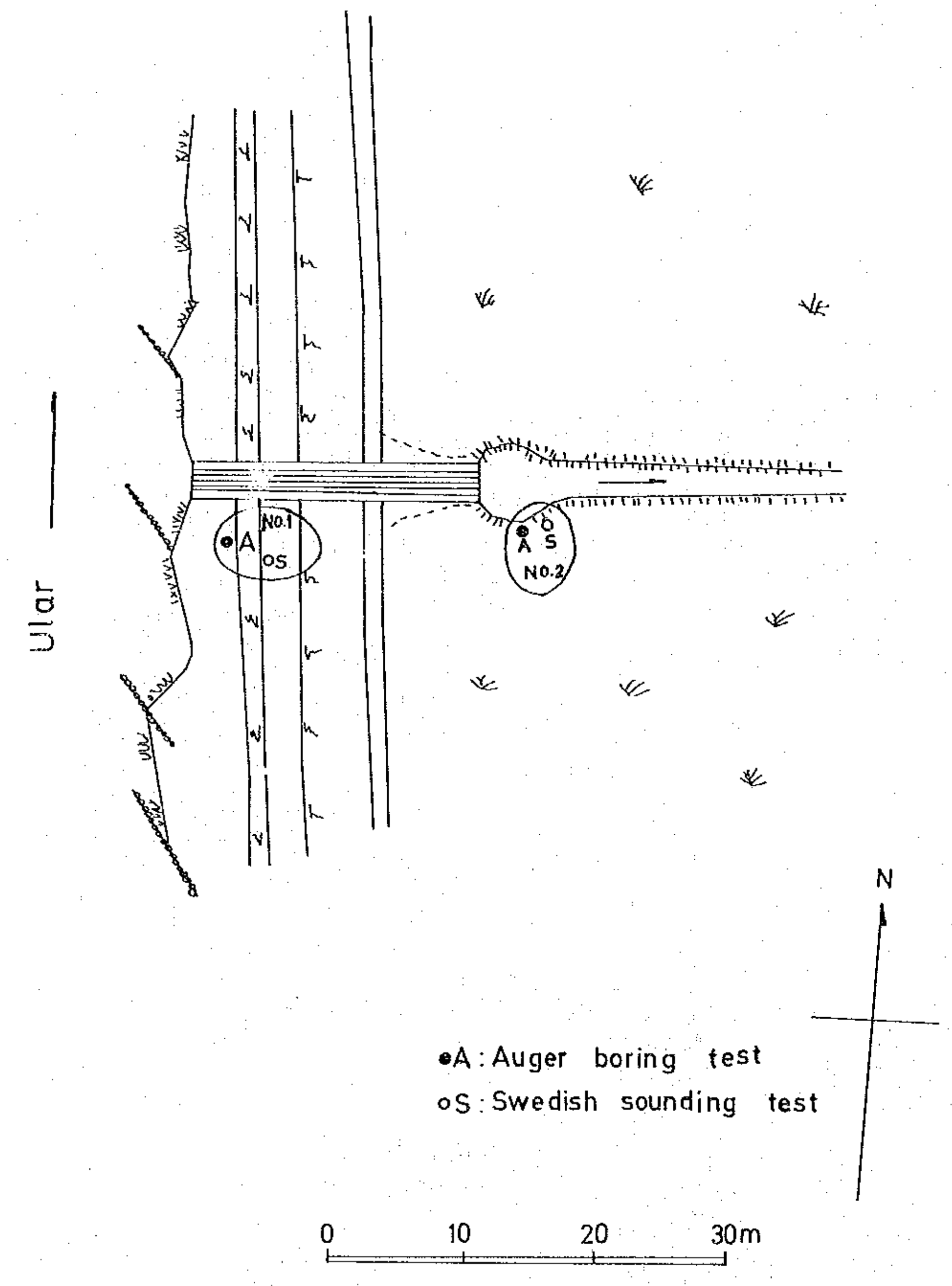


Fig. 2-2-4 Bendang Intake

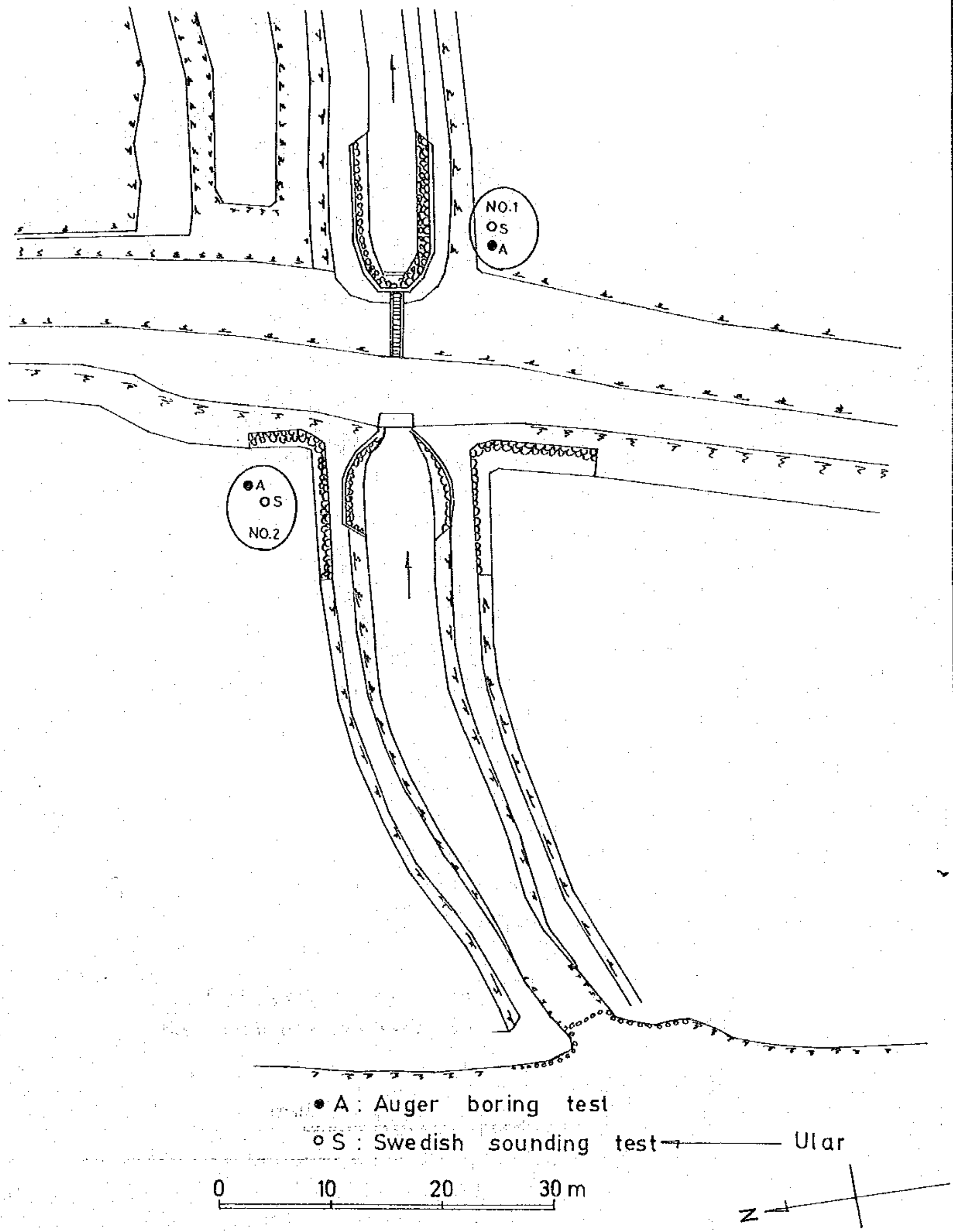


Fig. 2-2-5 Sumber Rejo Intake

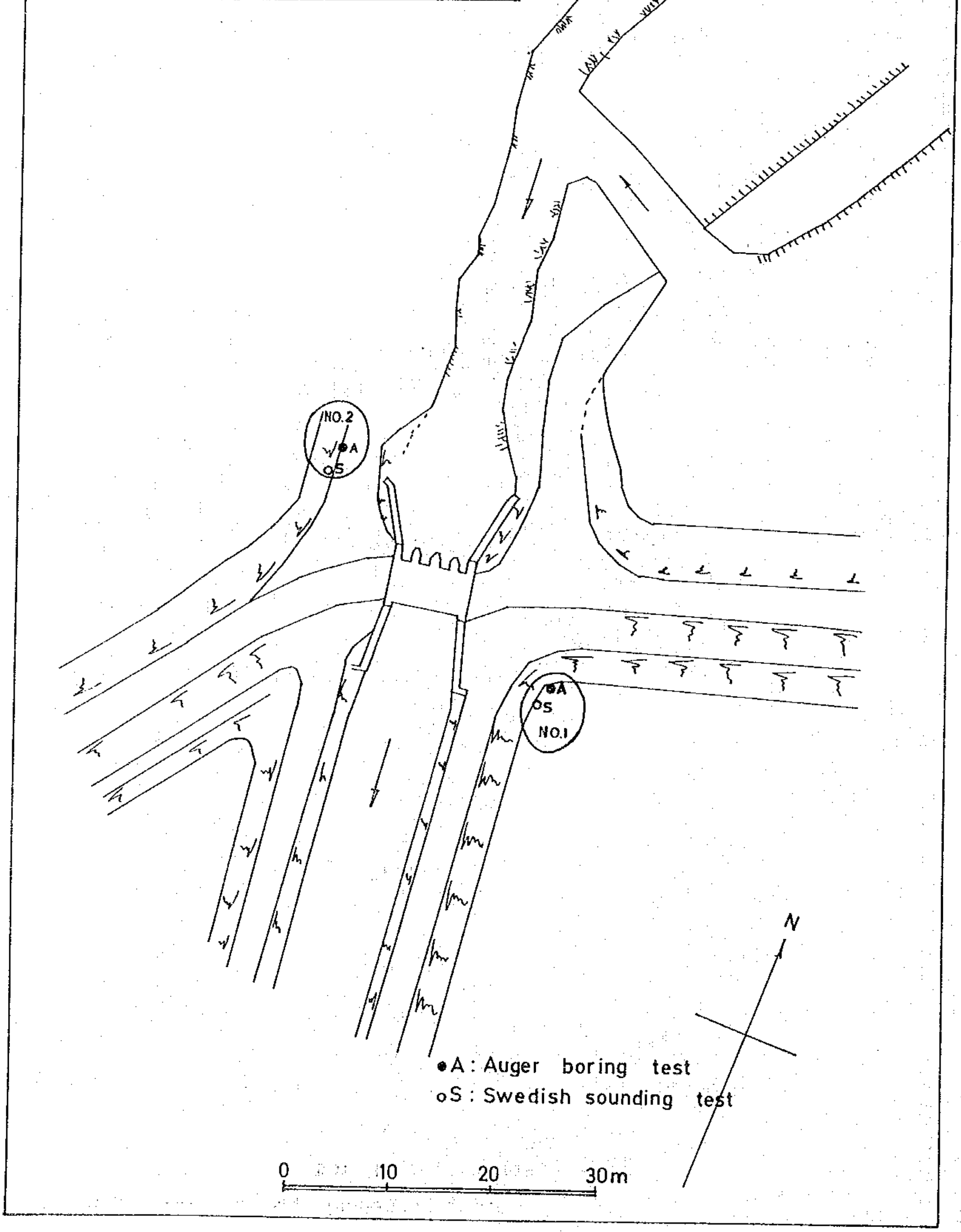


Fig. 2-2-6 Perbaungan Intake

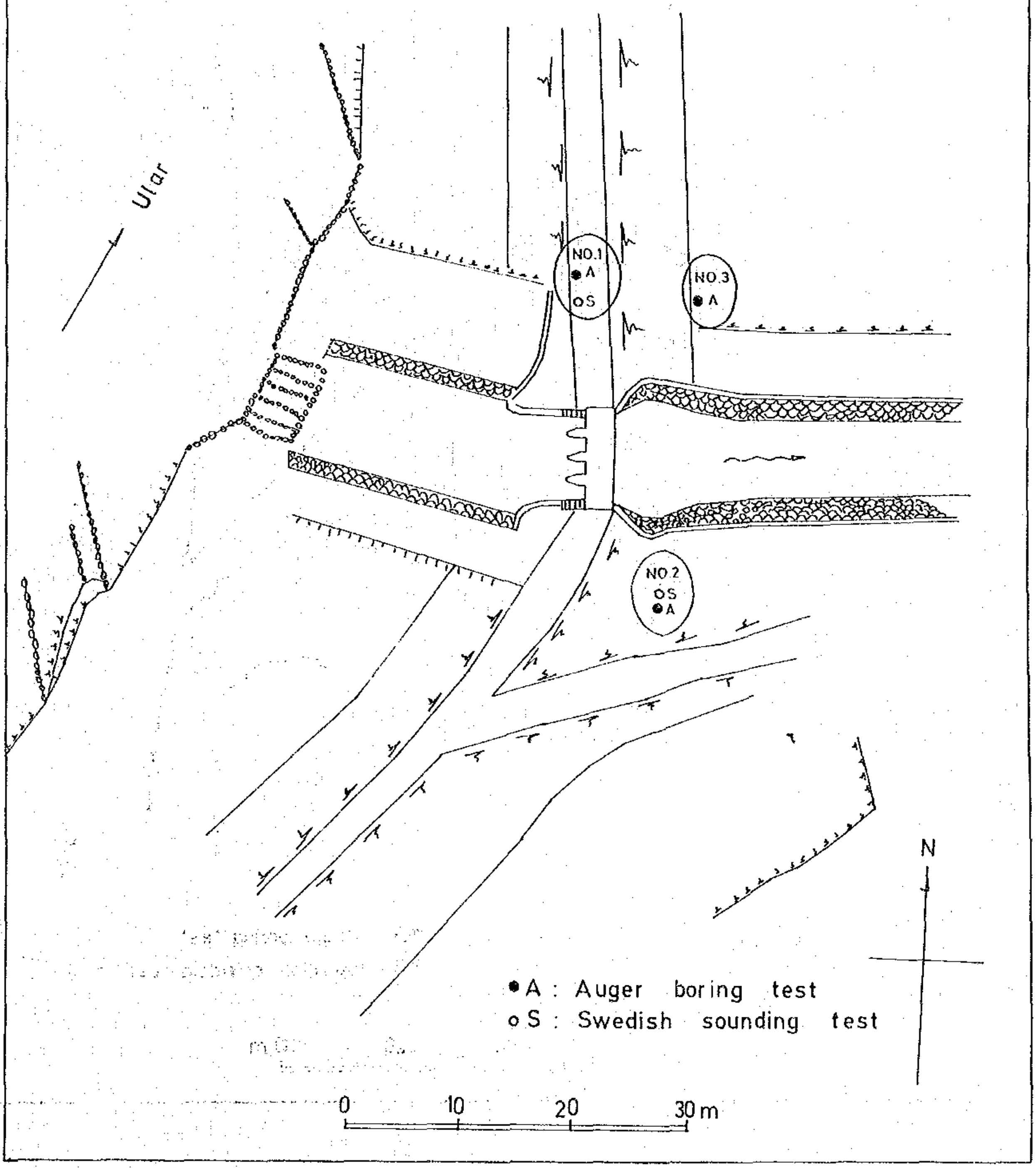


Fig. 2-2-7 Timbang Deli Intake

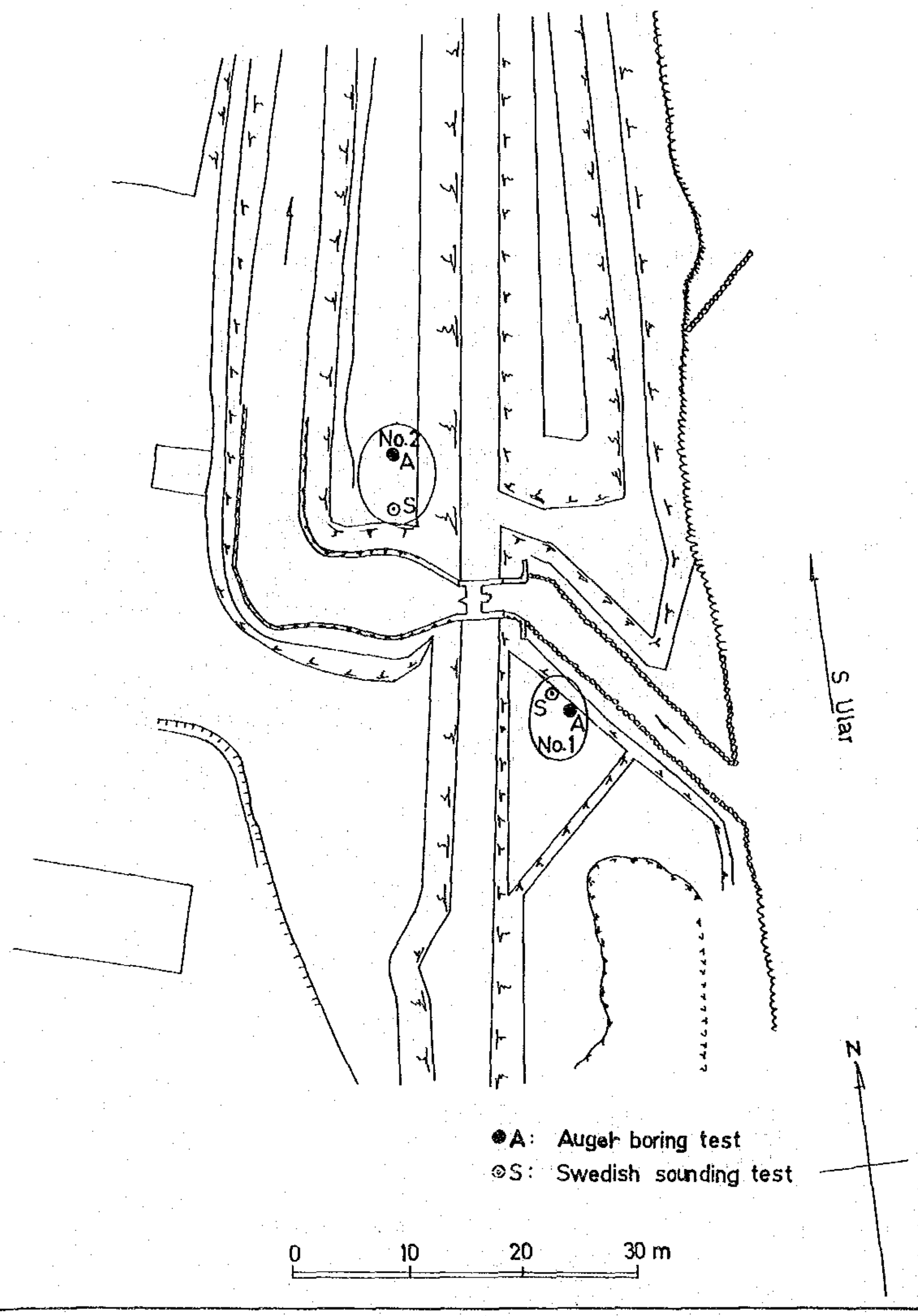


Fig. 2-2-8 Sei. Buluh Intake

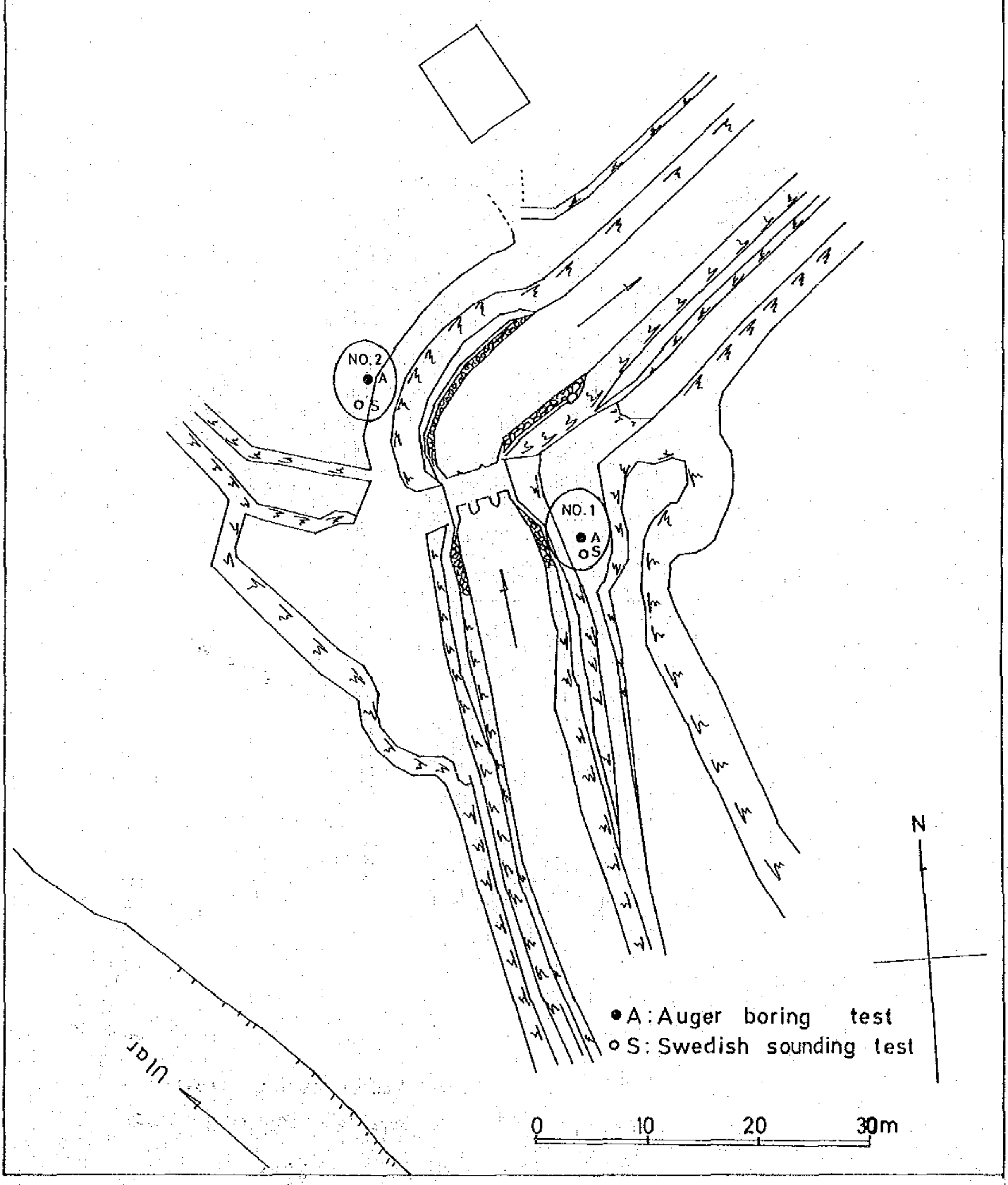


Fig. 2-2-9 Swadaya Intake

