3.2 Seismicity

The available record of earthquakes in the Philippines covers the years since 1585. The present analysis, however, utilizes only the records from 1907 to 1976, which have reliable descriptions on approximate locations of the epicenters and magnitudes.

Equations used for the analysis are as follows:

- Kawasumi's formula

$$I_J = M_k - 0.00183\ (d - 100) - 4.605\ \log\frac{d}{100}\ \text{(when } d \geq 100 \text{ km)} \quad .. \ (1)$$

$$I_j = M_k + 4.605\ \log\frac{Do}{D} + 2k\ (D - Do)\ \log e\ \ \text{(when } d < 100 \text{ km)} \quad .. \ (2)$$

where,

I_J: Intensity in JMA (Japan Meteorological Agency) scale

M_k: Magnitude in Kawasumi's scale, that is, JMA intensity at the distance of 100 km from the epicenter. (Magnitude in Richter scale $M = 4.85 + 0.5\ M_k$)

d : Distance from the epicenter (km)

D : Distance from the focus (km)

Do: Distance from the focus to the point of d = 100 km

k : Damping rate of 5-wave (0.0192/km)

The analysis is made in the following procedure.

For all the earthquakes in the record, intensities in JMA scale that could be felt at the damsite are calculated by the Kawasumi's formula.

Frequency of earthquakes in each grade of JMA intensity, within 70 years from 1907 to 1976, is converted into frequency in 100 years, and from the relation between the intensity (I_J) and the cumulative number of frequency (Nc), the expected maximum intensity and hence the expected maximum acceleration in a probable return period of 100 years are obtained.

Earthquake Intensity and Frequency

Intensity (I_j)	Frequency in 70 years	Frequency in 100 years	Cumulative number for 100 years (Nc)
0 (less than 0.6)	28	40.0	124.3
1 (0.6 - 1.5)	32	45.7	84.3
2 (1.6 - 2.5)	13	18.6	38.6
3 (2.6 - 3.5)	12	17.1	20.0
4 (3.6 - 4.5)	2	2.9	2.9
5 (4.6 - 5.5)	0	0	0
6 (5.6 - 6.5)	0	0	0
Total	87	124.3	

Plotting the above on the I_j - log Nc coordinates, and by the minimum square method, the relation between I_j and Nc is given as below.

$$\log Nc = 2.252 - 0.389\ I_j$$

For the case of Nc = 1, the expected maximum intensity in a probable return period of 100 years is obtained as $I_j = 5.8$.

According to Kawasumi, the relation between the intensity I_j and the maximum acceleration of the earthquake motion is very closely approximated by the relation

$$\alpha = 0.45 \times 10^{0.5 I_j}\ \text{(gal)}$$

where α is the geometrical mean value of α as observed empirically. Accordingly, the expected maximum acceleration in a probable return period of 100 years is 357 gal or 0.36 g.

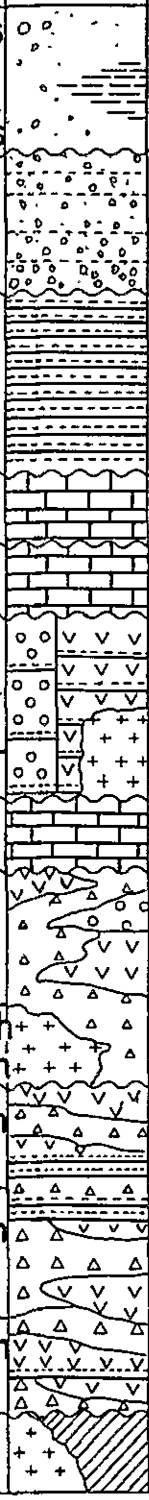
Practically, such a large value is not taken for design, because the maximum acceleration of an earthquake motion will act only for a fraction of a second so that it cannot cause such significant deformation as expected when it is assumed to be a lasting static force.

Most engineers in the United States, who use a pseudo-static method of seismic stability analysis, adopt some empirical value for the design seismic coefficient; generally, this value is in the range of 0.05 to 0.15 (H. Bolton-Seed & G.R. Martin, 1966). Japanese National Committee on Large Dams proposed a design criteria for dams (1971), which prescribes the ground seismicity coefficient to be in the range of 0.12 to 0.20 for concrete and rockfill dame in the seismically active region in the Japanese archipelago where the 100 year acceleration is in the range of 200 to 600 gals. There is no definitive logical basis for selecting those design value but for the experience that numerous dams have been constructed by the use of seismicity coefficients within the said range without any serious damages.

Considering the resemblance of seismic situation between the Philippines and Japan, recommendable design value for the ground seismicity coefficient is 0.15.

However, NPC and NIA requested to adopt the coefficient of 0.20 for the sake of caution in the official minutes made in December 1983. Therefore the upstream and downstream slopes of the Matuno dam were designed by the surface sliding method as shown in Paragraph 5.7.2. But in the stage of the detailed design, it is recommended to make a detailed stability analysis on the oscillation response of the dam by the "Finite Element Method", because such analysis normally clarifies the required slopes steeper than the simple two-dimensional sliding method. It may save the dam volume required. The recent experiments reported that no resonance happened between earthquake and rockfill dam, because the earthquake has shorter vibration cycle of less than 1 second while the rockfill dam has a longer own oscillation cycle of around 2.2 second.

Table 3-1 STRATIGRAPHY OF CARABALLO MOUNTAINS

PERIOD	EPOCH	FORMATIONS	SCHEMATIC COLUMN	ROCK FACIES
Quaternary	Holocene	River deposits Deposits of flood plain. Talus deposits		Sand and gravel, silt and clay, Loose deposits
Quaternary	Pleistocene	Terrace deposits		Sand and gravel, silt and clay, dense deposits
Tertiary	Pliocene	Matuno Formation		Alternation of sandstones and mudstones
Tertiary	Miocene	Aglipay F.		Limestones
Tertiary	Miocene	Macde limestones		Limestones
Tertiary	Miocene	Nat-bang Form / Palali Form. / Patali Batholith		Natbang F : Conglomerate with alternating sandstones & mudstones. Palali F : Basalt and andesite lavas, mudstones, sandstones, pyroclastics. Palali Batholith : Syenite, monzonite.
Tertiary	Oligocene	Santa Fe Formation		Limestones
Tertiary	Oligocene	Mamparang Formation		Andesitic & basaltic lavas & pyroclastics Conglomerate
Tertiary	Eocene	Coastal Batholith Dupax Batholith		Diorite
Tertiary	Eocene	Caraballo Group: Formation III		Andesitic lava & tuff breccia with alternating sandstones, shales, tuffs
Tertiary	Paleocene	Caraballo Group: Formation II		Tuffaceous sandstones & shales Basaltic lava & pyroclastics, dolerites, w/alternating siliceous sandstones & shales.
Tertiary	Paleocene	Caraballo Group: Formation I		Andesitic pyroclastics & lava with alternating sandstones & shales
Cretaceous		Basement complex		Tonalites, schists

Source : Geology and Mineral Resources of the Philippines, 1981, by the Bureau of Mines and Geo-Science.

Table 3-2 RECORD OF EARTHQUAKE

(Continue)

DATE	LATITUDE (DEGREE N)	LONGITUDE (DEGREE E)	DEPTH (km)	DISTANCE FROM SITE (km)	MAGNITUDE	INTENSITY AT SITE I_J
1907, 4. 18	14	123	-	348.6	7.6	2.5
	13.5	123.5	-	426.9	7.4	1.6
1925, 5. 25	12.5	122.5	-	468.4	7.3	1.1
1927, 4. 13	16	120.5	140	79.1	6.75	3.75
	16	120.5	140	79.1	6.25	2.75
4. 19	16	120	100	127.2	6.75	3.3
1928, 6. 15	12.5	121.5	-	442.5	7.0	0.7
8. 5	16	119.5	-	70.8	6.25	3.0
1930, 12. 21	20	122.25	170	417.1	6.9	0.7
1931, 3. 19	18.3	120.2	-	226.6	6.9	2.2
10. 28	17.5	121.5	-	127.2	6.25	2.3
1932, 6. 13	18	119.25	-	264.4	6.25	0.6
6. 14	18.3	120.2	80	226.6	6.5	1.4
	18	120	40	208.4	6.0	0.6
7. 18	14	120	100	296.7	6.25	0.3
8. 24	16.5	120.5	-	61.5	6.25	2.8
1933, 3. 3	15.5	120	120	157.6	6.5	2.3
6. 6	14	120	-	296.7	6.25	0.3
9. 20	13	121	100	384.1	6.5	0.1
1934, 2. 14	17.5	119	-	256.4	7.6	3.3
11. 26	14.2	120.2	-	267.7	6.25	0.5
1937, 3. 16	18	121	100	172.6	6.5	2.1
8. 20	14.5	121.5	-	222.8	7.5	3.5
1938, 5. 23	18	119.5	80	244.0	7.0	2.3
1939, 5. 6	13.5	121.2	110	328.8	6.5	0.5
1940, 4. 28	13.5	120	160	348.6	6.75	0.8
1941, 5. 7	14	123	-	348.6	6.75	0.8
11. 5	12.5	123	100	490.4	6.9	0.2
1942, 4. 8	12.5	120	-	455.0	7.7	2.0

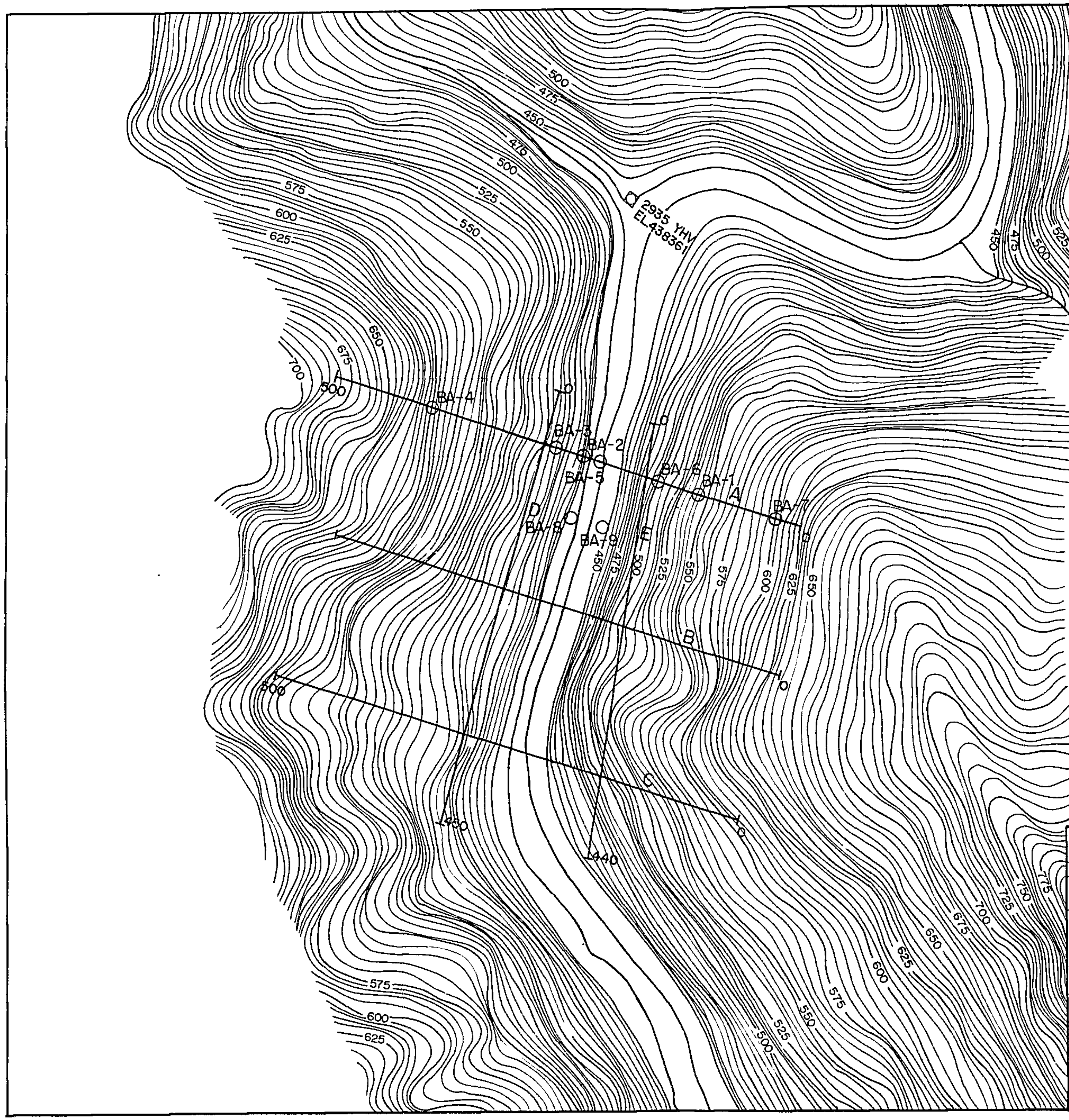
(After UNDP Seismological Programme for Southeast Asia)

(Continue)

DATE	LATITUDE (DEGREE N)	LONGITUDE (DEGREE E)	DEPTH (km)	DISTANCE FROM SITE (km)	MAGNITUDE	INTENSITY AT SITE Ij
1943, 5. 3	12.5	125	-	621.8	7.4	0.5
1948, 3. 3	18.5	119	-	322.7	7.2	1.9
1949, 12. 29	17.5	121.5	-	127.2	7.2	4.2
1950, 1. 3	18	121.5	-	179.7	6.5	2.0
1953, 12. 22	16	119.25	-	206.5	5.75	0.2
1954, 3. 29	19	121	33	283.9	6.5	0.9
7. 2	13	124	60	384.1	6.75	0.6
9. 7	20.7	120.8	-	473.9	6.8	0.1
1956, 2. 12	18.9	119.7	33	311.4	6.5	0.6
2. 14	17	120	-	131.9	6.2	2.1
7. 19	15	120.5	-	172.6	5.75	0.6
10. 27	13.5	120.5	100	334.0	6.75	1.0
10. 28	14	123.5	-	385.7	6.75	0.6
11. 10	16	121	-	50.4	6.0	2.8
1957, 6. 11	18	120.5	44	183.1	6.7	2.3
1959, 7. 18	15.5	120.5	150	122.2	6.6	3.1
1960, 9. 19	16	120	25	127.2	5.5	0.8
1961, 2. 26	16.1	121.6	32	72.6	6.1	2.6
1962, 6. 23	17.1	121.4	-	82.2	6.3	3.0
6. 30	16.4	122.3	-	139.3	5.75	1.1
12. 21	15.9	121.8	-	103.5	5.0	0.2
1963, 3. 15	16.75	121	-	33.9	4.75	0.7
5. 17	15.7	120.1	99	134.7	5.6	0.8
1966, 2. 3	16.7	119.9	71	131.0	5.3	0.3
1967, 10. 17	17.3	121.8	41	126.2	5.5	0.8
1968, 8. 1	16.3	122.1	31	118.1	5.9	1.7
	15.8	121.8	33	110.5	5.0	0.1
8. 3	16.5	122.3	52	139.3	6.1	1.8
8. 6	15.7	122.0	48	134.7	5.3	0.2

(Continue)

DATE		LATITUDE (DEGREE N)	LONGITUDE (DEGREE E)	DEPTH (km)	DISTANCE FROM SITE (km)	MAGNITUDE	INTENSITY AT SITE Ij
1968,	8. 14	15.1	122.5	15	171.9	5.5	0.1
	8. 28	15.6	122.0	42	141.9	5.7	0.9
	8. 29	15.9	121.75	50	99.1	5.3	0.9
	9. 22	15.7	121.9	47	126.2	5.3	0.4
	11. 2	16.1	121.9	41	102.3	5.0	0.3
	11. 22	16.2	122.2	60	131.0	5.3	0.3
1969,	5. 15	16.0	121.9	62	107.1	5.0	0.1
1970,	4. 7	15.8	121.7	40	102.3	6.5	3.3
		15.5	122.4	47	183.8	5.6	0.1
	4. 8	15.59	121.72	31	121.4	5.3	0.5
		15.4	121.75	31	140.5	5.7	0.9
	4. 12	15.08	122.01	25	186.2	5.8	0.5
1971,	7. 2	15.9	120.3	60	103.5	5.0	0.2
	7. 4	15.6	121.9	20	133.8	5.6	0.9
		15.6	121.85	50	129.9	5.5	0.7
	7. 20	15.26	120.26	33	159.0	5.4	0.1
	10. 27	17.3	120.4	50	119.1	5.5	0.9
1972,	2. 29	17.8	120.6	33	158.4	5.5	0.3
	3. 16	16.2	121.6	25	67.3	5.4	1.3
	4. 25	13.4	120.5	-	345.0	7.0	1.4
	5. 22	16.6	122.3	-	140.1	6.5	2.6
1973,	8. 23	16.4	122.0	64	105.9	5.1	0.4
1974,	2. 9	16.2	120.1	-	109.3	5.5	1.1
	7. 6	16.4	120.7	0	39.4	4.5	0.2
1975,	4. 3	16.9	120.5	80	79.1	5.5	1.3
	10. 31	17	121	-	61.5	5.3	1.2
1976,	2. 13	15.67	121.70	47	113.0	5.4	0.8
	5. 28	16.97	120.50	95	84.3	4.9	0.1
	9. 22	15	120	-	199.3	5.8	0.3



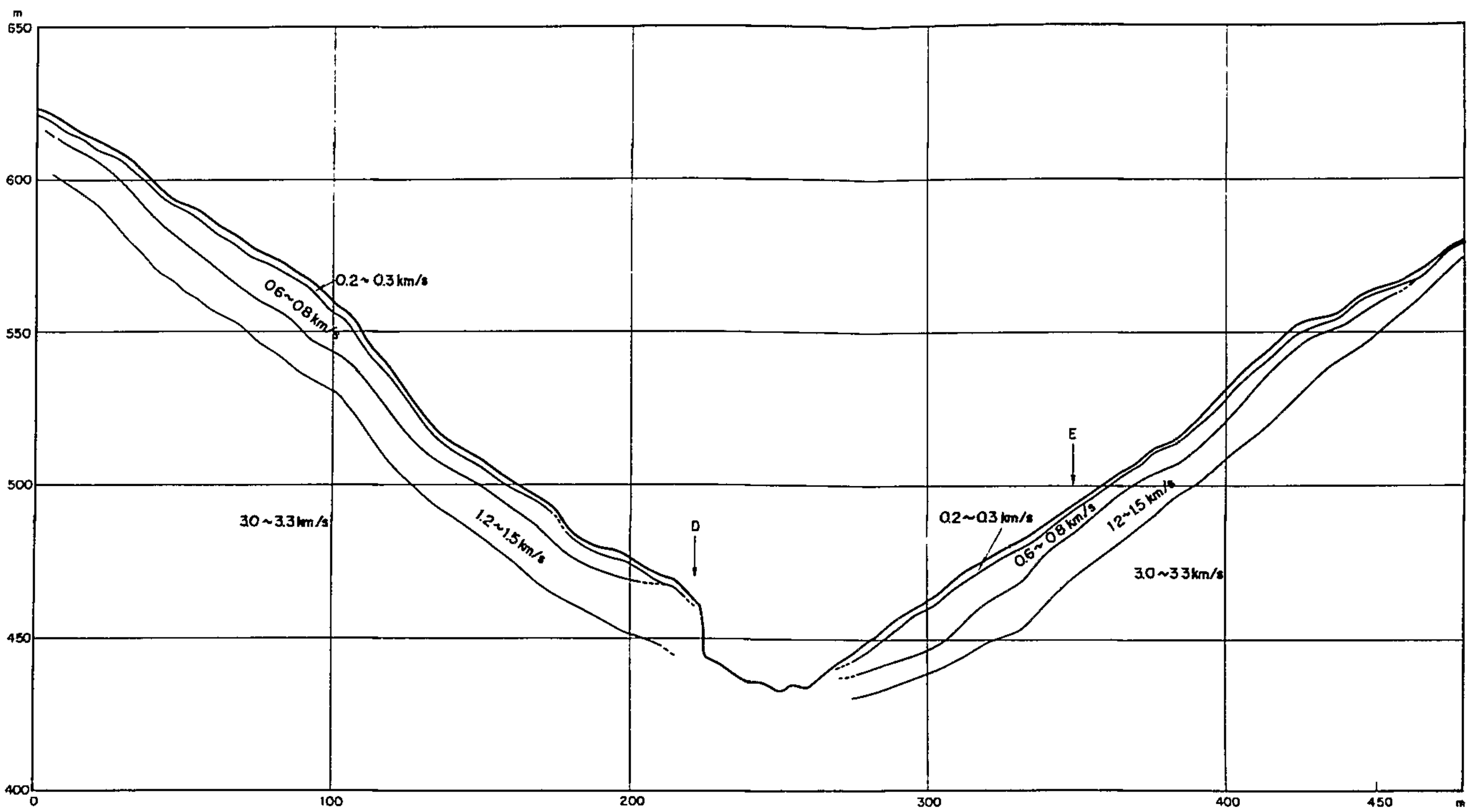
REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION.

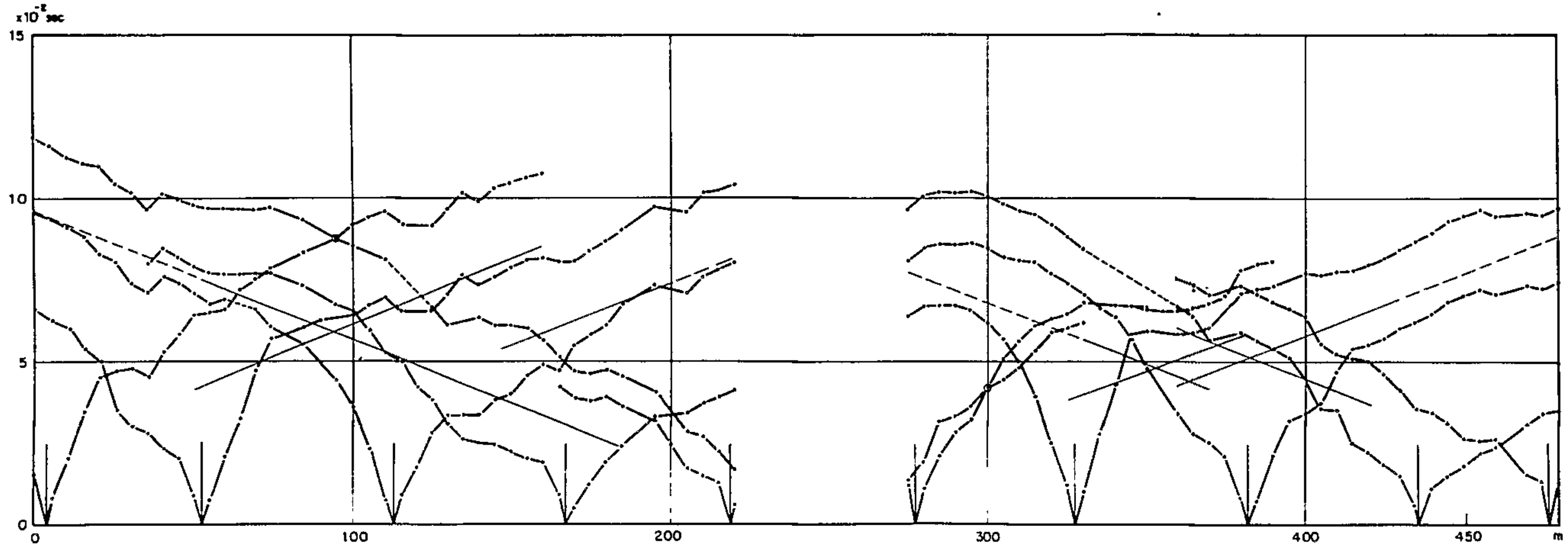
MATUNO RIVER DEVELOPMENT PROJECT

LOCATION MAP OF
SEISMIC LINE & BORING POINT
A DAMSITE

DATE		FIG. No	3 - 1

JAPAN INTERNATIONAL COOPERATION AGENCY





LEGEND

0.2 ~ 0.3 km/s	Top soil
0.6 ~ 0.8 km/s	Talus deposit or Decomposed rock
1.2 ~ 1.5 km/s	Weathered rock
3.0 ~ 3.3 km/s	Fresh rock

SCALE

0 50 100m

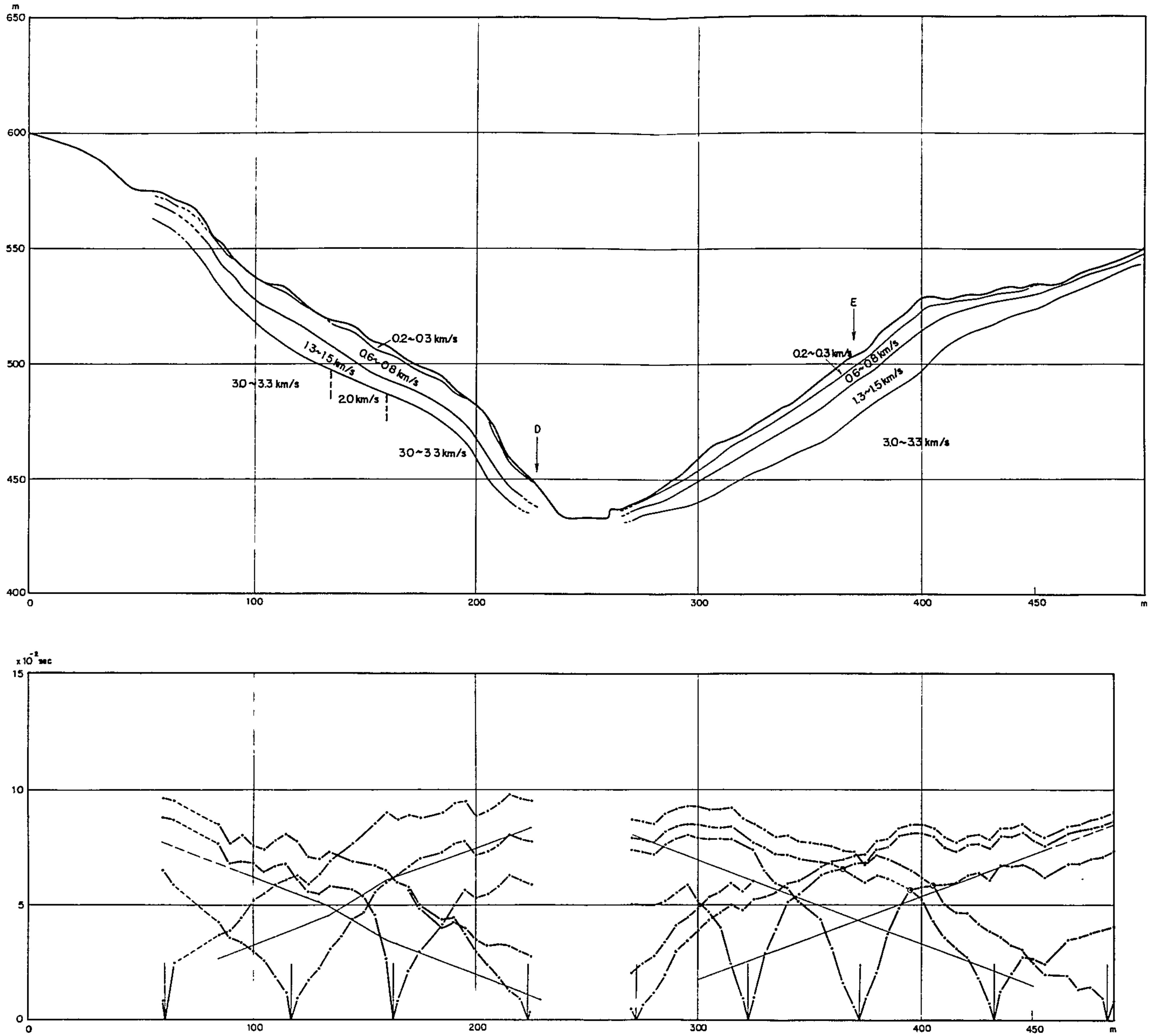
REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

MATUNO RIVER DEVELOPMENT PROJECT

SEISMIC EXPLORATION
TIME-DISTANCE CURVE & VELOCITY LAYER SECTION
LINE B A DAM SITE

DATE		FIG. NO.	3 - 2

JAPAN INTERNATIONAL COOPERATION AGENCY



LEGEND

0.2 ~ 0.3 km/s	Top soil
0.6 ~ 0.8 km/s	Talus deposit or Decomposed rock
1.2 ~ 1.5 km/s	Weathered rock
3.0 ~ 3.3 km/s	Fresh rock

SCALE

0 50 100m

REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

MATUNO RIVER DEVELOPMENT PROJECT

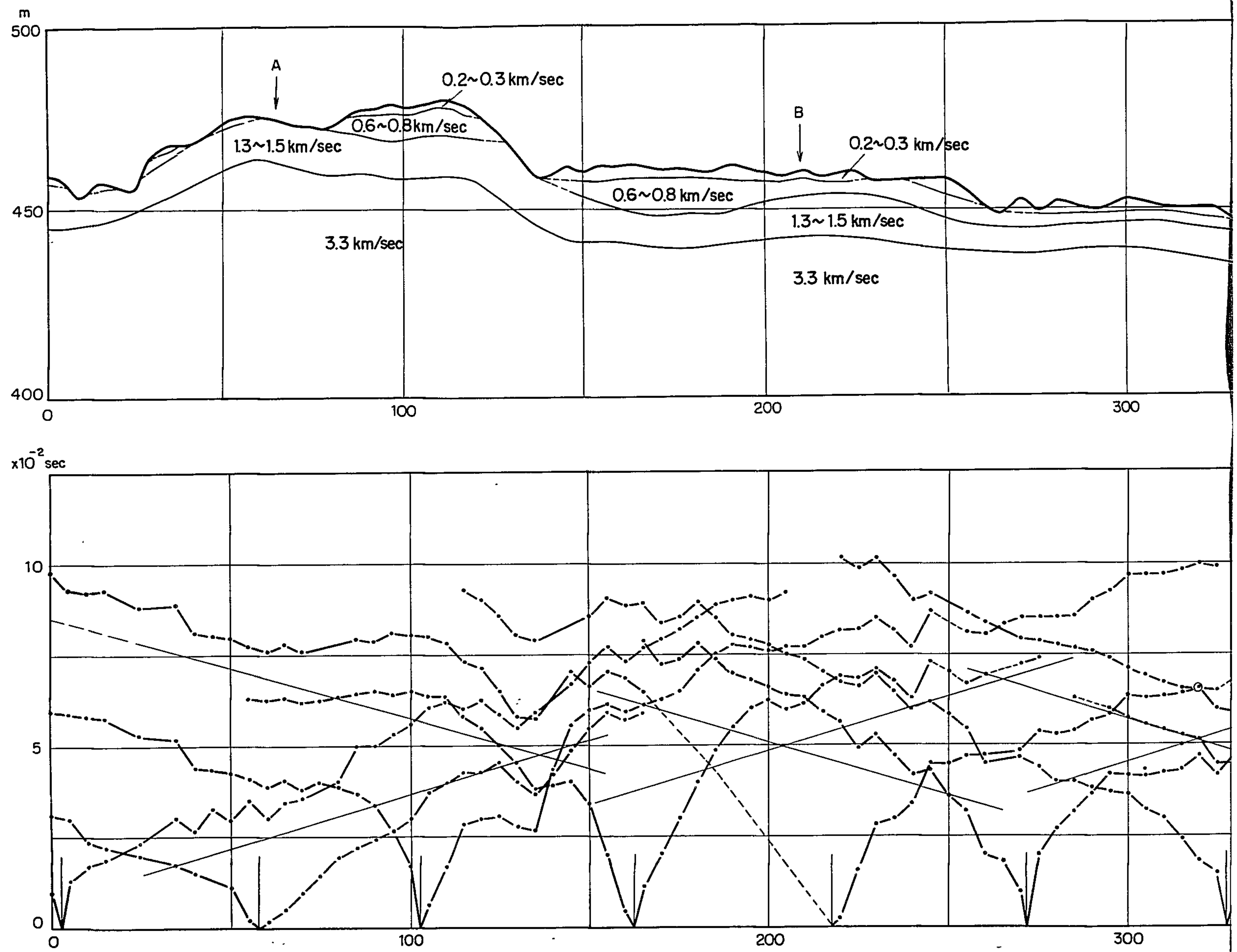
SEISMIC EXPLORATION

TIME-DISTANCE CURVE &
VELOCITY LAYER SECTION

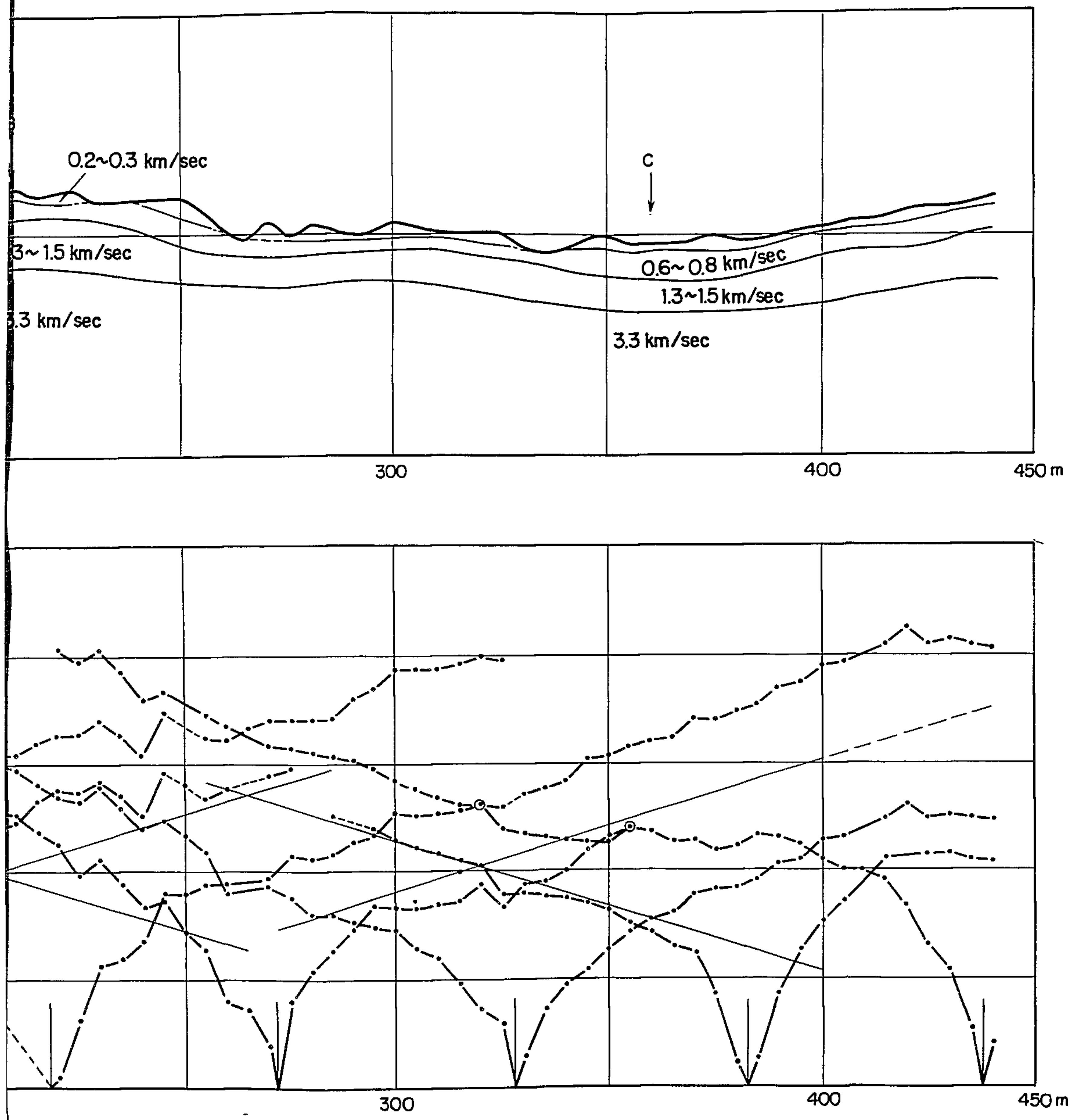
LINE C A DAM SITE

DATE		FIG. NO	3-3

JAPAN INTERNATIONAL COOPERATION AGENCY



m
500
450
400
A
B
0.2~0.3 km/sec
0.6~0.8 km/sec
1.3~1.5 km/sec
3.3 km/sec
0.2~0.3 km/sec
0.6~0.8 km/sec
1.3~1.5 km/sec
3.3 km/sec
0
100
200
300
x10^-2 sec
10
5
0
0
100
200
300



LEGEND

0.2 ~ 0.3 km/s	Top Soil
0.6 ~ 0.8 km/s	Tarrace deposit or Decomposed rock
1.3 ~ 1.5 km/s	Weathered rock
3.3 km/s	Fresh rock

SCALE

0 50 100m

REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

MATUNO RIVER DEVELOPMENT PROJECT

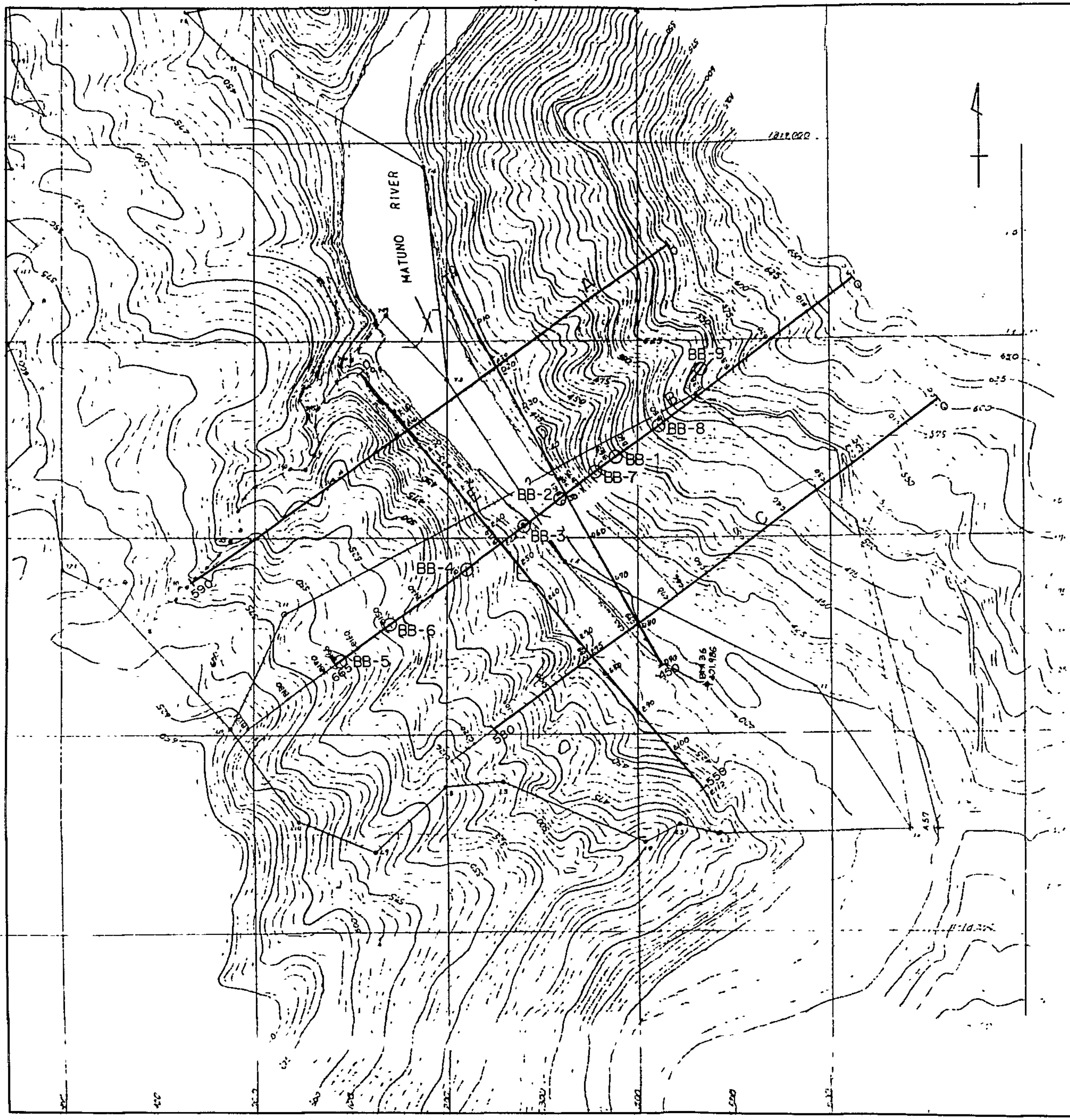
SEISMIC EXPLORATION

TIME-DISTANCE CURVE & VELOCITY LAYER SECTION

LINE-D A DAM SITE

DATE		FIG. NO.	3-4

JAPAN INTERNATIONAL COOPERATION AGENCY



LEGEND

SEISMIC EXPLORATION LINE

o BORING POINT

SCALE

0 100 200 300m

REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

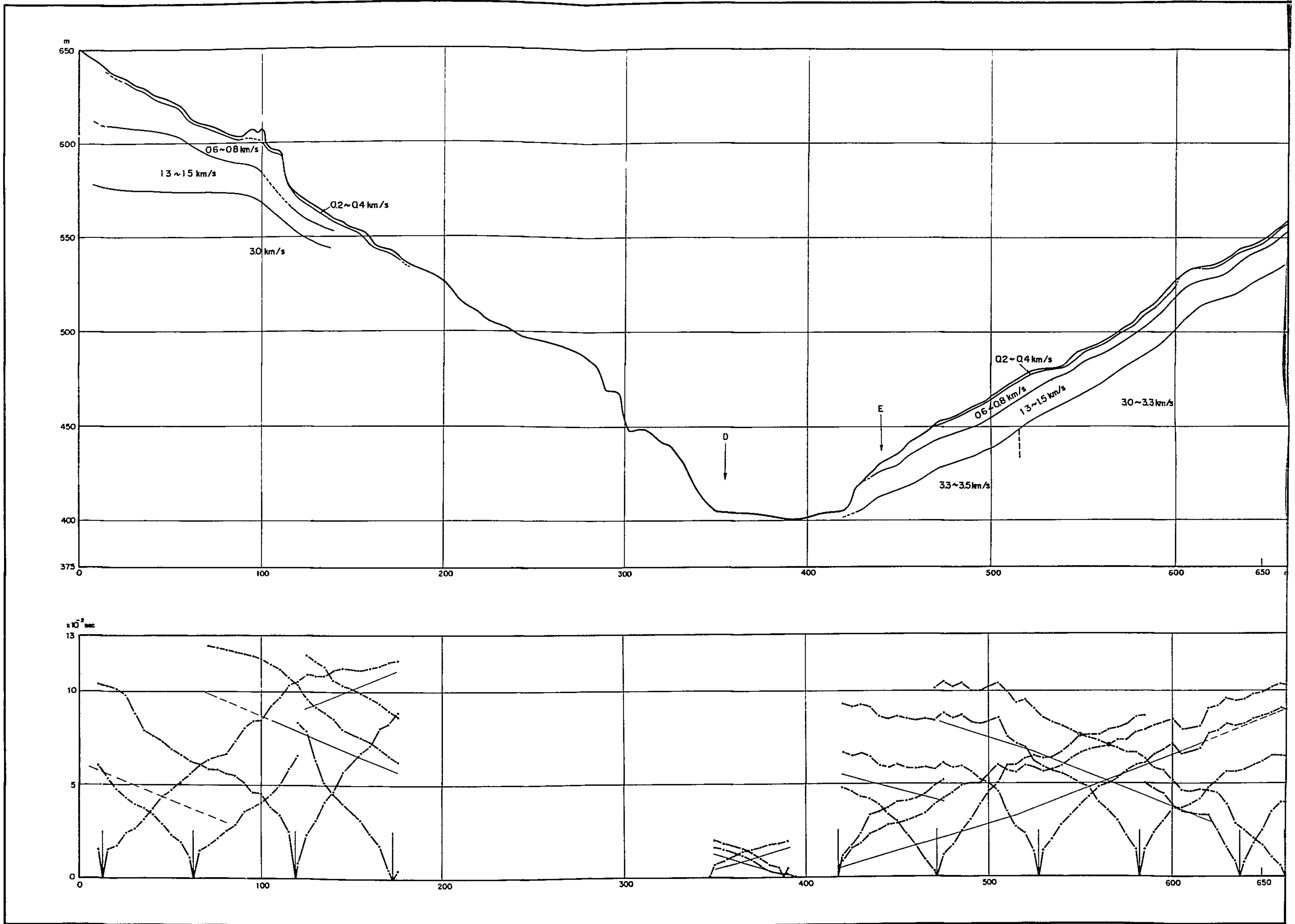
MATUNO RIVER DEVELOPMENT PROJECT

LOCATION MAP OF SEISMIC LINE & BORIN POINT

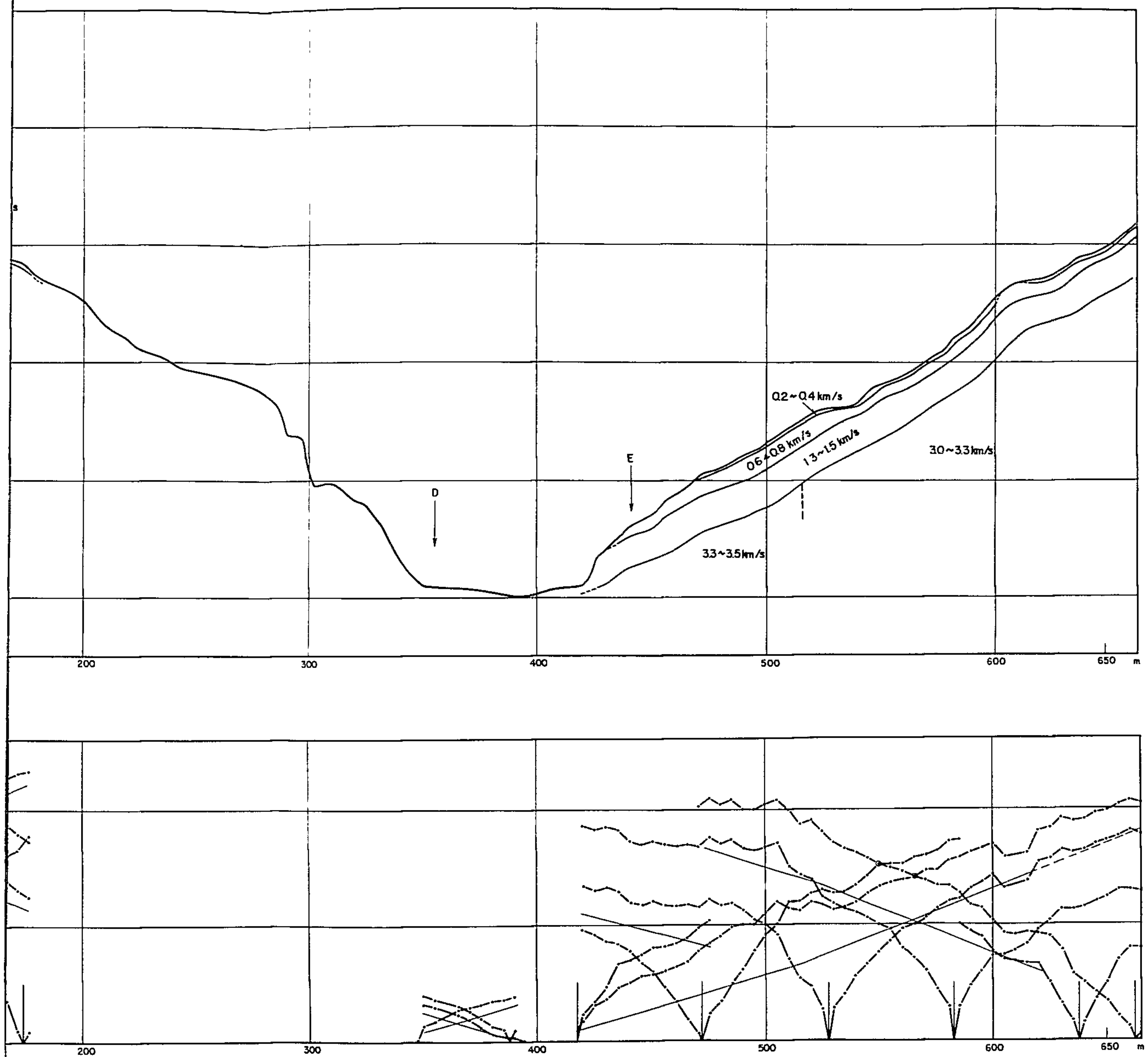
B1 DAMSITE

DATE		FIG. NO.	3-5

JAPAN INTERNATIONAL COOPERATION AGENCY



m
650
600
550
500
450
400
375
0.6~0.8 km/s
1.3~1.5 km/s
0.2~0.4 km/s
3.0 km/s
D
E
0.2~0.4 km/s
0.6~0.8 km/s
1.3~1.5 km/s
3.0~3.3 km/s
3.3~3.5 km/s
0
100
200
300
400
500
600
650
$\times 10^{-2}$ sec
13
10
5
0



LEGEND

0.2 ~ 0.3 km/s	Top soil
0.6 ~ 0.8 km/s	Talus deposit or Decomposed rock
1.3 ~ 1.5 km/s	Weathered rock
3.0 ~ 3.3 km/s	Fresh rock

SCALE

0 50 100m

REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

MATUNO RIVER DEVELOPMENT PROJECT

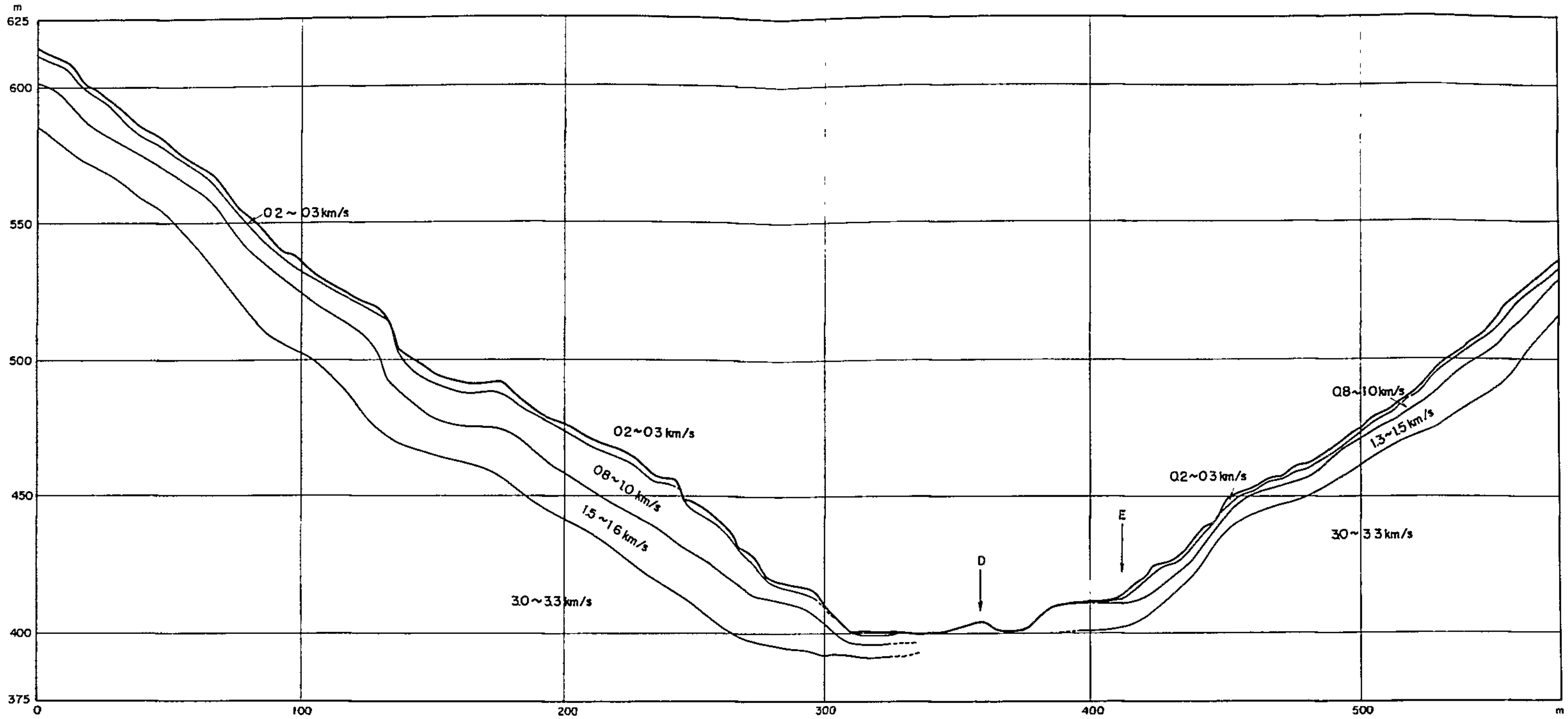
SEISMIC EXPLORATION

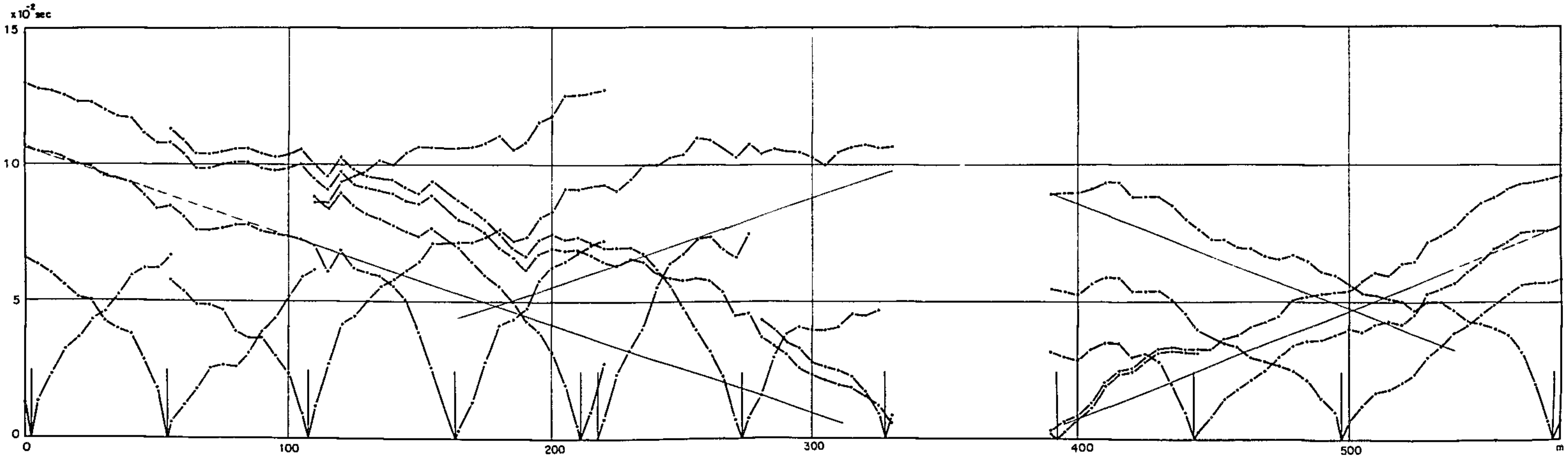
TIME-DISTANCE CURVE & VELOCITY LAYER SECTION