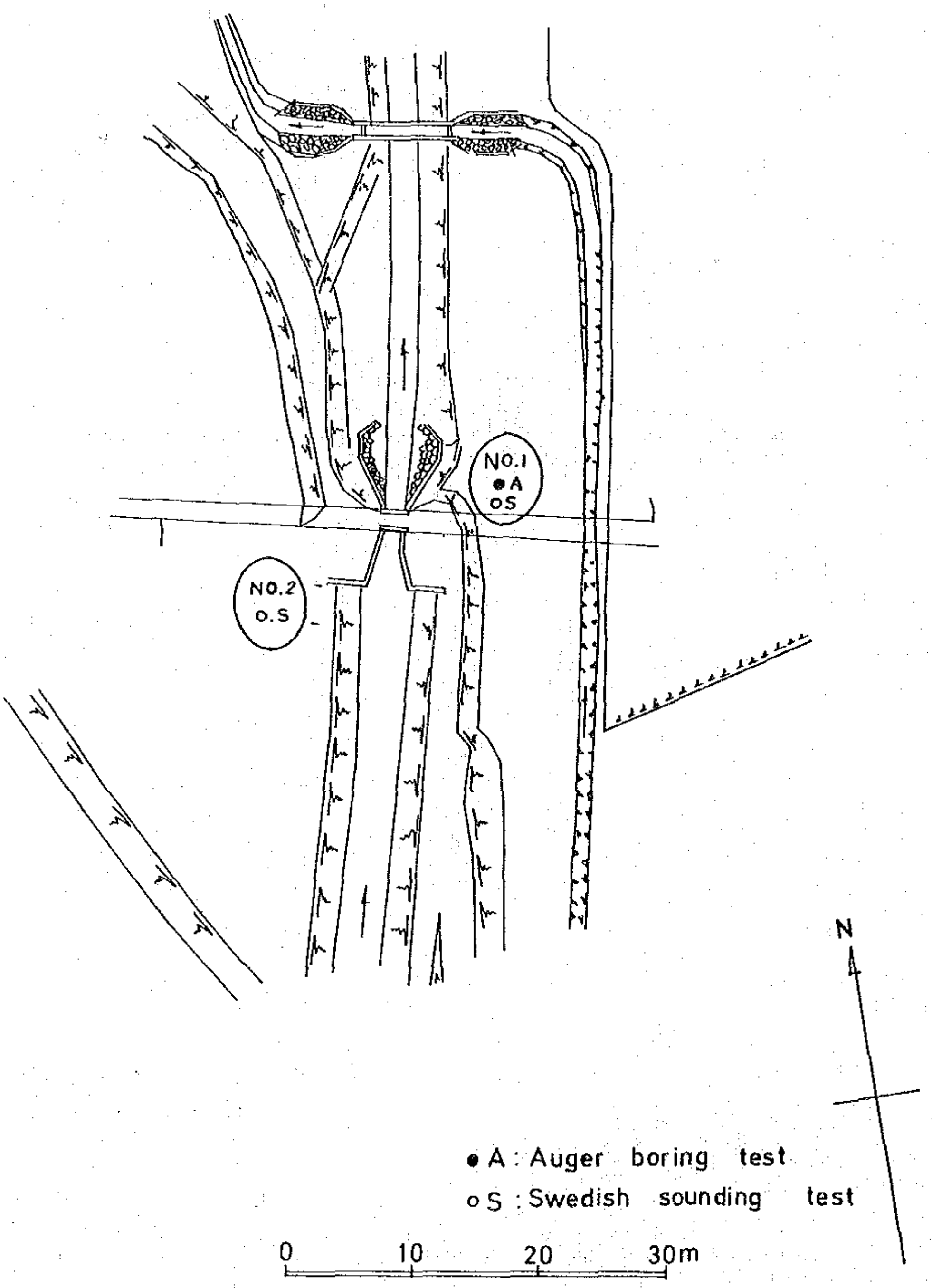


Fig. 2-2-10 Pulau Gambar Intake

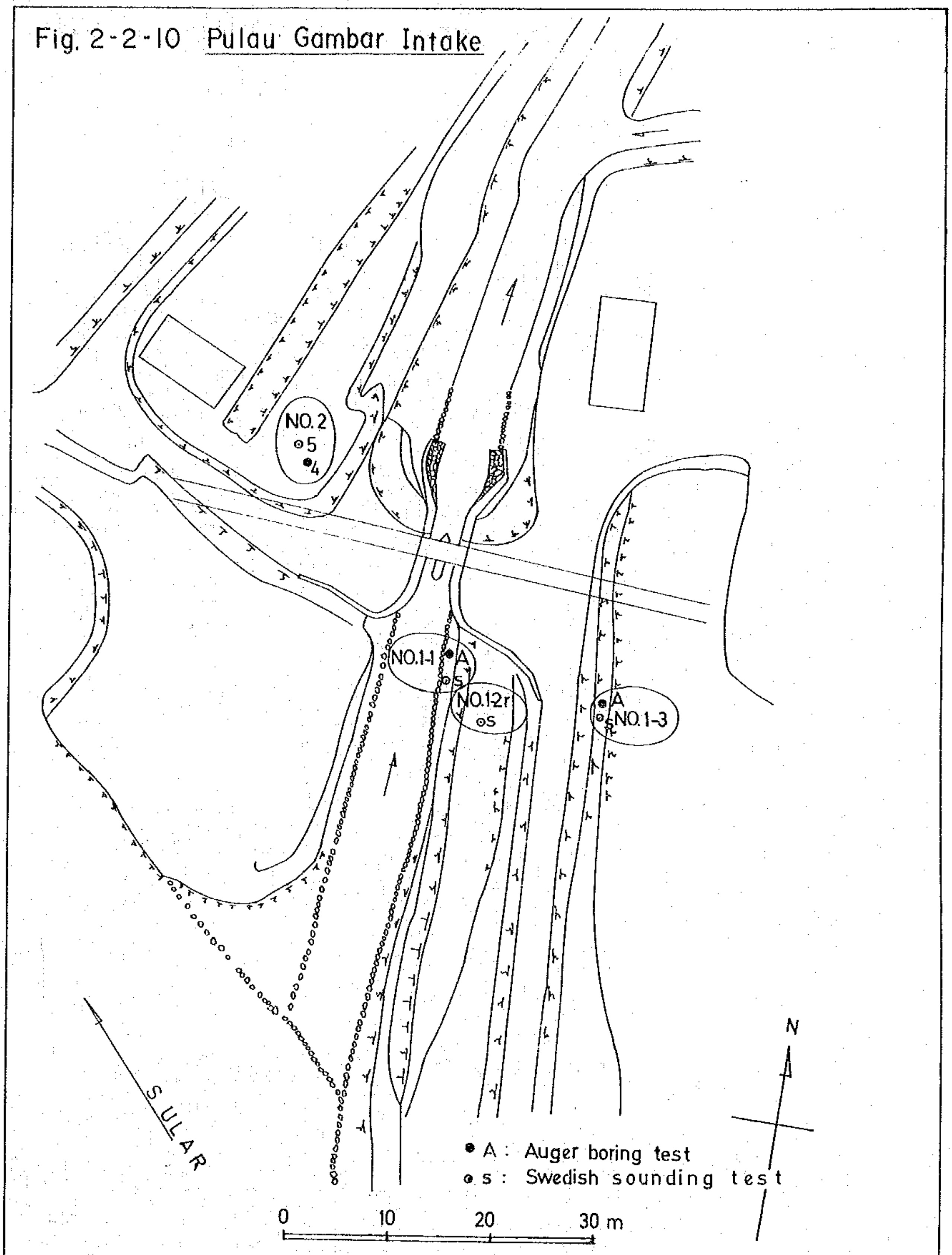


Fig. 2-2-11 Soil-survey Site at Confluence of Pulau Gambar Canal

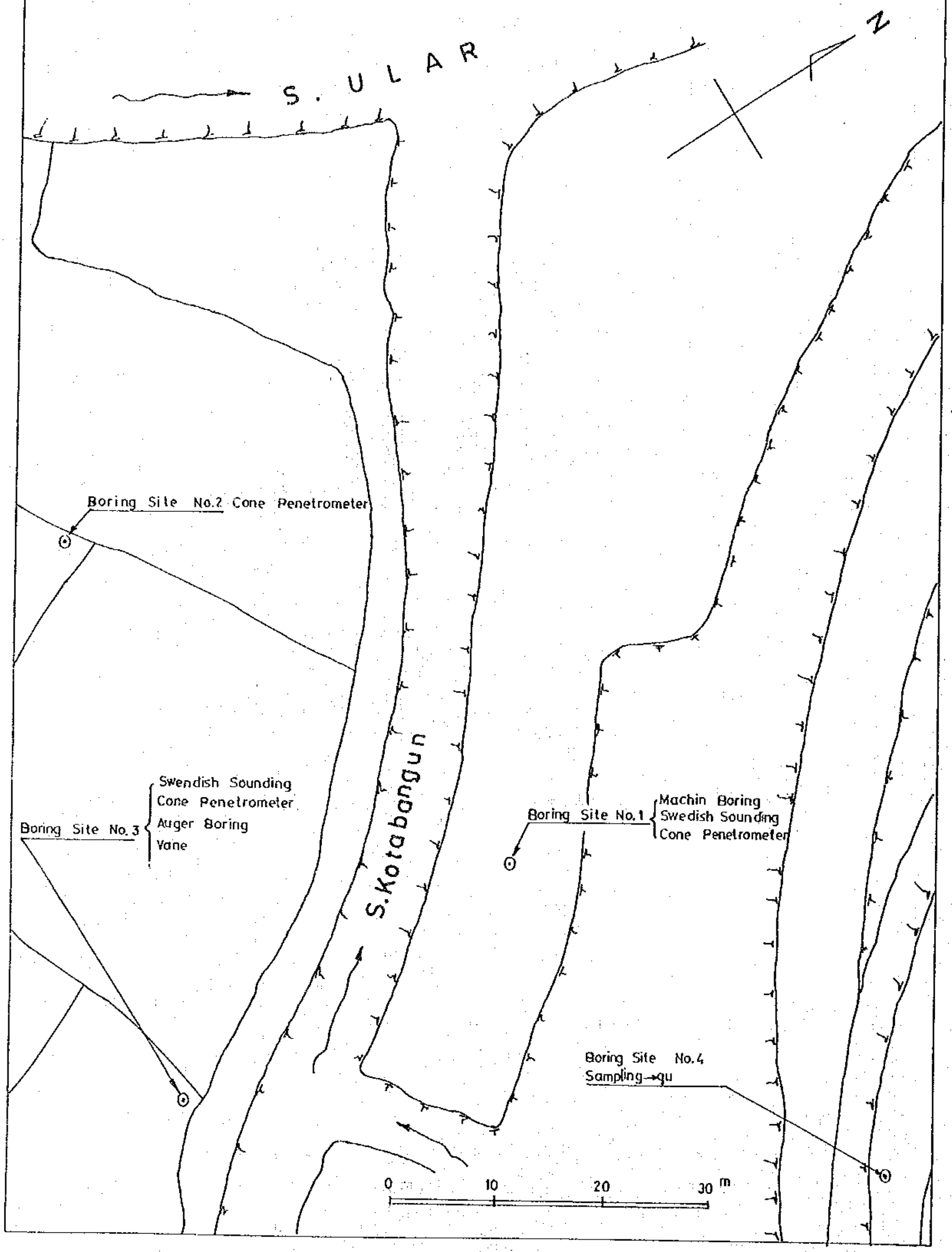


Fig. 2-3-1 LEGEND OF SOIL PROFILE

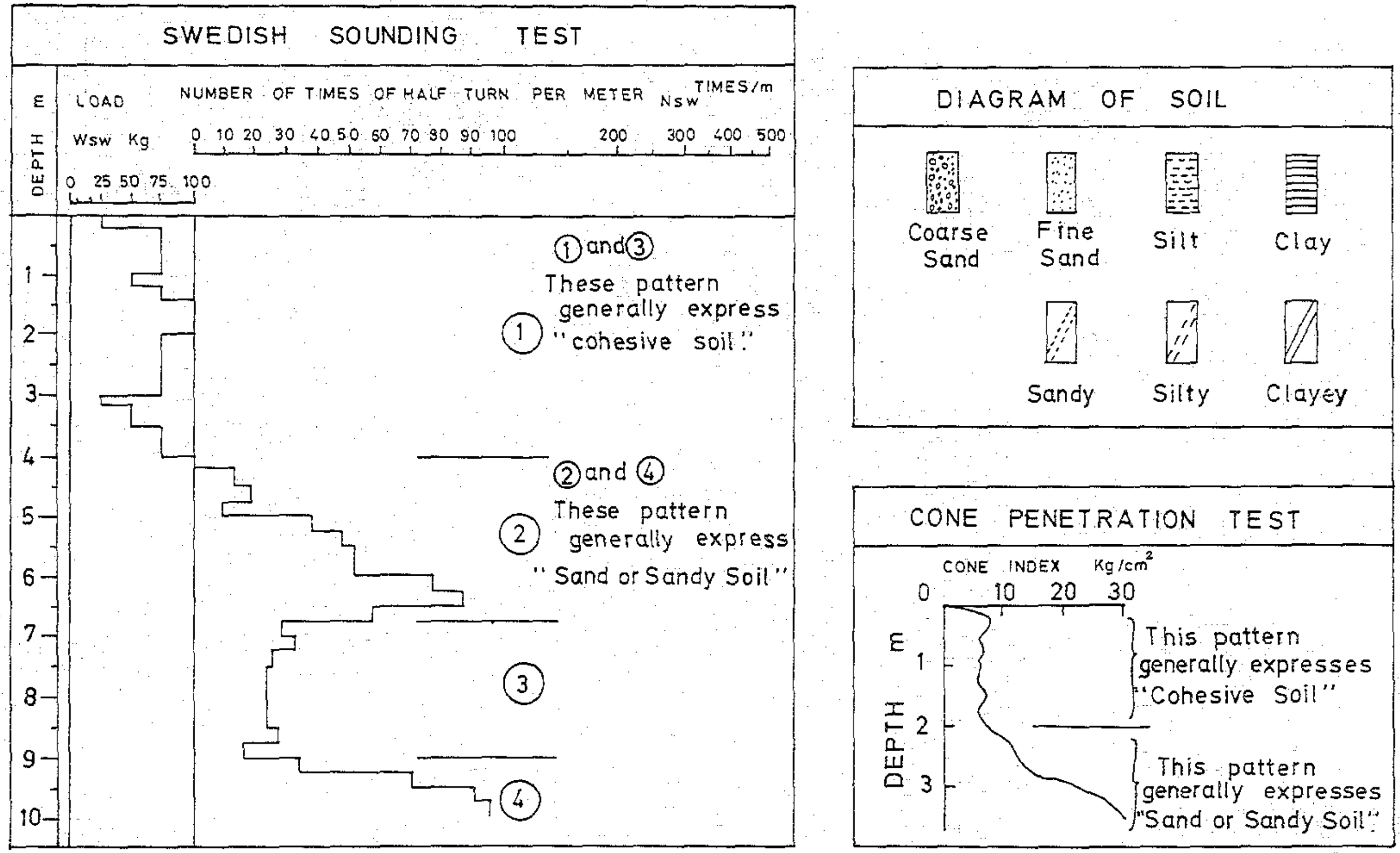


Fig. 2-3-2 Soil Profile

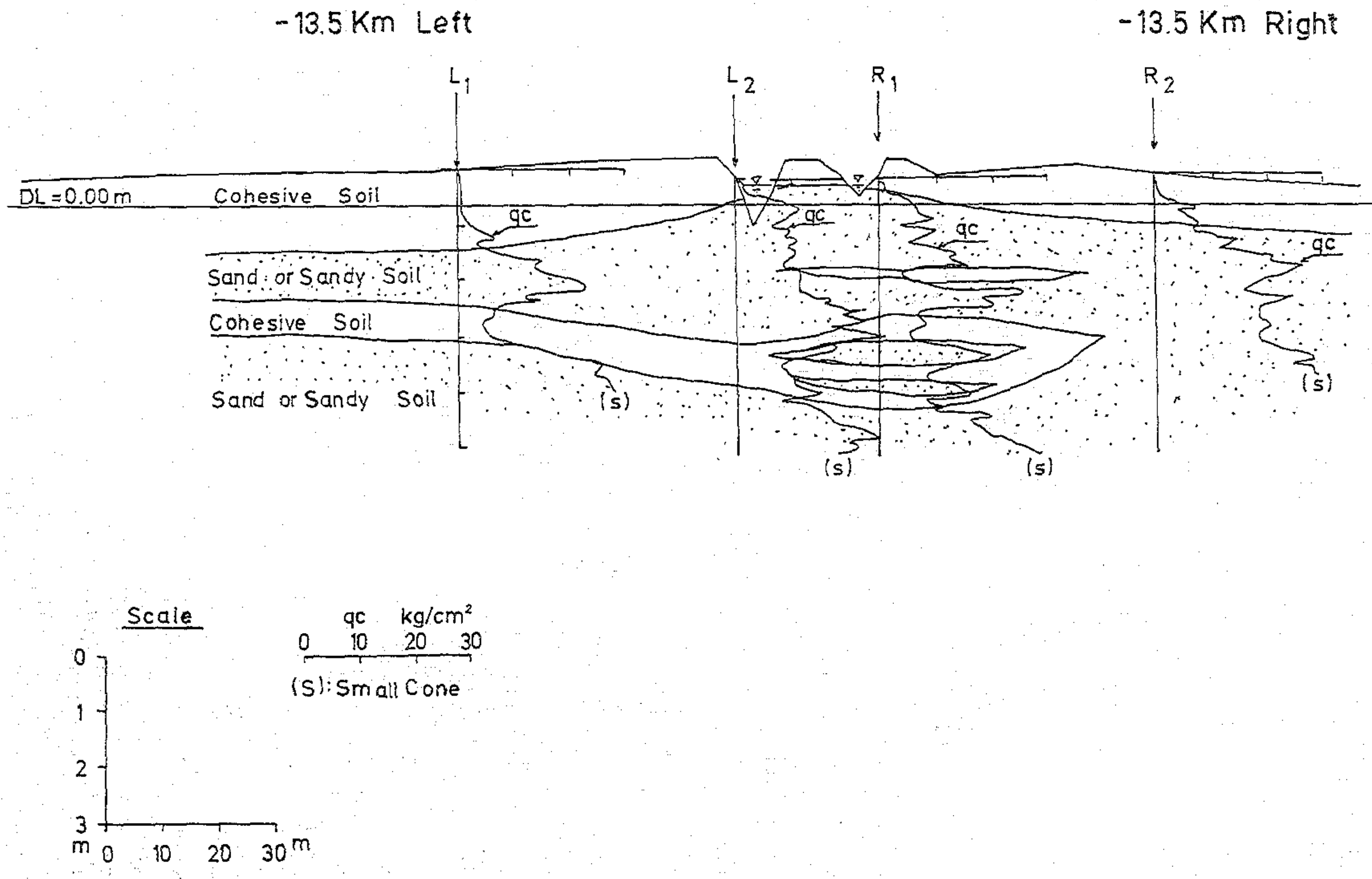
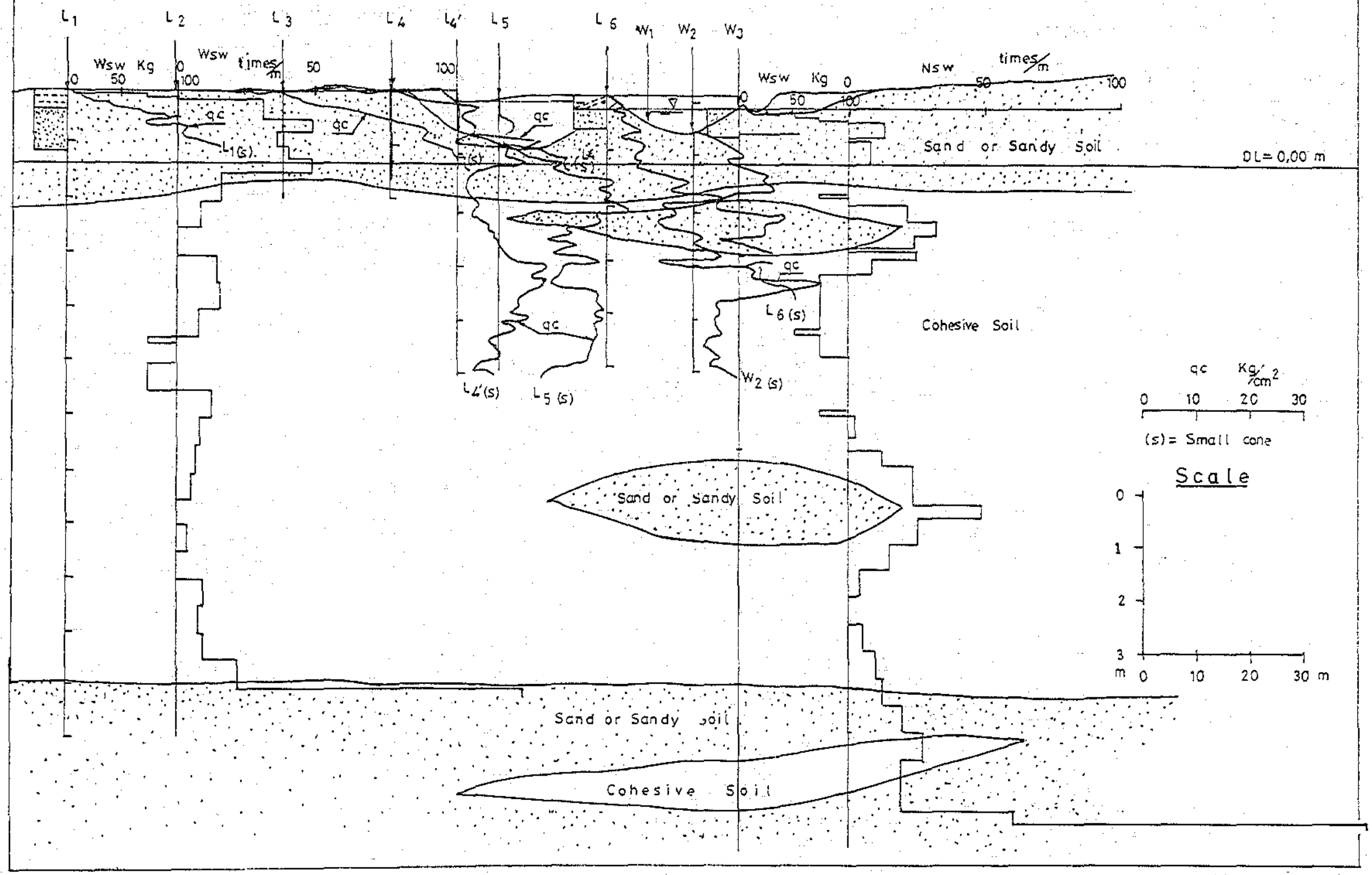