LINE B B1 DAM SITE

DATE		FIG. NO.	3-6

JAPAN INTERNATIONAL COOPERATION AGENCY





LEGEN

0.2~0.3km/s

0.6~0.8km/s

1.3~1.5km/s

3.0~3.3km/s

S

0

REPUBLIC OF
NATIONAL IRRIGA

MATUNO RIVER DE

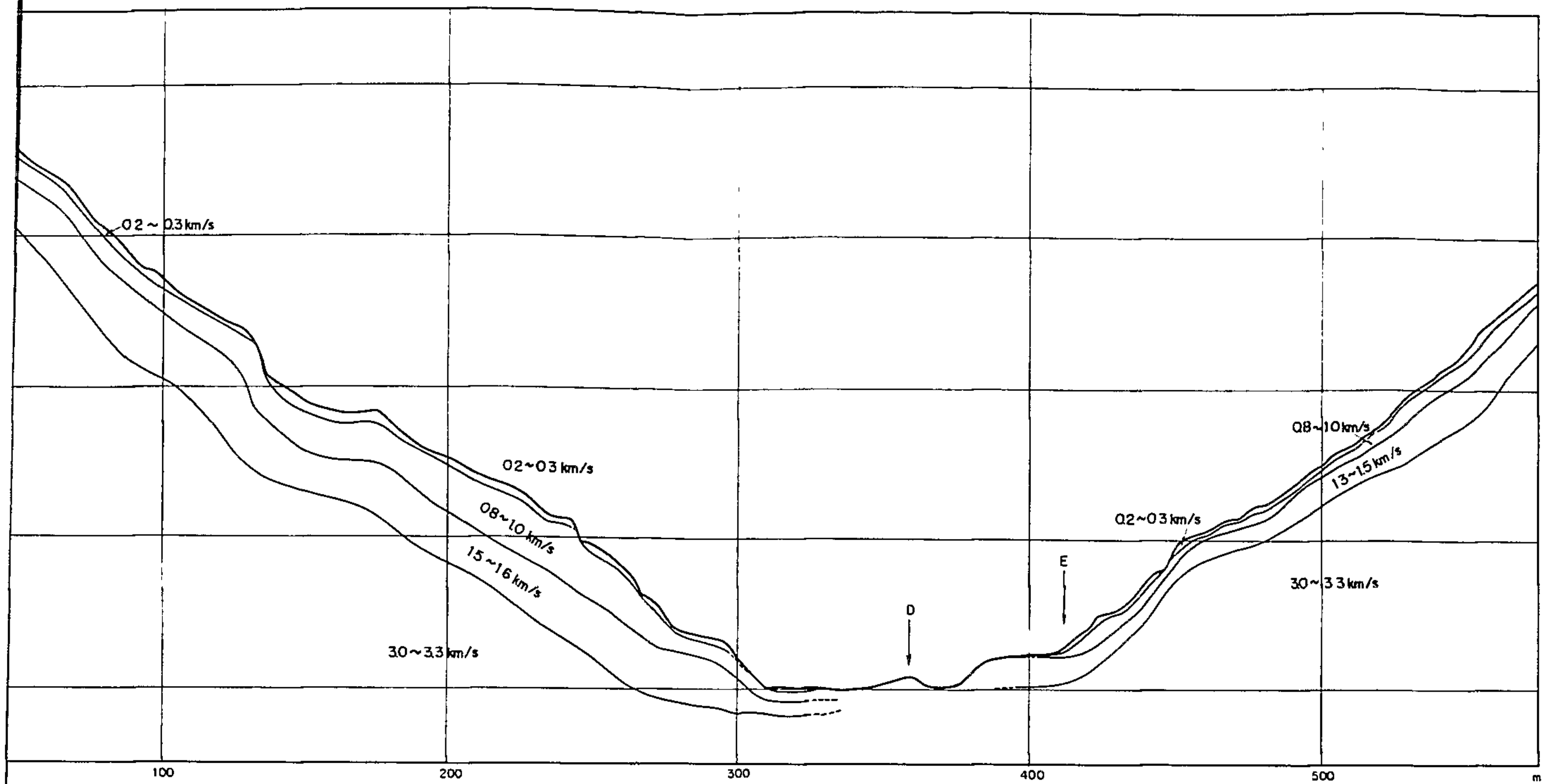
SEISMIC

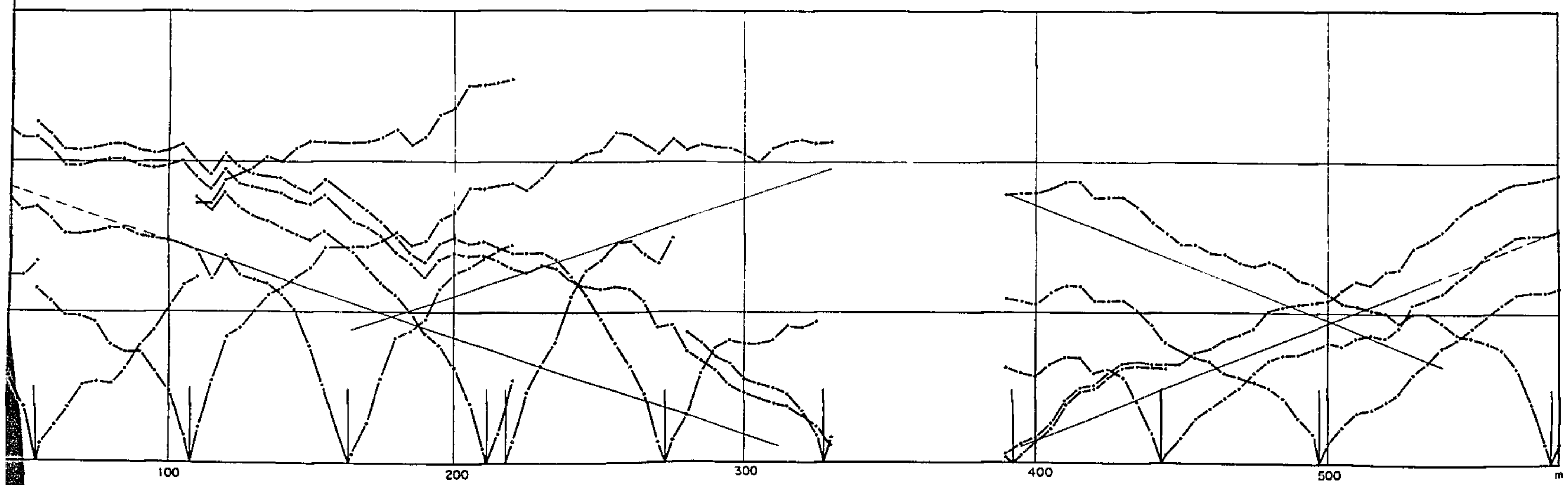
TIME-DISTANCE
VELOCIT

LINE C

DATE

JAPAN INTERNATONA





LEGEND

0.2 ~ 0.3 km/s	Top soil
0.6 ~ 0.8 km/s	Talus deposit or Decomposed rock
1.3 ~ 1.5 km/s	Weathered rock
3.0 ~ 3.3 km/s	Fresh rock

SCALE

0 50 100m

REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

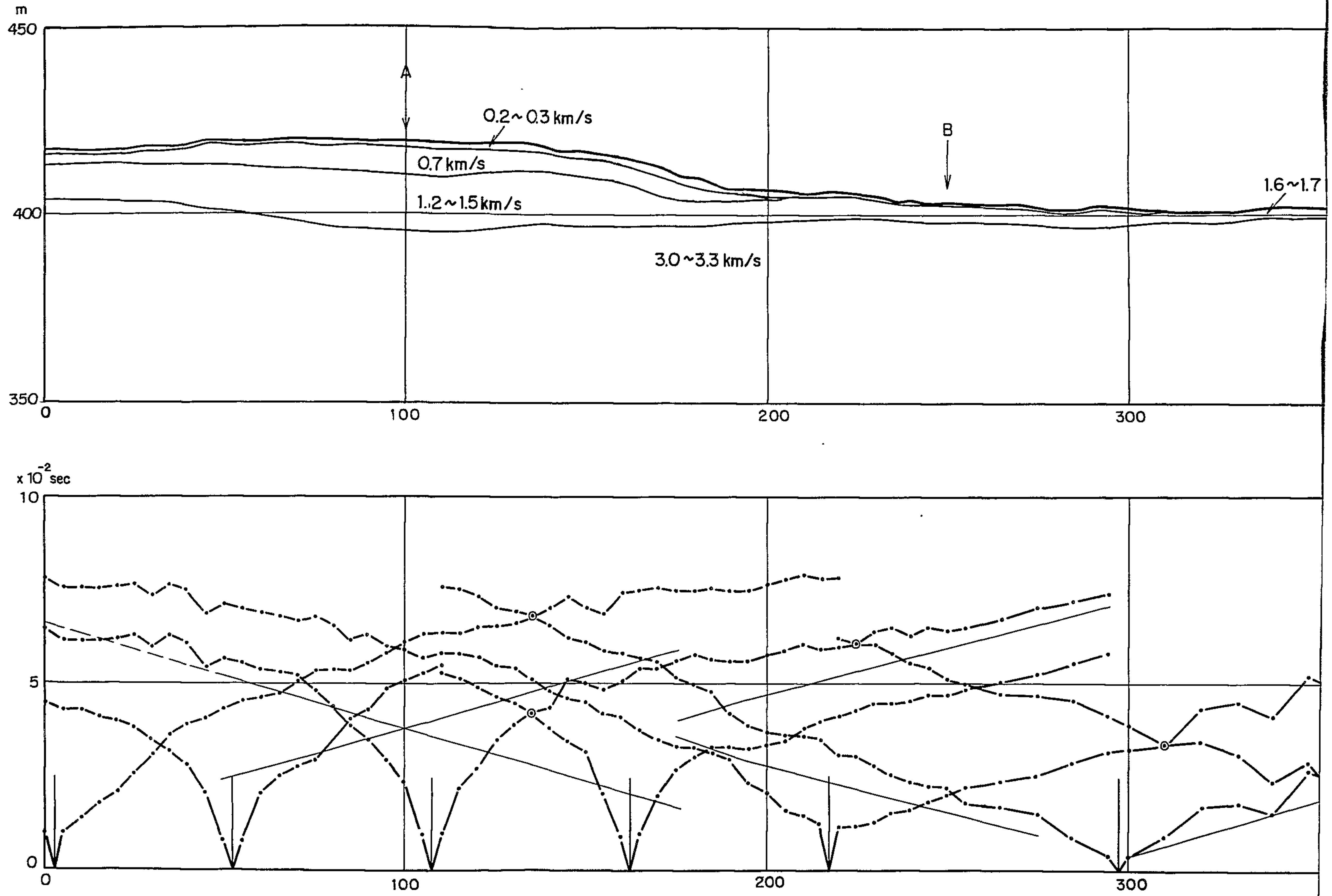
MATUNO RIVER DEVELOPMENT PROJECT

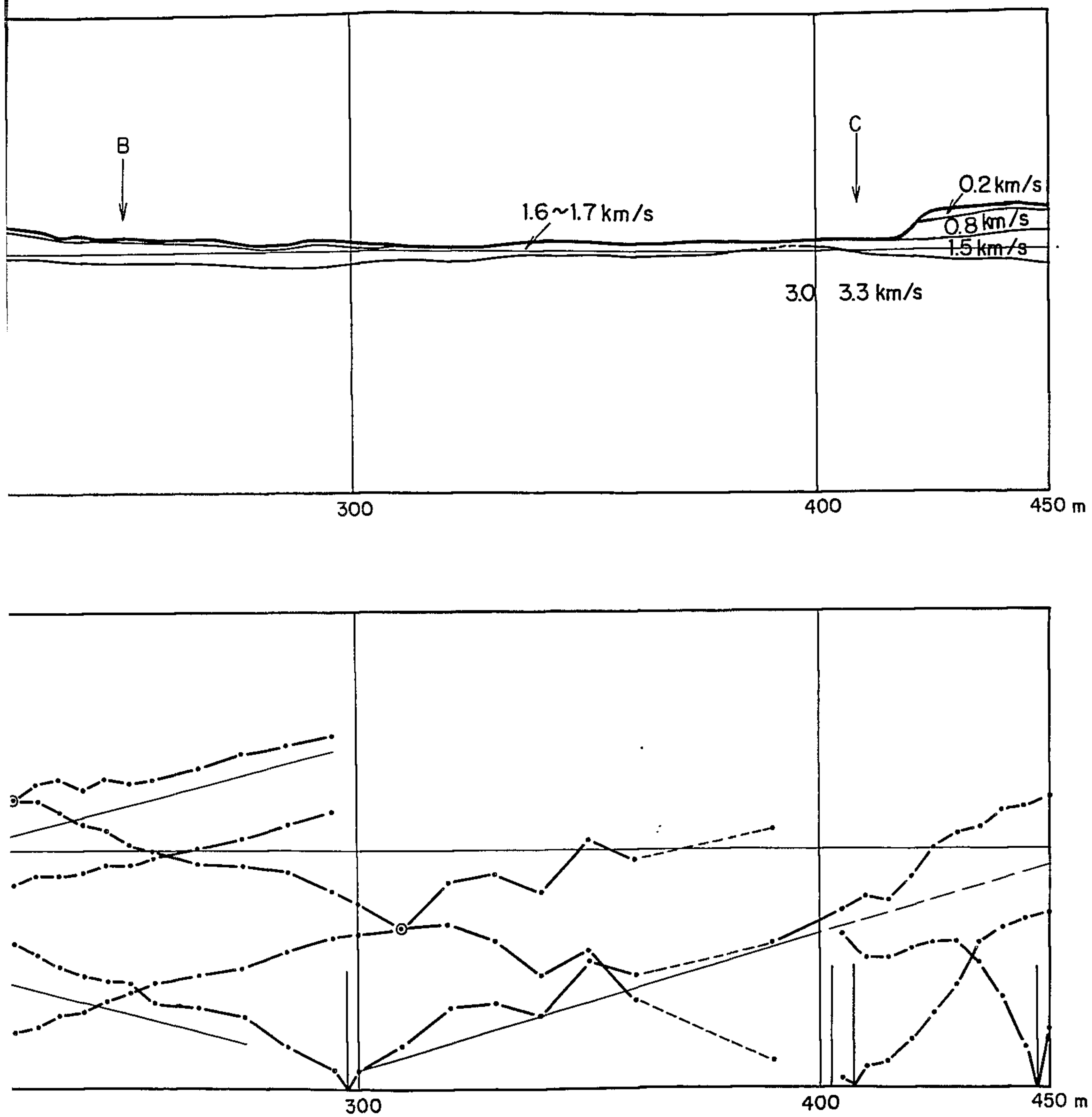
SEISMIC EXPLORATION
TIME-DISTANCE CURVE & VELOCITY LAYER SECTION

LINE C B1 DAM SITE

DATE		FIG. NO.	3-7

JAPAN INTERNATONAL COOPERATION AGENCY


m
450
400
350
A
B
0.2 ~ 0.3 km/s
0.7 km/s
1.2 ~ 1.5 km/s
3.0 ~ 3.3 km/s
1.6 ~ 1.7
0
100
200
300
x 10⁻² sec
10
5
0
0
100
200
300



LEGEND

0.2 ~ 0.3 km/s	Top soil
0.7 ~ 0.8 km/s	Terrace deposit or Decomposed rock
1.2 ~ 1.5 km/s	Weathered rock or River deposit
3.0 ~ 3.3 km/s	Fresh rock

SCALE

0 50 100m

REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

MATUNO RIVER DEVELOPMENT PROJECT

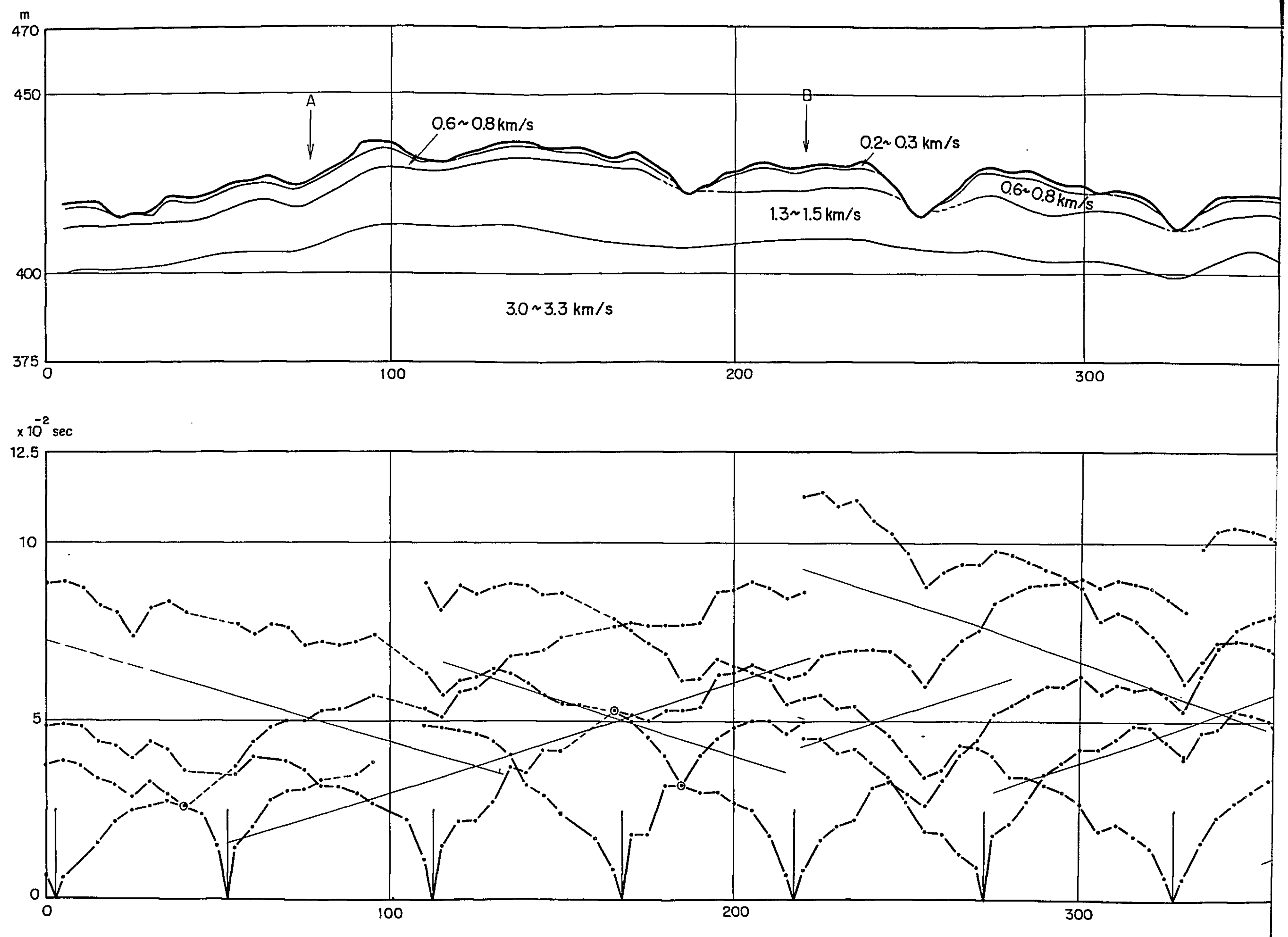
SEISMIC EXPLORATION

TIME-DISTANCE CURVE &
VELOCITY LAYER SECTION

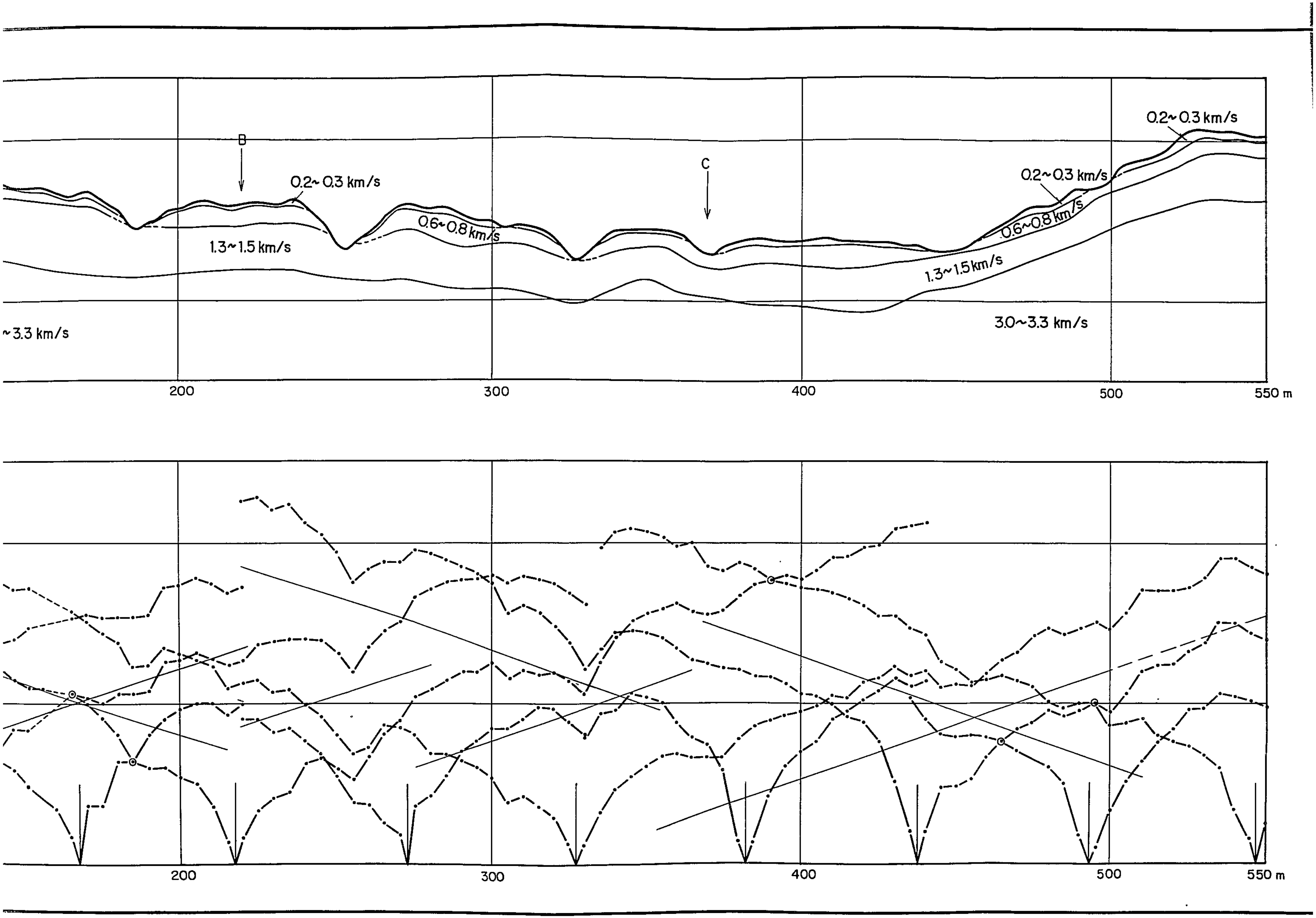
LINE D B1 DAM SITE

DATE		FIG. NO.	3-8

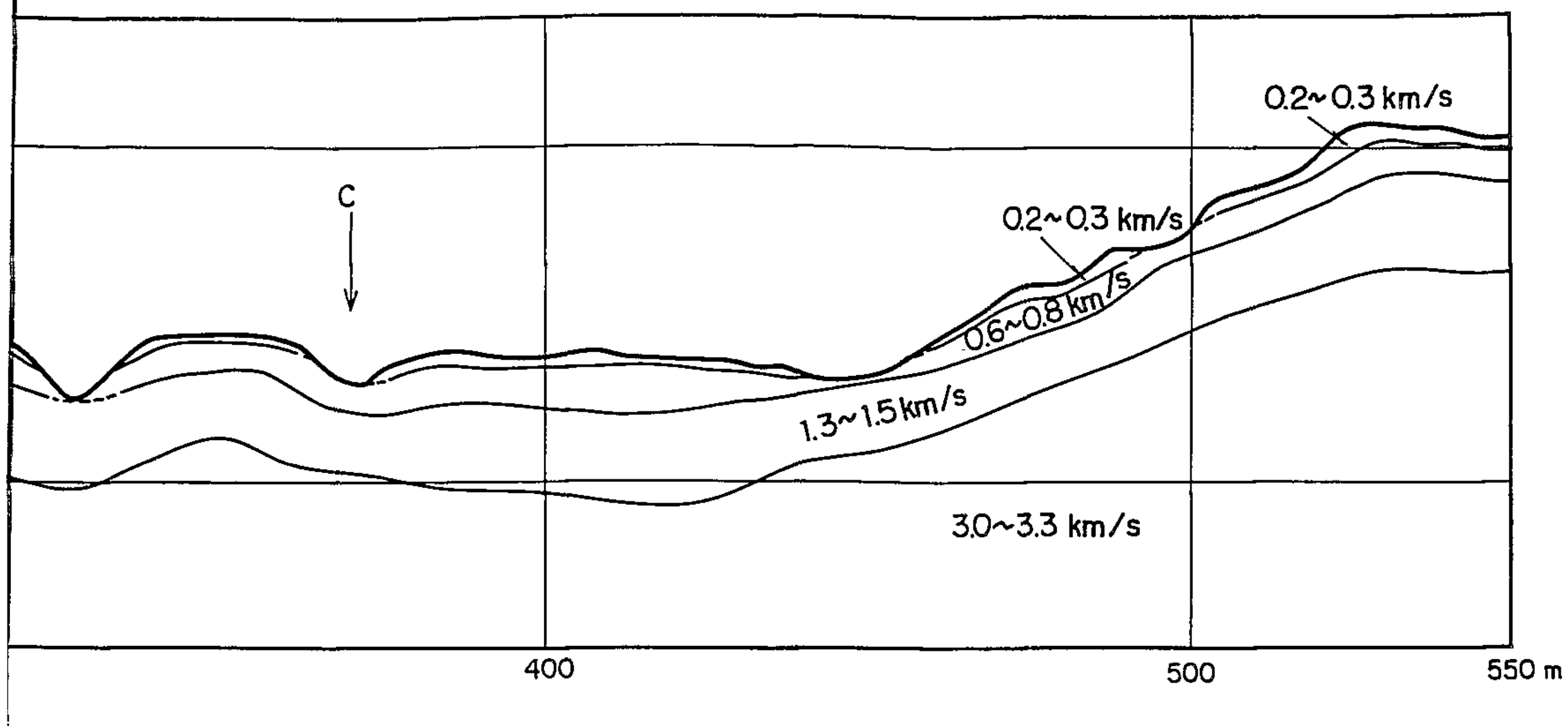
JAPAN INTERNATIONAL COOPERATION AGENCY

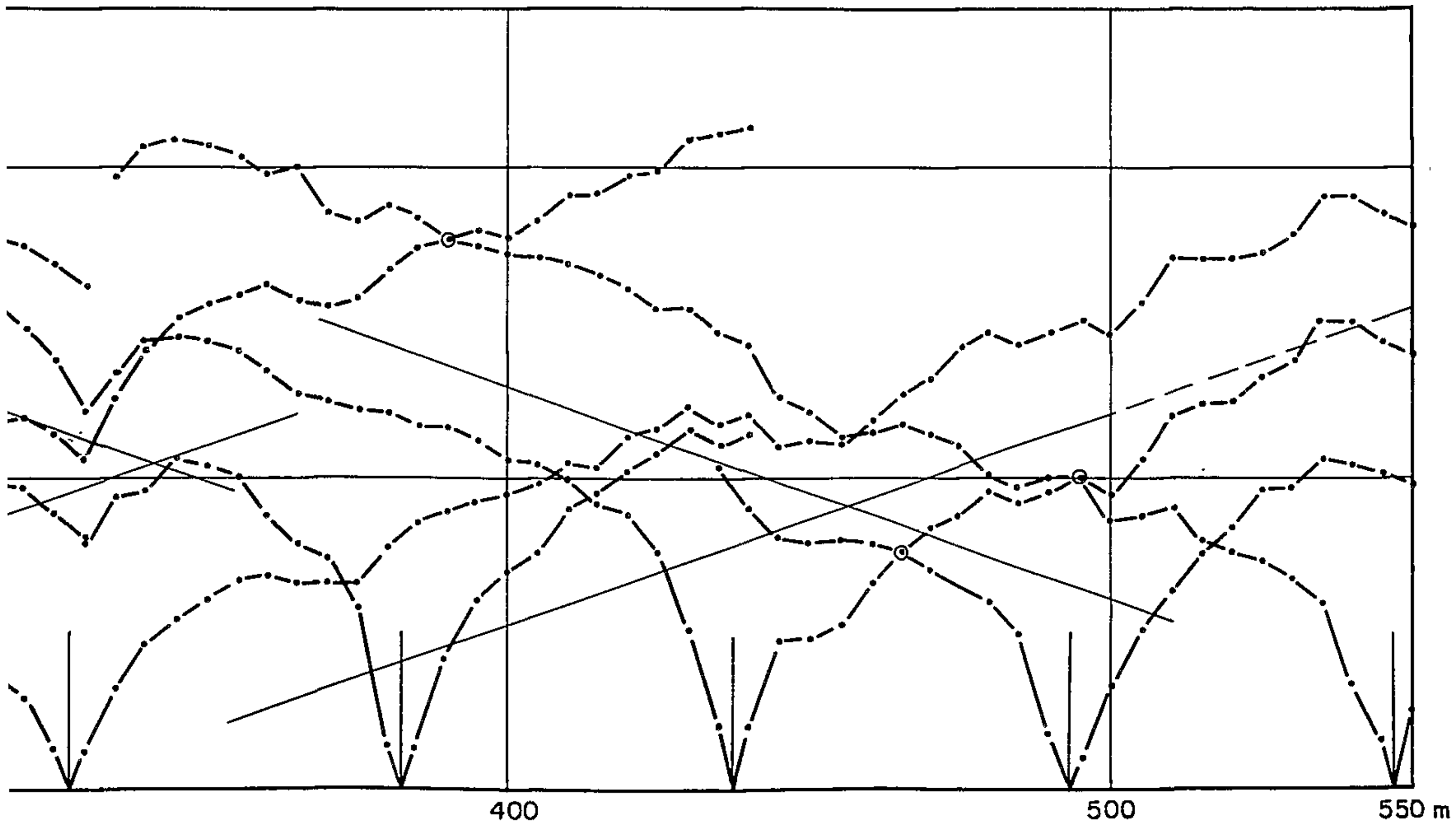


m
470
450
400
375
A
B
0.6~0.8 km/s
0.2~0.3 km/s
0.6~0.8 km/s
1.3~1.5 km/s
3.0~3.3 km/s
0
100
200
300
x 10^{-2} sec
12.5
10
5
0
0
100
200
300



B
C
0.2~0.3 km/s
0.2~0.3 km/s
0.2~0.3 km/s
0.6~0.8 km/s
0.6~0.8 km/s
1.3~1.5 km/s
1.3~1.5 km/s
~3.3 km/s
3.0~3.3 km/s
200
300
400
500
550 m
200
300
400
500
550 m

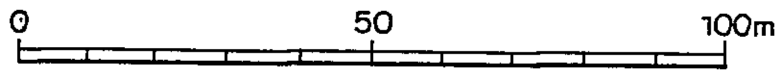




LEGEND

0.2 ~ 0.3 km/s	Top soil
0.7 ~ 0.8 km/s	Terrace deposit or Decomposed rock
1.2 ~ 1.5 km/s	Weathered rock or River deposit
3.0 ~ 3.3 km/s	Fresh rock

SCALE



REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

MATUNO RIVER DEVELOPMENT PROJECT

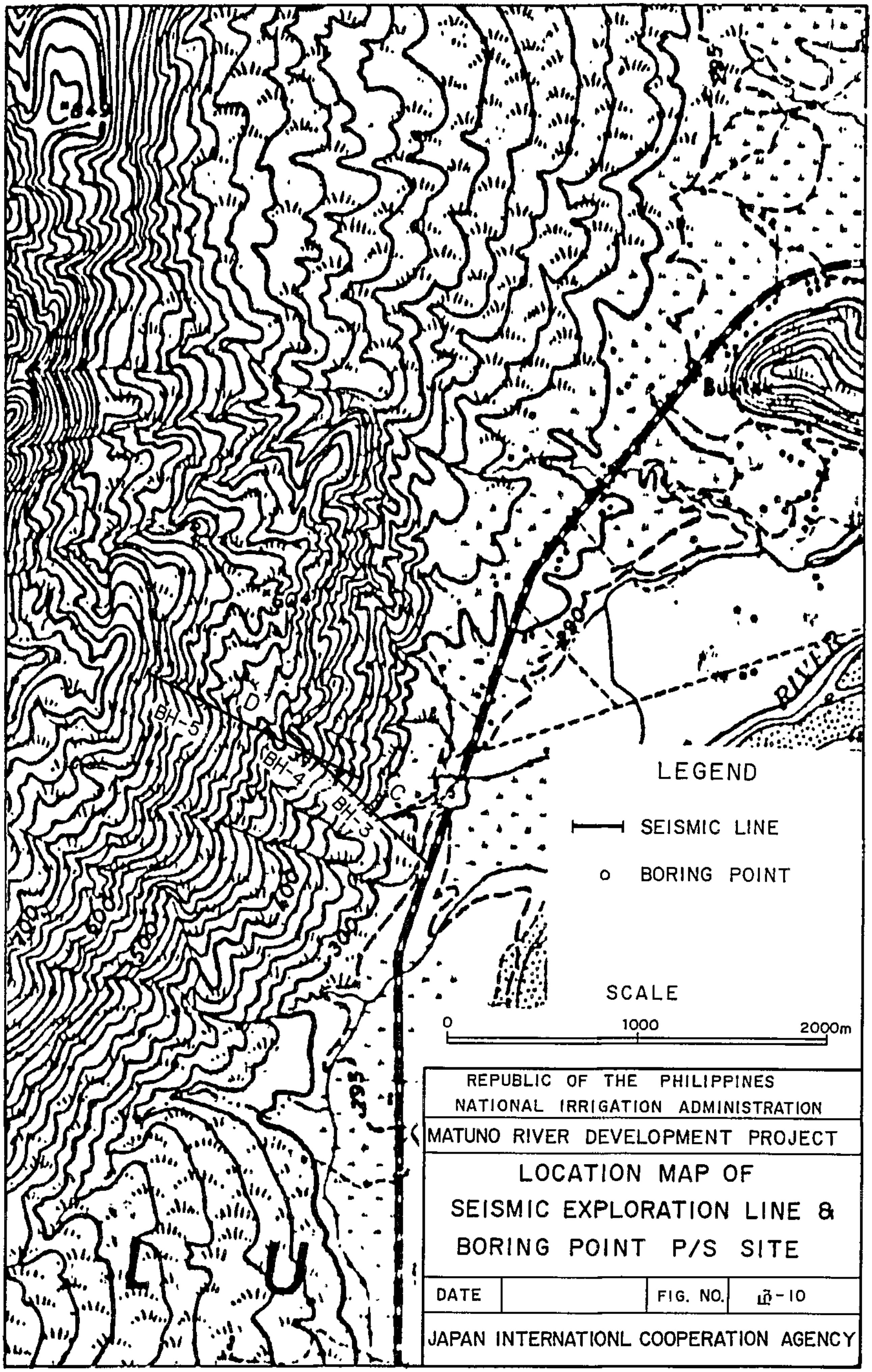
SEISMIC EXPLORATION

TIME-DISTANCE CURVE & VELOCITY LAYER SECTION

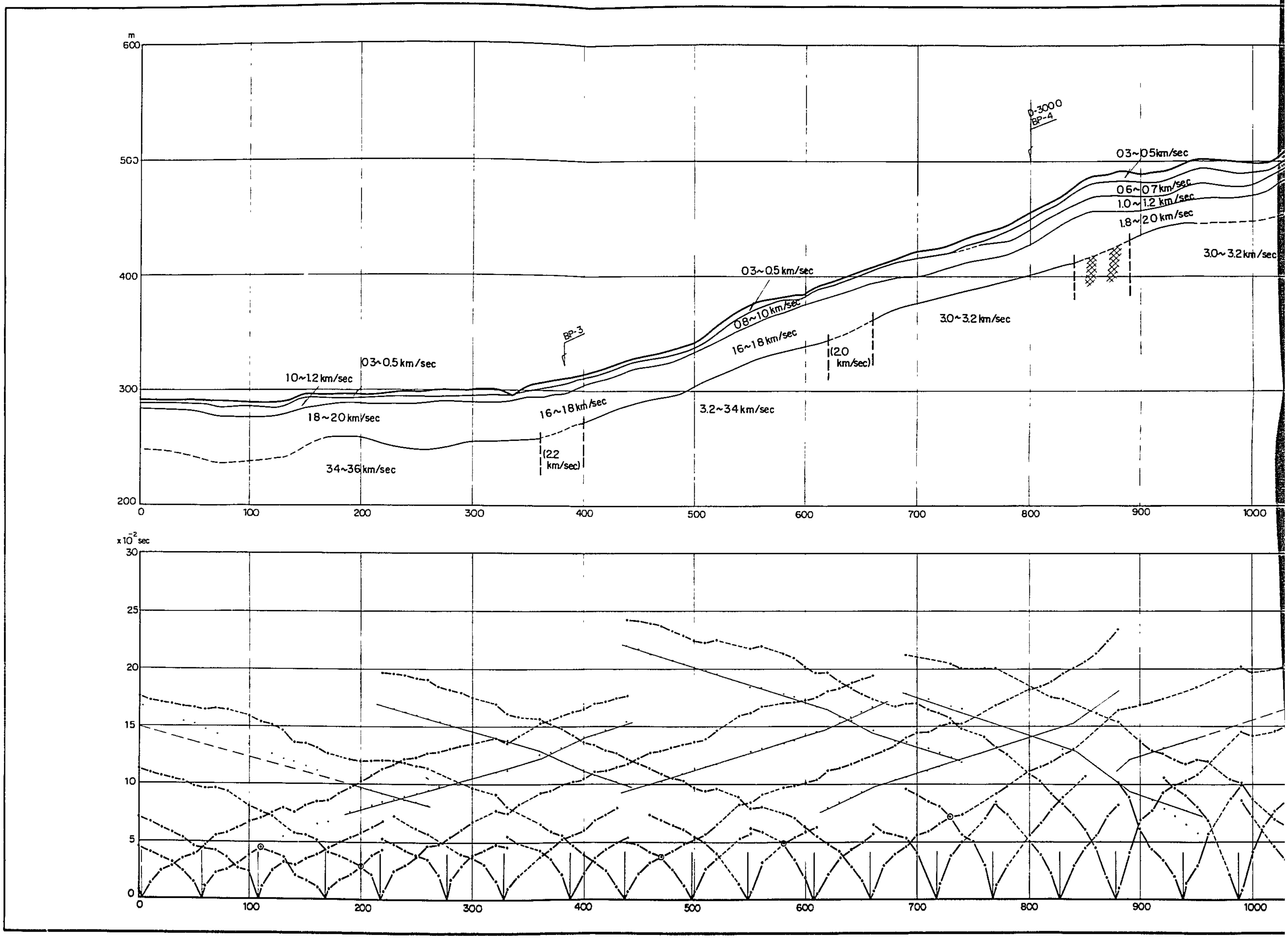
LINE E — B1 DAM SITE

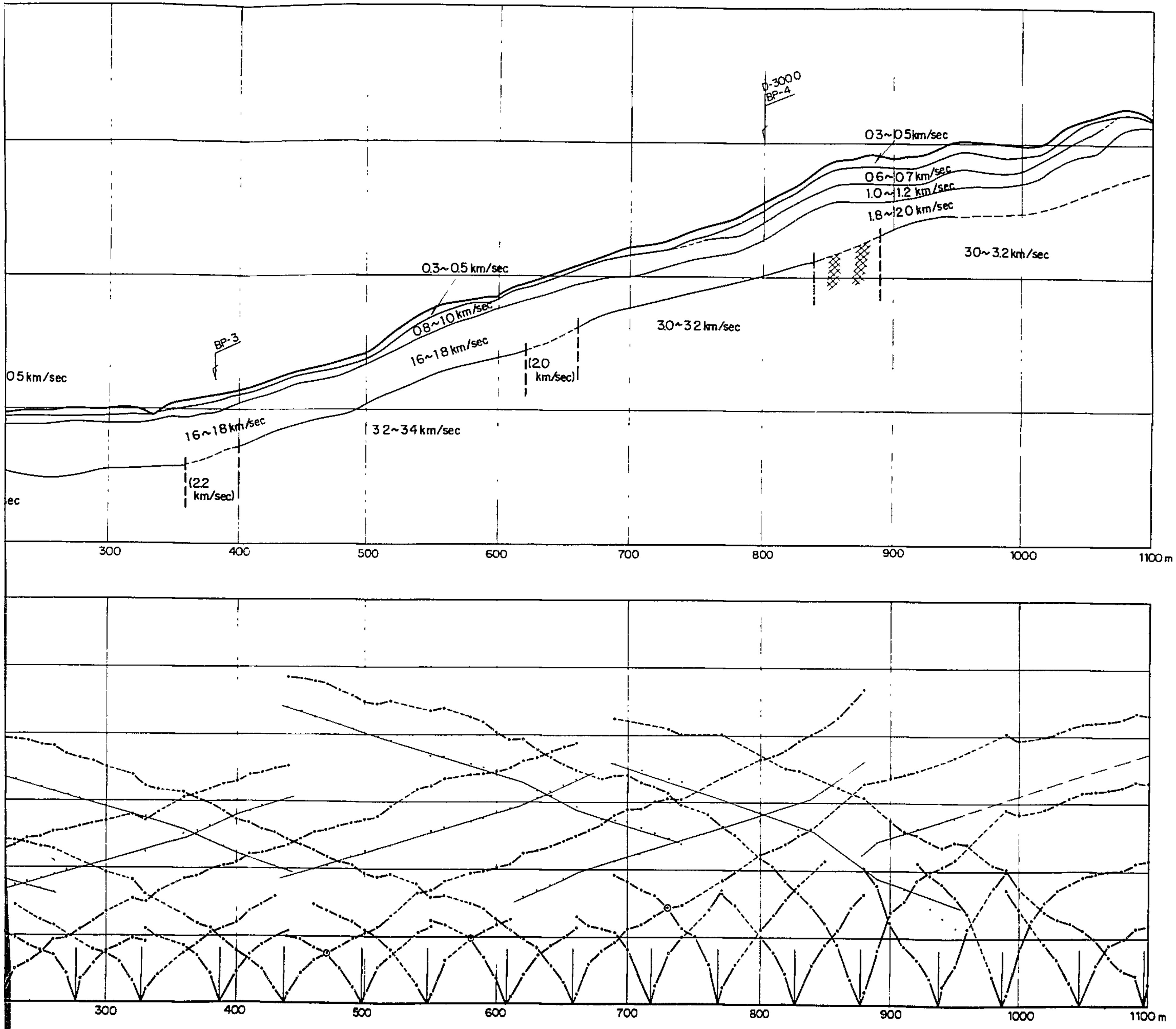
DATE		FIG. NO.	3 - 9

JAPAN INTERNATIONAL COOPERATION AGENCY



BH-5
BH-4
BH-3
D
C
RIVER
LEGEND
SEISMIC LINE
BORING POINT
SCALE
0
1000
2000m
REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION
MATUNO RIVER DEVELOPMENT PROJECT
LOCATION MAP OF
SEISMIC EXPLORATION LINE &
BORING POINT P/S SITE
DATE
FIG. NO. Ⅲ-10
JAPAN INTERNATIONL COOPERATION AGENCY


m
600
500
400
300
200
0
100
200
300
400
500
600
700
800
900
1000
0-3000
BP-4
BP-3
03~05km/sec
06~07km/sec
1.0~1.2 km/sec
1.8~20 km/sec
3.0~3.2 km/sec
03~0.5 km/sec
08~10 km/sec
16~18 km/sec
(2.0
km/sec)
3.0~3.2 km/sec
03~0.5 km/sec
10~1.2 km/sec
16~18 km/sec
18~20 km/sec
3.2~34 km/sec
(2.2
km/sec)
34~36 km/sec
x10⁻² sec
30
25
20
15
10
5
0



LEGEND

0.3~0.5 km/s	Top soil
0.6~0.7 km/s 0.8~1.2 km/s	Talus deposit or Decomposed rock
1.6~2.0 km/s 2.4~2.6 km/s	Talug deposit, River deposit or Wethered rock
3.0~3.4 km/s	Fresh rock
(hatched)	Fracture zone

SCALE

0 100 200m

REPUBLIC OF TH PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

MATUNO RIVER DEVELOPMENT PROJECT

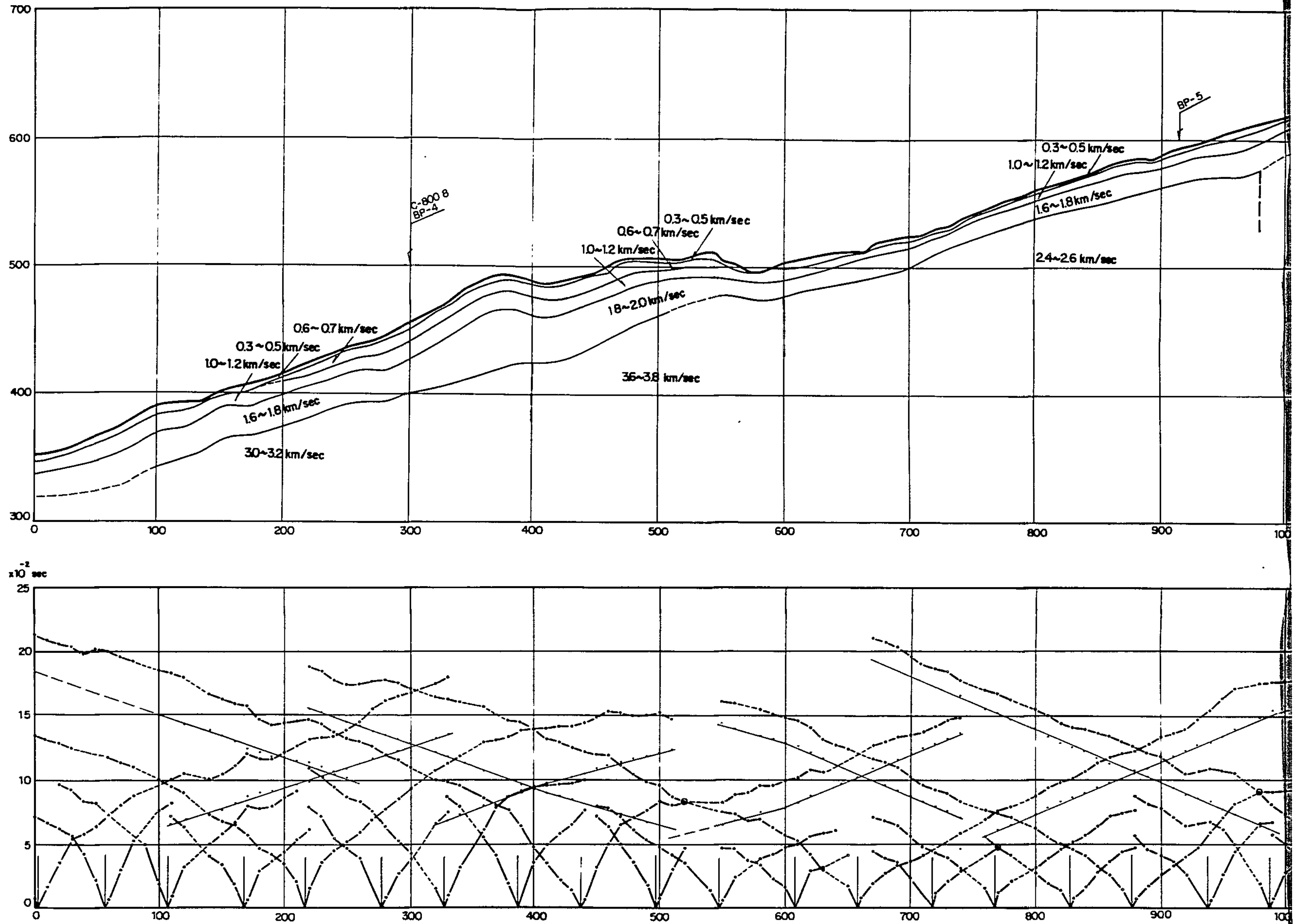
SEISMIC EXPLORATION

TIME-DISTANCE CURVE & VELOCITY LAYER SECTION

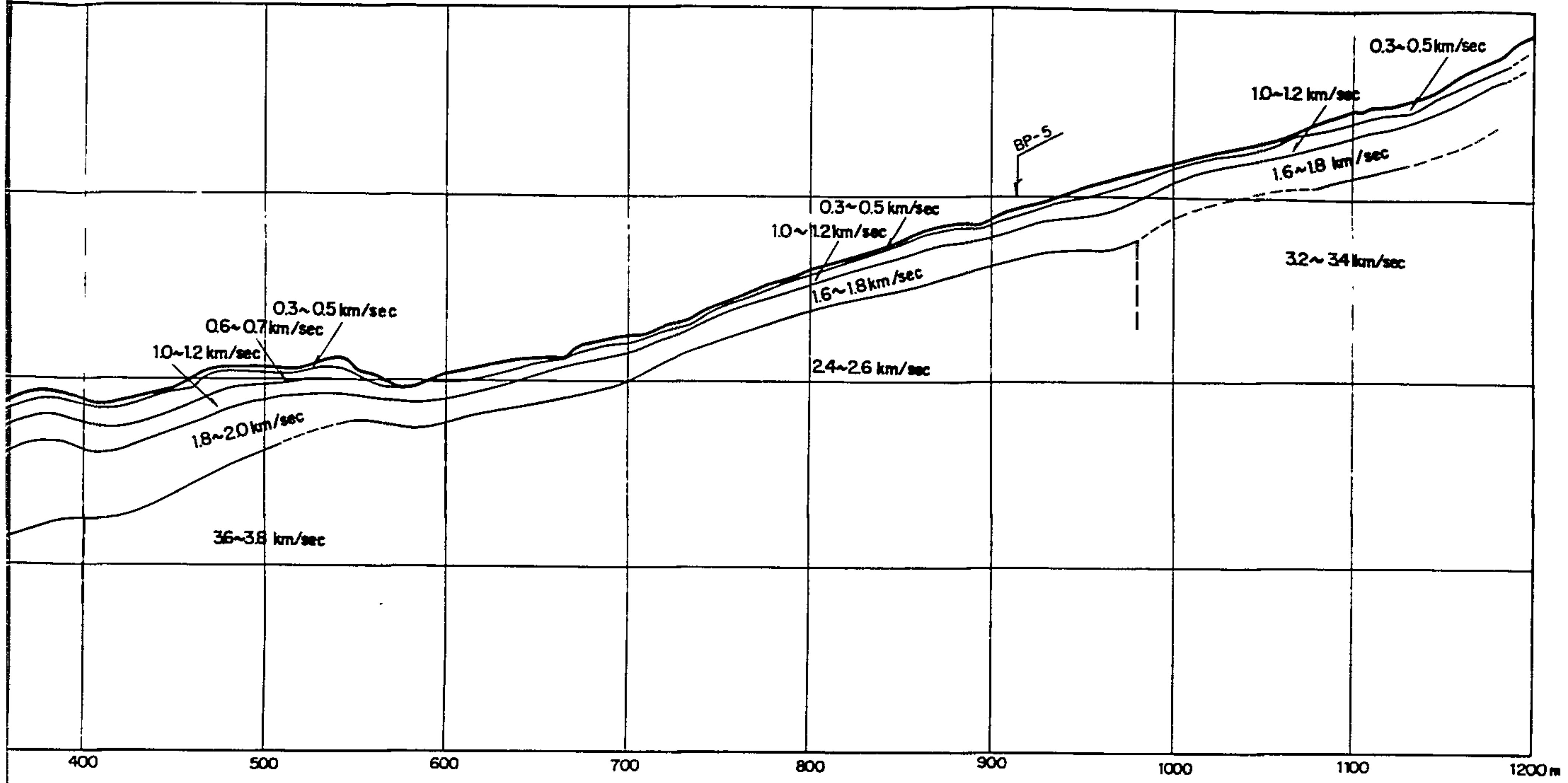
LINE - C P/S SITE

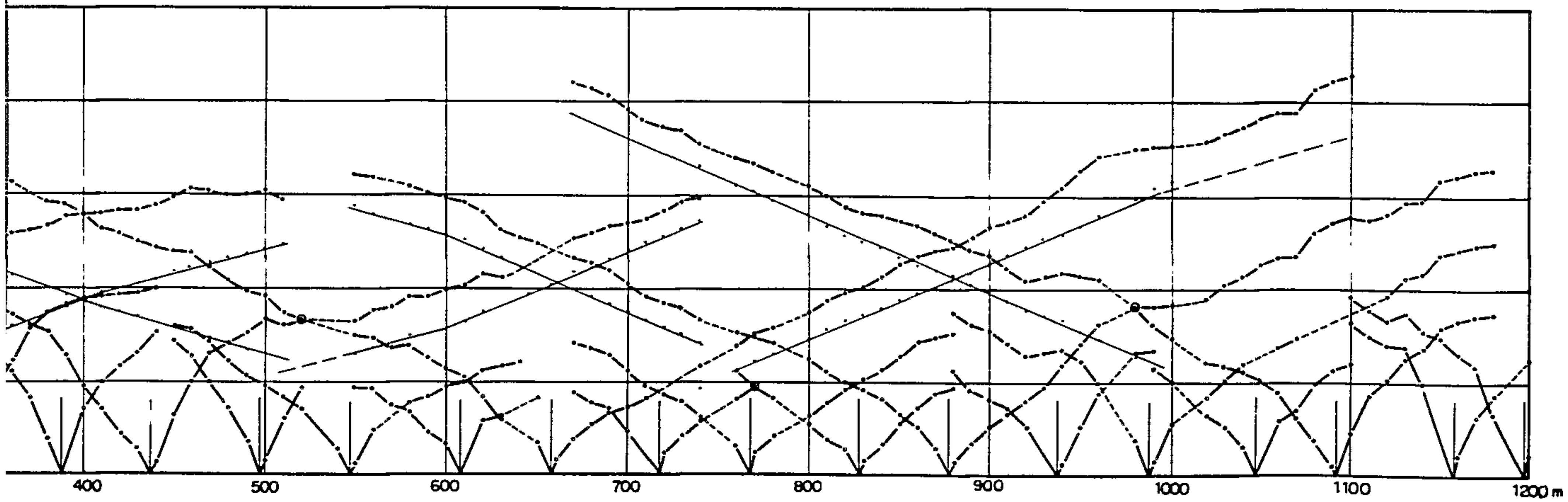
DATE		FIG. NO.	3-11

JAPAN INTERNATIONAL COOPERATION AGENCY



700
600
500
400
300
C-800 8
BP-4
BP-5
0.3~0.5 km/sec
0.6~0.7 km/sec
1.0~1.2 km/sec
1.6~1.8 km/sec
1.8~2.0 km/sec
2.4~2.6 km/sec
3.0~3.2 km/sec
3.6~3.8 km/sec
0
100
200
300
400
500
600
700
800
900
x10⁻² sec
25
20
15
10
5