Fig. 2-3-3 Soil Profile

−13 Km Left



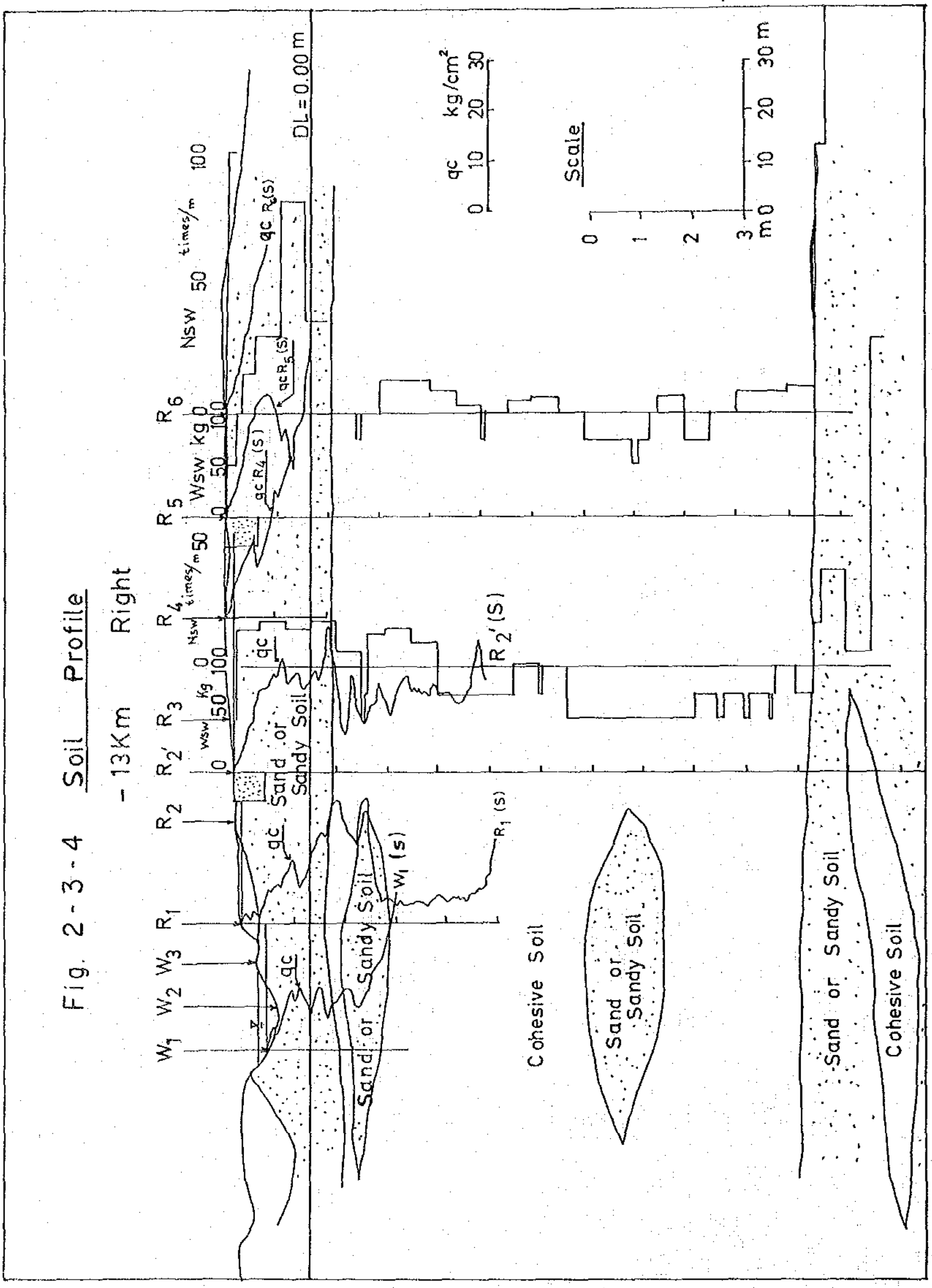
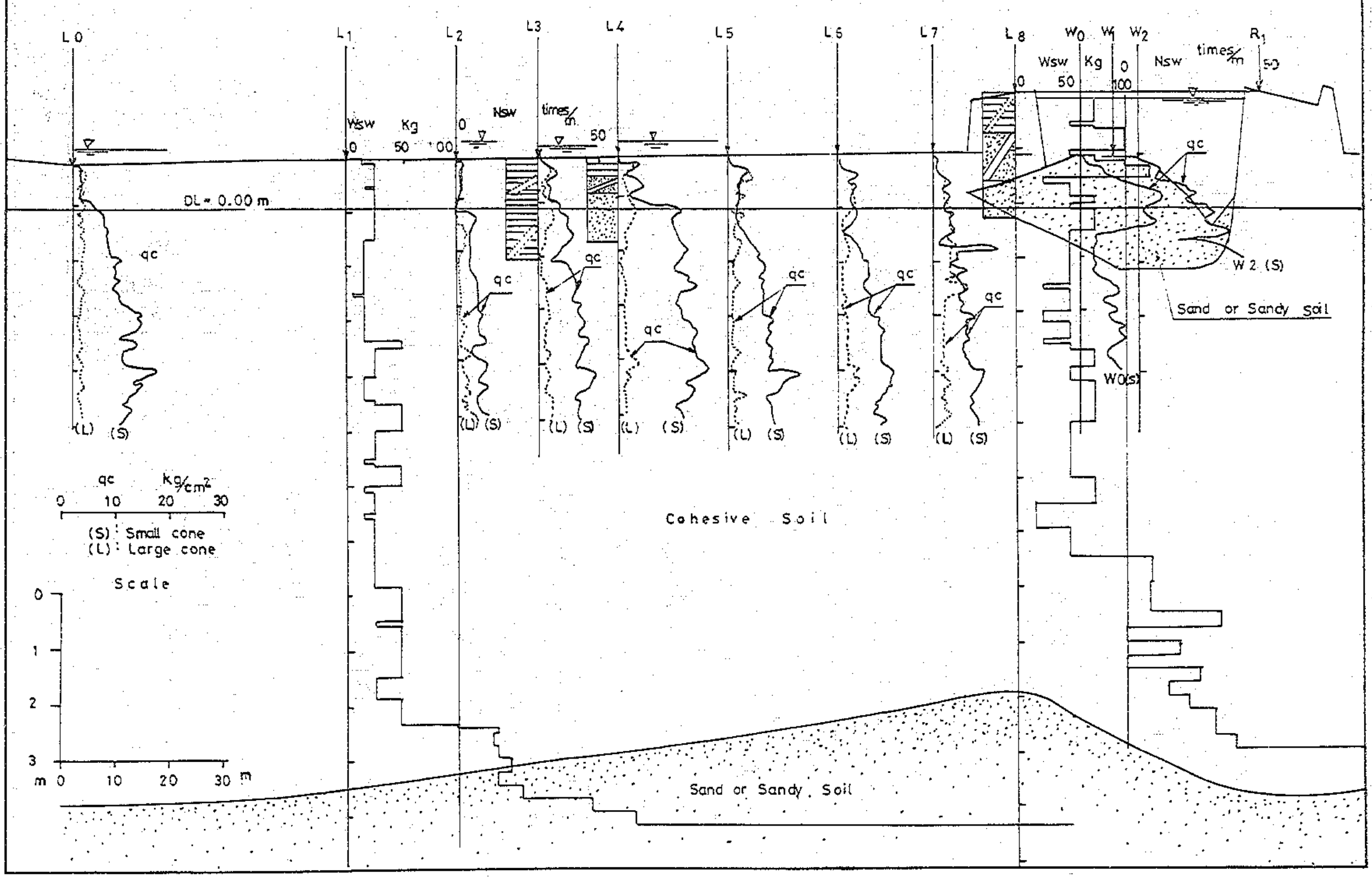