LEGEND

0.3 ~ 0.5 km/s	Top soil
0.6 ~ 0.7 km/s 0.8 ~ 1.2 km/s	Talus deposit or Decomposed rock
1.6 ~ 2.0 km/s 2.4 ~ 2.6 km/s	Talus deposit, River deposit or Wethered rock
3.0 ~ 3.4 km/s	Fresh rock

SCALE

0 100 200 m

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NATIONAL IRRIGATION ADMINISTRATION

MATUNO RIVER DEVELOPMENT PROJECT

SEISMIC EXPLORATION

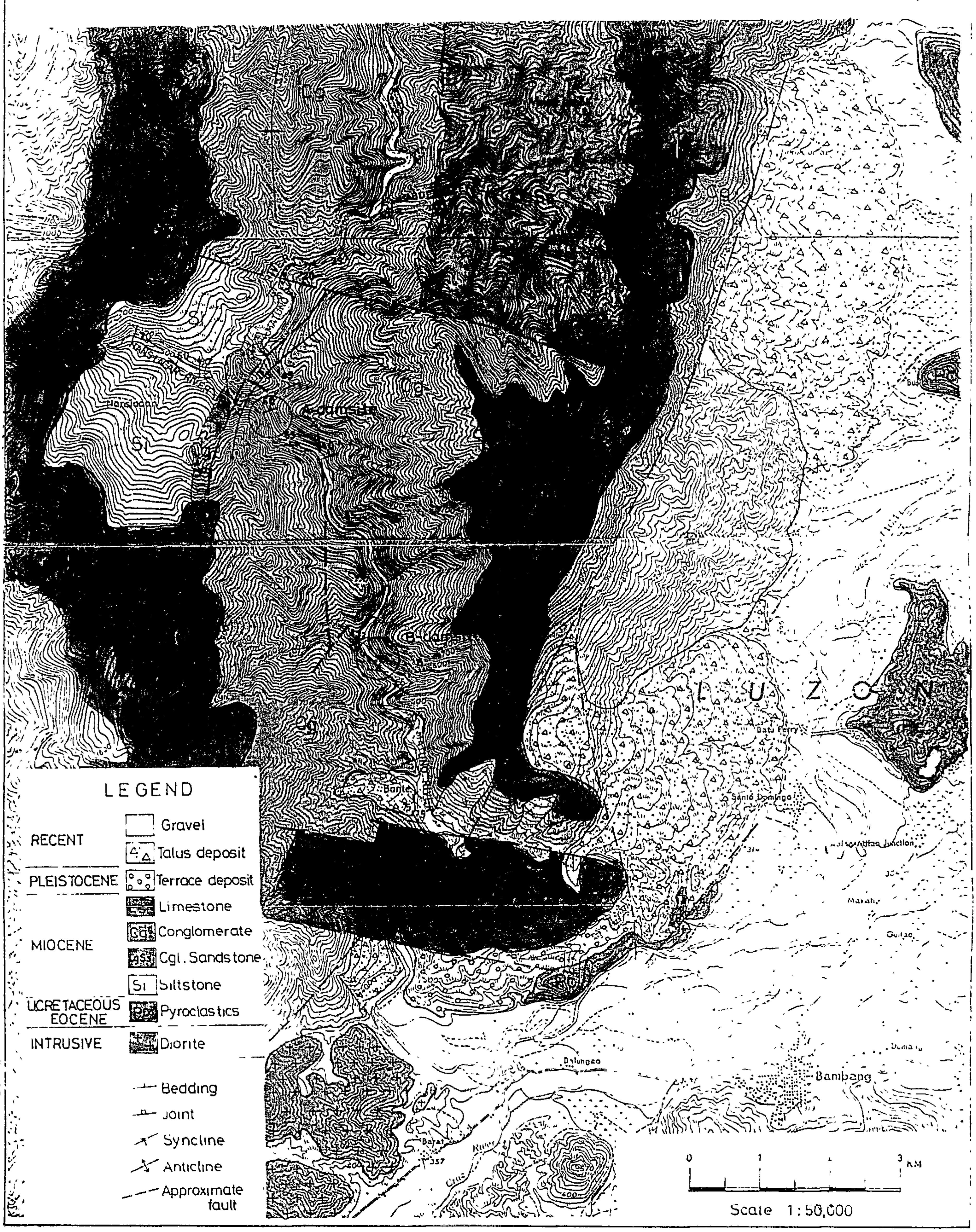
TIME-DISTANCE CURVE &
VELOCITY LAYER SECTION

LINE - D P/S SITE

DATE		FIG. NO.	3 - 12

JAPAN INTERNATIONAL COOPERATION AGENCY

FIG. 3-13 GEOLOGICAL MAP OF THE PROJECT AREA



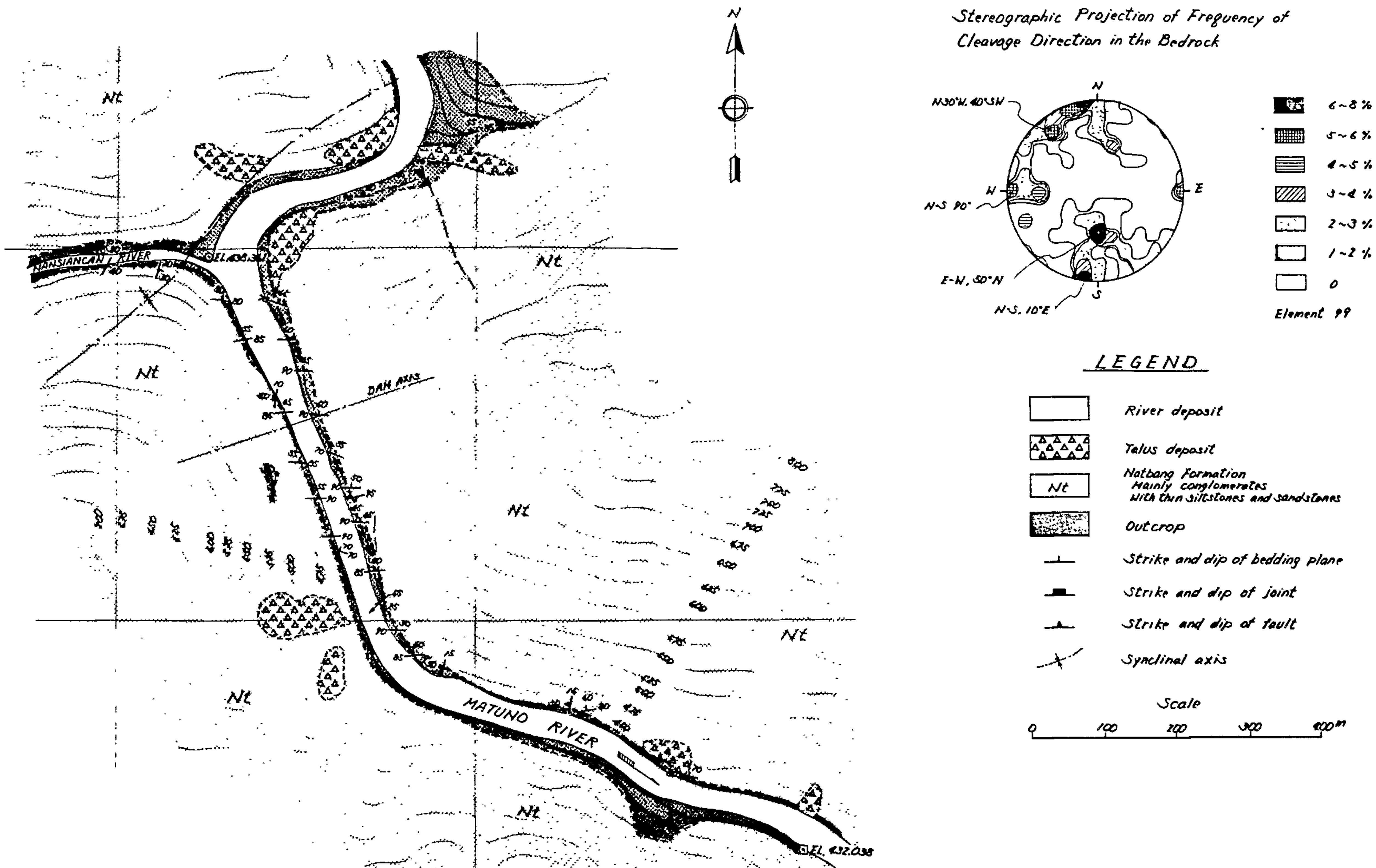
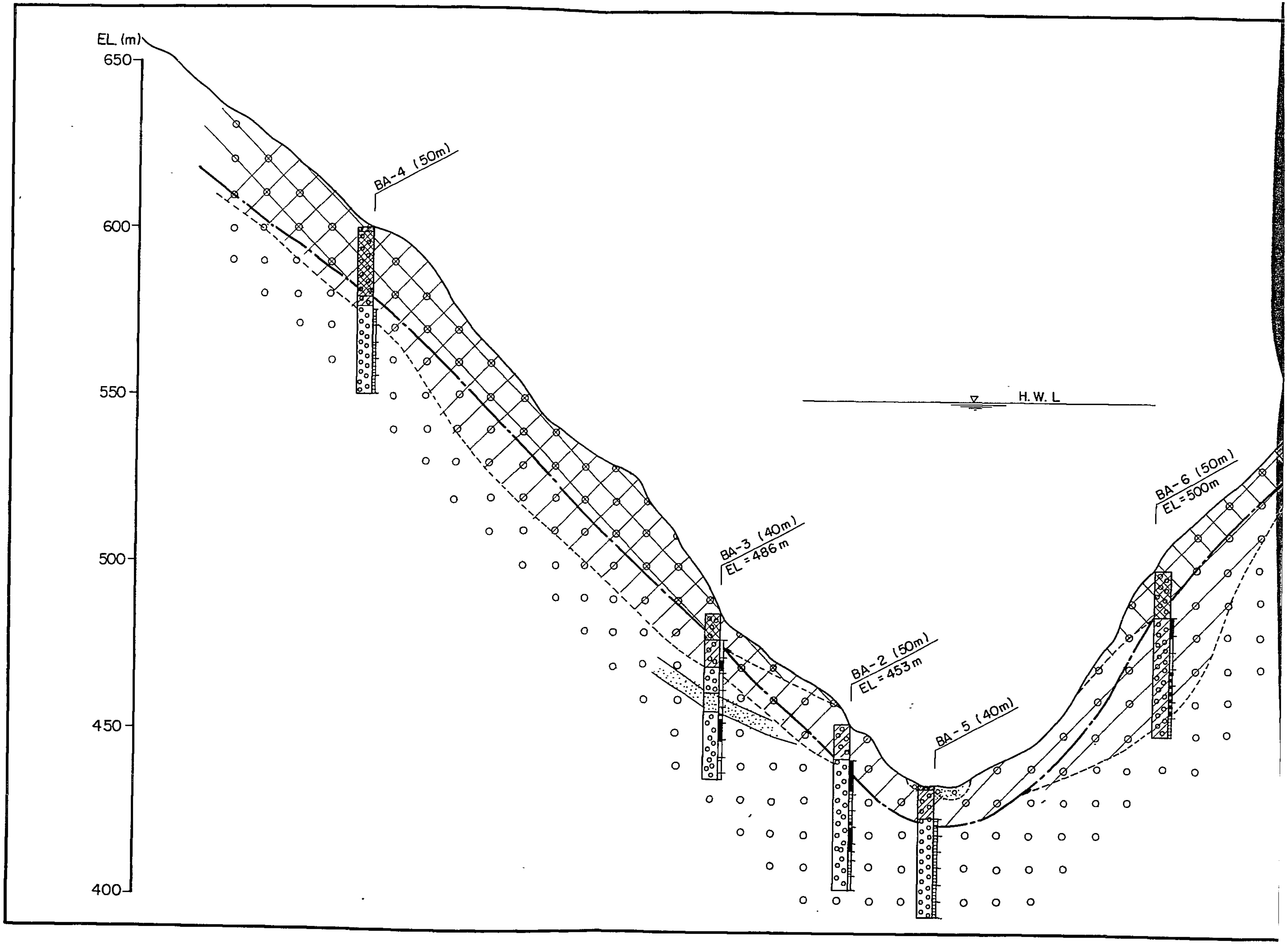
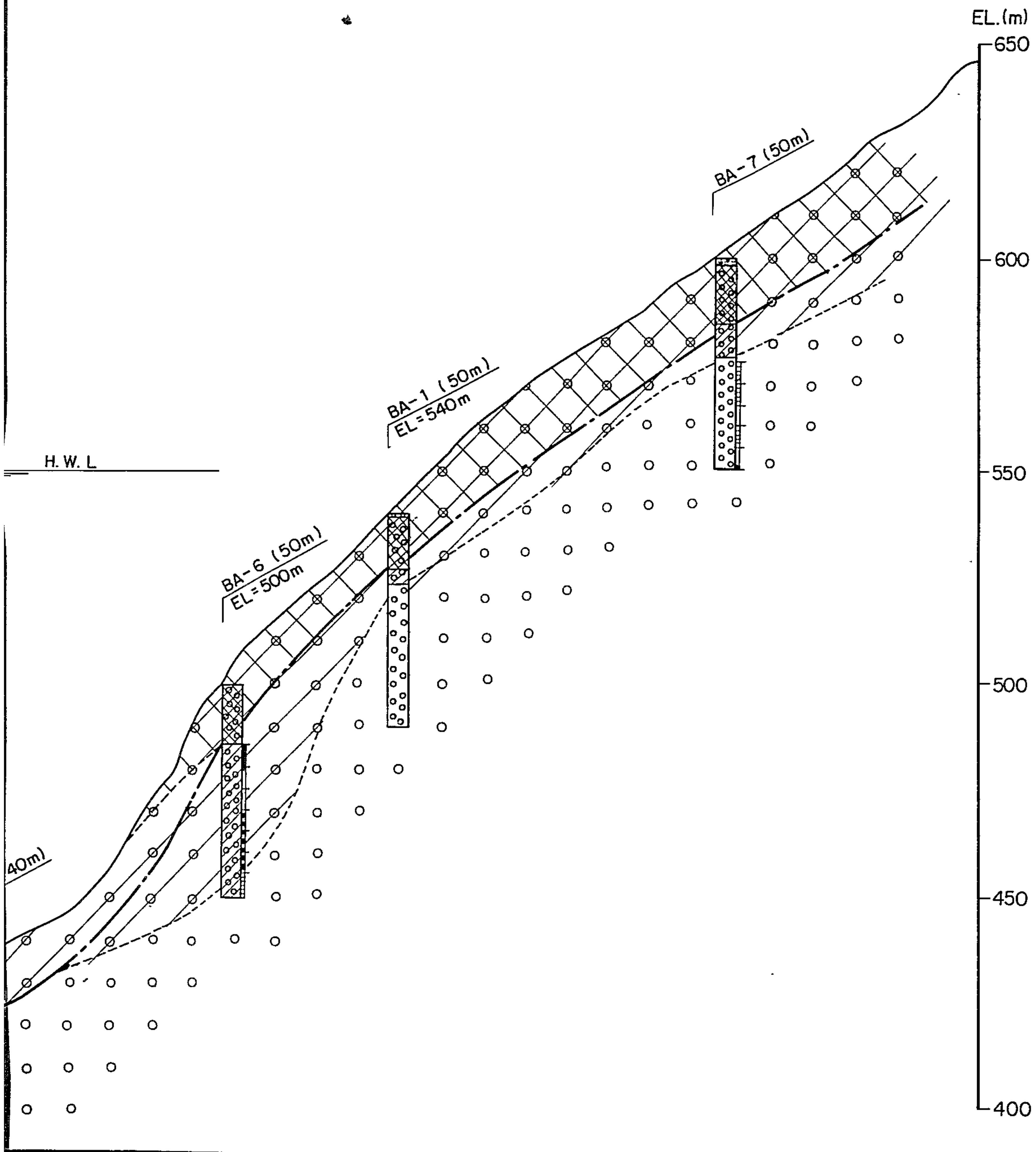


FIG.3-14 GEOLOGIC MAP OF A DAMSITE



EL.(m)
650
600
550
500
450
400
BA-4 (50m)
BA-3 (40m)
EL=486m
BA-2 (50m)
EL=453m
BA-5 (40m)
BA-6 (50m)
EL=500m
H.W.L



LEGEND

- RIVER DEPOSITS
- TOP SOIL TALUS AND SOREE
- CONGLOMERATE
- SANDSTONE
- DECOMPOSED OR INTENSIVELY WEATHERED ROCK
- WEATHERED ROCK

EXCAVATION LINE FOR FOUNDATION OF IMPERVIOUS CORE ZONE OF FILL DAM

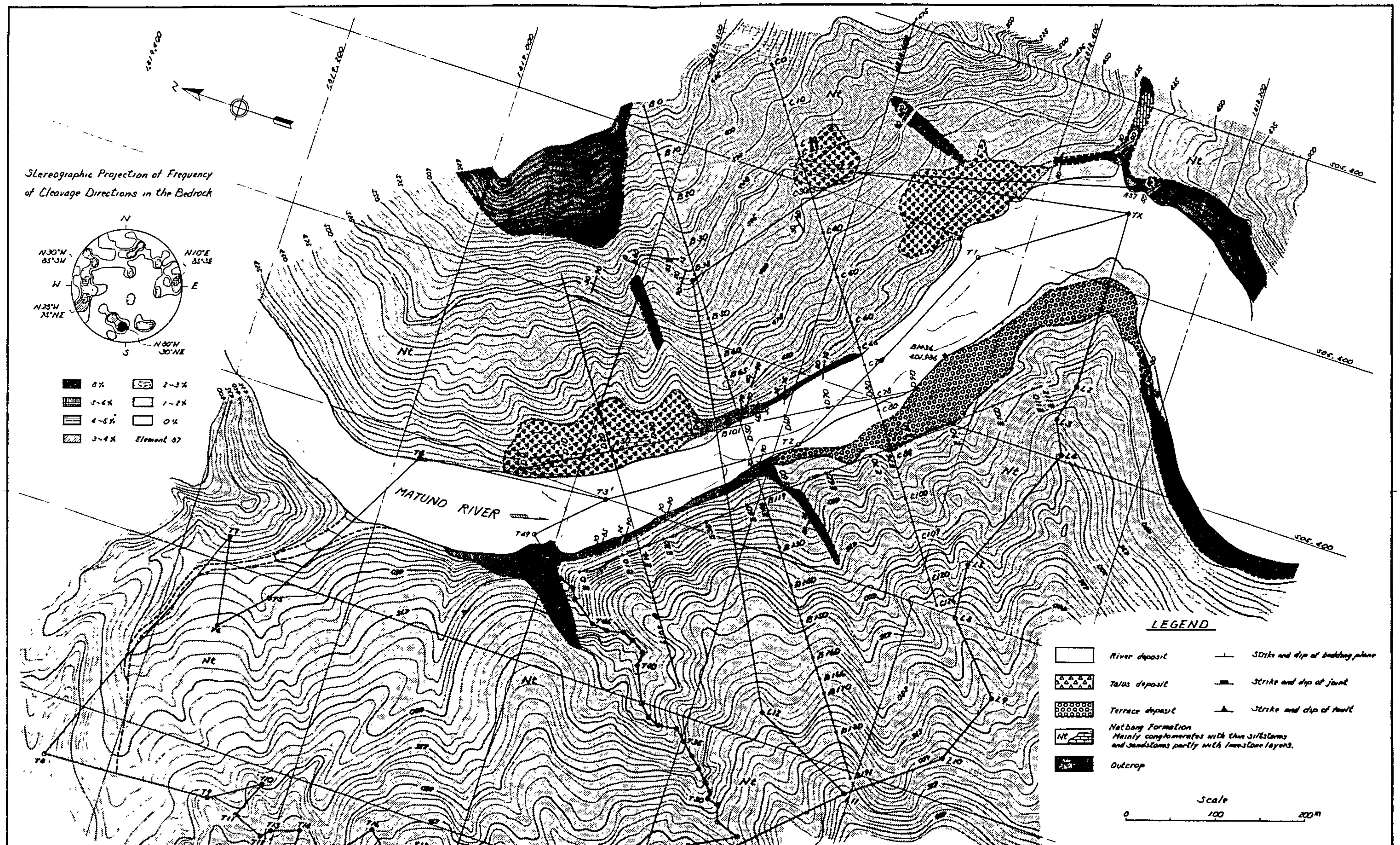
PERMEABILITY IN LUGEON UNIT

- 5≧Lu
- 10≧Lu>5
- 20≧Lu>10
- Lu>20

SCALE

0 50 100m

REPUBLIC OF THE PHILIPPINES NATIONAL IRRIGATION ADMINISTRATION.			
MATUNO RIVER DEVELOPMENT PROJECT			
GEOLOGICAL PROFILE A. DAMSITE LINE - A			
DATE		FIG. NO	3 - 15
JAPAN INTERNATIONAL COOPERATION AGENCY			



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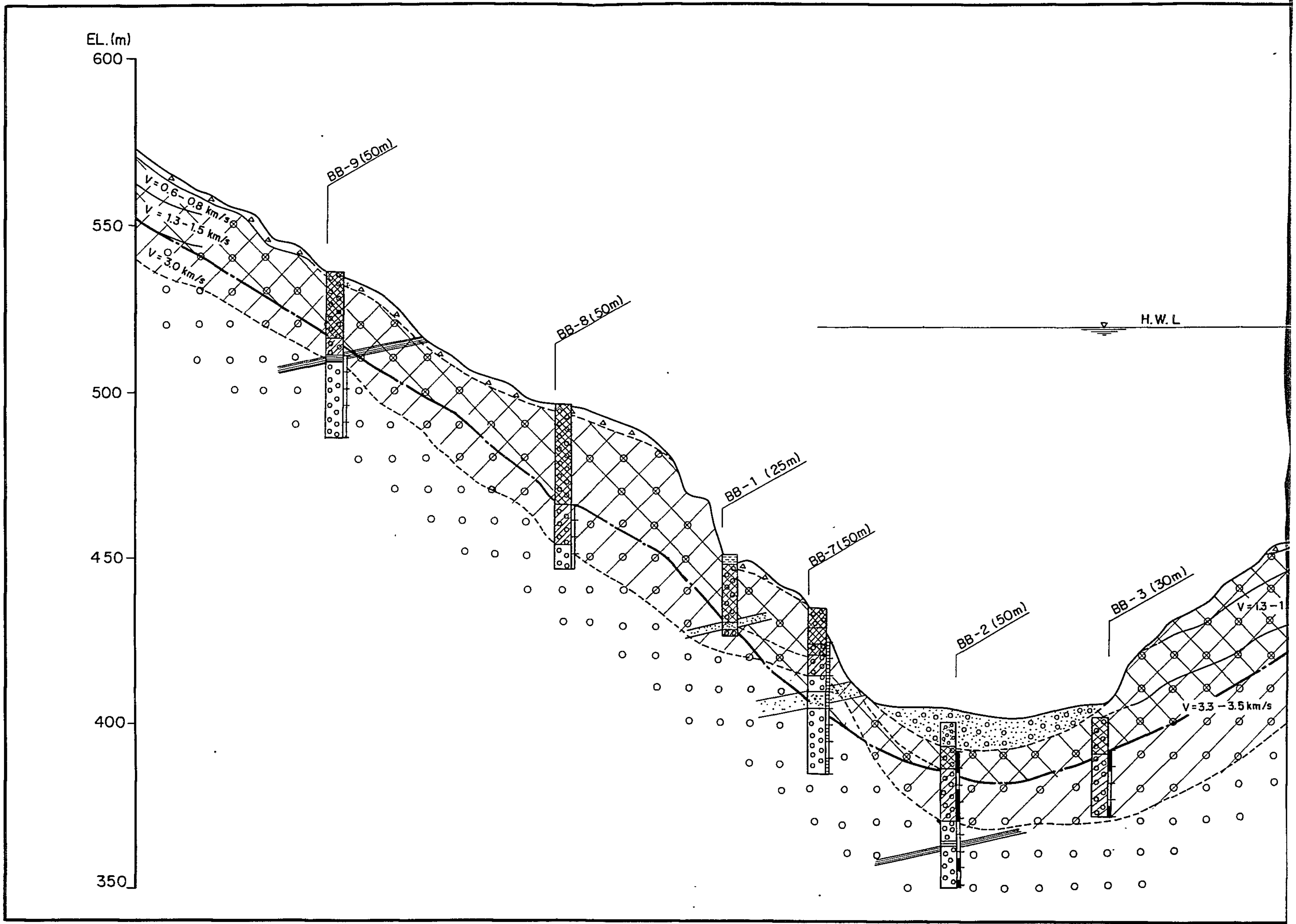
NATIONAL IRRIGATION ADMINISTRATION

MATUNO RIVER DEVELOPMENT PROJECT

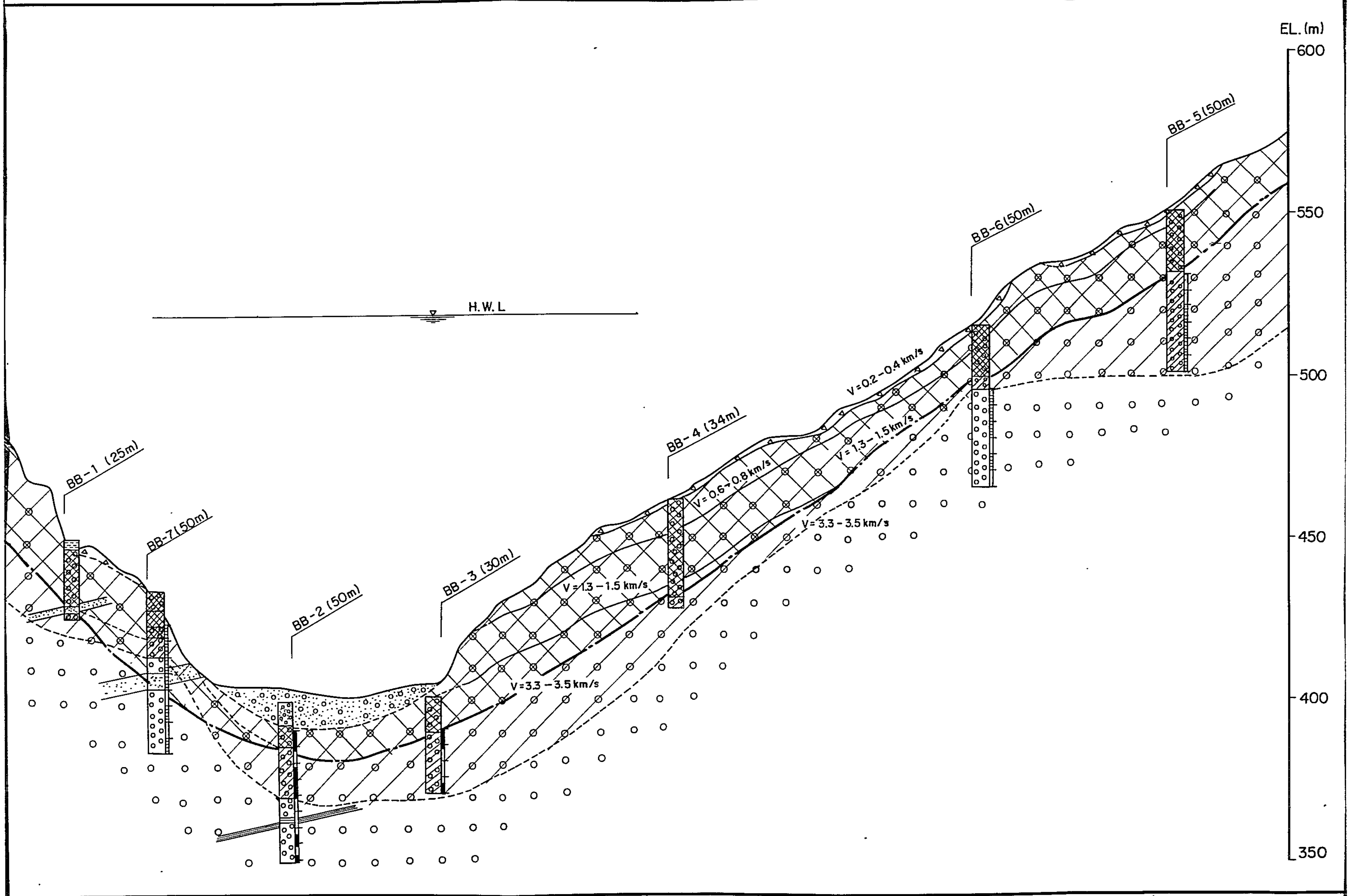
GEOLOGIC MAP OF B1 - DAMSITE

DATE		FIG.NO	3 - 16

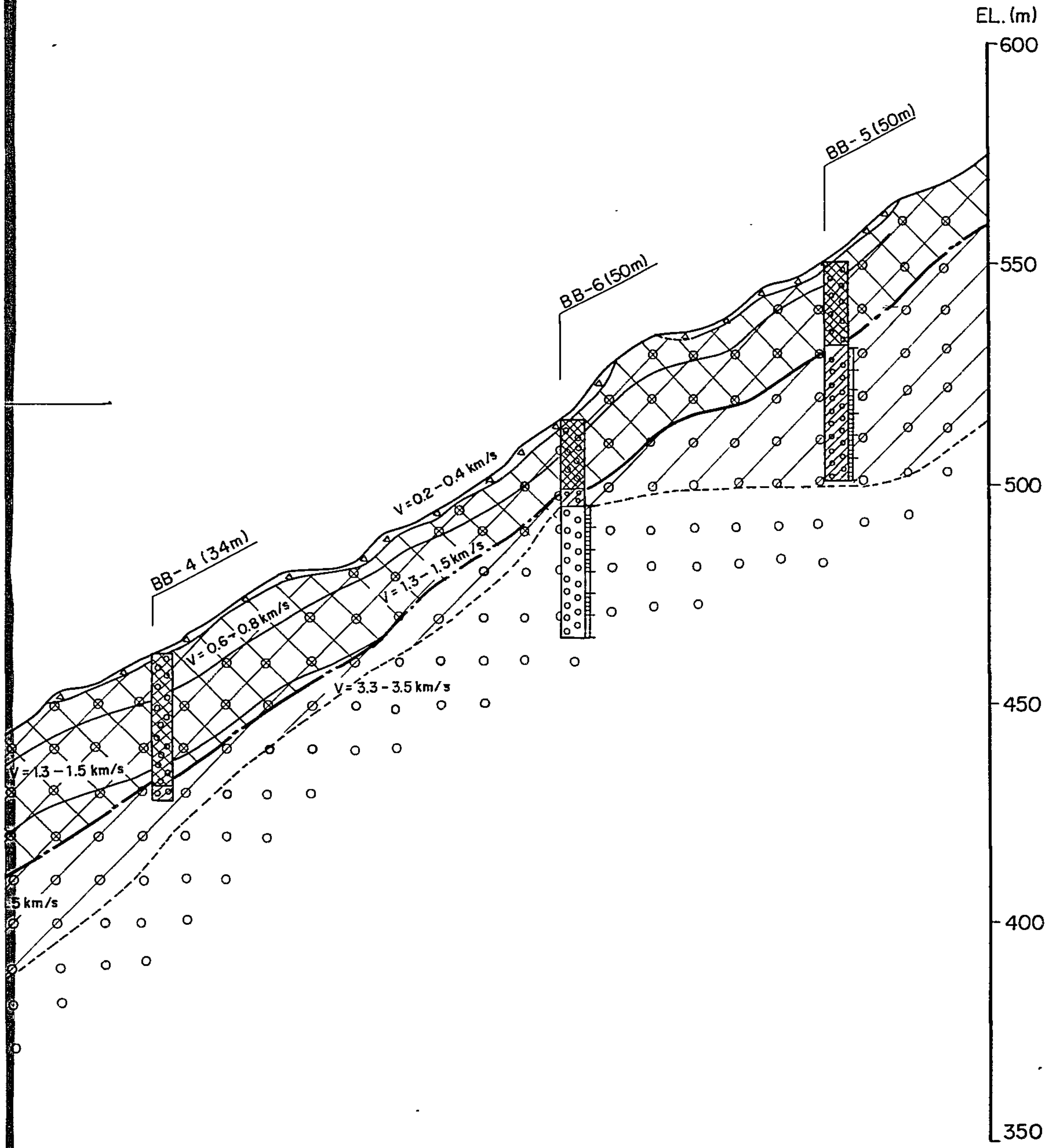
JAPAN INTERNATIONAL COOPERATION AGENCY



EL.(m)
600
550
500
450
400
350
BB-9 (50m)
BB-8 (50m)
BB-1 (25m)
BB-7 (50m)
BB-2 (50m)
BB-3 (30m)
H.W.L
V = 0.6 - 0.8 km/s
V = 1.3 - 1.5 km/s
V = 3.0 km/s
V = 3.3 - 3.5 km/s



EL. (m)
600
550
500
450
400
350
H.W.L
BB-1 (25m)
BB-7 (50m)
BB-2 (50m)
BB-3 (30m)
BB-4 (34m)
BB-6 (50m)
BB-5 (50m)
V=0.2-0.4 km/s
V=0.6-0.8 km/s
V=1.3-1.5 km/s
V=1.3-1.5 km/s
V=3.3-3.5 km/s
V=3.3-3.5 km/s



L E G E N D

- RIVER DEPOSITS
- TOP SOIL TALUS AND SOREE
- CONGLOMERATE
- SLATY SHALE
- SANDSTONE
- DECOMPOSED OR INTENSIVELY WEATHERED ROCK
- WEATHERED ROCK
- V = 30 - 3.5km/s VELOCITY OF ELASTIC WAVES
- EXCAVATION LINE FOR FOUNDATION OF IMPERVIOUS CORE ZONE OF FILL DAM
- PERMEABILITY IN LUGEON UNIT: 5 ≧ Lu; 10 ≧ Lu > 5; 20 ≧ Lu > 10; Lu > 20

SCALE

0 50 100m

REPUBLIC OF THE PHILIPPINES NATIONAL IRRIGATION ADMINISTRATION.			
MATUNO RIVER DEVELOPMENT PROJECT			
GEOLOGICAL PROFILE B1 DAMSITE LINE - B			
DATE		FIG.NO	3-17
JAPAN INTERNATIONAL COOPERATION AGENCY.			

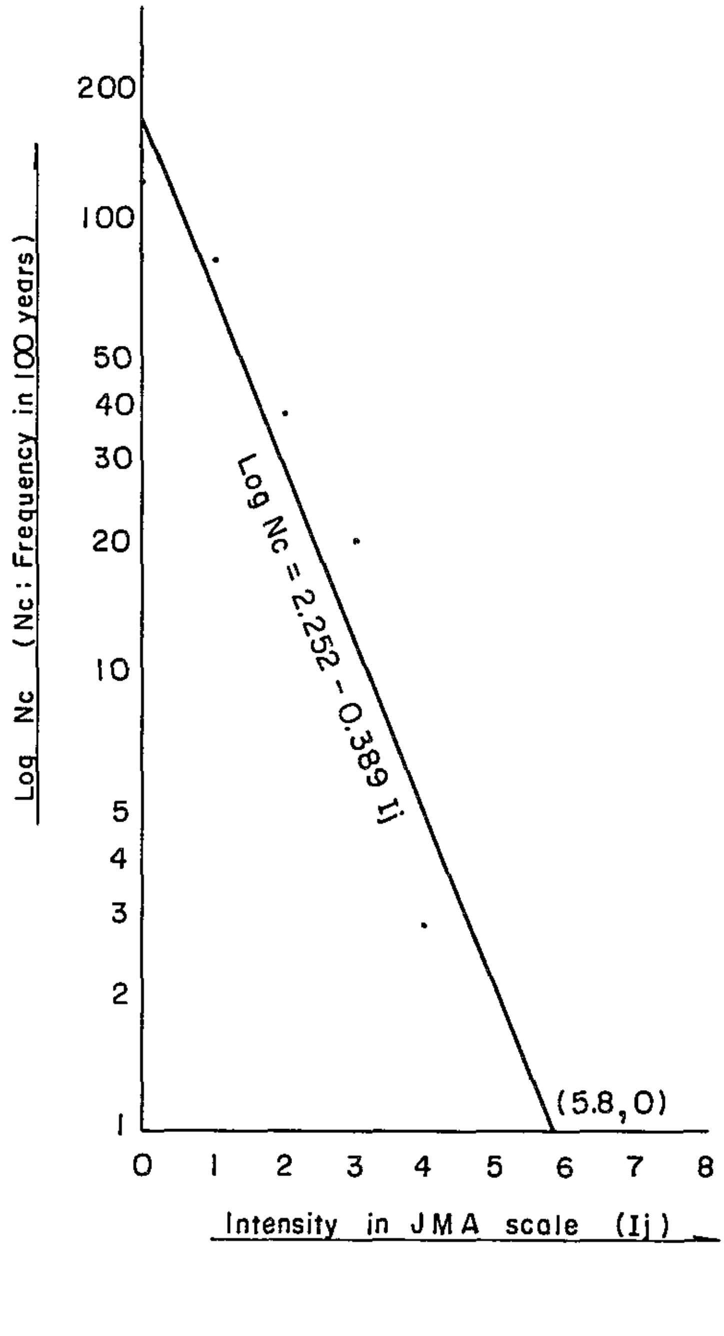


FIG. 3-18 FREQUENCY OF EARTHQUAKES

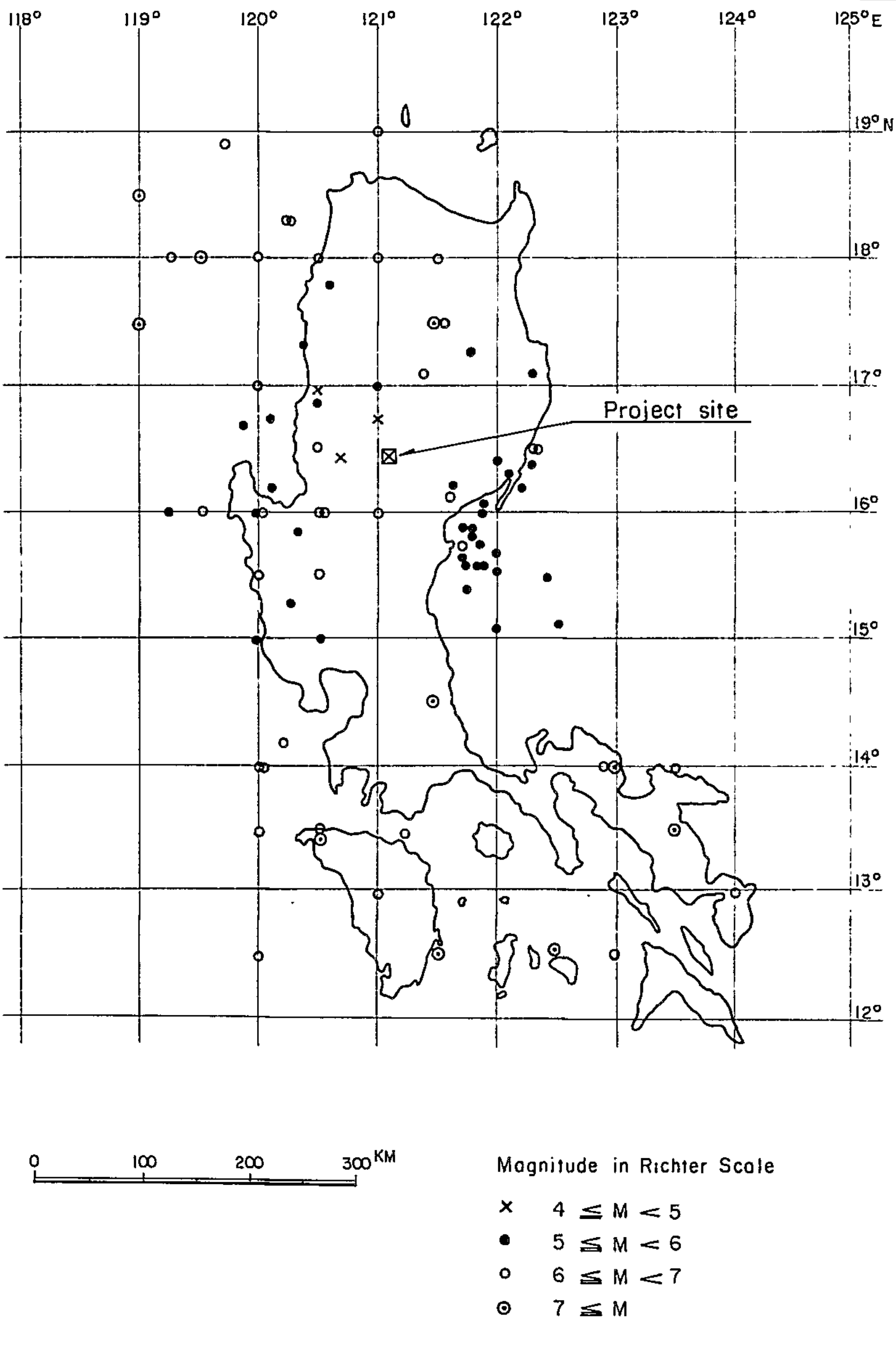


FIG. 3-19 EPICENTERS OF EARTHQUAKES (1907-1976) WHICH WERE INFLUENTIAL TO THE PROJECT SITE

II-4 CONSTRUCTION MATERIALS SURVEY AND TESTS

II-4 CONSTRUCTION MATERIALS SURVEY AND TESTS

4.1 Required Materials and Their Quantities

As reported in the Interim Report submitted to NIA (thru. JICA) on April 1, 1983, the B_1 dam site is finally selected as the most suitable dam site. The general investigation for the construction materials was made in the first year in a wide area covering the following alternative plans:

Case I : A high storage dam at "A" damsite with a low afterbay dam at "C" damsite and relevant two power stations.

Case II : A high storage dam at "A" damsite with a low afterbay dam at "B_1" damsite and relevant two power stations.

Case III : A high storage cam at "B_1" damsite, instead of "A" damsite, with one power station.

Case IV : A high storage dam at "A" damsite with a low afterbay dam at "B_2" damsite and relevant two power stations.

After the detailed reconnaissance and geological investigation, the above Case I has been discarded because of probable water leakage problem of "C" dam due to karstic limestone and expensive construction cost for the dam and power tunnel presumed.

From the point of view of construction materials, the Case II and Case IV is quite similar in the kinds of required materials and their quantities, because the "B_1" damsite and "B_2" damsite are only 800 m apart each other, although the "B_2" damsite has a narrower section than that of the "B_1" damsite.

Therefore, it was recommended to carry out the investigations for construction materials concentrating into Case II and Case III in the Detailed Program for Construction Materials Investigation submitted on November 22, 1982.

As the hydrological study and economic study progressed on one hand, it was gradually clarified that the dam type for a high dam at "A" damsite would be a concrete gravity from the safety against sudden high floods during construction. As for a low afterbay dam at "B_1" or "B_2" damsite, it is also suitable to adopt a concrete gravity dam in order to let the floods safely overflow on the dam.

As for the dam type for "B_1" high dam, it is revealed that the rockfill dam is more economical than the concrete gravity dam.

Based upon the above engineering and economic points of view, comparative studies were made on the different heights of dam at "A" and "B_1" damsites in case of a high dam. At the start of investigations, it was not clear yet that what height of the dam is most economical, but it was approximately presumed that the most optimum height may fall within the range of 130 to 170 m.

Therefore, the investigation of construction materials was planned as to cover the maximum quantities of required materials in case of the highest scale of approximately 170 m.

Based upon the preliminary design made so far, the maximum required quantities of respective materials were estimated as shown below:

Material	Required Max. Volume (m^3)	Case	Conditions
Concrete aggregates	3,300,000	II or IV	"A" site concrete gravity dam (160m high) --- power house (2x45MW) --- "B" site afterbay dam (53.5 m high) --- tunnel --- power house (2x38MW)
Impervious core (Soil Mat'l)	2,500,000		"B_1" site rockfill dam (171m high) --- tunnel --- power house (2x100MW)
Filter (Sand & gravel)	550,000		
Concrete aggregates (Sand & gravel)	510,000		
Ramdom rock	2,350,000 (quarried)		
(Inner shell)	900,000 (excavated)		
Rock	7,800,000 (quarried)		
(Outer shell)	500, 00 (excavated)		

Based on the field reconnaissance, the six borrow areas for soil materials were selected, five quarries were selected for promising potential quarries for rock materials, and sand and gravel borrow area was selected upstream from Batu Ferry Bridge as shown on the attached Location Map Fig. 4-1.

In addition to the above materials the chemical analysis of water samples taken from Matuno River and Magat River was made. The concrete strength tests with their design mix proportions were referred to the results of concrete tests made by NIA laboratory in the Magat Project Construction Office, where 1.1 million m^3 of concrete had been placed in 100 m high concrete gravity dam for spillway during 1978 to 1982.

Among the numerous concrete tests in Magat Project, 115 samples were listed and reviewed. Those test results can be deemed same as the concrete to be made for Matuno Project because the source of the sand and gravel is on the same Magat River though the distance is some 30 km away.

4.2 Sand and Gravel for Concrete

The sand and gravel deposits in Matuno River upstream from the proposed "C" damsite are scattered at many places, but the total quantity is not sufficient as the deposit at one place seems only several thousand cubic meters. The upstream deposits contain also much oversized boulders and gravels with less sand content not suitable for concrete. Those materials may be used for road surface metalling or rough concrete for temporary structures of small scale, but it will require many temporary access roads to collect the materials from river bed.

The wide deposits extend over the flood plains downstream from the river bend after the proposed "C" damsite along the Matuno and Magat Rivers.

Therefore at first 10 test pits (TP-SL-1 to TP-SL-10) were dug along this area for about five km upstream from the Batu Ferry Bridge. From the view-point of convenient transportation, the deposits along the left bank were examined for sieve analysis, specific gravity, nature of original rocks, etc.

The test pits with the depth of 2 to 3 meters were dug and representative samples were taken from approximately each one meter depth. All the results of tests are shown in the attached Table 4-1.

The tests on the first 10 pits revealed that the upstream deposits contain much higher content of boulders larger than 150 mm with less gravels. Therefore additional three test pits (TP-SL-11 to TP-SL-13) were dug in the downstream area as shown in Fig. 4-1.

Engineering Judgement on Test Results

a) The Nature of Mother Rocks

As shown in the following table, the sand and gravel from those deposits are mainly composed of andesite (65%) and basalt (15%) with several percents each of diorite, quartzite, diorite, limestone, conglomerate, sandstone and siltstone. All those particles have been washed for many years into round shape without any weathered parts.

Most particles are sufficiently hard as the results of Los Angeles abrasion test proved that wear rate at 500 revolution is less than 19% and at 100 revolution less than 5%. It is therefore judged that those materials were sound enough for any use.

Geology Classification of Sand and Gravel

Rock Type	Approximate Percent in Volume
A. Volcanic rock	(80%)
Andesite	65
Basalt	15
B. Plutonic rock	(9%)
Diorite	7
Quartz-Diorite	2
C. Sedimentary rock	(11%)
Limestone	4
Conglomerate	3
Sandstone	2
Siltstone	2

b) Specific Gravity and Absorption

The specific gravity of sand and gravel distributes in a normal range of 2.6 to 2.8, but the little lower values of sand shall be confirmed in future during the detailed design stage.

The absorption rate of sand is also in normal range of 2.0 to 3.0% and that of gravel is in range of 0.4 to 0.9%.

c) Gradation

On an average of the whole deposits, it contains 17% of over-sized boulders larger than 150 mm, approximately 51% of gravels between 5 and 150 mm, 31% of sand and 1% of fines.

The sieve analysis of gravels (5 to 150 mm) shows the following gradation:

Size	Content Rate (%)
5 mm to 20 mm	40
20 mm to 40 mm	29
40 mm to 80 mm	24
80 mm to 150 mm	7

The particle size distribution of sand on an average is as follows:

Size	Content Rate (%)
0.074 mm to 0.42 mm	24
0.42 mm to 2.0 mm	50
2.0 mm to 4.0 mm	26

The fineness modulus of sand ranges between 2.93 and 3.63 which implies a little high content of coarse sand. Especially the middle reaches have higher content of coarse sand. Therefore it is recommended to collect the sand mainly from the downstream area.

All the above results satisfy the required quality of fine and coarse aggregates for concrete recommended by the guideline of U.S.B.R. and JSCE. In addition, those deposits have a well graded mix of rounded sand and gravel with excellent mechanical and chemical resistance. Therefore it is recommended to screen only the coarse aggregate into the sizes of larger than 150 mm, 80 to 150 mm, 40 to 80 mm, 20 to 40 mm and 5 to 20 mm for respective use.

The sand smaller than 5 mm requires no further screening except some special case, such as for use of mortar grouting with fine sand. However, it is recommended that some additional investigation shall be made during the detailed design stage in further downstream area from the Batu Ferry Bridge. But the actual collection program should be carefully planned not as to give any bad effect on the scouring of the pier foundation of this bridge and nearby banks.

The total requirement of the sand and gravel in quantity is easily met by excavation of an area of 3,000 m x 500 m width x 3 m depth within this borrow area including the filter materials.

All the gradation curves are summarized in the attached Fig. 4-3.

4.3 Sand and Gravel for Filter Materials

About 550,000 m^3 of filter materials will be required in case of a rockfill dam at the maximum. The above test results of sand and gravel are checked by the standard requirement of filter materials as mentioned below.

The standard criteria for filter materials are as follows:

a) $$\frac{\text{15\% particle size of filter material (F15)}}{\text{15\% particle size of core material (C15)}} > 5$$

b) $$\frac{\text{15\% particle size of filter material (F15)}}{\text{85\% particle size of core material (C85)}} < 5$$

c) It is desirable that gradation curve of filter materials is approximately in parallel with that of the core materials.

d) If the core materials contain coarse materials, the above conditions (a) and (b) shall be applied to the materials under 25 mm size.

e) Filter materials shall not be cohesive and shall not contain more than 5% of finer particles passing No.200 (0.074 mm) sieve.

According to the results of tests on core materials, the value of C15 ranges between 0.01 to 0.05 mm and that of C85 falls within 1.5 to 20 mm. Then the sand and gravel gradation tests revealed that the value of F15 is in a range from 0.8 to 2.5 mm. This F15 range fully satisfies the above conditions (a) and (b). In addition, the grading curve of sand and gravel shown in Fig. 4-3 is nearly parallel with that of core materials shown in the attached Fig. 4-4, which satisfies the condition (c).

The content of the finer particles than 0.074 mm in the filter materials tested is less than 2%, which satisfies the above condition (e).

Therefore the sand and gravel materials in the same borrow area for concrete aggregates can be used for the filter materials as well with sufficient safety. However, it should be noted that when actual field operation of filter layer laying, minimum seggregation of filter materials should be attained.

4.4 Concrete Tests and Mix Proportion

In the Magat Dam Project about 1.1 million m^3 of concrete was placed into a concrete gravity dam for spillway. Concrete and cement quality control has been made all through the construction period by the field laboratory in the Project Office. Among the numerous concrete tests the representative results of 115 were referred as shown in the attached Table 4-3.

The design mix types were classified in 8 classes from "A" to "H" type. The Table 4-1 shows the design mix of "A" to "F" types and the 28-days strength of the actual tests for A, D, E and F types as the representatives.