Fig. 2-3-4 Soil Profile
-13Km Right

Fig. 2-3-5 Soil Profile

-12 Km Left



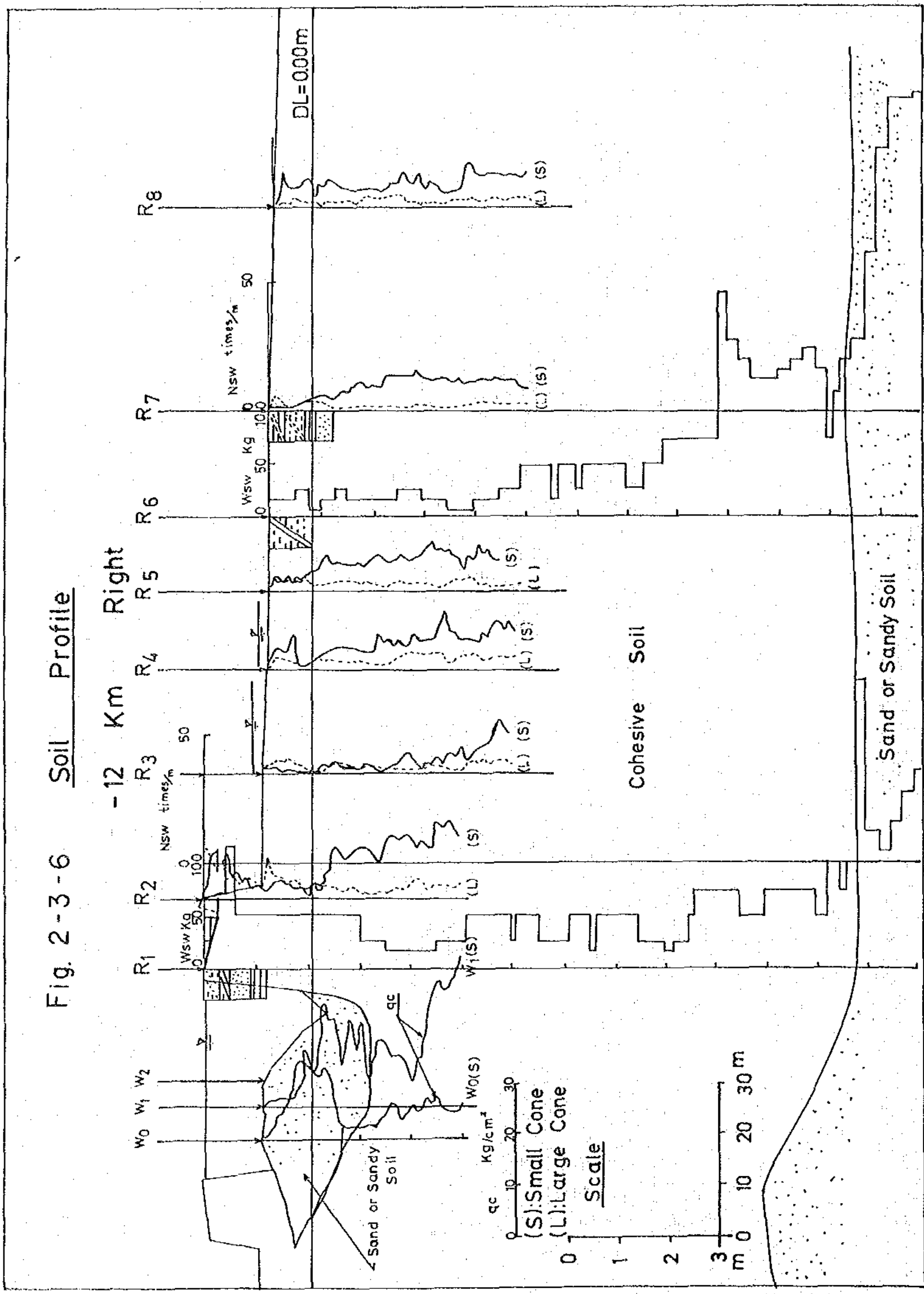


Fig. 2-3-6 Soil Profile
-12 Km Right

Fig. 2-3-7 Soil Profile

-12 Km Left

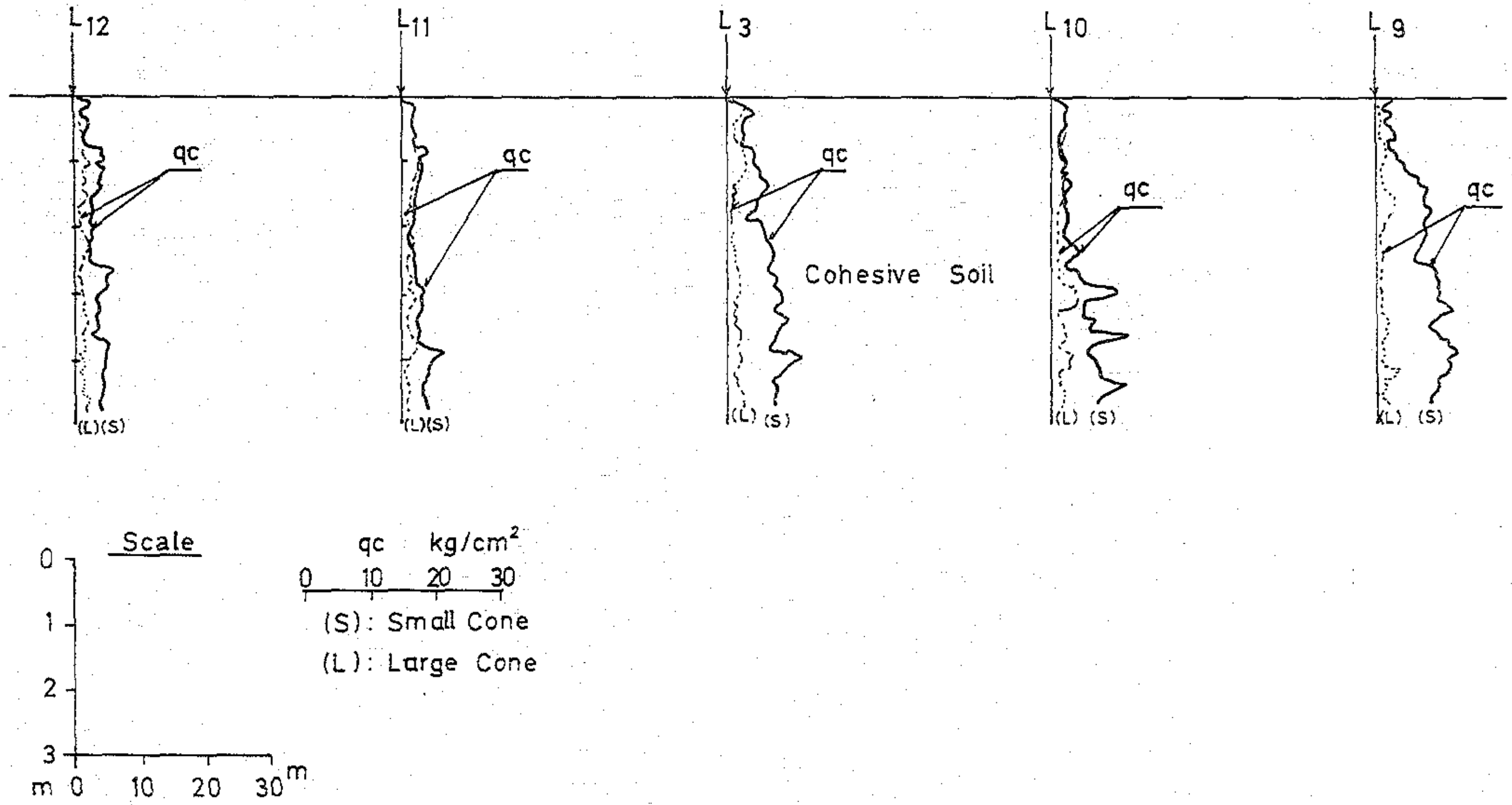


Fig. 2-3-8 Soil Profile

-11.25 Km Right

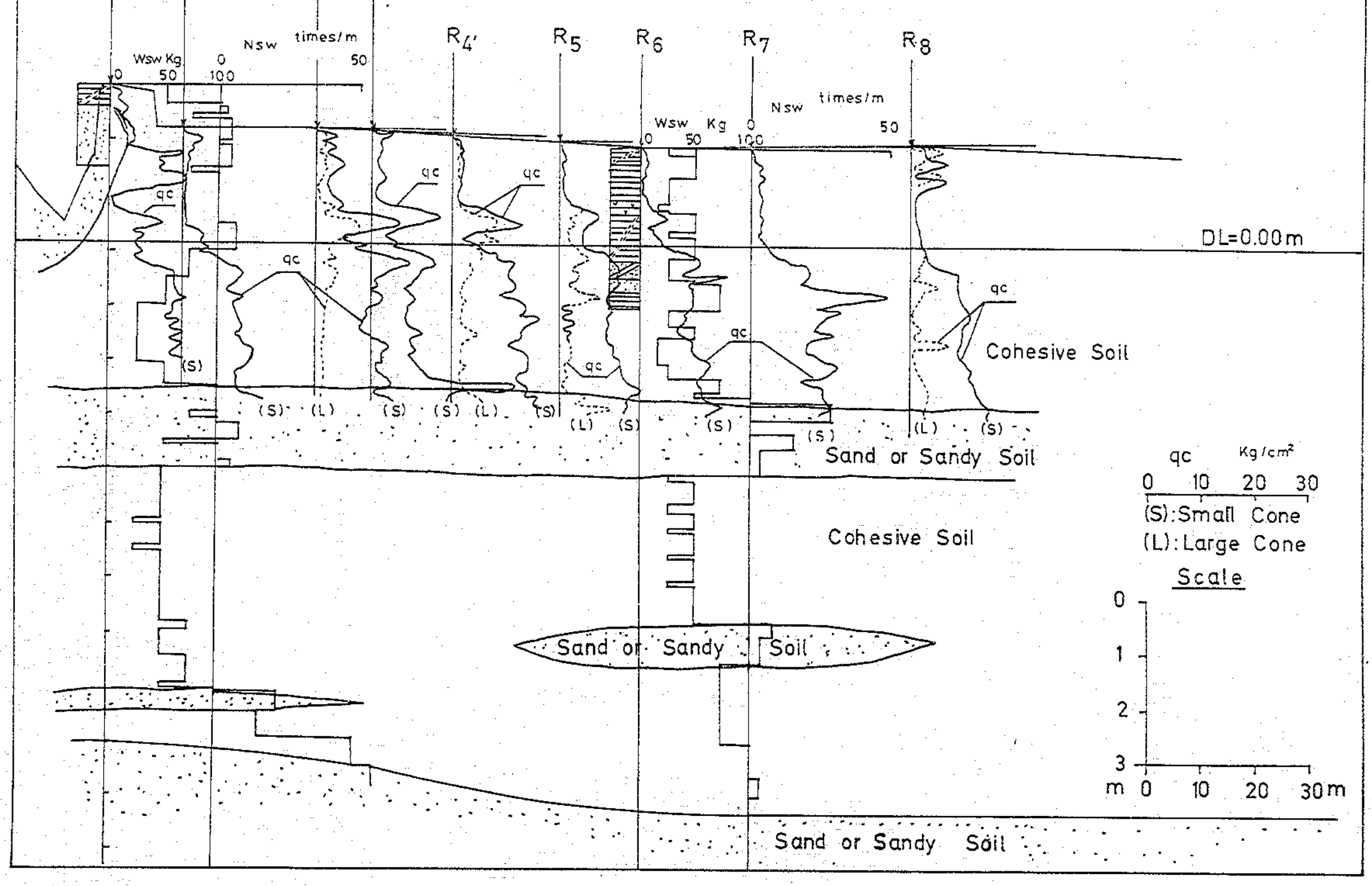


Fig. 2-3-9 Soil Profile

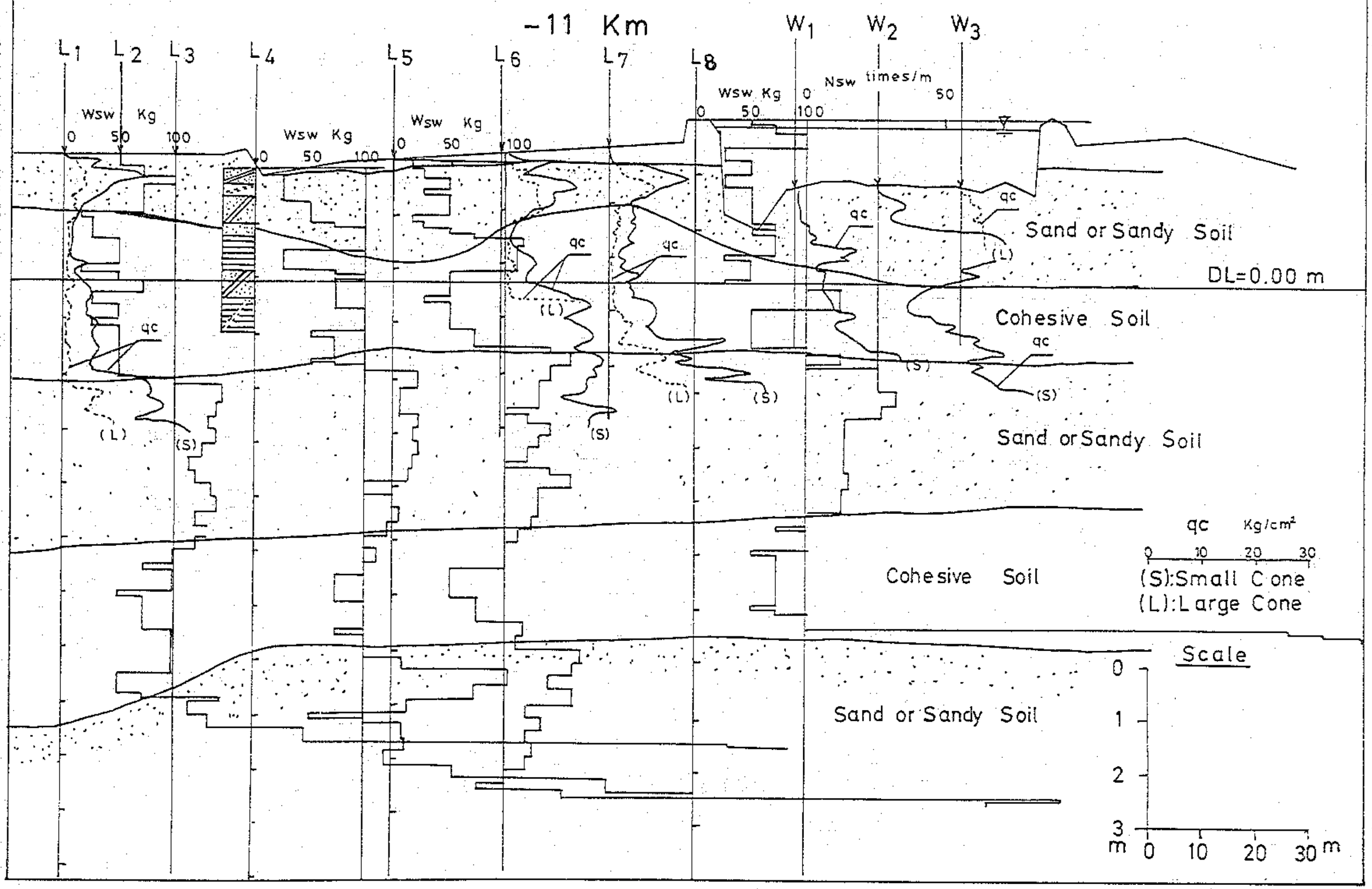


Fig. 2-3-10 Soil Profile

Left

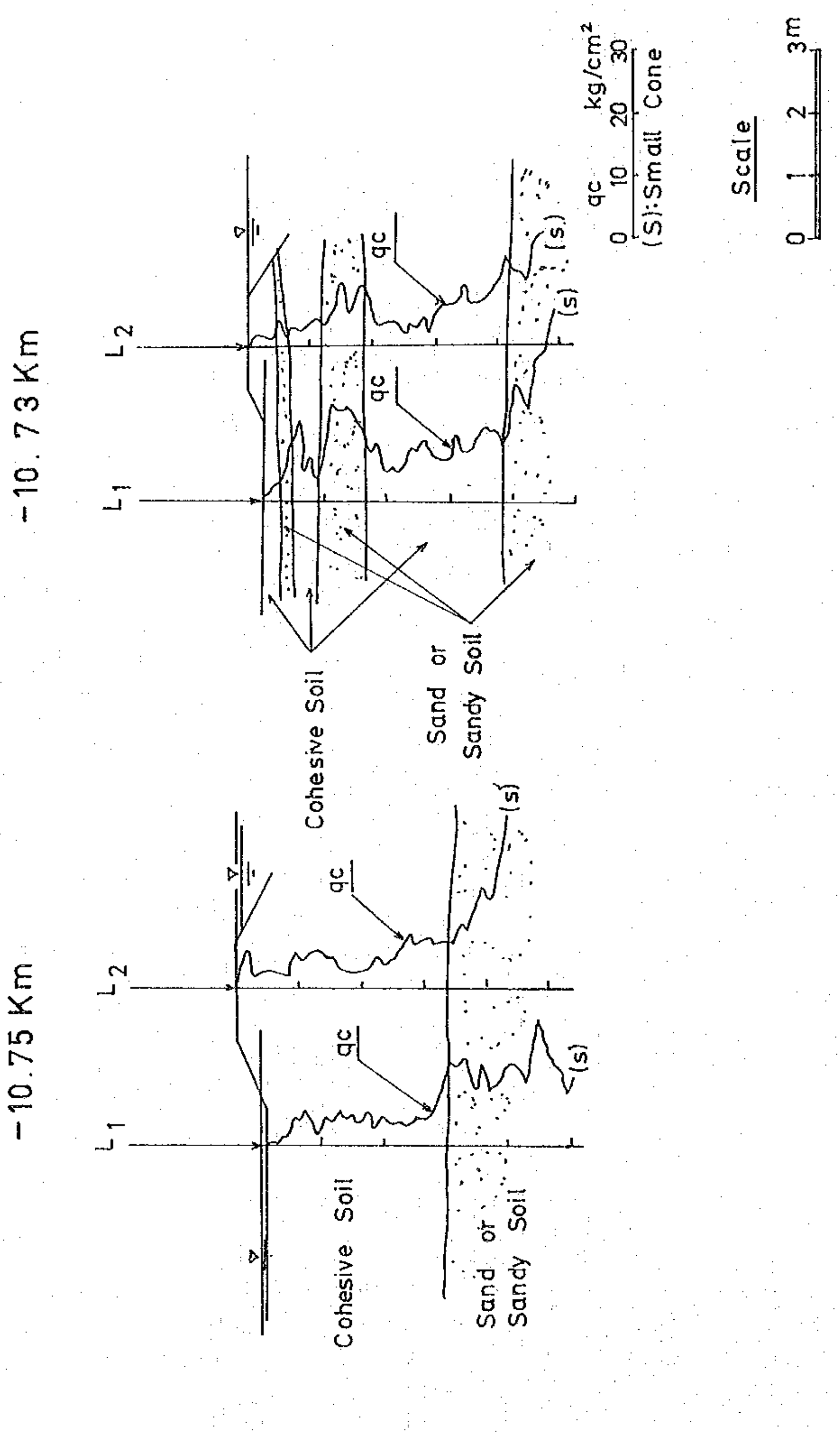


Fig. 2-3-11 Soil Profile
-10 Km Left

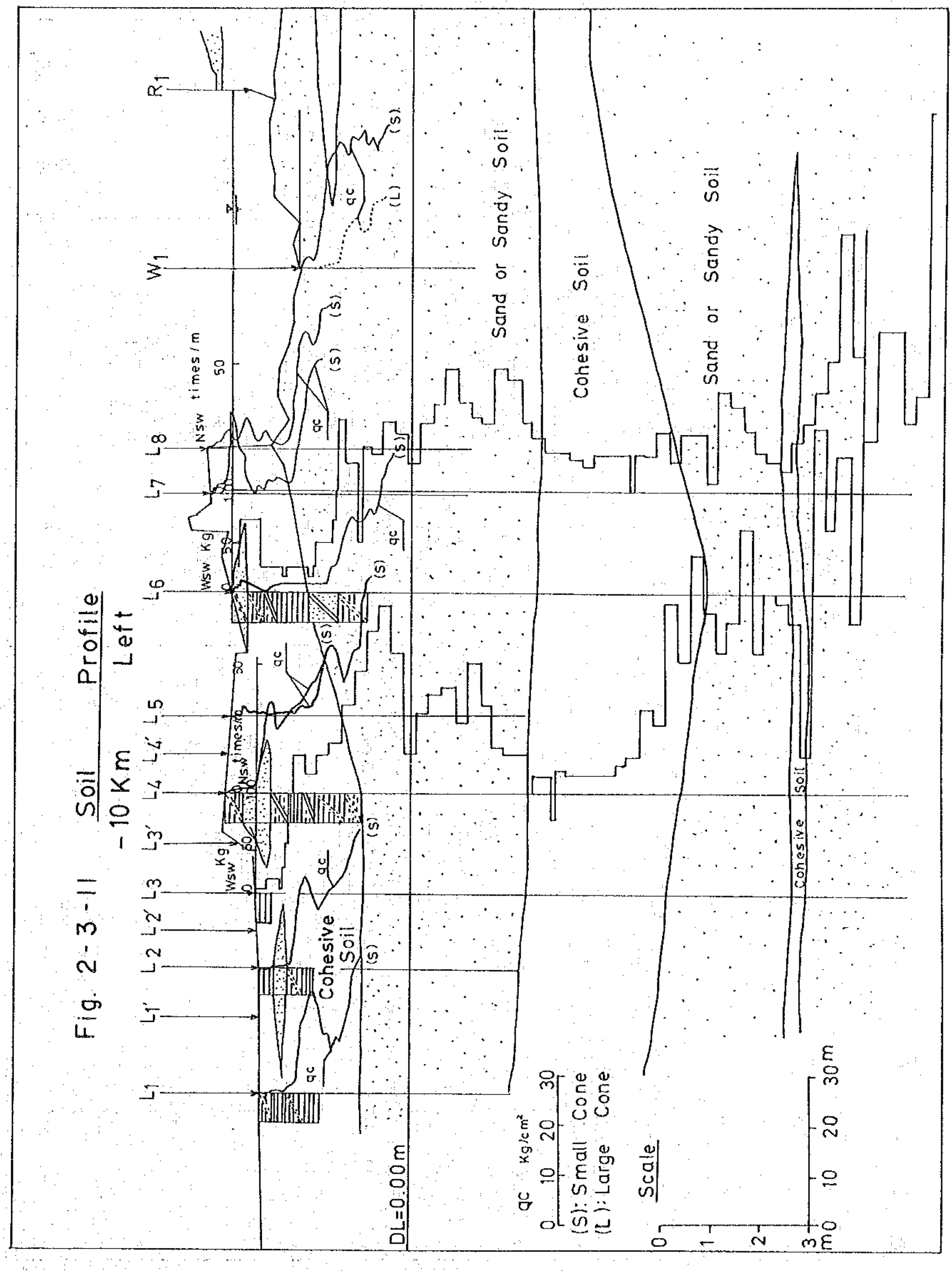


Fig. 2-3-12 Soil Profile

-10 Km Right

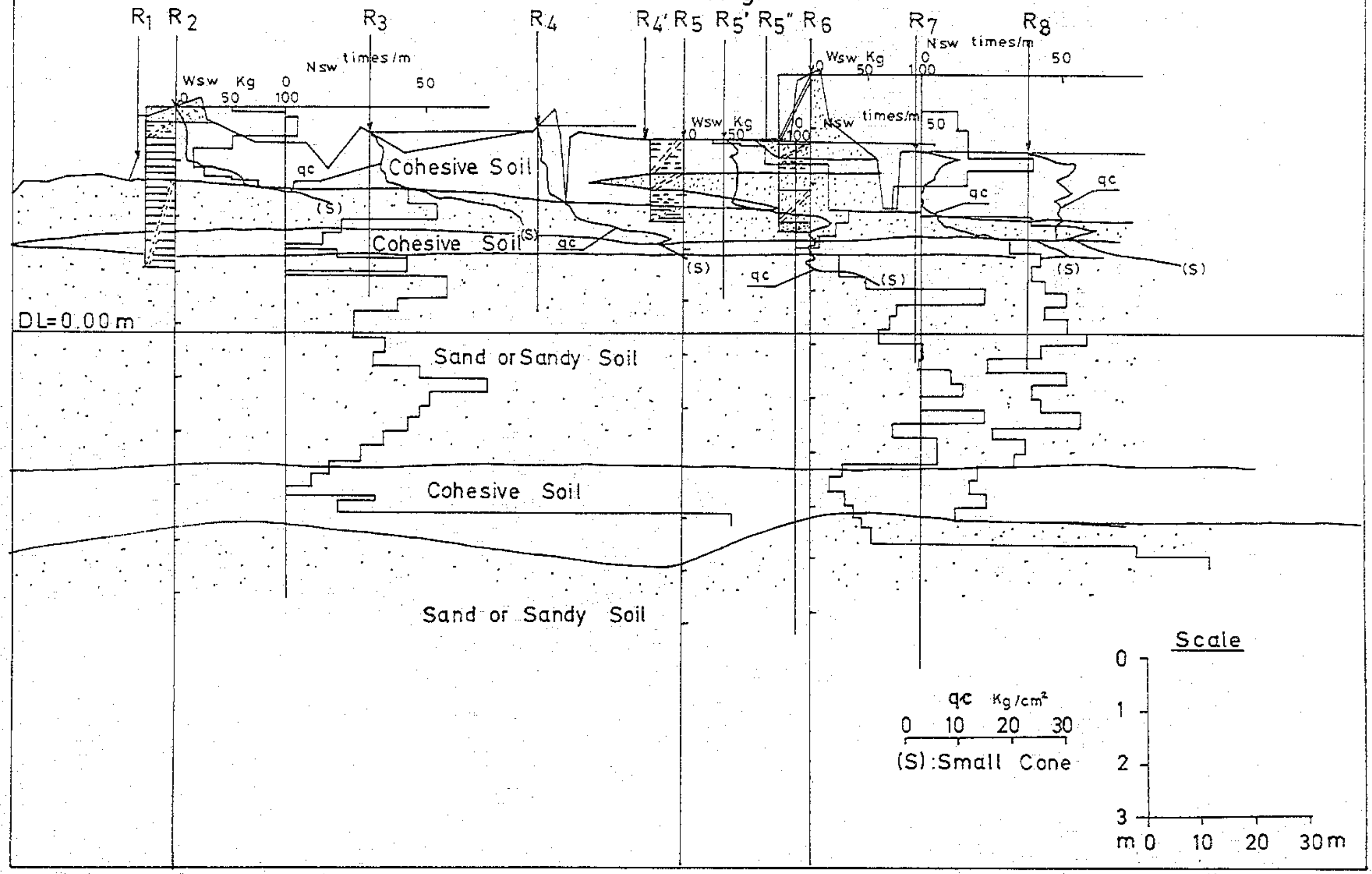


Fig. 2-3-13 Soil Profile

- 10 Km Left

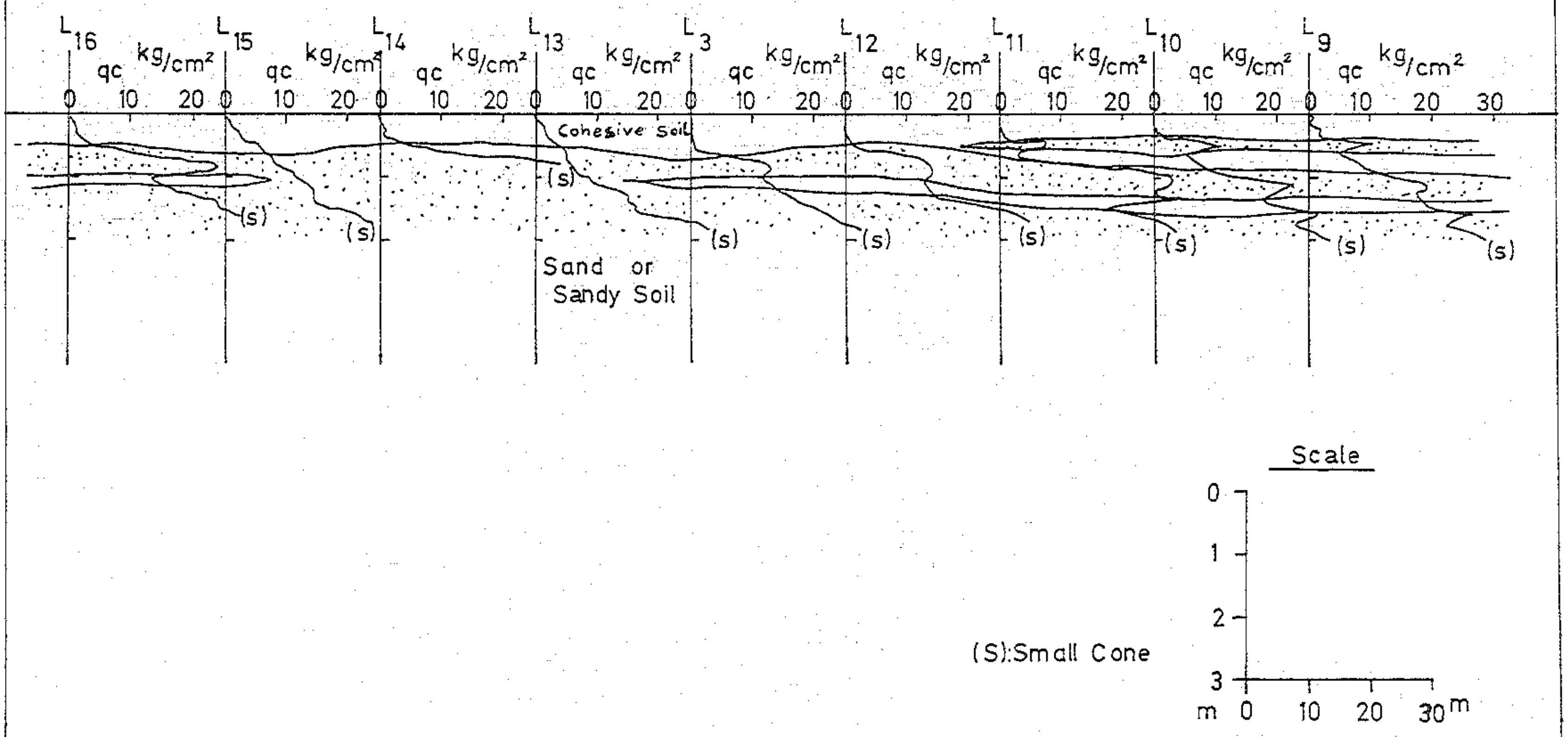


Fig. 2-3-14 Soil Profile

-9 Km Right (1)

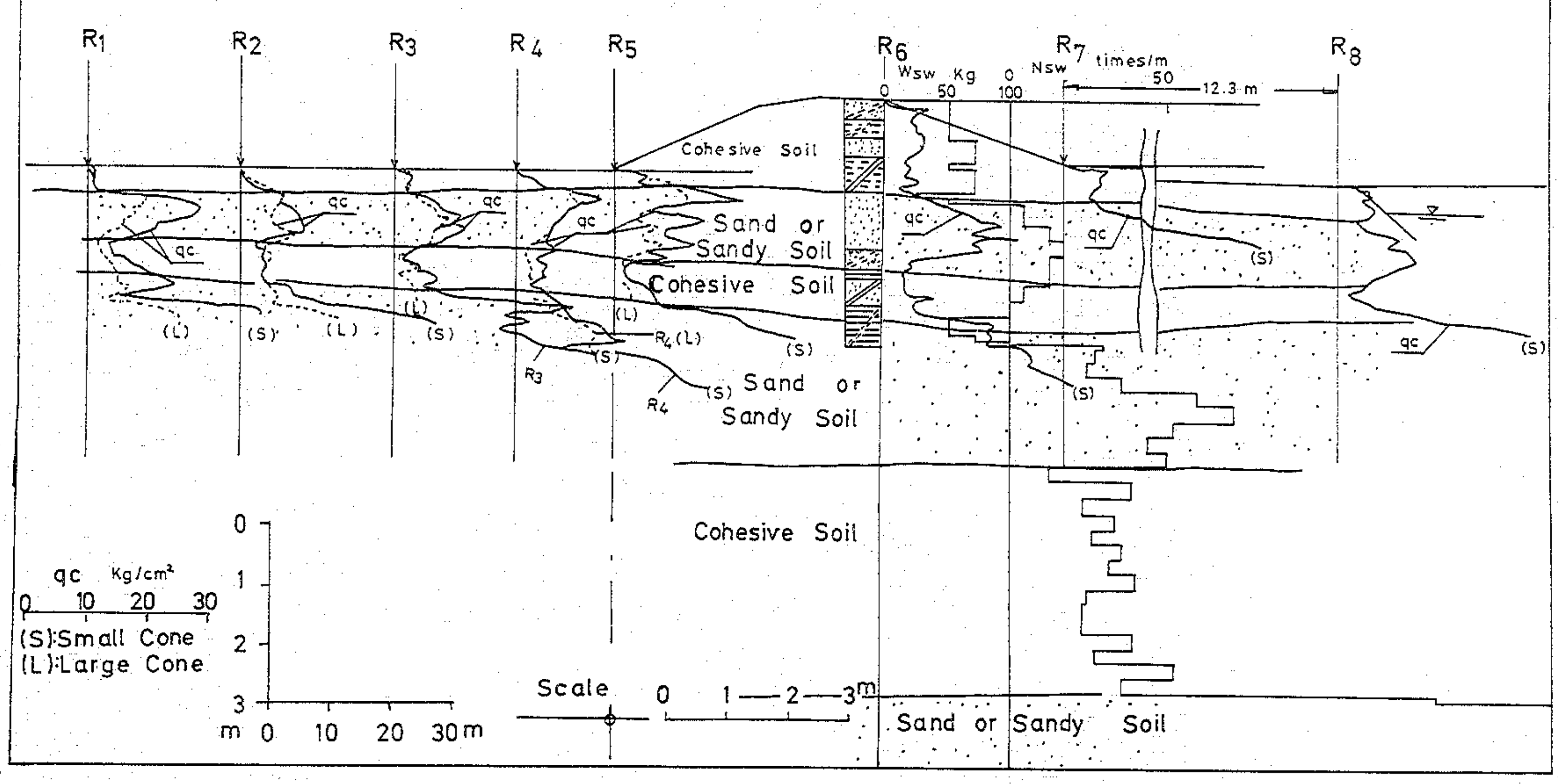


Fig. 2-3-15 Soil Profile

-9 Km Right (2)