The "A" and "B" type concrete aiming at more than 4,000 psi (281 kg/cm^2) with different maximum sizes of gravels achieved the average 28-day strength of 4,213 psi (296 kg/cm^2). Also the average strengths of Type D, E and F showed 10% higher strength than the design strength.

The cement content and water-cement ratio for respective type of concrete were well designed for the respective purpose. All the aggregates used for those concrete were collected from the Magat River bed materials about 30 km downstream from the proposed borrow area for Matuno Project. Only the slight difference of the fineness modulus of sand is identified between Magat sand and Matuno sand, but it may not affect so much on the concrete strength although some difference may occur in consistency of concrete.

All those test results proved that there is no problem for the concrete strength.

The cement used for the Magat Dam is local made cement produced by Filippinas Cement Co., Northern Cement Co., and Hi-Cement Co. The results of cement tests are shown in the attached Table 4-4. All the test values are within the normal standard of ordinary Portland Cement assuring a sound quality.

Chemical analysis was made on 7 water samples from Matsuno River and two samples from Magat River, the results of which are shown in the attached Table 2-1. Both river waters are chemically quite clean and can be used for any purpose.

The slightly higher content of sodium and chlorine in Magat River water than the Matuno River is attributable to the existence of salt spring near Salinas, the upstream of the Sta. Cruz River joining to Magat.

The pH values fall within a range from 7.6 to 8.5 showing slightly weak alkalinity. The calcium content of 17 to 38 ppm is attributable to the existence of limestone in the basin.

Thus all the concrete materials were proved to be sound.

In order to estimate the construction cost, the tentative design mix of concrete was studied, the results of which are shown in the attached Table 4-5.

The six types of concrete may be required for respective use as shown in the table from type "A" to "F". The type "F" is for mass concrete for a dam in case of concrete gravity dam.

4.5 Soil Materials for Impervious Core

Seven promising borrow areas for soil materials were selected by the reconnaissance as shown on the attached Fig. 4-1, three areas upstream from "A" damsite and four areas downstream from "B" damsite. Before JICA survey started from February 1982, NIA and Philtech Inc. have made a preliminary investigation on soils as reported in the Report on the Matuno River No.1 Multipurpose Project Pre-Feasibility Study (by Philtech, 1981) and the Reconnaissance Geology Report on the Matsuno Reservoir Project (by NIA, July 1981). The three soil borrow areas recommended by Philtech are shown on the same location map as Borrow-I, Borrow-II and Borrow-III.

The Philtech has dug three test pits in Borrow-II and five pits in Borrow-I, both are located at upstream talus deposits from the proposed "A" damsite. The Philtech also dug 11 test pits in Borrow-III area located at gentle hill just behind the Barangay Sto. Domingo.

The results of the soil tests carried out by the Philtech on 20 samples taken from those test pits are shown in the attached Table 4-6.

The JICA staff selected additional four borrow areas, namely one upstream from "A" damsite called as Borrow-A for "A" dam, and three downstream from "B_1" damsite called as Borrow-B-I, B-II and B-III. Six test pits were added mainly in B-I to B-III borrow areas because the possibility of "B_1" high dam with rockfill type seems most promising.

The 14 soil samples from the above test pits were tested in NIA Cabanatuan Material and Central Laboratory. The results are shown in the attached Table 4-7. The representative gradation curves are shown in Fig. 4-4.

The sample number tested is as follows:

a)	Natural moisture content:	47 samples
b)	Specific gravity:	
	Minus No.4 sieve	34 samples
	Plus No.4 sieve	18 samples
c)	Gradation test:	34 samples

d) Liquid limit and plastic limit test: 34 samples
e) Absorption test (Plus No.4 sieve): 18 samples
f) Compaction test: 25 samples
g) Cone penetration test: 5 samples
h) Permeability test: 5 samples

All the above tests were based on ASTM Standard except the compaction test, which is carried out based on the modified ASTM: D1557 Method A (6.89 cm. kg/cm^3, or 1.2 times of standard compaction energy Ec).

For the permeability test, the test samples were prepared at the optimum moisture content and at the maximum dry density.

Engineering Judgement on Soil Materials

a) Physical Properties

The natural moisture content of soils from Borrow-B-I, B-II and B-III ranges from 10.1% to 22.8% being averaged at 17.2%. But those samples were taken under wet condition in the rainy season. Therefore it may be lowered in the dry season by 2 or 3%. The optimum moisture content is 13.5% on an average.

Whereas, the natural moisture content of soils taken from Borrow-I and II upstream from "A" damsite is approximately 31%. This high content may be mainly attributable to the nature of the mother rocks, namely those soils are talus deposits originated from siltstone composed of fine particles, consequently containing high silt and clay content. The optimum moisture content for those soils is 23.4% on an average. It means strong drying operation will be required for embankment of core zone.

The natural moisture content of soils taken from Borrow-III is as high as 45% while the optimum moisture content is 24.3%. Those soils are talus deposits originated from limestone. At the same time this talus area has very high groundwater level.

From the above test results, it is obvious that the soils from Borrow-I, II and III are not suitable for impervious core zone.

On the contrary, the soil materials in Borrow B-I, B-II and B-III are originated from conglomerate with coarse grains and some pebbles, consequently having low moisture content favorable for use.

The plasticity index (Ip) of those soils ranges from 10 to 18% being averaged at 14.1%. It can be classified into medium plastic soils resistible against piping action.

The content rate of gravel, sand and fines components is 18, 27 and 55% on an average. The maximum gravel size is 75 mm to 100 mm which are decomposed to some extent as the specific gravity is as low as 2.41 with absorption rate of 5.5%. Those materials can be easily excavated by bulldozer with ripper.

The above soils from Borrow B-I, II and III are mostly classified into GC, SM and CL in the Unified Soil Classification System. The depth of test pits are three meters only but it seems that the suitable materials can be obtained even at the depth of 8 to 13 m by the eye witness on the core samples taken by borings made in those borrow areas. It seems that as the depth increases the soils contain more decomposed cobbles and pebbles, may be classified into mainly GM and GC.

b) Mechanical Properties

The compaction test was carried out in accordance with ASTM D-1557, Method A modified. The materials used for this test were Minus No.4 sieve, and compaction energy was 6.89 cm.kg/cm^3 which is 1.2 times energy of Standard Proctor test.

Borrow Area	d max (g/cm^3)	(mean)	O.M.C. (%)	(mean)
Borrow I, II	1.435-1.725	(1.558)	15.3-29.7	(23.4)
Borrow III	1.313-1.807	(1.512)	14.6-30.6	(24.3)
Borrow-B-I, II, III	1.784-1.895	(1.818)	11.1-15.2	(13.5)

As shown in the above table, the dry densities of the soil materials taken from Borrow I, II and III are low due to high optimum moisture content and very rich fine silt and clay. It is therefore recommended to use the soils originated from conglomerate in Borrow B-I, B-II and B-III.

If the "A" dam requires soil materials it can be recommended to take materials from Borrow A, the soils of which are also originated from conglomerate although no soil tests have been carried out.

The results of cone penetration test shows that the penetration resistance of the compacted samples under the optimum moisture content ranges from 93 to 128 kg/cm^2. Those high resistance values assure an excellent trafficability for the heavy construction equipment without any trouble.

The permeability test made on the compacted samples under the optimum moisture content clarified that those samples can have very low permeability in an order of 1×10^{-7} cm/sec which assures sufficient water tightness of core zone.

From all the above engineering points of view, those soil materials in Borrow B-I, B-II, B-III and probably Borrow A will offer very suitable materials for embankment of core zone for a rockfill dam. The quantity is also sufficient even for the maximum requirement.

4.6 Rock Materials

As for the rock materials to be used for a rockfill dam, abundant fresh conglomerate around the "A" and "B_1" damsites offers good sources of rock materials.

Fresh conglomerate rocks were tested for its compressive strength as shown below:

Compressive Strength (kg/cm^2)

Condition	Sample No.1	Sample No.2	Sample No.3	Sample No.4
Under natural condition	625	780	597	-
Saturated	539	350	-	345

As observed in the above table the fresh conglomerate has a quite sufficient strength even under saturated condition as high as more than 300 km/cm^2. It has enough soundness for use in any rockfill zones. Also the limestone extending in this area has enough hardness for any use.

The selection of rock quarries was therefore made mainly from the view-point of easy transportation to damsite. As shown on the attached Fig. 4-1 (Location Map) one quarry for "A" damsite is selected at Quarry A located about 3 km upstream from the damsite.

Three promising quarries are selected for "B_1" dam, namely Quarry B-I, B-II and B-III which are located within three km range from the "B_1" dam.

The Quarry C is selected as a reserve for the sake of caution if the riprap materials are short in quantity. The Quarry C is located at two km downstream from the proposed "C" damsite and composed of hard massive diorite.

Table 4-1 CONSTRUCTION MATERIALS INVESTIGATION
SUMMARY OF RESULTS - SAND AND GRAVEL-

TEST PIT NO.	TOTAL DEPTH EXCAVATED (M)	THICKNESS OF TOP SOIL & ORGANIC (M)	WATER LEVEL (-M)	SAMPLE NO.	SAMPLE DEPTH (M)	SPECFIC GRAVITY		ABSORPTION (%)		GRADATION					ORGA IMPU IN SA
						MINUS NO.4	PLUS NO.4	MINUS NO.4	PLUS NO.4	MAX.SIZE (CM)	PERCENTAGE RANGE >150 MM (COBBLES)	150 ~ 5MM (GRAVELS)	< 5MM (SAND)	FINENESS MODULUS	
TP-SL-1	2.50	0.70	NONE	TP-SL-1A	0.7 ~ 1.7	2.62	2.81	2.80	0.40	23	28	60	12	3.53	-
				TP-SL-1B	1.7 ~ 2.5	2.62	2.70	3.00	0.85	24	33	54	13	3.66	-
TP-SL-2	2.20	0.30	1.20	TP-SL-2A	0 ~ 1.0	2.66	2.76	2.80	0.51	20	26	47	27	3.63	-
				TP-SL-2B	1.0 ~ 2.2	2.64	2.79	2.80	0.48	28	30	53	17	3.15	-
				RS-TP-SL-2	0 ~ 2.2	2.65	2.74	2.80	0.55	38	27	40	33	3.34	NON
TP-SL-3	2.00	NONE	1.50	TP-SL-3A	0 ~ 1.0	2.62	2.78	2.70	0.45	23	23	48	29	3.18	-
				TP-SL-3B	1.0 ~ 2.0	2.62	2.77	2.80	0.69	23	24	44	32	3.32	-
				RS-TP-SL-3	0 ~ 2.0	2.64	2.75	2.80	0.65	26	21	52	27	3.22	NO
TP-SL-4	2.00	1.00	1.70	TP-SL-4A	1.0 ~ 2.0	2.64	2.76	2.40	0.63	20	31	41	28	2.93	-
TP-SL-5	2.00	NONE	NONE	TP-SL-5A	0 ~ 1.0	2.63	2.75	2.40	0.61	25	22	45	33	3.25	-
				TP-SL-5B	1.0 ~ 2.0	2.60	2.72	3.00	0.78	23	33	41	26	3.01	-
TP-SL-6	2.00	0.70	1.60	TP-SL-6A	0.7 ~ 2.0	2.65	2.74	2.10	0.63	20	25	42	33	3.18	-
TP-SL-7	2.00	NONE	1.90	TP-SL-7A	0 ~ 1.0	2.62	2.74	3.00	0.61	20	19	47	34	3.10	-
				TP-SL-7B	1.0 ~ 2.0	2.62	2.82	2.80	0.39	23	27	45	28	3.62	
TP-SL-8	2.00	NONE	1.70	TP-SL-8A	0 ~ 1.0	2.64	2.81	2.80	0.39	23	20	53	27	3.19	
				TP-SL-8B	1.0 ~ 2.0	2.61	2.78	2.90	0.43	48	27	47	26	3.02	
TP-SL-9	2.00	NONE	NONE	TP-SL-9A	0 ~ 1.0	2.60	2.77	2.80	0.45	20	26	42	32	2.95	
				TP-SL-9B	1.0 ~ 2.0	2.62	2.74	2.70	0.81	21	18	46	36	2.83	
TP-SL-10	3.00	NONE	NONE	TP-SL-10A	0 ~ 1.0	2.63	2.76	2.70	0.90	18	11	58	31	3.04	
				TP-SL 10B	1.0 ~ 2.0	2.62	2.71	2.70	0.62	19	10	56	34	3.11	
				TP-SL-10C	2.0 ~ 3.0	2.65	2.76	2.00	0.90	18	16	56	28	3.07	-
				RS-TP-SL-10	0 ~ 3.0	2.63	2.71	2.30	0.60	28	12	60	28	3.40	NO
TP-SL-11	2.00	1.10	NONE	TP-SL-11A	1.0 ~ 2.0	2.65	2.75	2.80	0.62	28	25	45	30	3.15	
TP-SL-12	2.00	1.00	0.90	TP-SL-12A	1.0 ~ 2.0	2.63	2.77	2.60	0.69	20	19	40	41	3.02	
TP-SL-13	2.00	0.80	NONE	RS-TP-SL-13	0.7 ~ 2.0	2.60	2.78	2.80	0.39	25	7	62	31	2.95	NO

-1 CONSTRUCTION MATERIALS INVESTIGATION

SUMMARY OF RESULTS - SAND AND GRAVEL-

GRAVITY	ABSORPTION (%)		GRADATION					ORGANIC	SODIUM SULPHATE		LOS ANGELES		UNIT WEIGHT		REMARKS
PLUS	MINUS	PLUS	MAX.SIZE	PERCENTAGE RANGE			FINENESS	IMPURITY	SOUNDNESS	LOSS AFTER: 5 CYCLES	ABRASION		(TON/M³)		
NO.4	NO.4	NO.4	(CM)	>150 MM (COBBLES)	150 ~ 5MM (GRAVELS)	< 5MM (SAND)	MODULUS	IN SAND	SAND	GRAVEL MAX.SIZE: 1 1/2"	WEAR AT: 100 REV. %	WEAR AT: 500 REV. %	SAND	GRAVEL MAX.SIZE: 3"	
2.81	2.80	0.40	23	28	60	12	3.53	–	–	–	–	–	–	–	
2.70	3.00	0.85	24	33	54	13	3.66	–	–	–	–	–	–	–	
2.76	2.80	0.51	20	26	47	27	3.63	–	–	–	–	–	–	–	
2.79	2.80	0.48	28	30	53	17	3.15	–	–	–	–	–	–	–	
2.74	2.80	0.55	38	27	40	33	3.34	NONE	12.0	7.0	2.8 ~ 4.9	14.1 ~ 19.0	2.219	2.214	
2.78	2.70	0.45	23	23	48	29	3.18	–	–	–	–	–	–	–	
2.77	2.80	0.69	23	24	44	32	3.32	–	–	–	–	–	–	–	
2.75	2.80	0.65	26	21	52	27	3.22	NONE	10.5	7.2	2.7 ~ 3.4	13.9 ~ 16.1	2.214	2.204	
2.76	2.40	0.63	20	31	41	28	2.93	–	–	–	–	–	–	–	
2.75	2.40	0.61	25	22	45	33	3.25	–	–	–	–	–	–	–	
2.72	3.00	0.78	23	33	41	26	3.01	–	–	–	–	–	–	–	
2.74	2.10	0.63	20	25	42	33	3.18	–	–	–	–	–	–	–	
2.74	3.00	0.61	20	19	47	34	3.10	–	–	–	–	–	–	–	
2.82	2.80	0.39	23	27	45	28	3.62	–	–	–	–	–	–	–	
2.81	2.80	0.39	23	20	53	27	3.19	–	–	–	–	–	–	–	
2.78	2.90	0.43	48	27	47	26	3.02	–	–	–	–	–	–	–	
2.77	2.80	0.45	20	26	42	32	2.95	–	–	–	–	–	–	–	
2.74	2.70	0.81	21	18	46	36	2.83	–	–	–	–	–	–	–	
2.76	2.70	0.90	18	11	58	31	3.04	–	–	–	–	–	–	–	
2.71	2.70	0.62	19	10	56	34	3.11	–	–	–	–	–	–	–	
2.76	2.00	0.90	18	16	56	28	3.07	–	–	–	–	–	–	–	
2.71	2.30	0.60	28	12	60	28	3.40	NONE	10.4	11.3	2.9 ~ 3.0	15.1 ~ 16.4	2.225	2.156	
2.75	2.80	0.62	28	25	45	30	3.15	–	–	–	–	–	–	–	
2.77	2.60	0.69	20	19	40	41	3.02	–	–	–	–	–	–	–	
2.78	2.80	0.39	25	7	62	31	2.95	NONE	11.6	8.3	3.2 ~ 3.6	15.8 ~ 17.5	2.473	2.278	

Table 4-2 ADDITIONAL TEST RESULT - SAND AND GRAVEL -

(MAGAT RIVER DOWN STREAM OF BATU FERRY BRIDGE)

TEST PIT & SAMPLE NO.	DEPTH (m)	SPECIFIC GRAVITY		ABSORPTION (%)		GRADATION				REMARKS
							PARCENTAGE RANGE			
		MINUS NO.4 (SAND)	PLUS NO.4 (GRAVEL)	MINUS NO.4 (SAND)	PLUS NO.4 (GRAVEL)	MAX. SIZE (cm)	$150^{m/m}-5^{m/m}$ % (GRAVEL)	$< 5^{m/m}$ % (SAND)	FINESS MODULUS OF SAND	
MG-10A	0.3 - 0.6	2.67	2.81	1.67	0.96	8	89	11	3.08	
MG-10B	0.9 - 1.2	2.65	2.86	1.88	1.28	8	84	16	2.82	
MG-14A	0.3 - 0.6	2.65	2.78	1.31	1.15	6	79	21	3.31	
MG-14B	0.9 - 1.2	2.63	2.64	1.57	1.10	8	85	15	2.98	
MG-18A	0.3 - 0.6	2.65	2.93	0.65	0.31	8	90	10	2.46	
MG-18B	0.9 - 1.2	2.63	2.75	1.86	1.42	10	85	15	2.96	
MG-22A	0.3 - 0.6	2.69	2.73	1.52	1.39	6	79	21	3.11	
MG-22B	0.9 - 1.2	2.68	2.75	1.88	0.78	10	83	17	2.94	
Average		2.66	2.78	1.54	1.05	6 - 10	84.2	15.8	2.96	

TABLE 4-3 Magat Project, Concrete Control Results

	Concrete Type	A	B	C	D	E	F
Design Mix and Specification	Strength σ_{28} Coeff. of variation	4,000 psi 15%		3,000 psi 15%			2,000 psi 15%
	Max. size of agg. (in)	1 1/2	3	3/4	1 1/2	3	6
	Slump ±1/2 (in)	2 1/2					
	Air content ± 1 (%)	4.5	3.5	6.0	4.5	3.5	3.0
	Sand, percent s/a (%)	33	22	41	35	26	22
	Water cement ratio w/c	0.48	0.46	0.59	0.58	0.58	0.72
	Water (kg)	131	107	148	133	113	98
	Cement (kg)	273	234	252	231	194	136
	Sand FM = 2.85 (kg)	682	475	825	742	582	519
	Coarse aggregate (kg)	1,370	1,691	1,175	1,352	1,632	1,810
Control Test Results	Period	July 20 to Aug. 16, 1981					
	Sample No. Volume m^3	25 3,943	15 3,338	3 333	54 15,099	65 17,086	16 5,991
	Slump av. cm	6.58	6.69	7.52	6.60	6.41	8.91
	Air content av. %	4.34	4.29	4.57	4.18	3.83	3.79
	Unit weight av. ton/m^3	2.459	2.510	2.431	2.453	2.504	2.556
	Period	July 20 to Aug. 16 '81, Dec. 16 '81 to Jan. 15 '82					
	Strength σ_{28} (psi) mean max. min.	4,213 4,622 3,959	- - -	- - -	3,330 3,641 2,873	3,355 3,694 3,052	2,397 3,141 2,313
	(sample no.)	(26)	-	-	(48)	(29)	(12)
	Coeff. of varia.	6.33	-	-	8.66	6.70	8.28

Table 4-4 CEMENT CONTROL RESULTS

DATE	BRAND OF CEMENT	BLAIN AIR FINENESS	NORMAL CONSIS-TENCY	VOL. WATER USED PER 500 GRAMS CEMENT	SETTING TIME INITIAL (MIN.)	SETTING TIME FINAL (HR-MIN)	NORMAL FLOW	COMPRESSIVE STRENGTH (PSI) 3 DAYS	7 DAYS	28 DAYS
A S T M STANDARD		MINIMUM 2,800	-	-	MINIMUM 60	MAXIMUM 10 HR.	100-115	-	-	-
12-16-81	Filipinas	3,001	25.6	128	146	4-48	102	1,298	2,198	3,598
do	do	3,046	25.6	128	148	4-46	102	1,383	2,392	3,610
12-19-81	do	3,184	25.6	128	151	4-51	102	1,314	2,298	3,599
do	do	3,098	25.6	128	129	4-48	102	1,384	2,343	3,600
12-21-81	do	2,998	25.6	128	135	5-15	102	1,336	2,286	-
12-21-81	Northern	3,106	25.2	126	138	5-10	102	1,349	2,314	-
12-23-81	do	3,100	25.6	128	140	5-00	106	1,298	2,286	-
12-24-81	Filipinas	3,010	25.0	128	142	5-02	106	1,341	2,243	-
12-26-81	do	3,314	25.6	128	146	4-52	108	1,343	2,292	-
12-27-81	do	3,261	26.0	13-	148	4-58	108	1,458	2,418	-
do	do	3,142	26.0	130	144	5-00	110	1,424	2,301	-
12-29-81	do	3,019	26.0	130	155	5-01	110	1,301	2,341	-
12-29-81	Northern	3,098	25.6	128	158	5-15	106	1,364	2,242	-
12-30-81	do	2,996	25.6	128	136	5-30	102	1,286	2,199	-
1- 2-82	Filipinas	3,098	25.6	128	141	5-10	100	1,343	2,298	-
do	do	3,041	25.2	126	130	4-48	100	1,446	2,416	-
1- 4-82	do	3,035	25.2	126	142	4-56	106	1,348	2,342	-
1- 6-82	do	3,000	25.2	126	142	4-58	106	1,342	2,296	-
do	do	3,016	25.6	128	148	5-01	102	1,298	2,198	-
do	do	3,016	25.6	128	148	5-01	102	1,314	2,284	-
3- 1-81	Hi-Cement	3,108	-	-	125	5-15	-	1,345	2,218	-
1-23-81	Filipinas	3,284	-	-	148	4-40	-	1,720	2,345	-
1- 3-81	Northern	3,016	-	-	192	5-10	-	1,475	2,318	-

Table 4-5 PROPOSED CONCRETE DESIGN MIX

TYPE		A	B	C	D	E	F
STRENGTH *-3	kg/cm^2	280	210	210	210	140	180
MAX. SIZE OF AGGREGATE	mm	40	20	40	80	80	150
SLUMP	cm	7	10	7	5	5	2.5
AIR CONTENT	%	4.0	5.0	4.0	0.5	0.5	0.2
SAND, PERCENT	(s/a) %	36.0	44.5	36.0	32.5	32.5	29.5
WATER-CEMENT *-3 RATIO	w/c	0.49	0.61	0.61	0.61	0.70	0.50
WATER	kg	150	180	150	139	109	102
CEMENT	kg	308	294	245	227	156	204
SAND *-1	(F.M=3.05) kg	692	809	712	690	737	666
COARSE AGGREGATE *-2	G20 kg	620	1,010	633	430	459	318
	G40 kg	620		633	430	459	318
	G80 kg				573	612	398
	G150 kg						556

*-1 Sand: Gs = 2.62 F.M = 3.05 ------------MATUNO Fine aggregate
*-2 Coarse aggregate: Gs = 2.76 ------------MATUNO Coarse aggregate
*-3 $\sigma 28 = -61 + 166.w/c$ (kg/cm^2): -------MAGAT Dam Control data

Air and water contents, the proportions of fine and coarse aggregate -------- U.S.B.R Concrete Manual

Concrete Types will be mainly used for as following:
Type A: water way structures, Type B: side walls, arch in tunnel lining, power house, Type C: general civil structures, Type D: heavy mass construction, Type E: foundation, Type F: concrete gravity dam body.