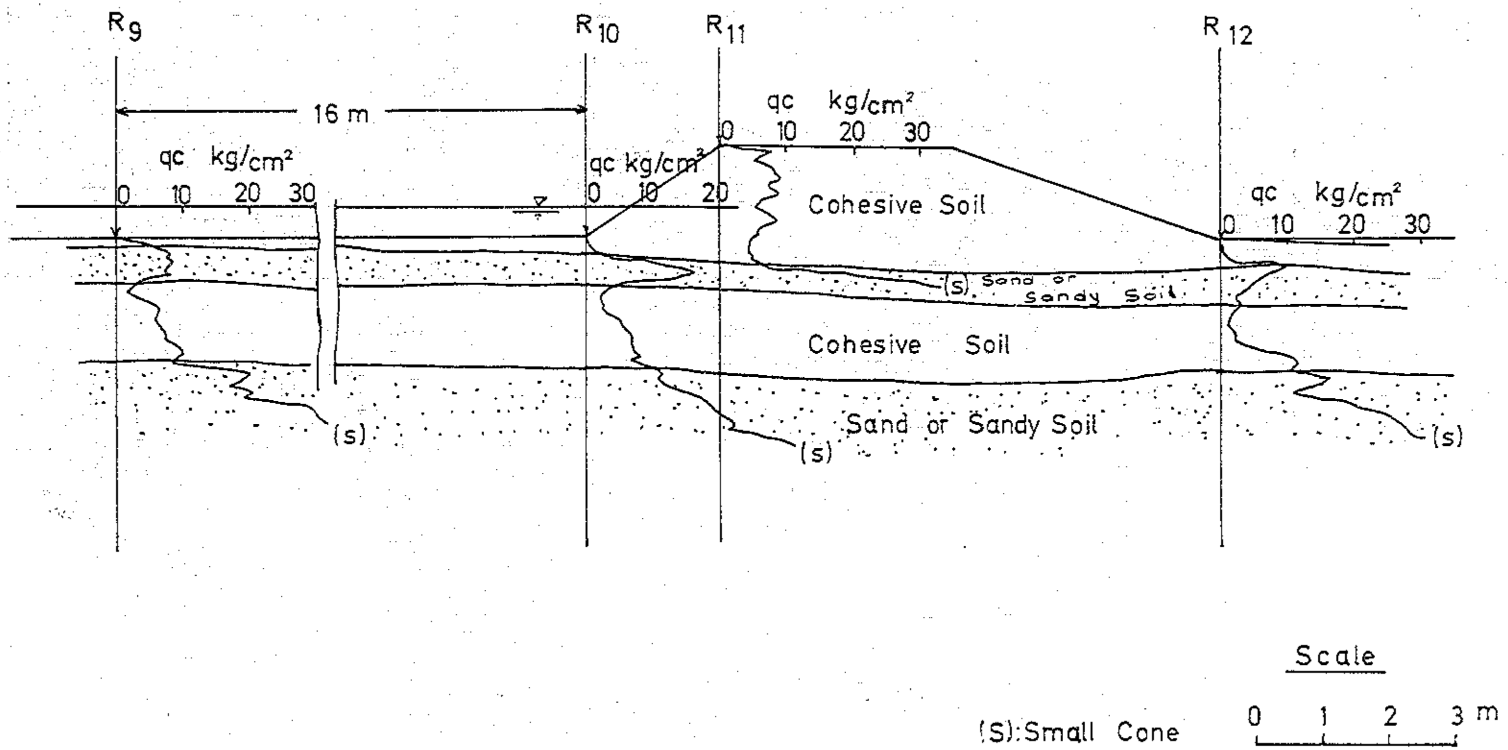


Fig. 2-3-16 Soil Profile

-8.25 Km Left

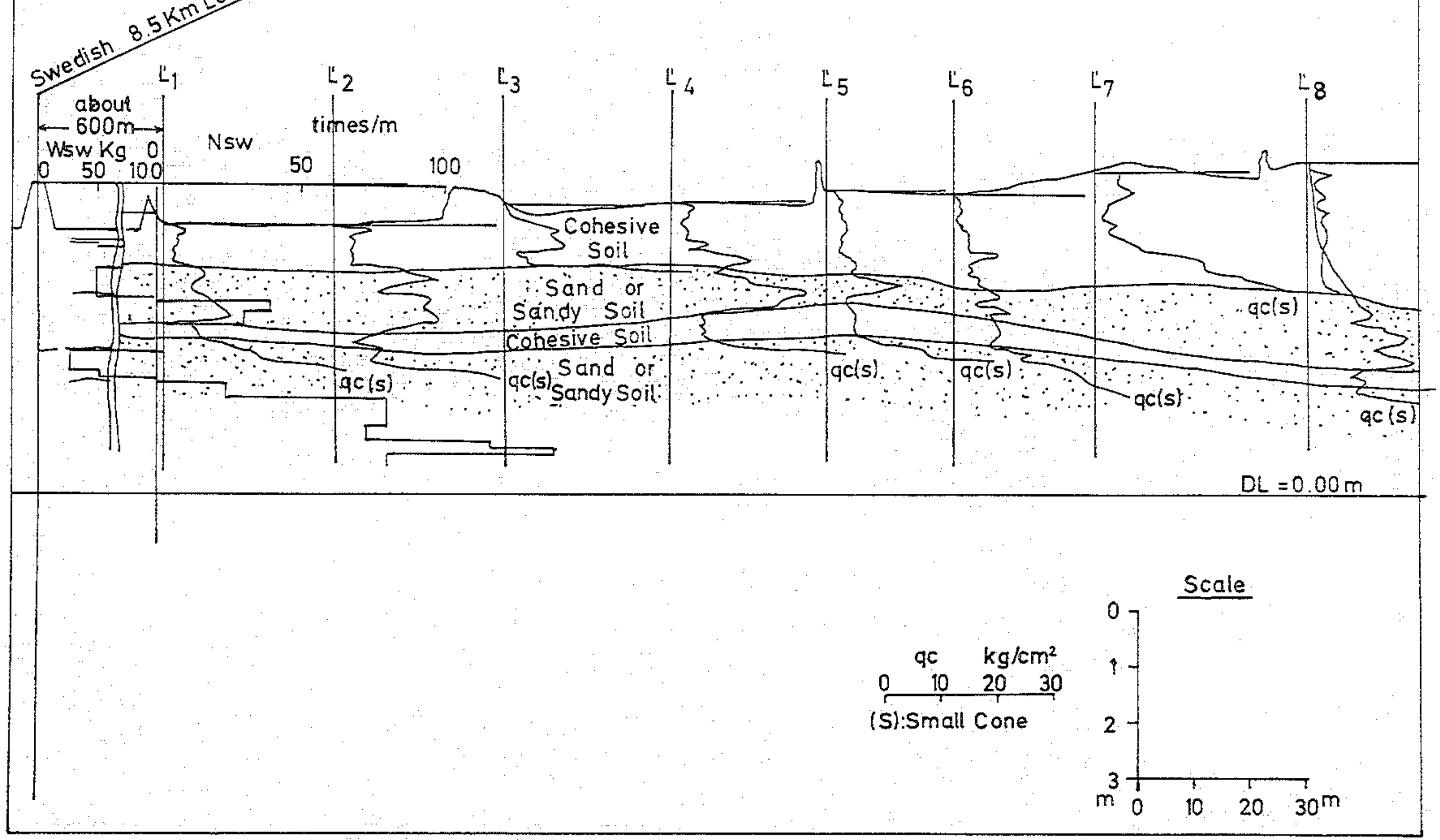


Fig. 2-3-17 Soil Profile

-7 Km Left

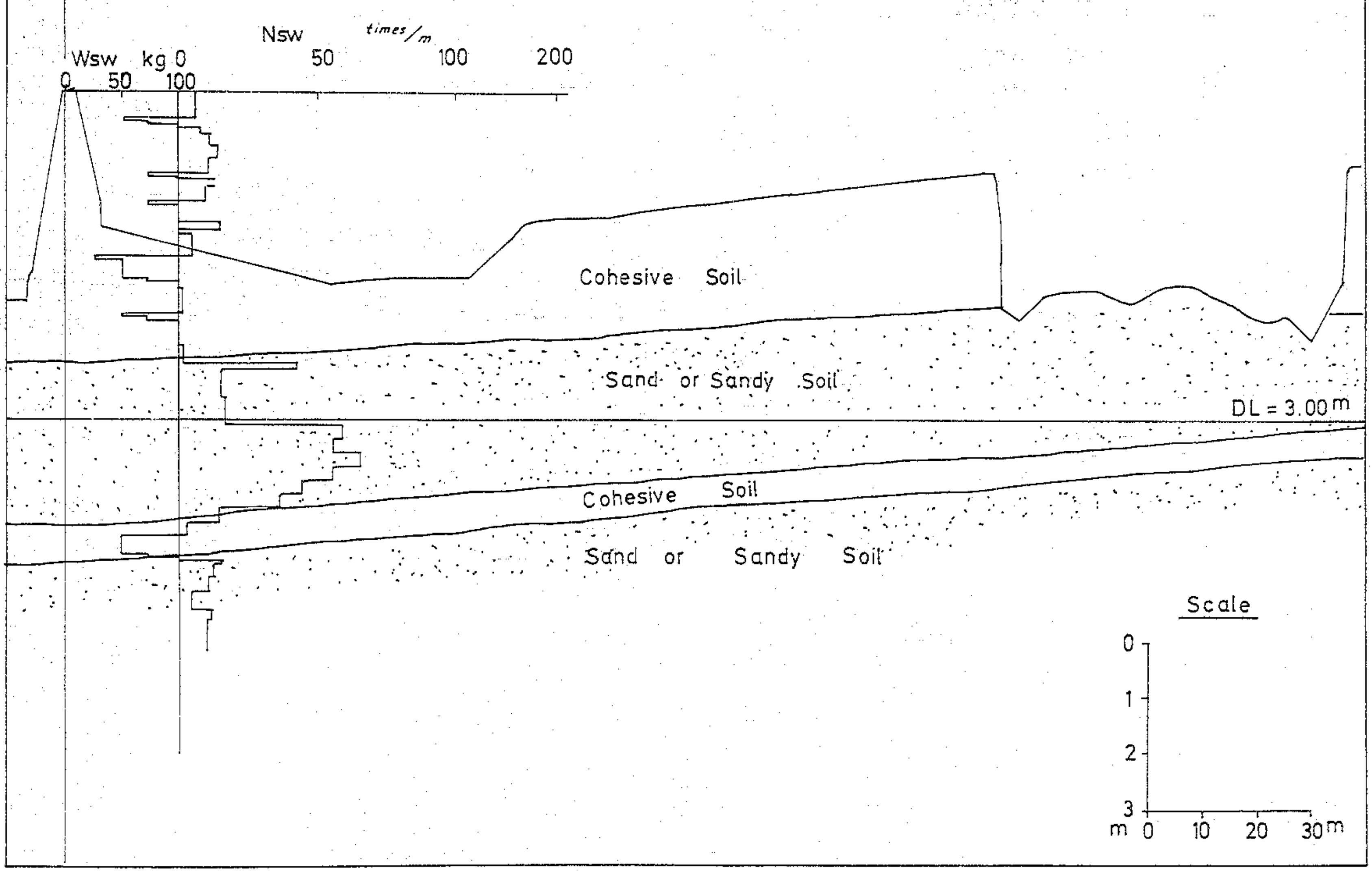


Fig. 2-3-18 Soil Profile

-7 Km Right

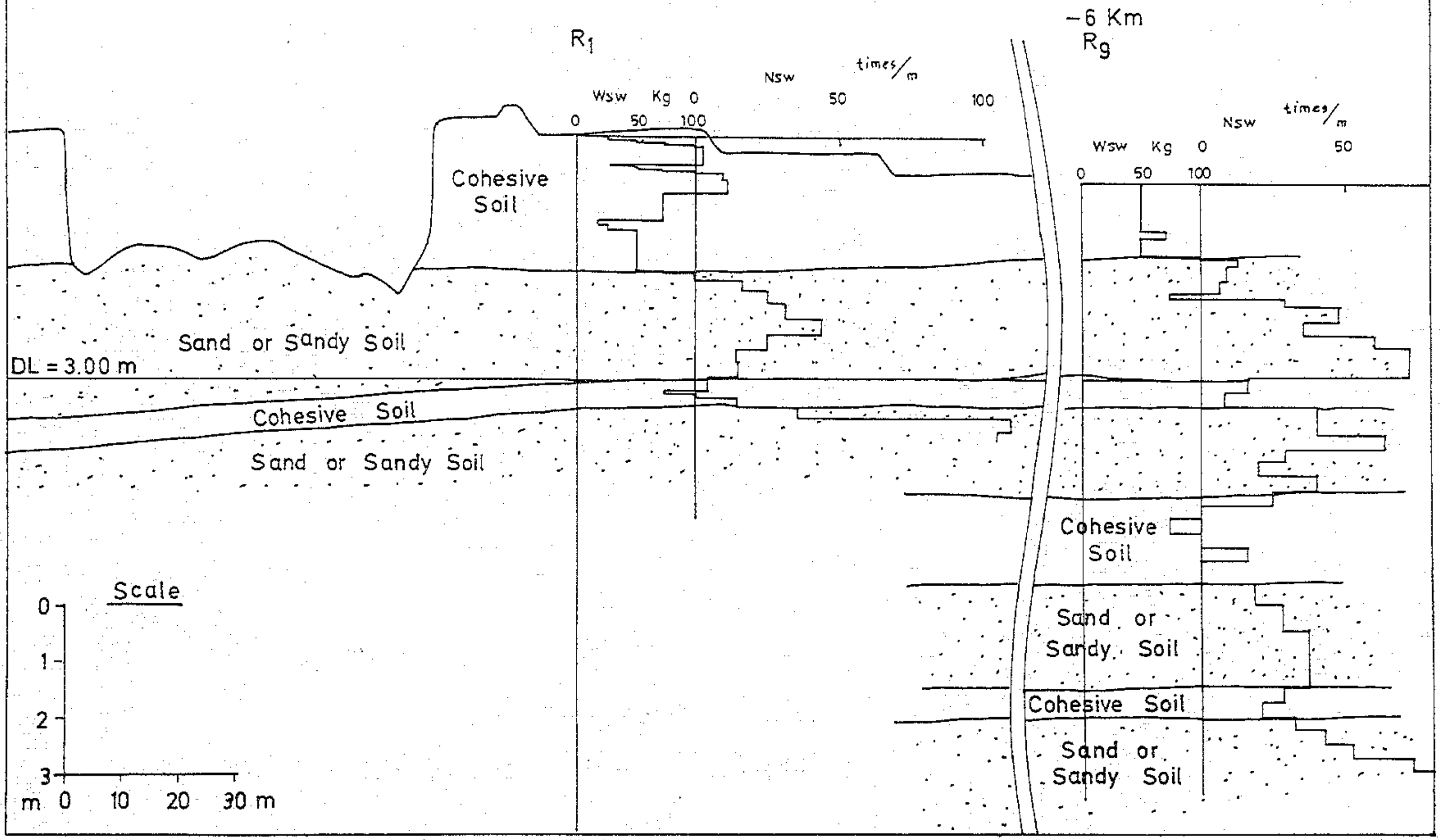
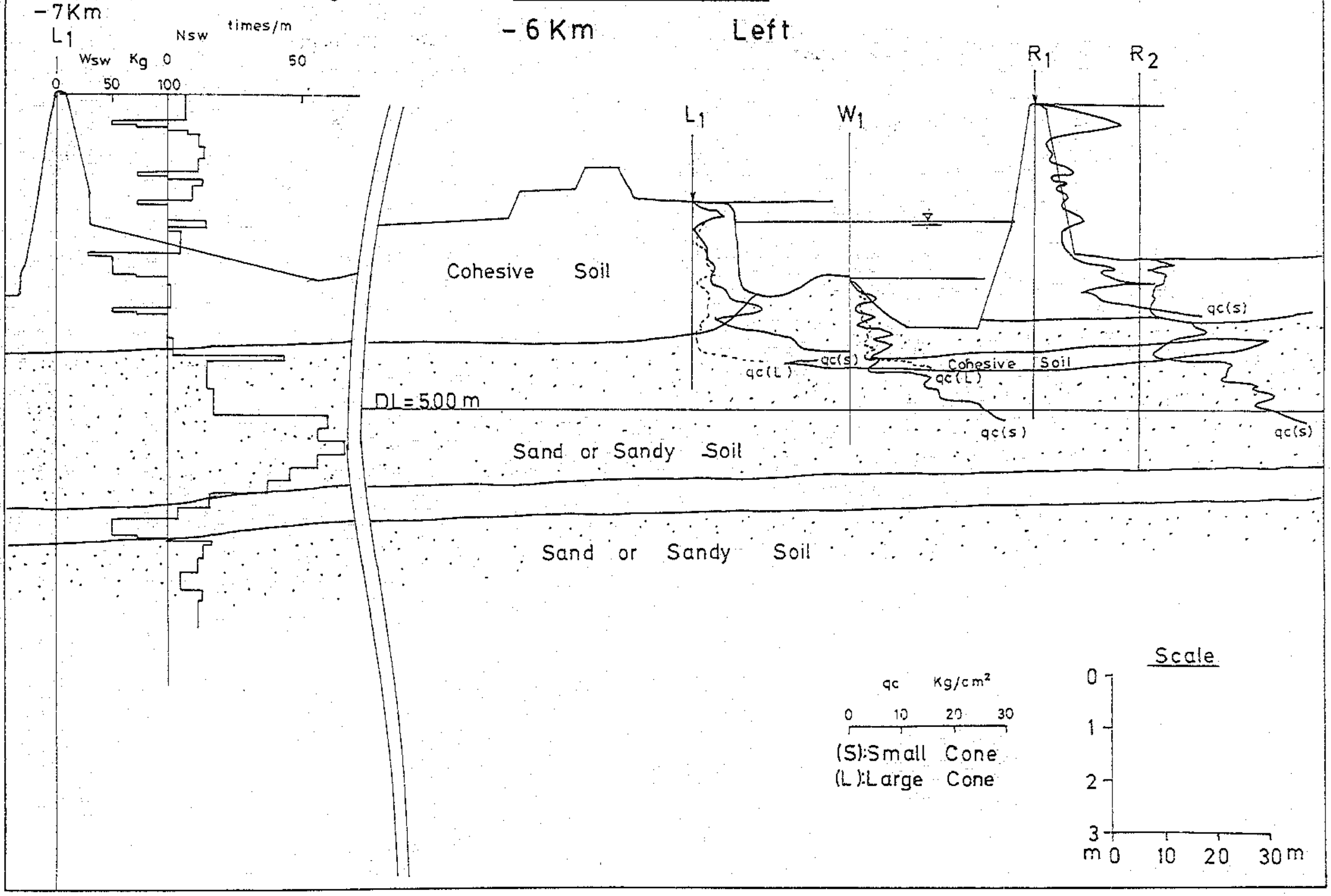


Fig. 2-3-19 Soil Profile
-6 Km Left



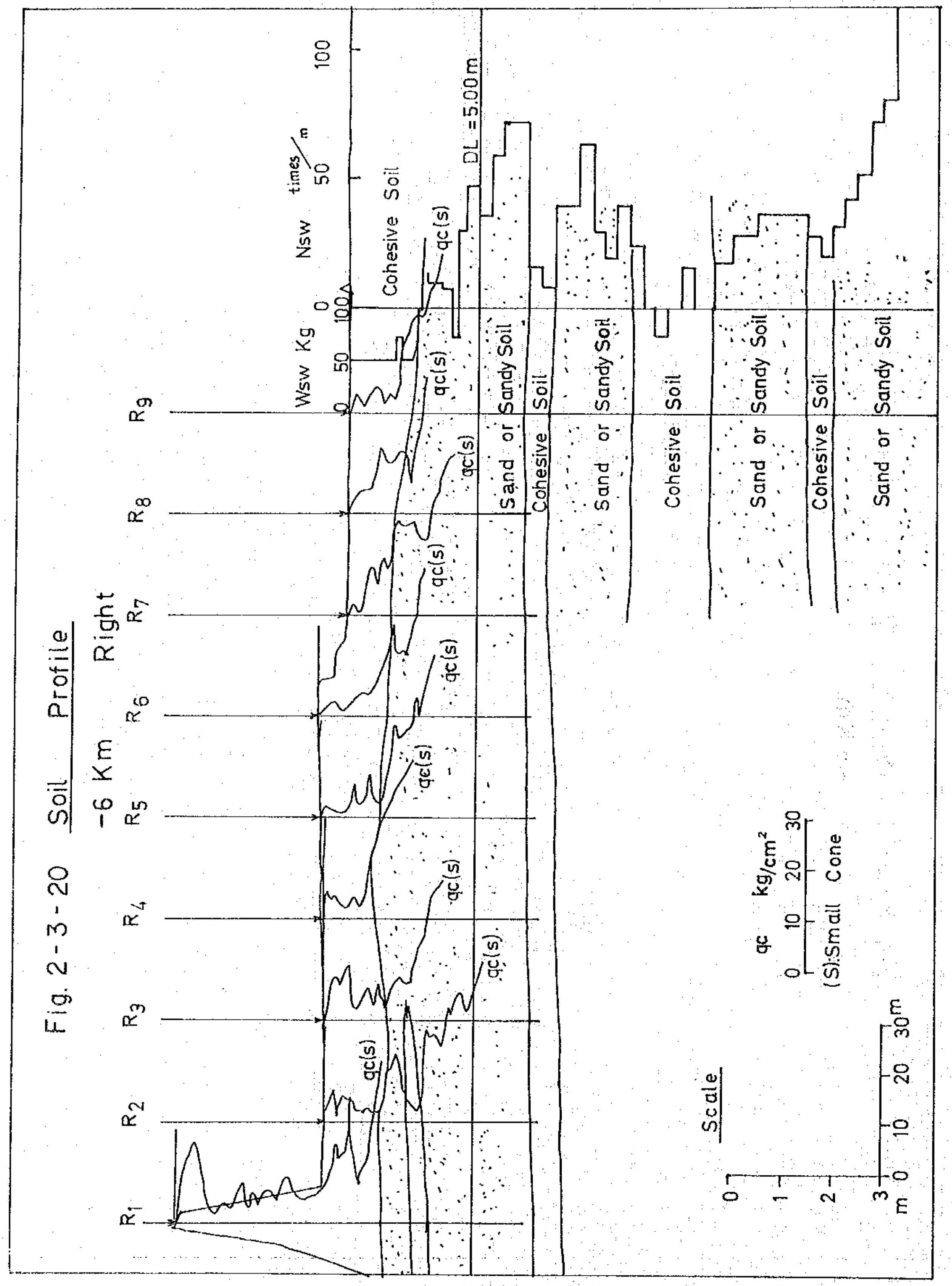
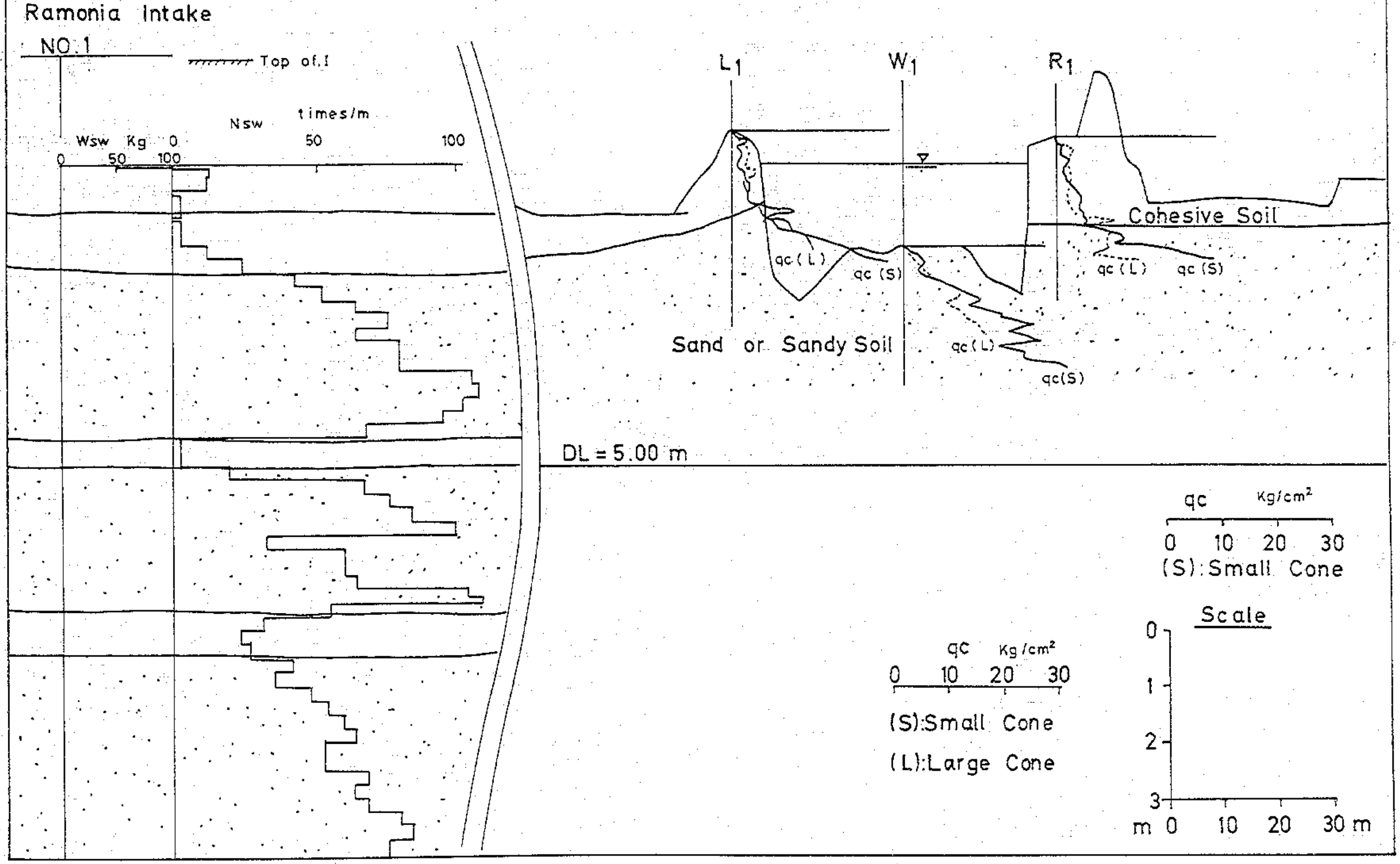


Fig. 2-3-20 Soil Profile -6 Km Right

Fig. 2-3-21 Soil Profile

-4 Km



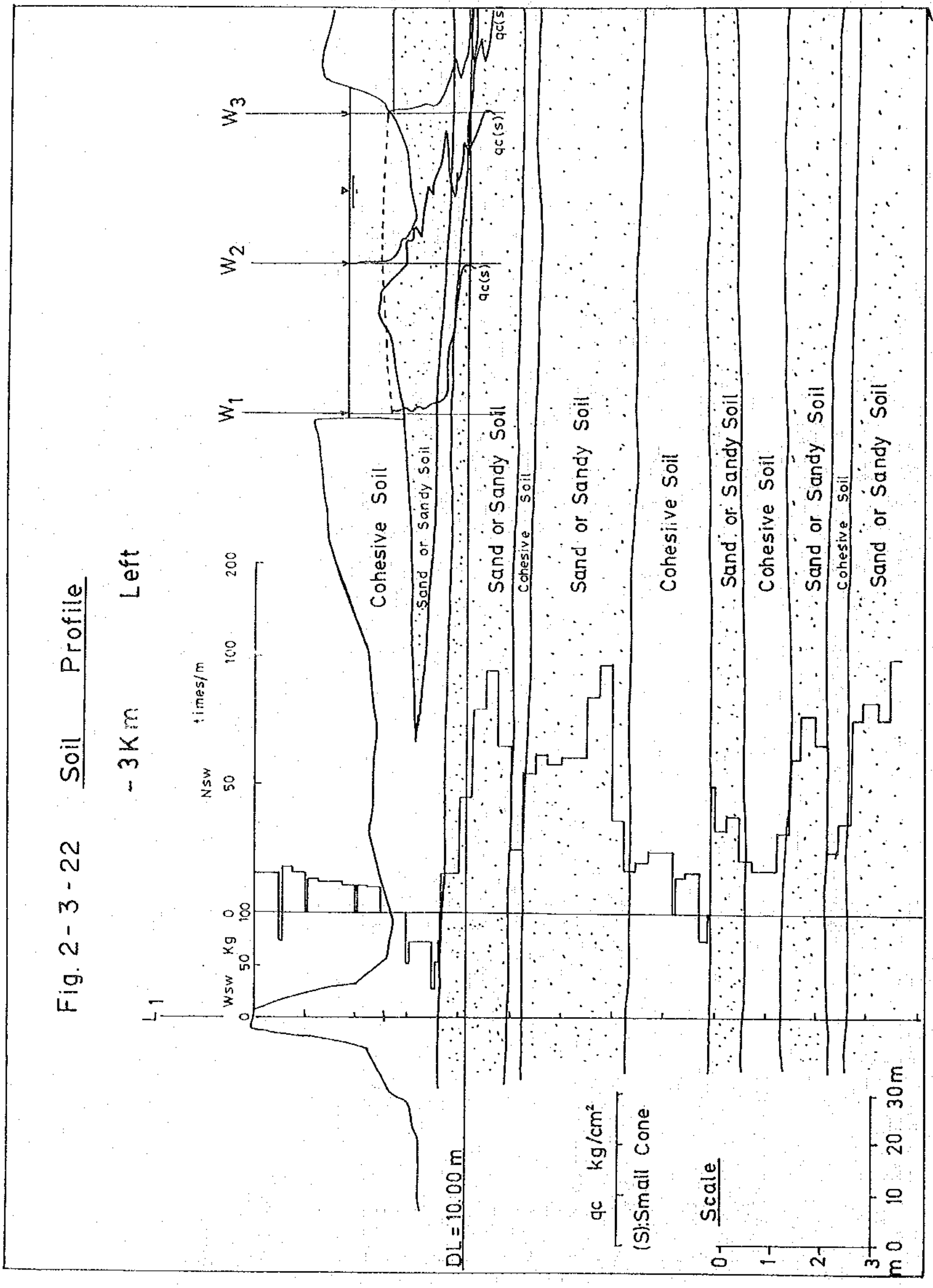
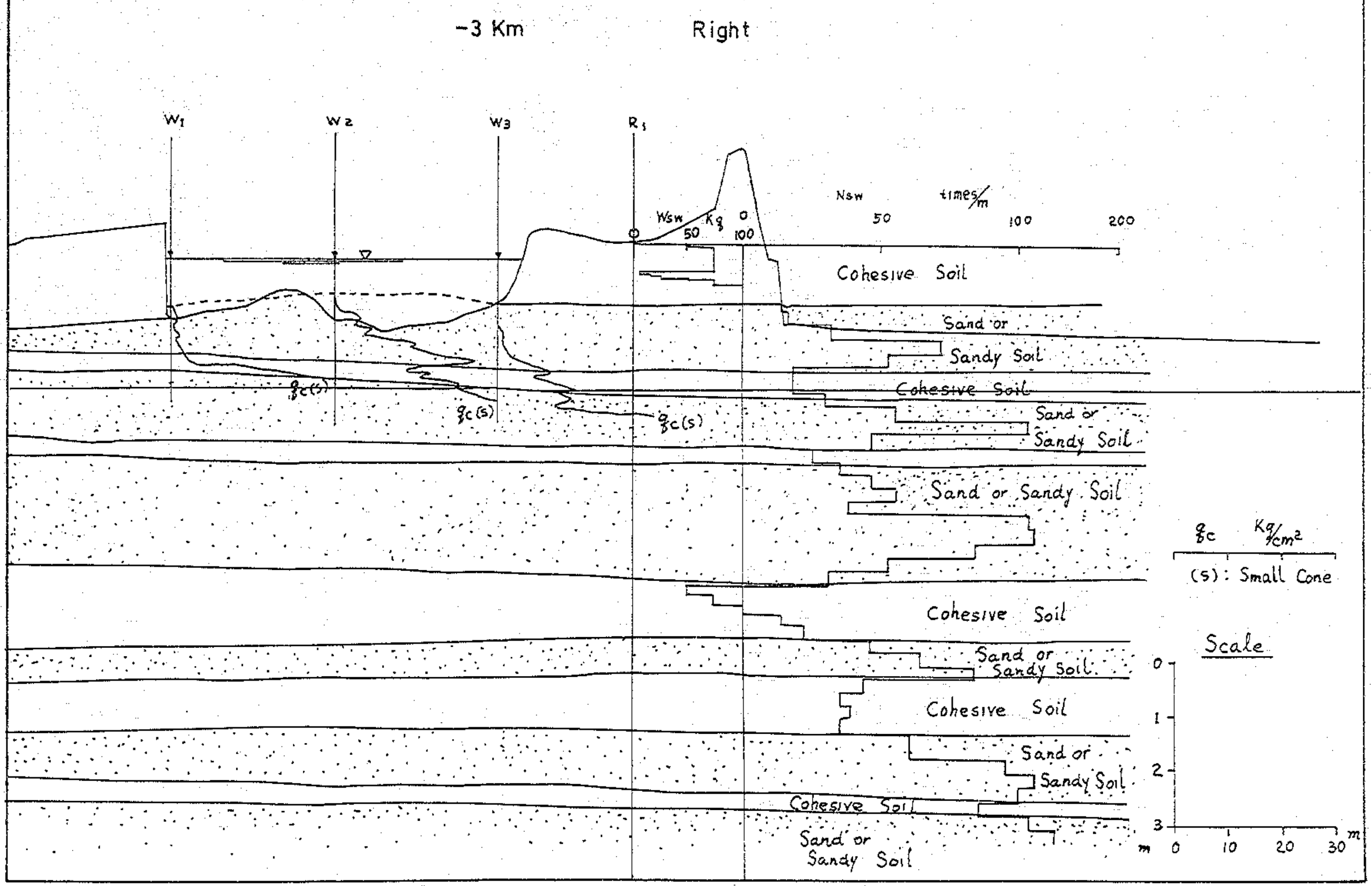


Fig. 2-3-22 Soil Profile

-3Km Left

Fig. 2-3-23 SOIL PROFILE



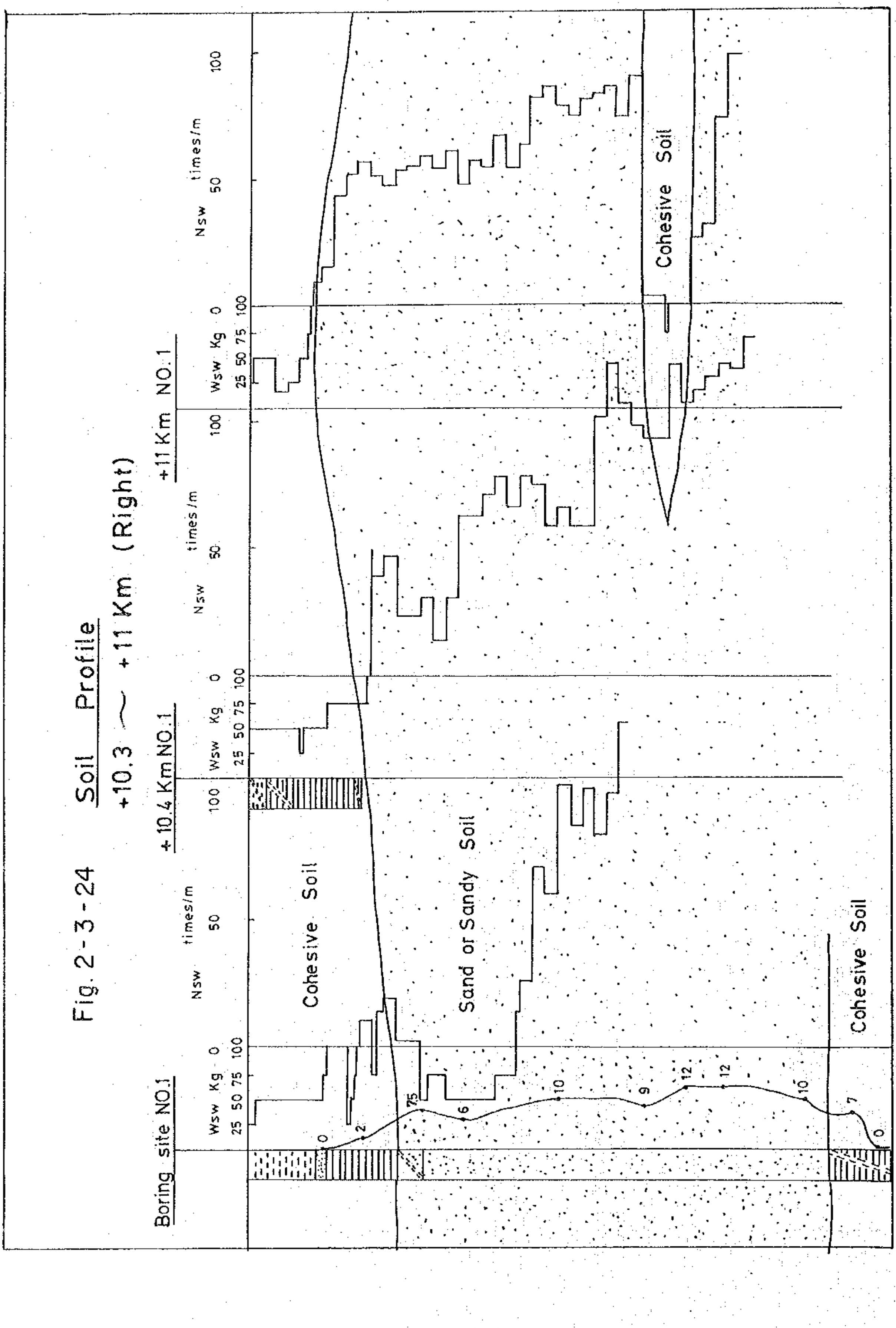
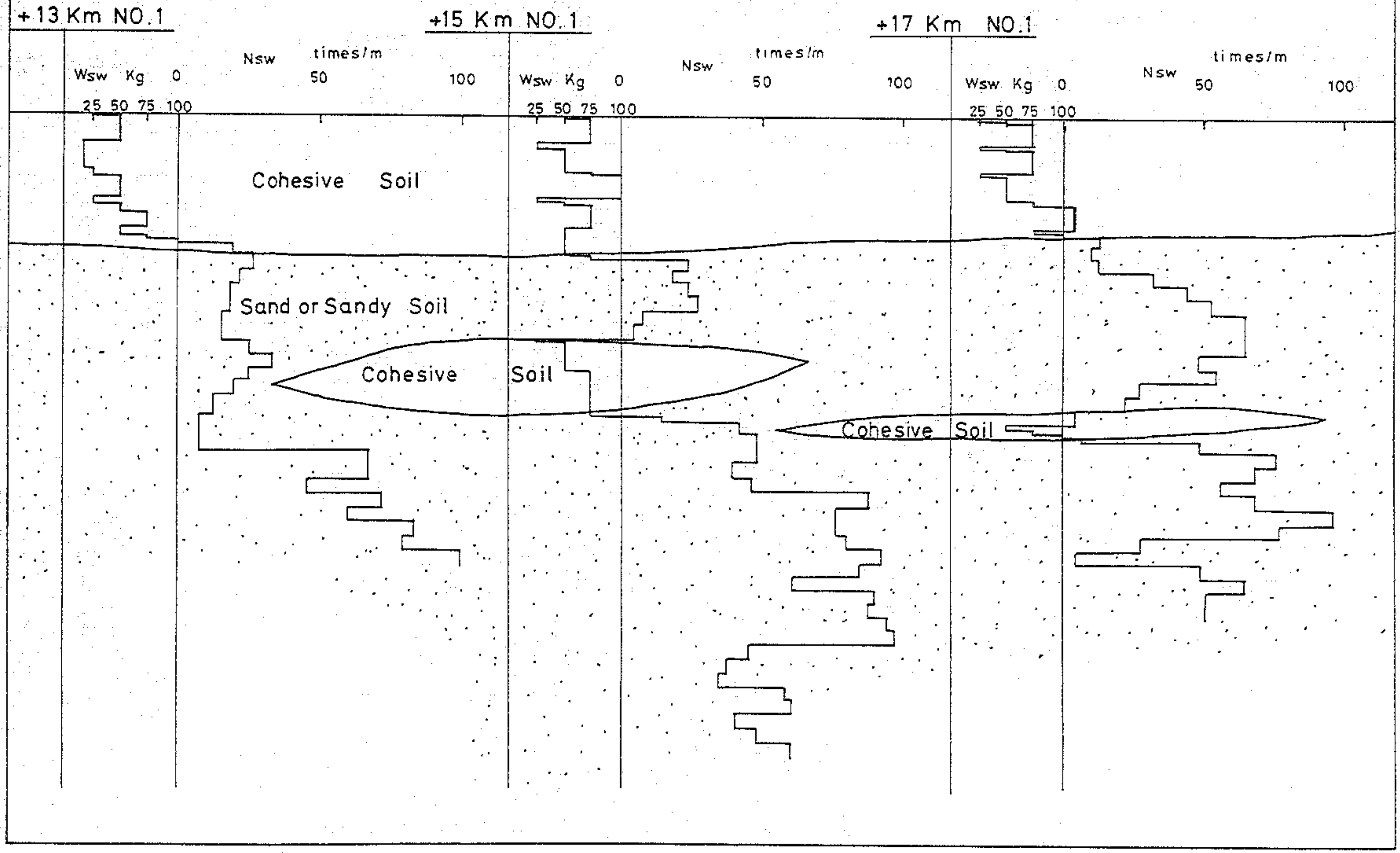


Fig. 2-3-24 Soil Profile
+10.3 ~ +11 Km (Right)

Fig. 2-3-25 Soil Profile
+13 ～ +17 Km (Right)



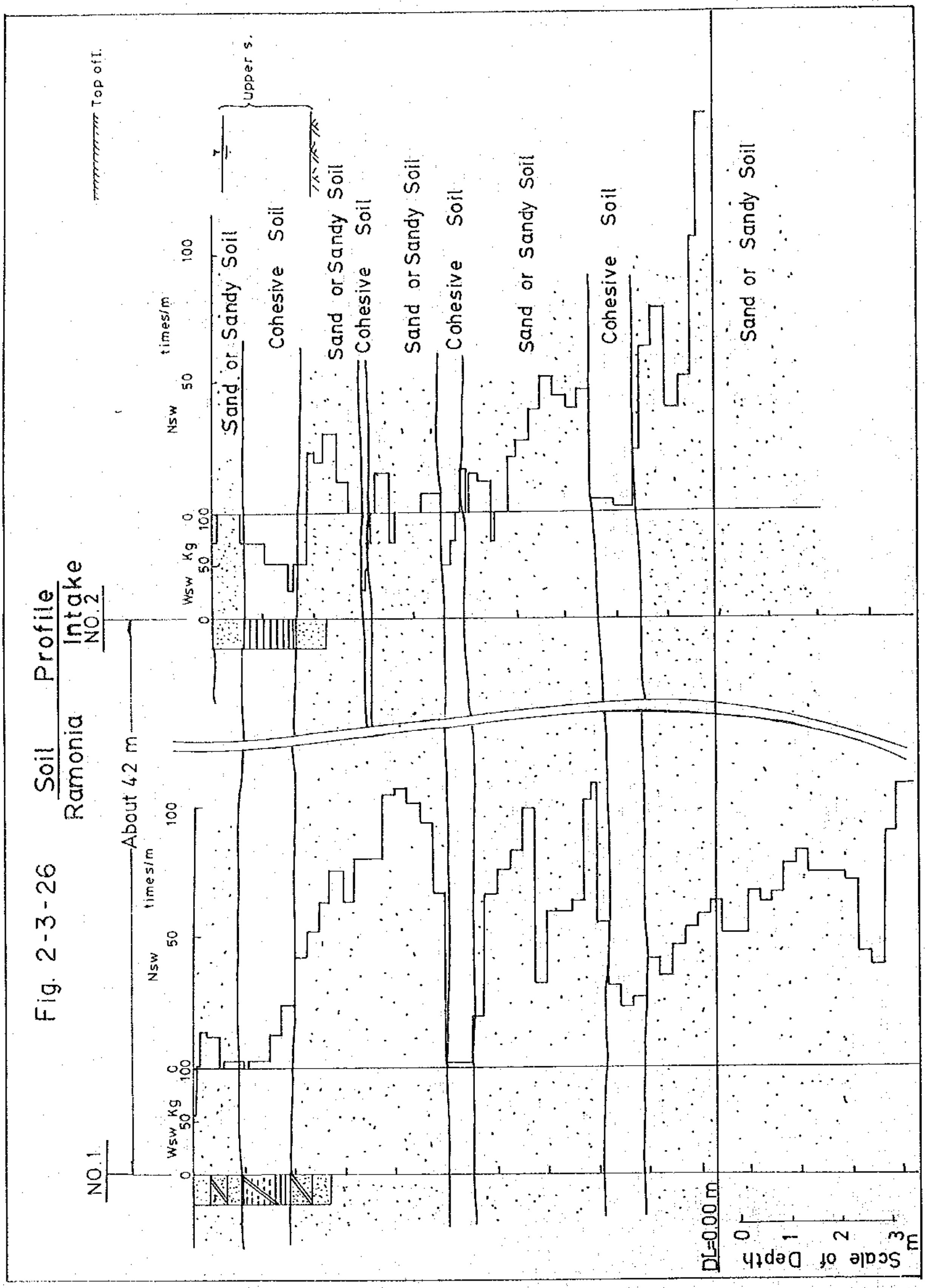


Fig. 2-3-26 Soil Profile Ramonia Intake

Fig. 2-3-27 Soil Profile
Singosari Intake

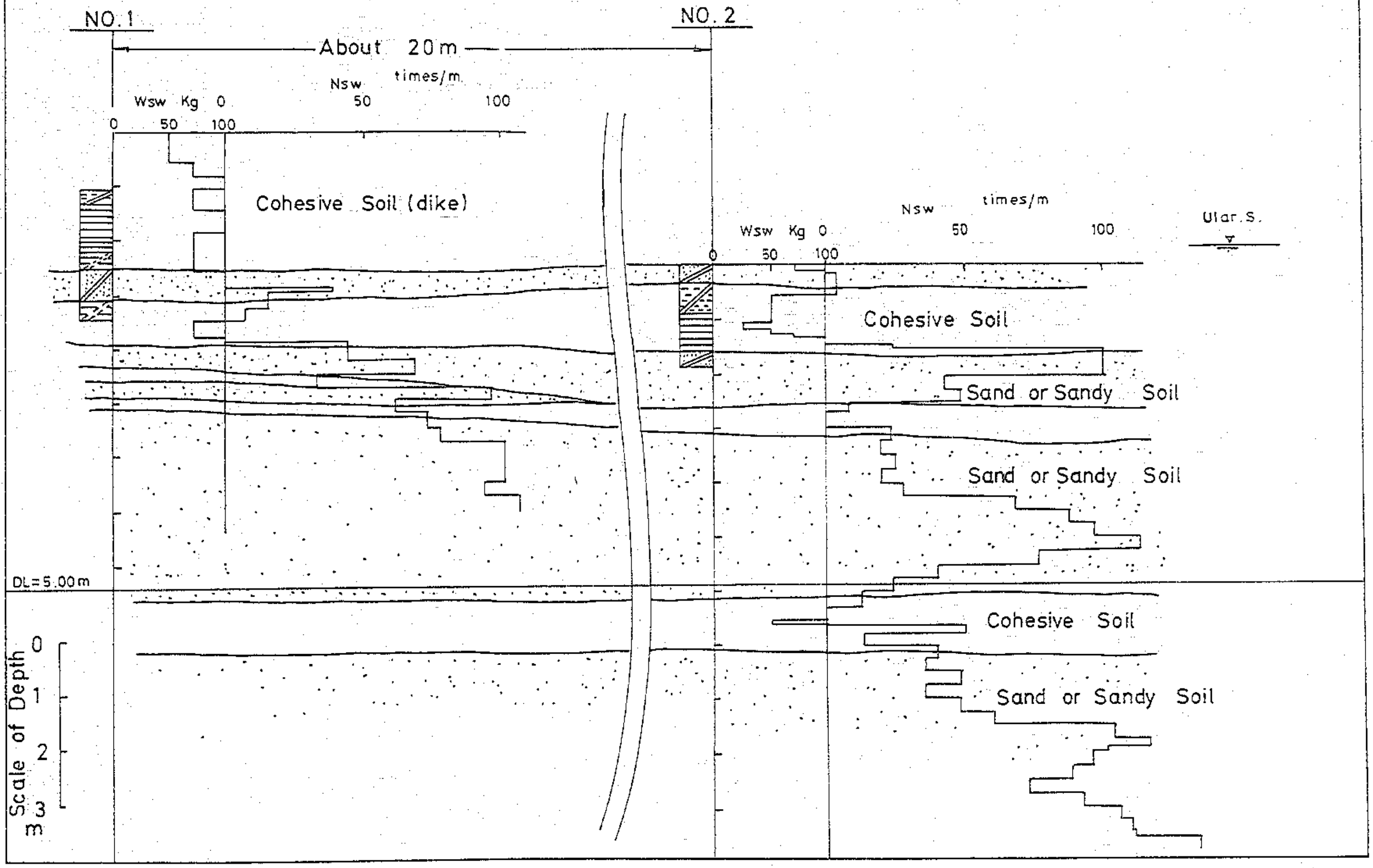


Fig. 2-3-28 Soil Profile

Bendang Intake

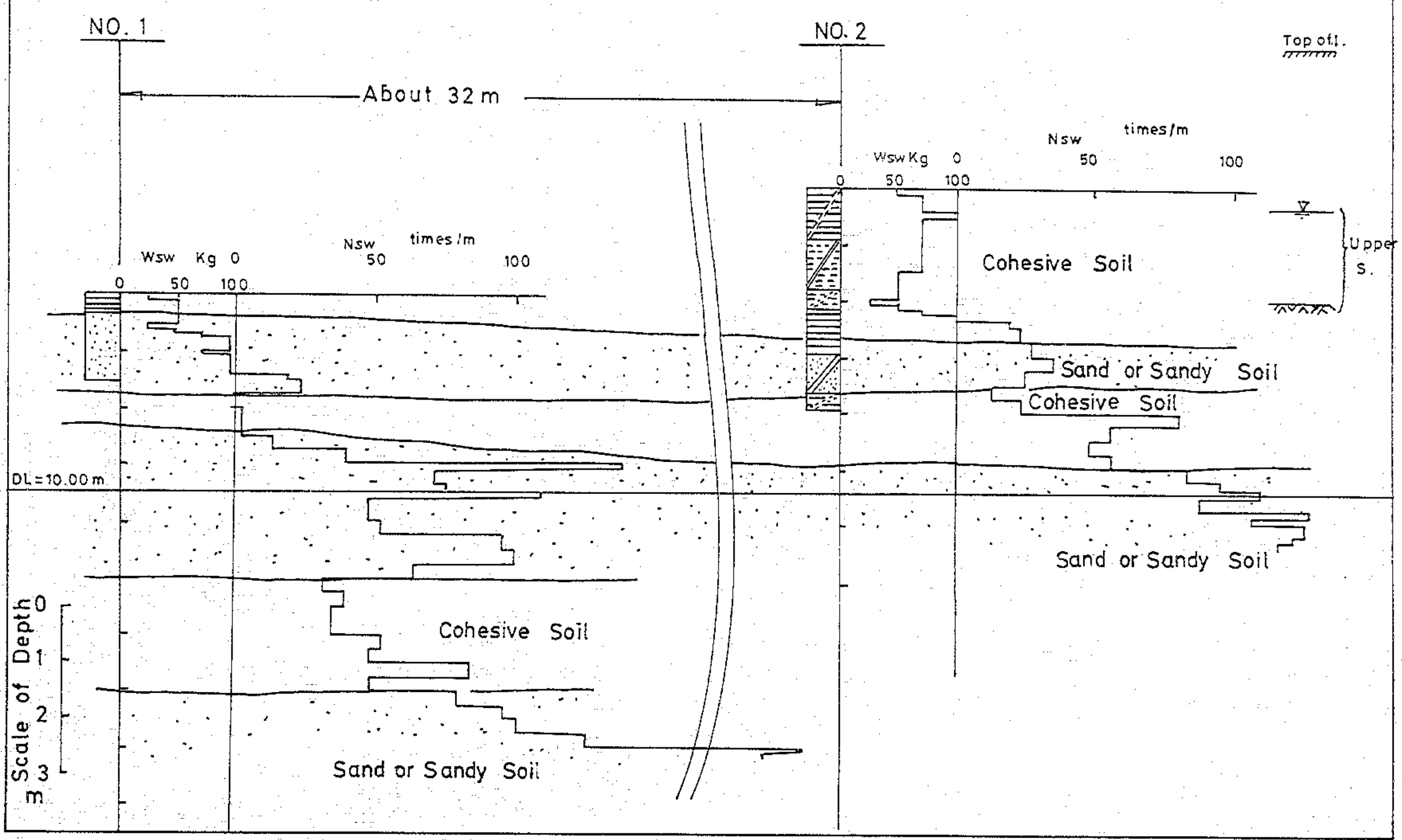


Fig. 2-3-29 Soil Profile

Sumber Rejo Intake

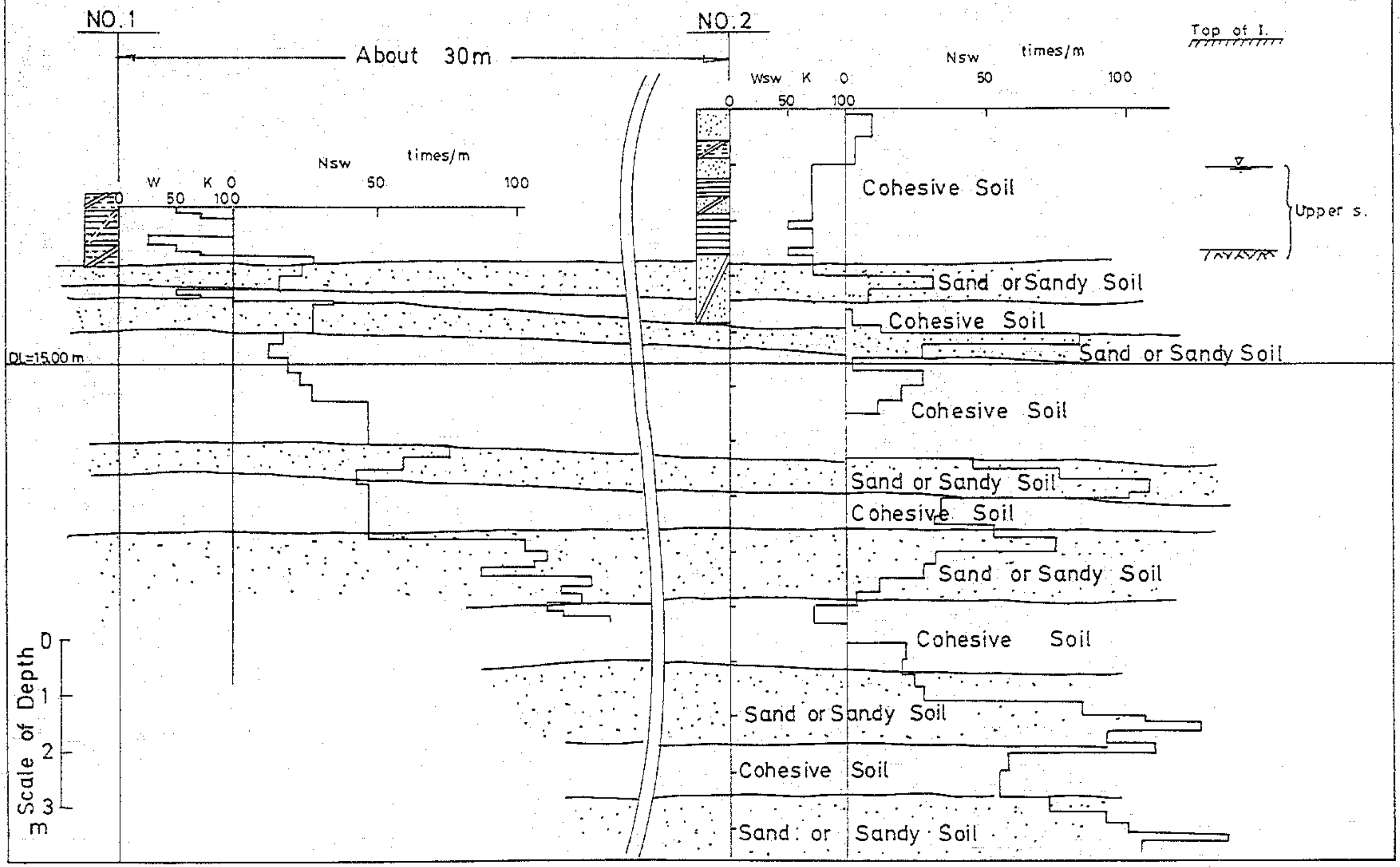


Fig. 2-3-30 Soil Profile
Perbaungan Intake

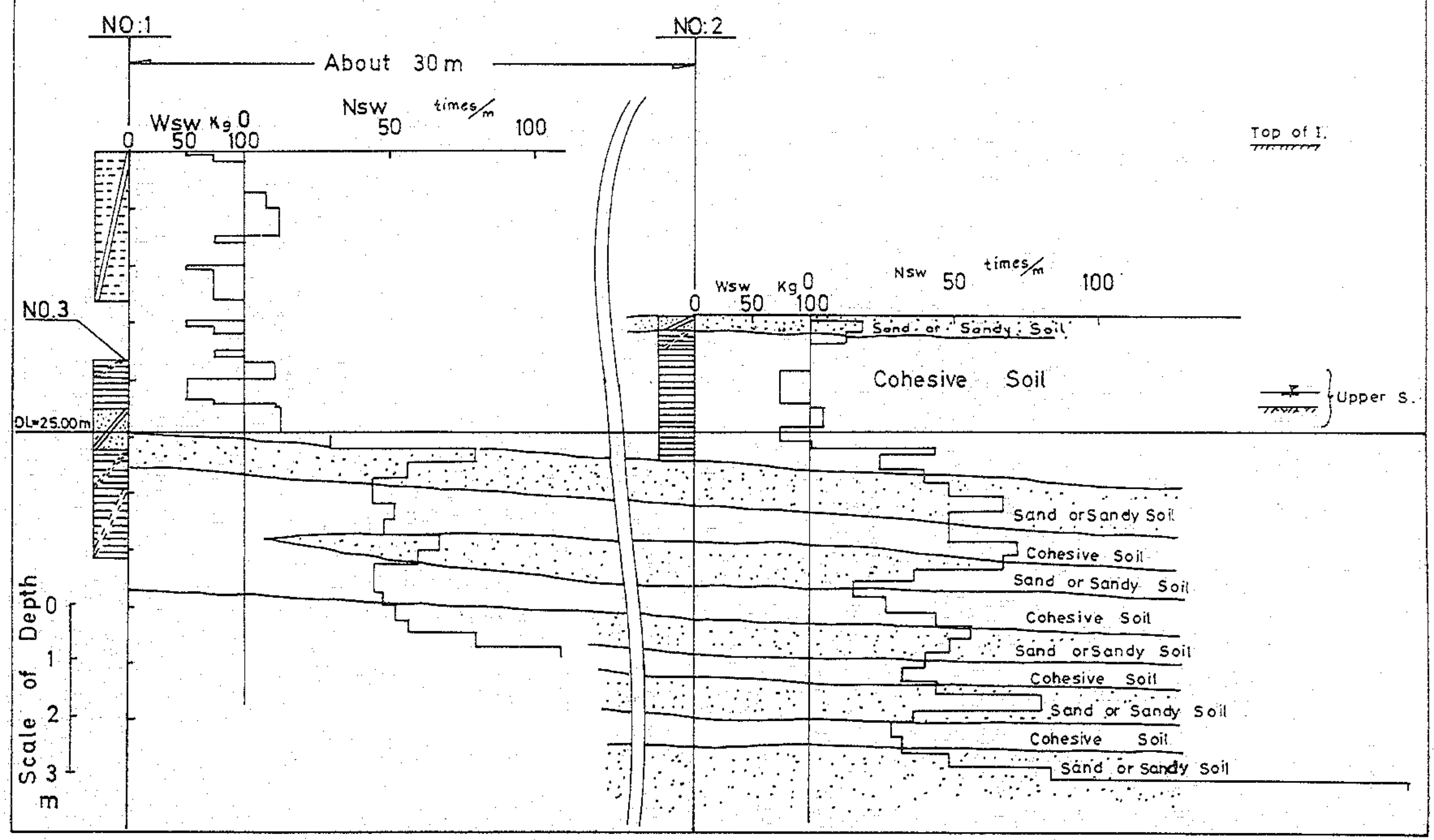


Fig.2-3-31 Soil Profile

Timbang Deli Intake

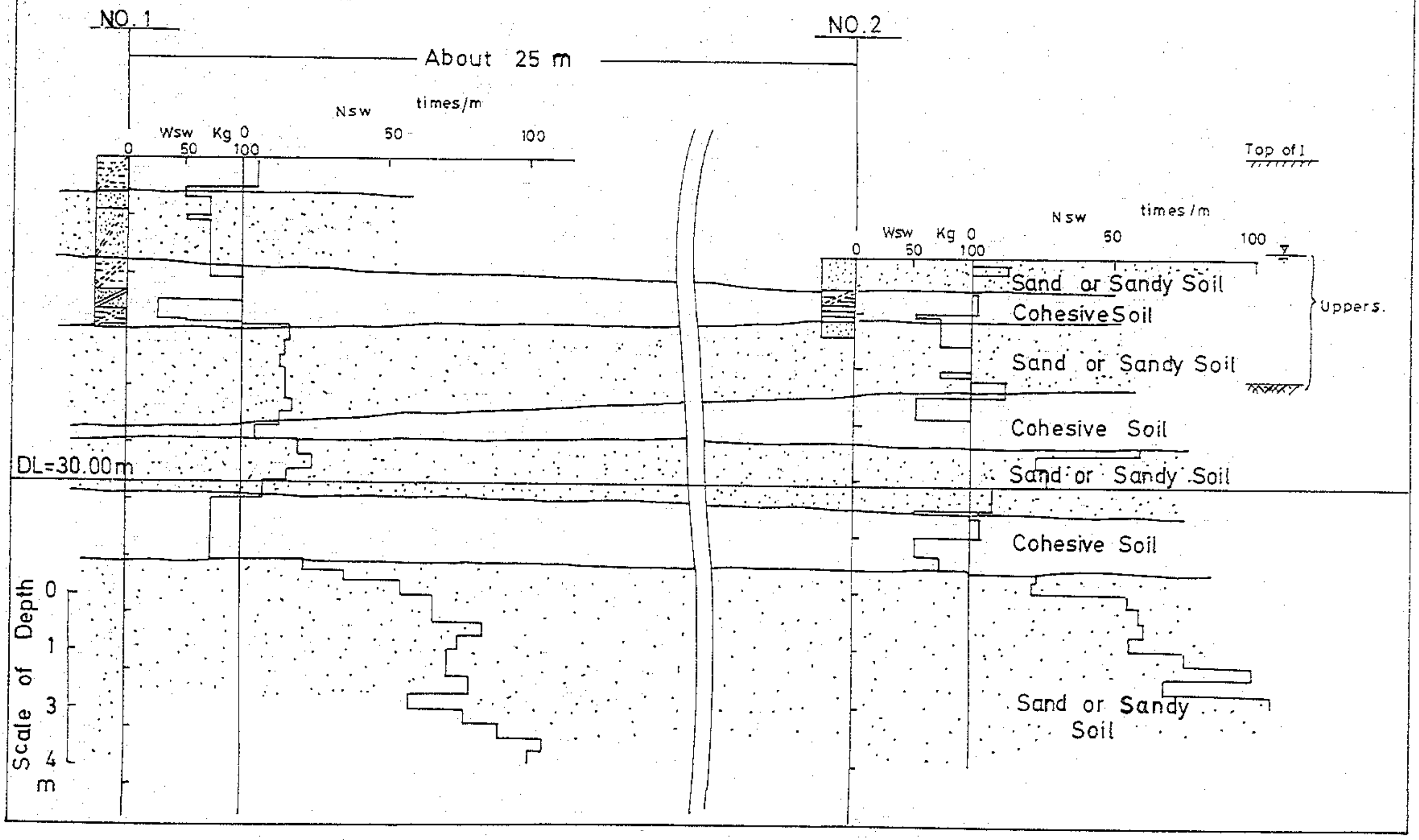


Fig. 2-3 -32 Soil Profile

S. Buluh Intake

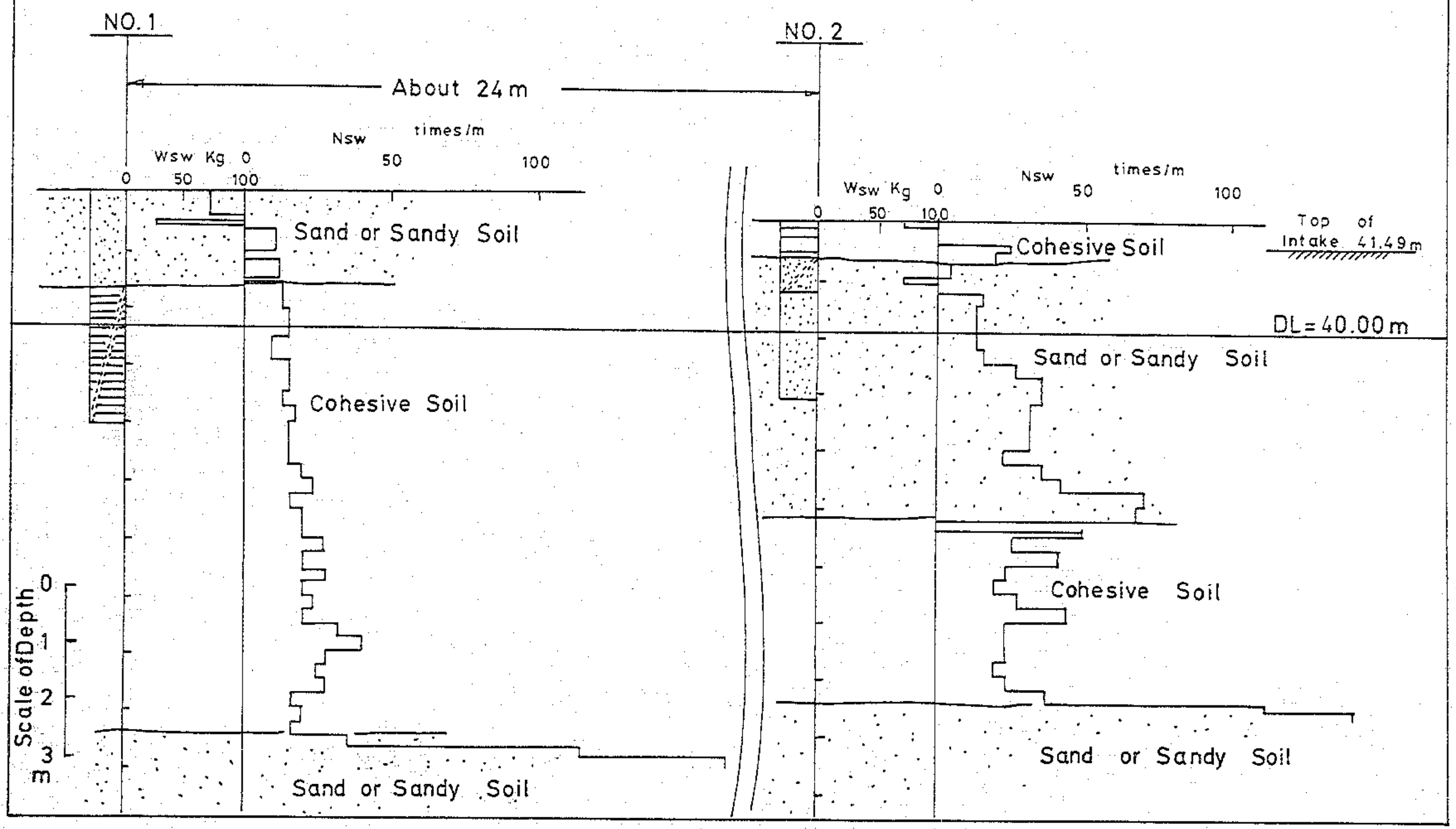


Fig. 2-3-33 Soil Profile

Swadaya Intake

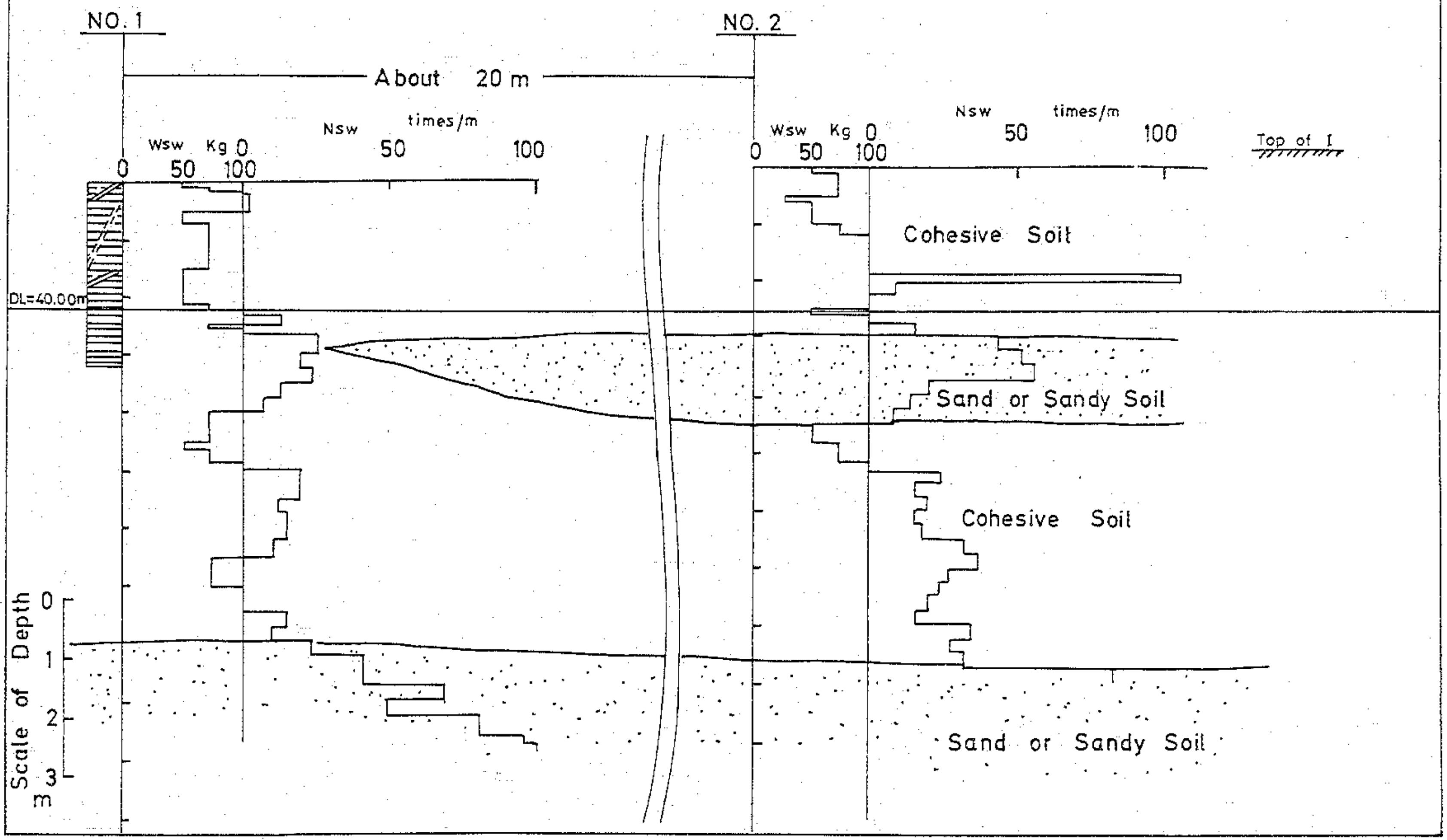


Fig. 2-3-34 Soil Profile

Pulau Gambar Intake

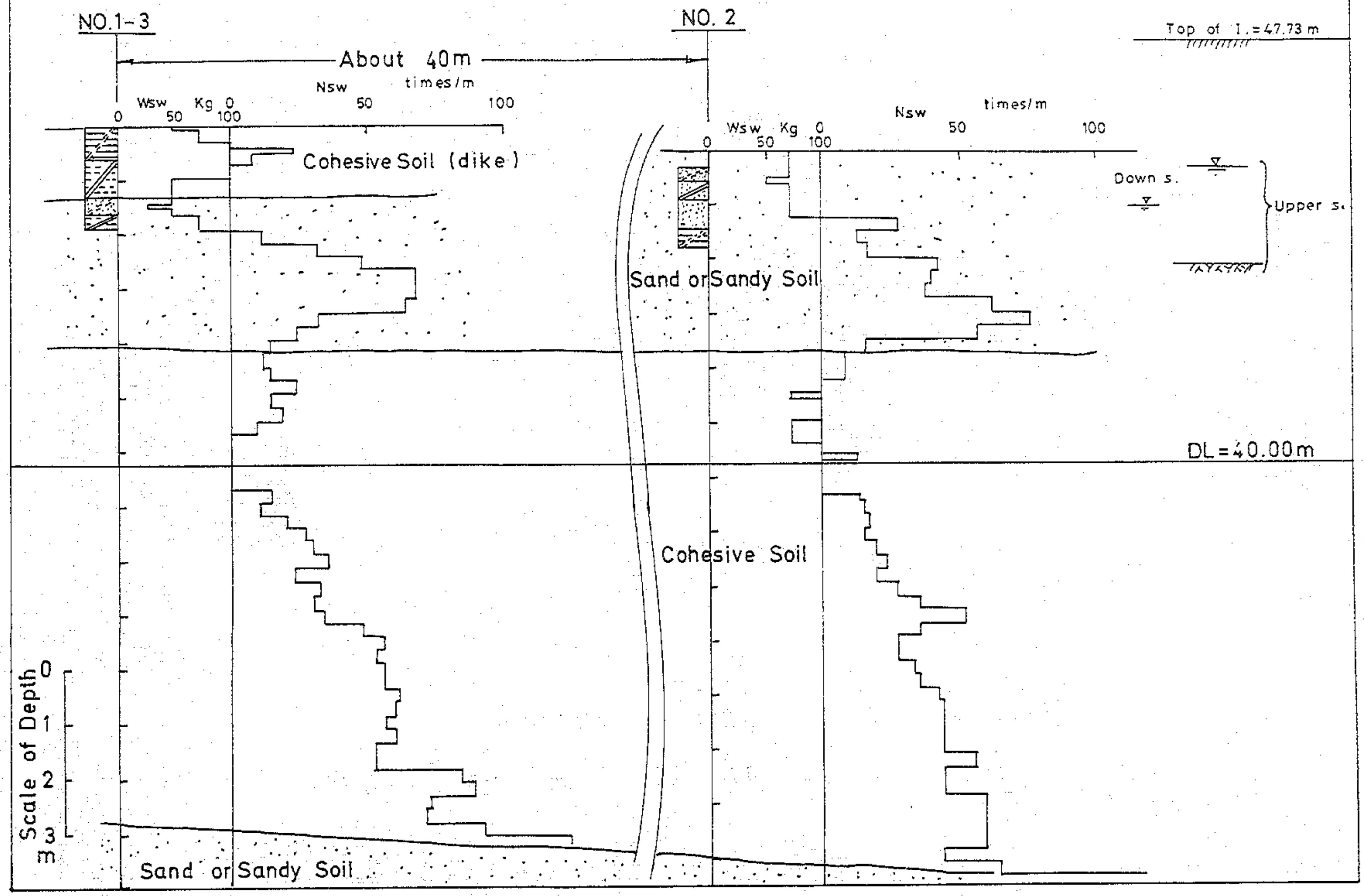


Fig. 2-3-35 Soil Profile Boring Site

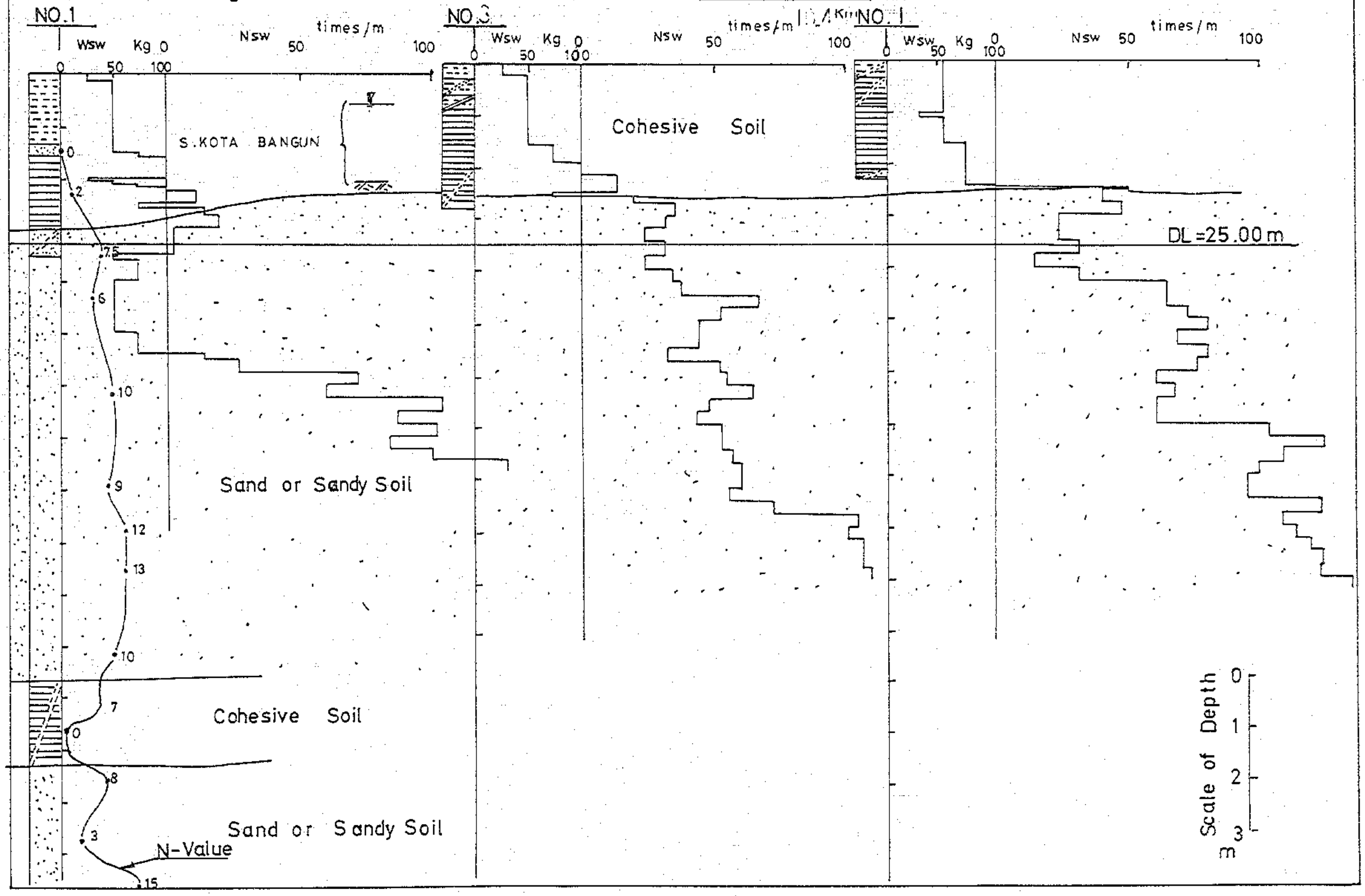


Fig. 2 - 3 - 36 BORING LOG

LOCATION: +10.3 Km Right — ELEVATION: 28.30 m — TERM: FROM 19 Sept. 1977 TO 31 Oct. 1977

BORE HOLE No: 1 — GROUND-WATER LEVEL: 0.6 m — DRILED: A GULTOM — TESTED BY: LOGGED & LABORATORY TEST A. HAKIM LUBIS

Soil profile

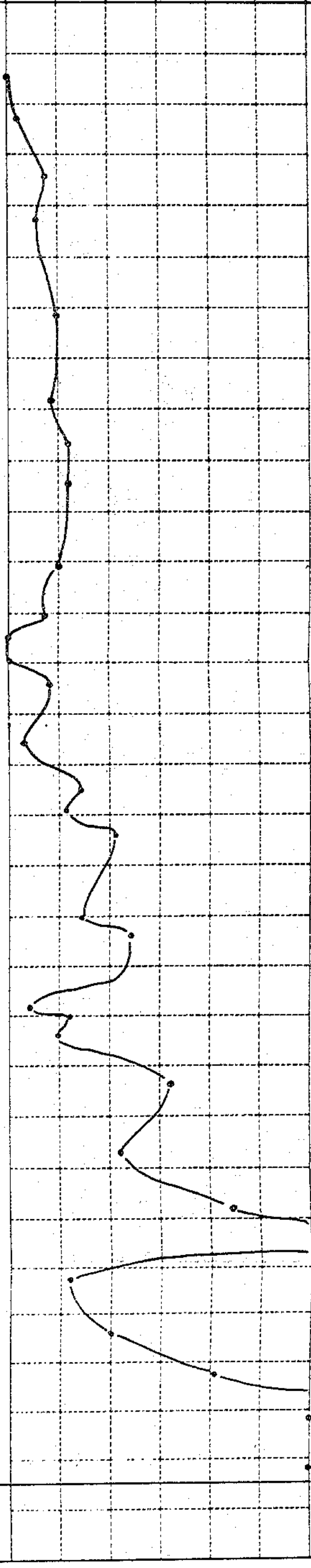
ELEVATION m	DEPTH m	STRATUM m	CLASSIFICATION	COLOR	DESCRIPTION OF MATERIALS & MOISTURE CONTENT Wn
28.30					
26.91	1.39	1.39	silt	brown	Surface soil Wn ≒ 55%
26.72	1.58	0.19	fine sand	gray	
25.30	3.00	1.42	clay	brown	Wn ≒ 55%
24.80	3.50	0.50	silty sand	brown	Wn ≒ 55%
16.75	11.55	8.05	sand	white to brown. mixed black	Wn = 20~30%
15.17	13.13	1.58	silty clay	gray	soft fat clay Wn = 50%. 12.20
8.40	19.90	6.77	sand	white to dark brown	15.70 18.20 Wn = 15~30% 20.00
			mixed fine Sand and Silt	gray Dark brawn	
2.70	25.60	5.70	peat	dark brown to black	Wn = 20~30%···Sand Wn ≒ 70%···Silt Wn ≒ 100%···Peat fitrous peat
0.20	28.10	2.50	sand	greenish gray	Wn = 25~30%
-1.20	29.50	1.40	tuff	light gray	Wn = 25~30%

Standard penetration test

Depth (m): 1.17, 1.76, 2.20, 2.63, 3.40, 3.60, 4.10, 4.40, 6.00, 6.30, 7.77, 8.07, 8.50, 8.80, 9.35, 9.65, 10.95, 11.25, 11.82, 12.15, 12.80, 13.27, 13.57, 14.45, 14.70, 15.32, 15.62, 15.90, 15.21, 16.51, 17.80, 18.10, 18.50, 19.70, 19.90, 20.30, 20.60, 21.20, 21.50, 22.60, 22.90, 23.70, 24.00, 24.45, 24.75, 25.15, 24.45, 26.25, 26.55, 27.15, 27.45, 28.15, 28.45, 29.15, 29.45

Blows by depth T/D: 0/59, 2/43, 5/20 (→ 7.5/30), 6, 10, 9, 12, 12, 10, 7/33, 0/60, 8, 3/25, 15, 11, 21, 15, 24, 3/20 (→ 4.5/30, → 12/30), 10, 32, 22, 45, 66, 12, 20, 41, 128, 107

N — VALUE



SAMPLING: SAMPLE No. / DEPTH m / SAMPLING METHOD

REMARKS:

SIGNS OF SAMPLING METHOD
● : THIN-WALLED SAMPLING
○ : STANDARD PENETRATION TEST
◉ : DENISSON TYPE SAMPLING
⊕ : FOIL SAMPLING
× : OTHERS

Fig.2-4-1 Section for - 12 Km

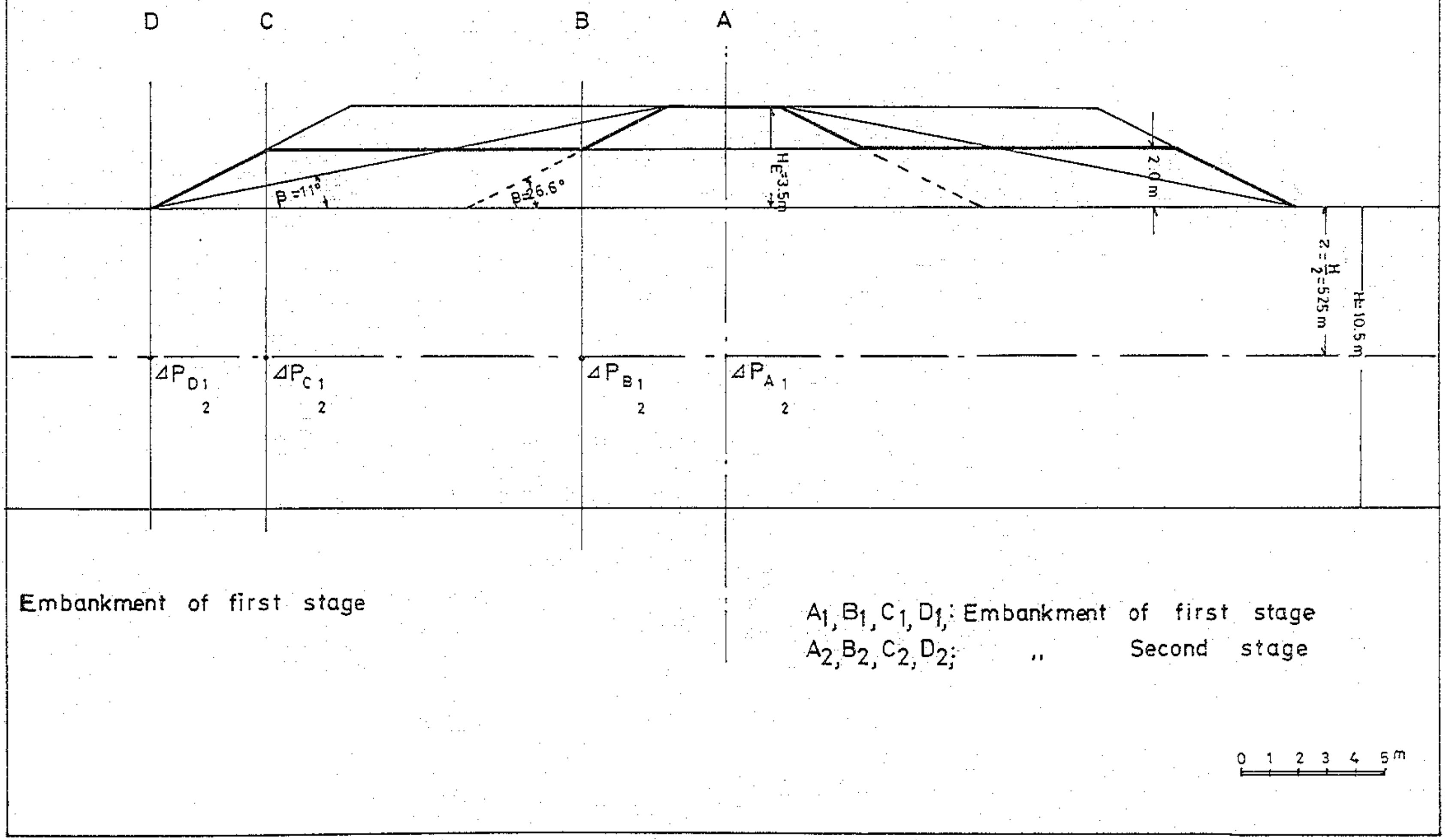


Fig. 2-4-2

Stability Analysis of Slopes in Cohesive Soils (ϕ =0)

by P.W. Tayler

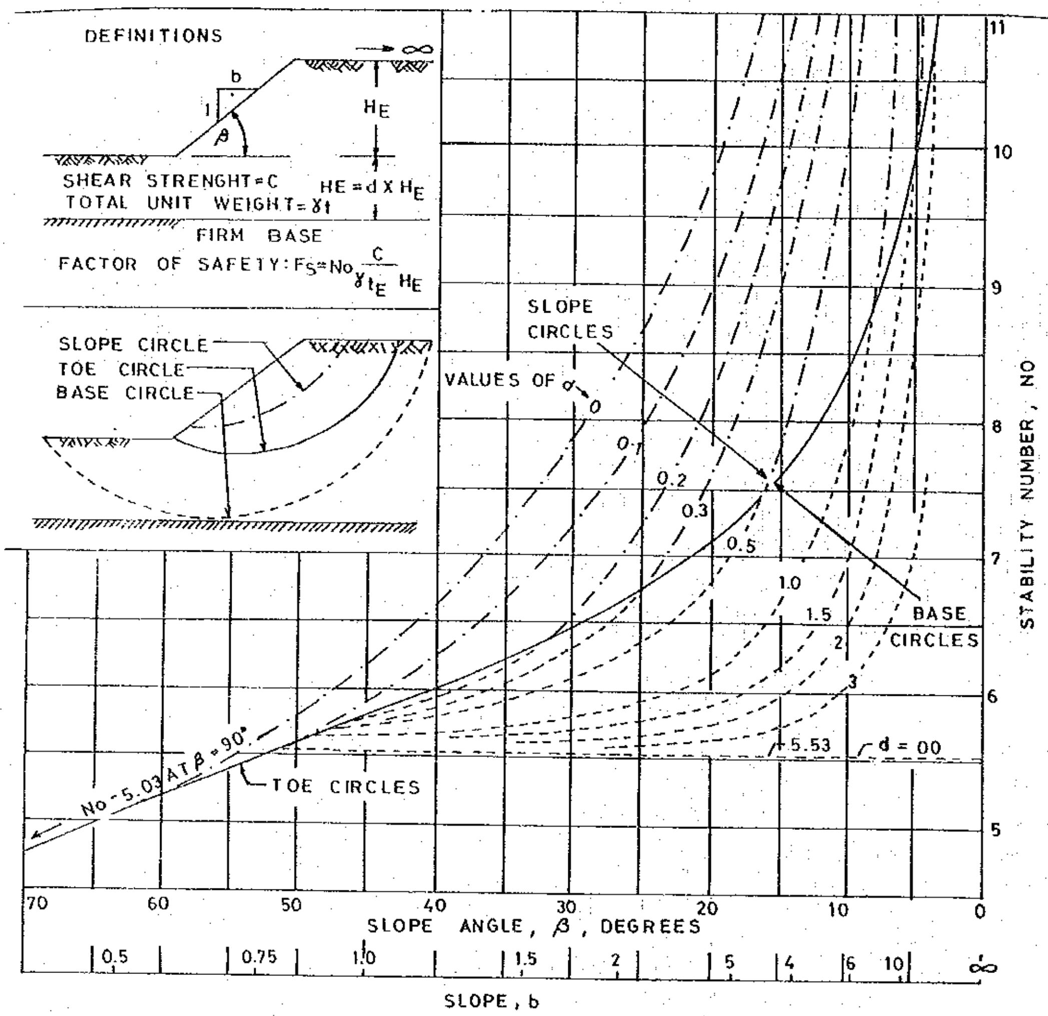


Fig. 2-4-3
Influence Value for Vertical Stress Under Embankment Load of Infinite Length
by J. Osterberg

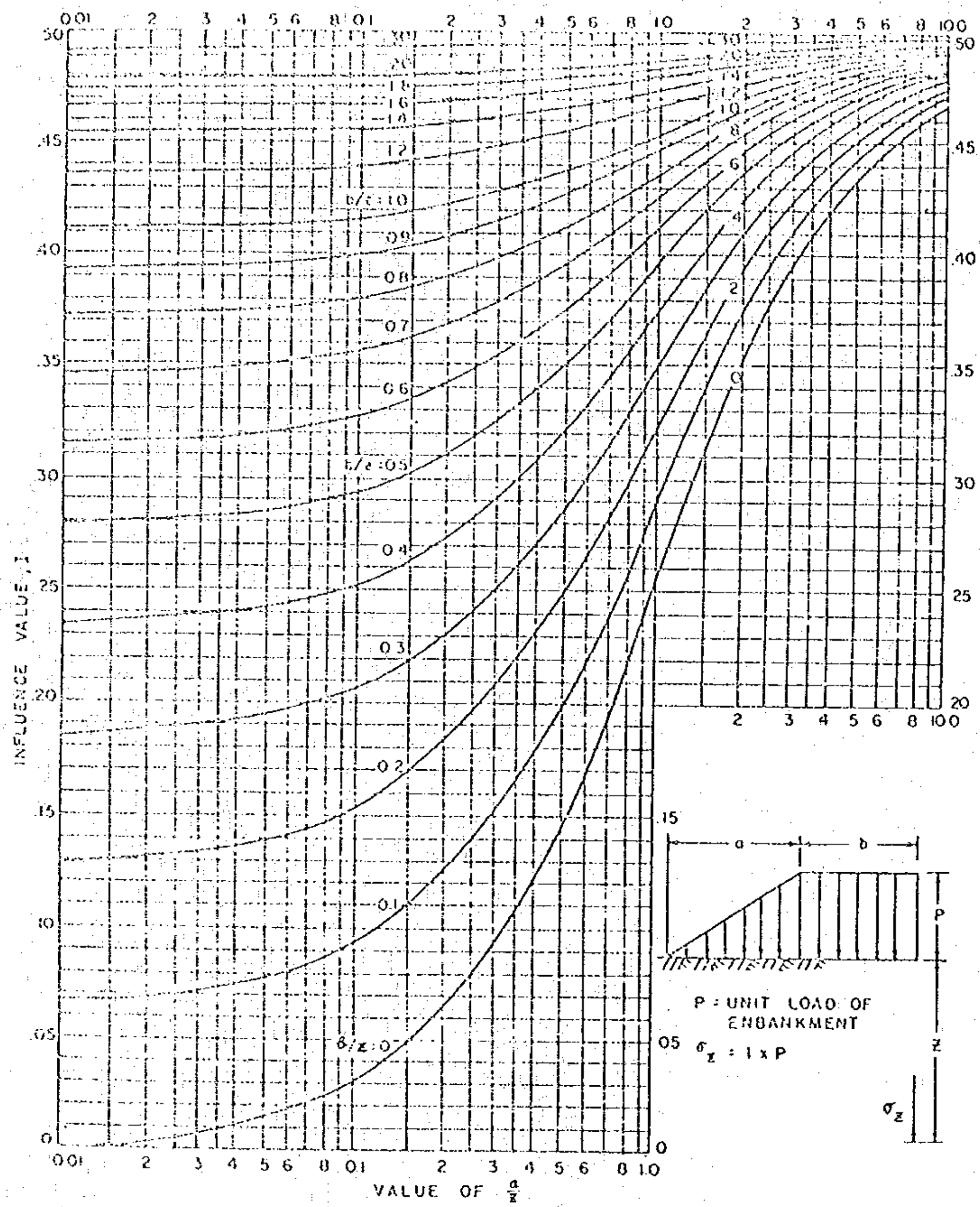


Fig. 2-4-4 Settlement

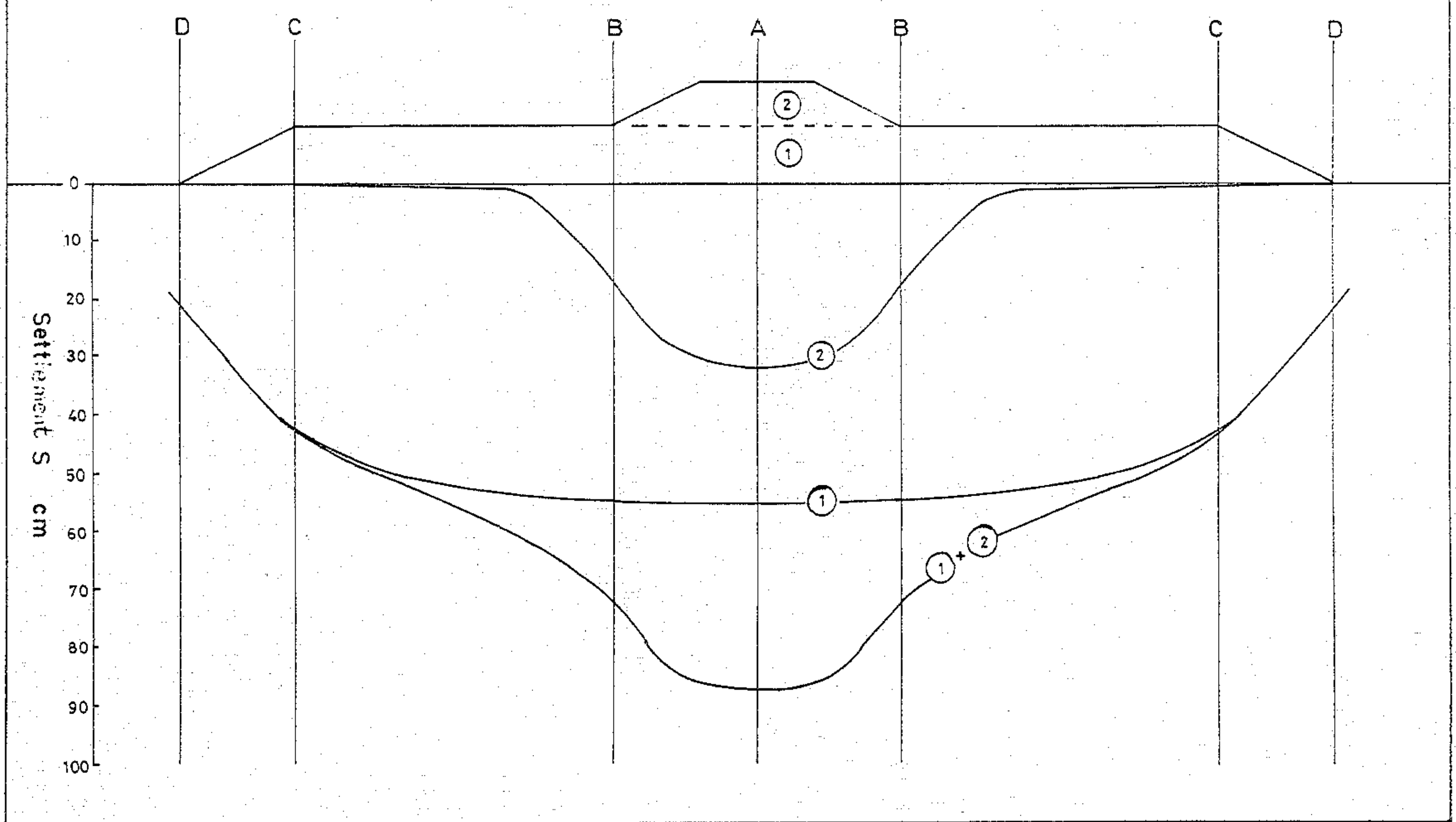


Fig. 2-4-5 Time ~ Settlement

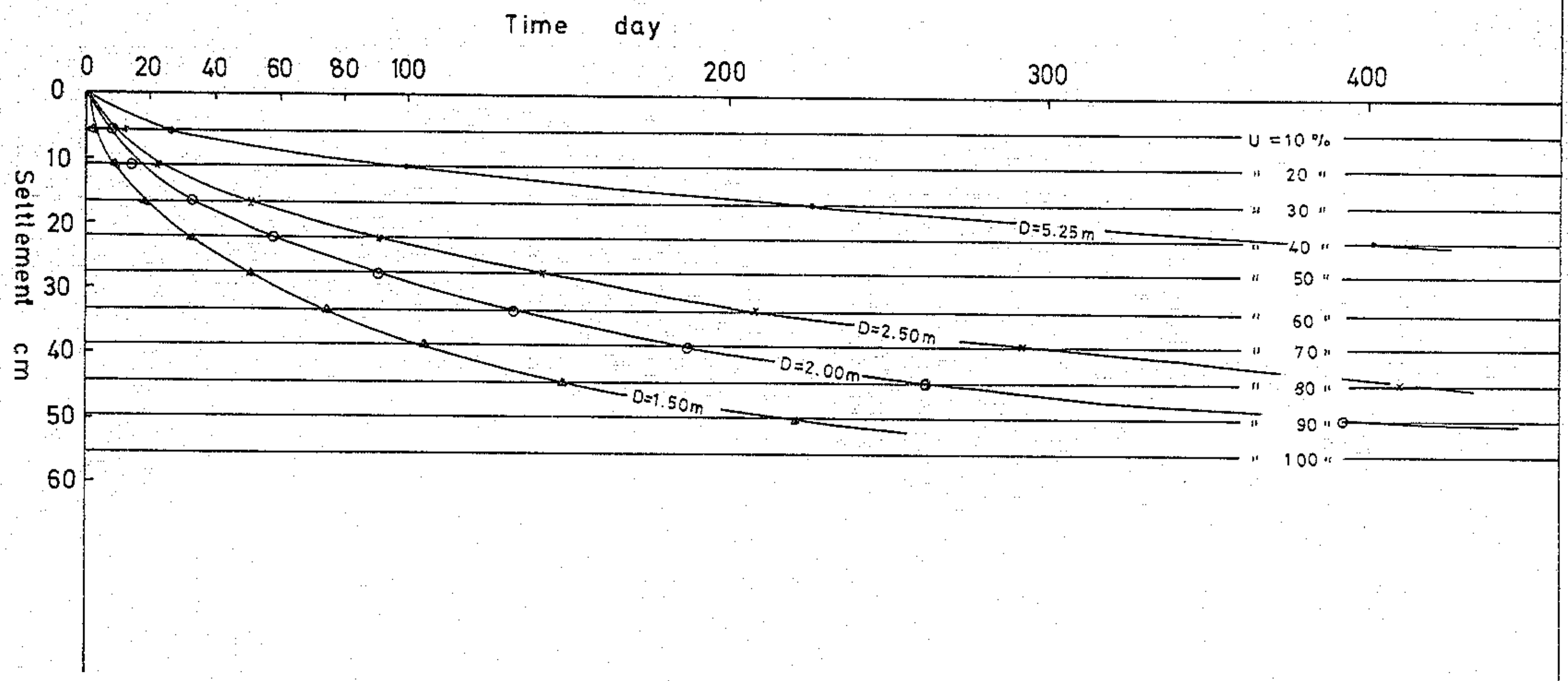


Fig. 2-4-6 Relation between Mean Grain Size and Station Number