Teble 4-6 SUMMARRY TEST RESULTS FOR CORE MATERIAL (1)

BORROW AREA	TEST PIT, SAMPLE NO.	DEPTH (M)	NATURAL MOISTURE CONTENT (%)	OPTIMUM MOISTURE CONTENT (1 Ec) %	DIFFERENCE BETWEEN NATURAL & OPTIMUM MOISTURE CONTENT %	MAXIMUM DRY DENSITY TON/M^3	SPECIFIC GRAVITY		GRADATION				ATTERBERG LIMIT		GROUP SYMBOL	ABSORPTION (COARSE) %
							MINUS NO.4	PLUS NO.4	MAX SIZE (MM)	GRAVEL %	SAND %	FINES %	LIQUID LIMIT %	PLASTICITY INDEX %		
I	I - 1	15 ~ 4.0	38.2	29.7	+ 8.5	1.435	2 63	–	5 0	0	1 3	87	55	25	MH	–
	I - 2	1 0 ~ 5 0	21.2 ~ 26.6	15 3	+11.3 ~ +5.9	1.725	2 72	2 51	75.0	1 3	2 2	65	40	1 5	CL	5.0
	I - 3	1.0 ~ 4.0	316 ~ 31.7	27 3	+ 4 4 ~ +4.3	1.495	2 66	–	5.0	0	1 6	84	49	1 6	ML	–
	I - 4	1 0 ~ 4.0	30.6 ~ 30.9	19 2	+11.7 ~ +11.4	1.676	2.73	2.51	75 0	1 7	30	53	41	1 7	CL	3.8
	I - 5	1 0 ~ 4.0	24 2 ~ 42 0	23.0	+19.0 ~ +1.2	1 580	2 72	–	5.0	0	2 6	74	45	20	CL	–
II	II - 1	1 0 ~ 4.0	337 ~ 36 1	25 5	+10.6 ~ +8.2	1 479	2 69	–	5 0	0	6	94	58	25	MH	–
	II - 2	1 0 ~ 5 0	25 7 ~ 48.0	25.7	+22 3 ~ ± 0	1.487	2.65	–	5.0	0	7	9 3	5 8	3 1	CH	–
	II - 3	1 0 ~ 4.0	19 3 ~ 38.7	21 6	+17 1 ~ -2.3	1 590	2.63	2.34	75.0	30	1 5	5 5	42	1 5	ML	34.9
III	III - 1	1.0 ~ 2 7	32 6 ~ 44 6	25 7	+18 9 ~ +6.9	1.528	2.68	–	5.0	0	5 3	47	45	1 5	SM	–
	III - 2	1 0 ~ 5 0	33 8 ~ 40.6	23 3	+17.3 ~ +10.5	1.515	2 56	–	5.0	0	3 7	63	46	1 4	ML	–
	III - 4	1 0 ~ 3.0	37 5	25 3	+12.2	1 479	2 51	–	5.0	0	3 1	69	39	1 4	CL	–
	III - 5	1.5 ~ 3.0	36.9 ~ 62 9	23.9	+39.0 ~ +13.0	1 568	2.73	2.59	50 0	23	1 4	6 3	5 9	2 9	MH	2 9
	III - 6	1 0 ~ 3 0	566 ~ 71.4	33 2	+38 2 ~ +23 4	1 313	2.60	–	5 0	0	1 4	8 6	105	59	MH	–
	III - 7A	1 0 ~ 2.0	65.5	28 5	+ 37 0	1.426	2 60	–	5 0	0	35	6 5	1 1 3	85	CH	–
	III - 7B	2.0 ~ 3 0	28 9	17 2	+ 1 1 7	1 670	2.61	2.43	75.0	42	1 7	4 1	5 3	20	GM	3.8
	III - 8	0 5 ~ 2 5	14.7 ~ 60.7	30 6	+30.1 ~ -15.9	1 346	2 63	–	5 0	0	7	9 3	95	50	MH	–
	III - 9	1.0 ~ 3 0	50.1	28 0	+22 1	1 410	2.65	2.34	100.0	1 0	2 4	6 6	4 1	6	ML	4.2
	III - 10A	1 0 ~ 2 5	74 7	19 3	+55.4	1.533	2.50	–	5.0	0	6	9 4	109	68	CH	–
	III - 10B	2 5 ~ 4.0	35.0	14 6	+20.4	1 807	2 79	2.59	75 0	4 6	20	3 4	55	30	GM	2.5
	III - 11	1 0 ~ 2 5	38.3	22.2	+16 1	1.556	2.56	2.30	37 5	1 6	4 6	38	37	8	SM	9.4

Table 4-7 SUMMARRY TEST RESULTS FOR CORE MATERIAL (2)

BORROW AREA	TEST PIT, SAMPLE NO	DEPTH (M)	NATURAL MOISTURE CONTENT (%)	*OPTIMUM MOISTURE CONTENT 1.2Ec (%)	DIFFERENCE BETWEEN NATURAL & OPTIMUM MOISTURE CONTENT	*MAXIMUM DRY DENSITY TON/M³	SPECIFIC GRAVITY MINUS NO.4	SPECIFIC GRAVITY PLUS NO.4	GRADATION MAX SIZE (MM)	GRADATION GRAVEL %	GRADATION SAND %	GRADATION FINES %	ATTERBERG LIMIT LIQUID LIMIT %	ATTERBERG LIMIT PLASTICITY INDEX %	GROUP SYMBOL	ABSORPTION (COARSE) %
B-I	TM-6-1	1.0	16.8	11.1	+7 9	1 810	2.679	2.557	100 0	42	16	42	36	12	GC	5.1
	TM-6-2	2.0	18 8				2.636	2 386	75.0	37	18	45	35	11	GC	5.0
	TM-6-3	3.0	21.3				2 678	2 409	75.0	38	16	46	37	13	GC	2.5
B-II	TM-1-1	1.0	21.4	15 0	+2 9	1 784	2 650	2.623	50 0	23	23	54	51	18	MH	6.2
	TM-1-2	2 0	17 7				2.622	2.519	50 0	11	19	70	40	13	ML	10.4
	TM-1-3	3.0	14.7				2.679	2.257	37.5	2	42	56	39	15	CL	5.2
	TM-2-1	1 0	17 9	15.2	+3 9	1 793	2.665	–	5 0	0	55	45	41	15	SM	–
	TM-2-2	2.0	16.6				2 650	–	5.0	0	20	80	29	10	CL	–
	TM-2-3	3.0	22.8				2.650	–	5.0	0	22	78	37	15	CL	–
	TM-3-1	1.0	20.2	12.0	+1.9	1.895	2 650	2.460	100 0	40	24	36	36	14	GC	2.7
	TM-3-2	2.0	11 3				2 665	–	5 0	0	42	58	35	17	CL	–
	TM-3-3	3.0	10 1				2 651	2 244	75.0	35	26	39	30	12	GC	2 5
B-III	TM-5-1	1 5	17 5 (1.0 m)	14.2	+2.0	1 810	2 665	2 240	50.0	30	16	54	40	17	CL	9.7
	TM-5-2	3 0	17 7 (2.0 m)				2.694		5.0	0	34	66	38	16	CL	–
			13.5 (3.0 m)													
AVERAGE VALUE			17 2	13.5	+3.7	1 818	2. 660	2.411		18 4	26.6	54 9	37.4	14 1	GC ~ CL	5.5

Remarks :

- Coeff. of permeability (k) (1 2 Ec, max dry density, minus NO.4 materal) :
  - TM-6 (1.2,3 mixed) 6.4×10^{-8} cm/sec
  - TM-1 (-do-) 2.0×10^{-8} cm/sec
  - TM-2 (-do-) 8.2×10^{-8} cm/sec
  - TM-3 (-do-) 2.0×10^{-8} cm/sec
  - TM-5 (1.2 mixed) 8.9×10^{-9} cm/sec
- *Compaction test : Dry to wet method, 150cm ø mold, minus NO.4 meterial used.
  1.2 Ec = 6.89cm kg/cm² compaction energy

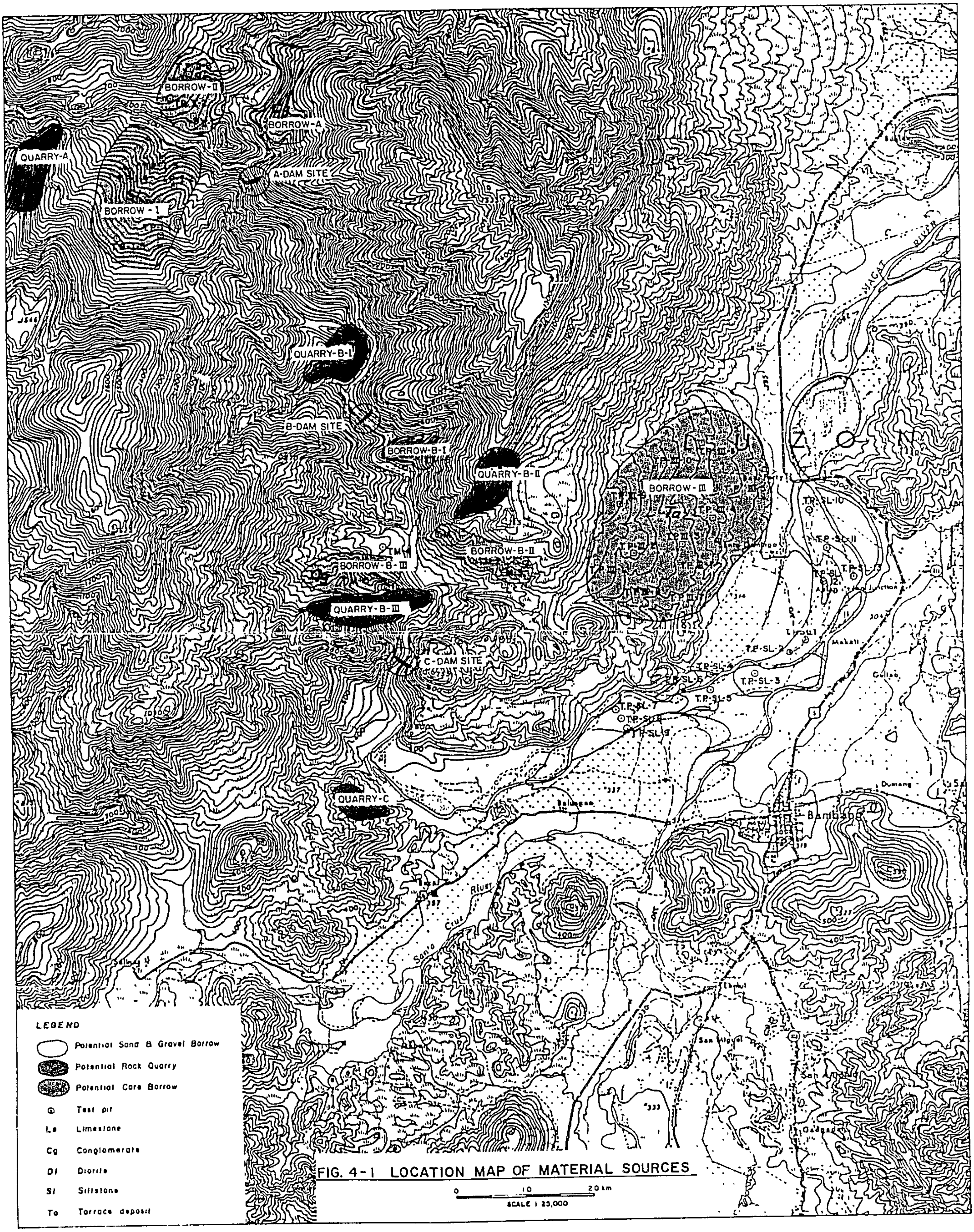


FIG. 4-1 LOCATION MAP OF MATERIAL SOURCES

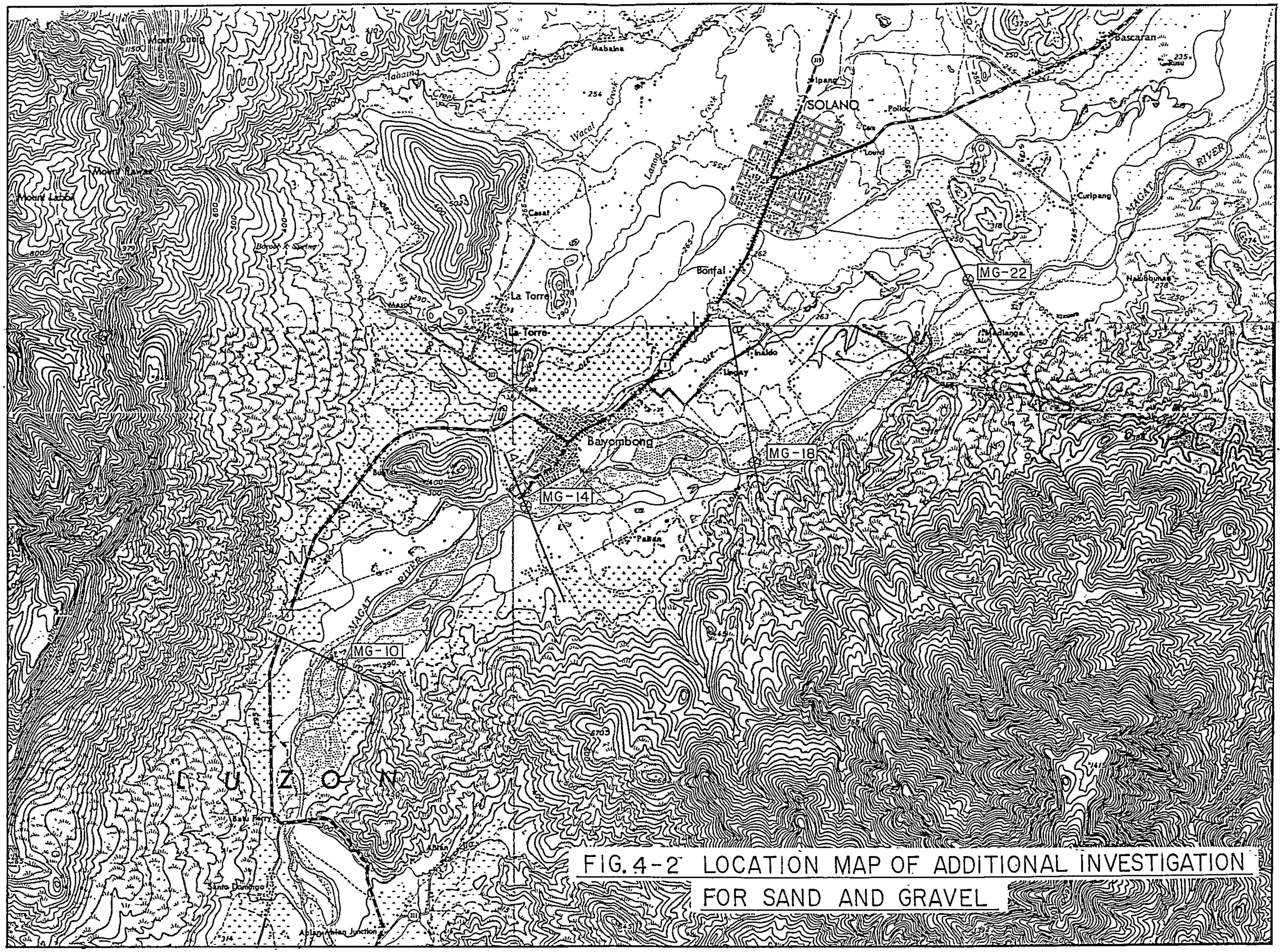


FIG. 4-2 LOCATION MAP OF ADDITIONAL INVESTIGATION FOR SAND AND GRAVEL

Fig.4-3 Comparison of Gradation for Sand and Gravel

GRAIN SIZE ACCUMULATION CURVE

粒径加積曲線

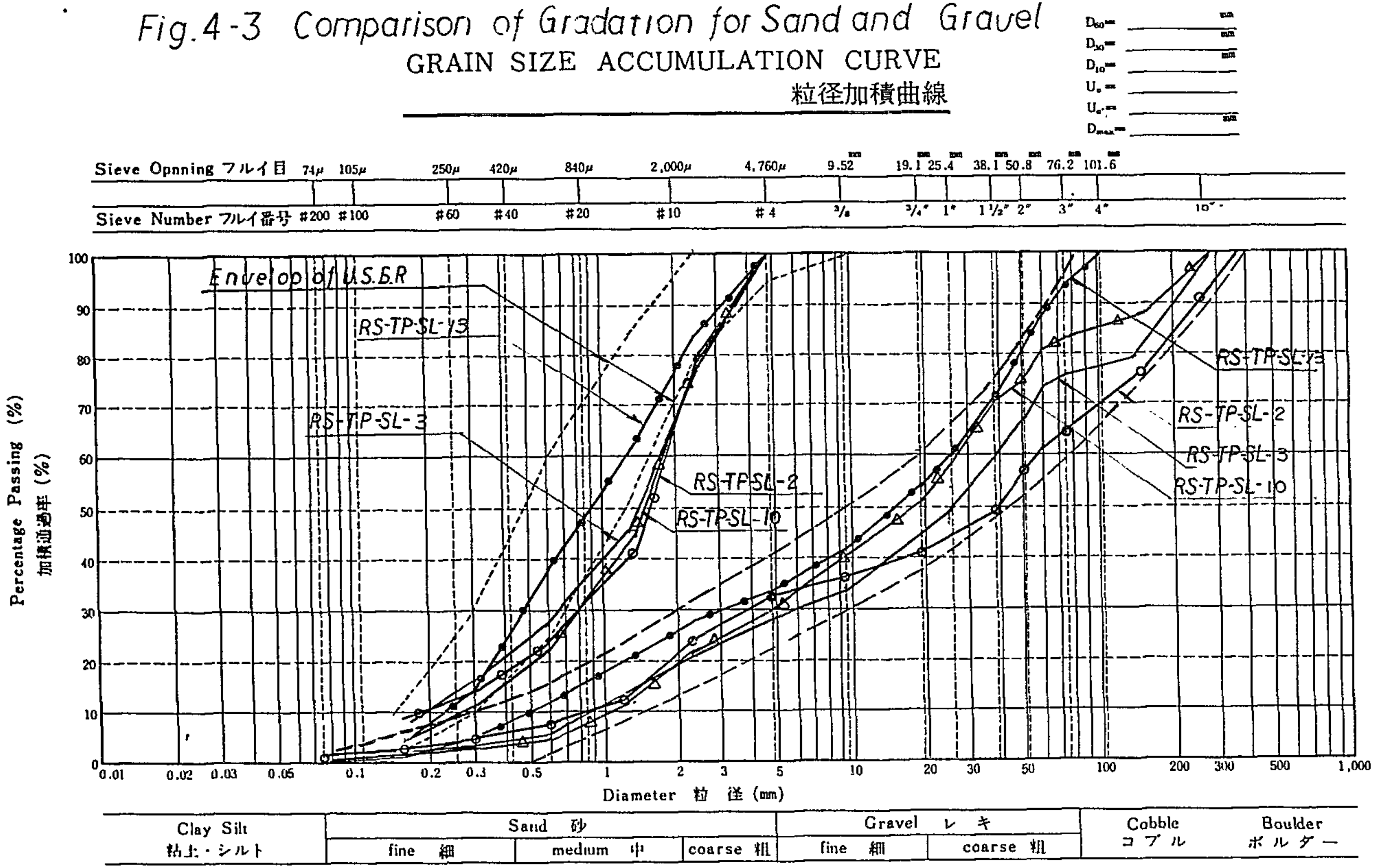


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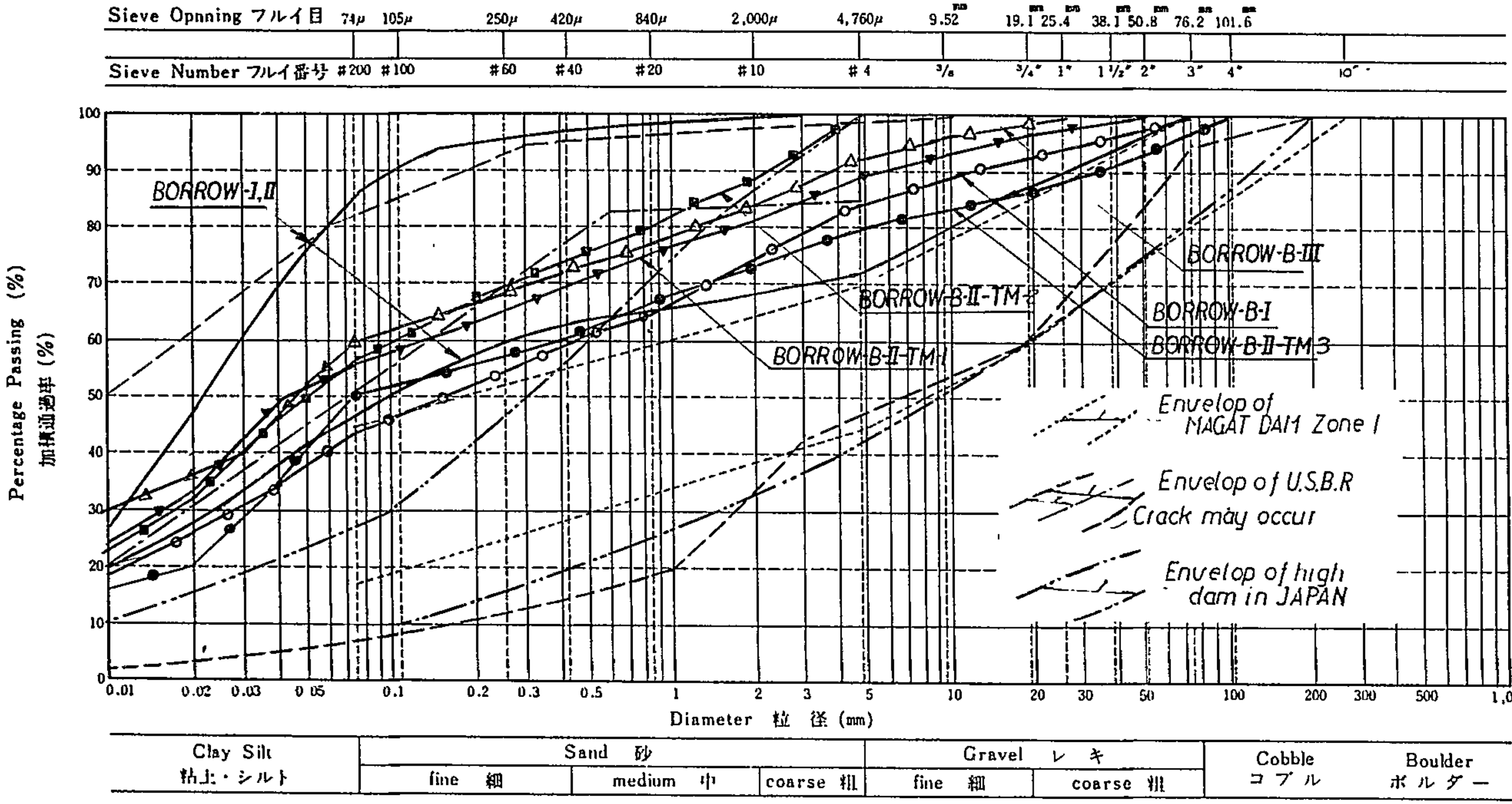
Fig. 4-4 Comparison of Gradation for Core Material

GRAIN SIZE ACCUMULATION CURVE

粒径加積曲線



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II-5 ENGINEERING STUDIES ON STRUCTURES

II-5 ENGINEERING STUDIES ON STRUCTURES

5.1 Study on Height of Non-Overflow Part of Dam

Freeboard of Dam

The height of the crest of non-overflow part of the B dam is calculated by using the following equations.

Type	
Gated Spillway	Hn + hw + he + 1.5 ..... (1)
"	Hs + hw + he/2 + 1.5 ... (2)
"	Hd + hw + 1.5 ... (3)

where,

Hn : normal high water level = El. 520.0 m

hw : surge height from R.W.L. due to wind (m)

he : surge height from R.W.L. due to earthquake (m)

Hs : surcharge water level by design flood = \$1. 524.7 m

Note: The largest value in the above equations should be selected.

The value of 1.5 m is a sum of 1.0 m of additional allowance for fill type dam and 0.5 m of rise of water level for the unexpected accident by the gate miss operation etc.

Calculation of surge height due to wind (hw)

$$hw = 0.00086V^{1.1}F^{0.45} \quad \text{(according to SMB method)}$$

where, V : mean wind velocity in 10 min. = 30 m/sec

F : fetch = 4,000 m

$$\therefore hw = 0.00086 \times 30^{1.1} \times 4{,}000^{0.45} = 1.51 \text{ m}$$

Calculation of surge height due to earthquake (he)

$$he = \frac{1}{2} \cdot \frac{K\tau}{\pi} \sqrt{gHo}$$

where, K : horizontal seismicity = 0.20

τ : period of seismic wave = 1.0 sec

Ho: reservoir water depth at normal high water level = 130.0 m

$$he = \frac{1}{2} \cdot \frac{0.20 \times 1.0}{\pi} \sqrt{9.8 \times 130.0} = 1.14 \text{ m}$$

Applying such values Hn, hw, he, Hs and Hd to the equations, water levels are obtained as follows:

from the equation (1) = 520.00 + 1.51 + 1.14 + 1.00 + 0.50 = El. 524.15

-do- (2) = 524.70 + 1.51 + 0.57 + 1.00 + 0.50 = El. 528.28

-do- (3) = 524.70 + 1.51 + 1.00 + 0.50 = El. 527.71

As a result, the El. 528.28 is the heighest value. Therefore the crest elevation of the dam is designed as EL. 529.0 m.

5.2 Hydraulic Study on Diversion Tunnel

5.2.1 Design discharge

The diversion design discharge is set at 3,200 m^3/sec, which is 20 years probable flood. On the other hand, the 5 years provable flood discharge at dam site is estimated at about 2,000 m^3/sec. Thus, there're two discharge requirement of diversion facilities as follows:

1) The 5 year provable discharge of 2,000 m^3/sec (in open channel flow in diversion tunnel); and

2) The design discharge of 3,200 m^3/sec (in pressure flow in diversion tunnel):

5.2.2 Diversion tunnel

The tunnel length is approx. 950 m of 2 lane in the circular section. The inlet invert level of No.1 and No.2 are set separately to EL. 405.0 and EL. 408.9 respectively. And the outlet invert level of both tunnel are set to EL. 401.80 and EL. 405.70 m respectively. The plan, profile and section of the tunnel is shown in Fig. 5-1. The principal dimensions finally adopted are tabulated below.

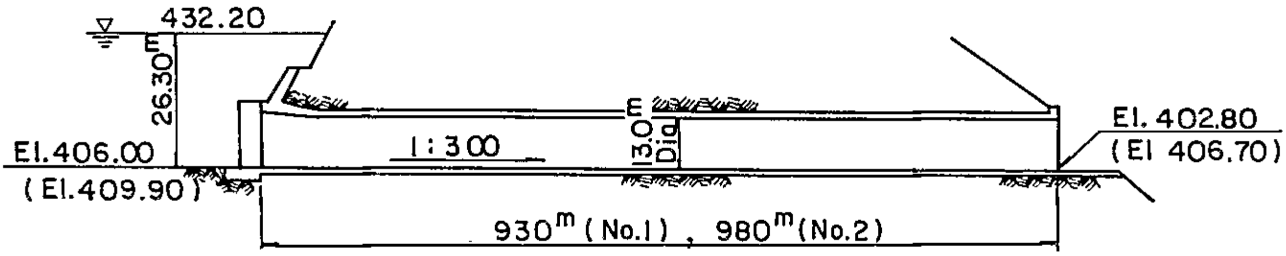
Items	Dimensions
Design discharge (m^3/sec.)	3,200
Inlet invert level (m)	El. 406.00, El. 409.90
Diameter of tunnel (m)	13.00
Length of tunnel (m)	930.0 (No.1), 980.0 (No.2)
Slope	1:300
Outlet invert level (m)	El.402.80, El.406.70

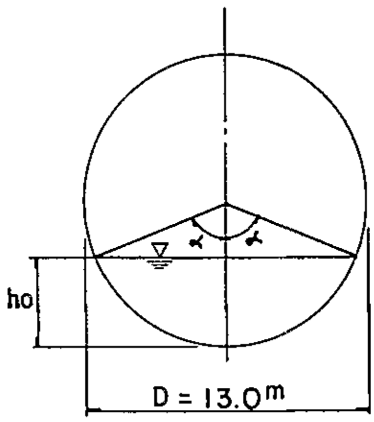
5.2.3 Discharge capacity

The discharge rating curve of the tunnel is worked out. The flow conditions in the tunnel are defined approximately by the ratio of water depth at orifice bottom sill and height of cross section at its exit. If the ratio is within 1.2, flow condition is taken as free flow, if it is between 1.2 and 1.5, flow condition is taken as mixed flow, and if it is over 1.5, then it is taken as pressure flow.

1) Free flow condition

Assuming that the flow is uniform, the hydraulic characteristic curves of 13.0 m - circular-section is obtained by Manning's equation as shown in Fig. 5-1. (Manning's roughness coefficient n is assumed to be 0.015.) The inlet reservoir water level is calculated by the following equation.





(Remarks)

In case of free flow, discharge calculation carried out using Manning's formula as mentioned below.

$$Q = A.V = A.\frac{1}{n}.R^{\frac{2}{3}}.\ I^{\frac{1}{2}}$$

where,

n: roughness coefficient = 0.015 as concrete lining

I: tunnel slope = 1:300

A: flow area = $\frac{D^2}{4}$ (α - SINα COSα) (α in radian)

P: perimeter = D

R: hydraulic radius = A/P

Also, the upper pondage water level considering the entrance loss is calculated by using the equation below.

$$PWL = IIL + d + (1 + f_e)\ \frac{v^2}{2g}$$

where, PWL: pondage water level (EL. m),

IIL: inlet invert level (EL. m),

d : water depth at the entrance (m),

f_e : entrance loss coefficient = 0.2,

v : velocity at the entrance (m/sec) and

g : the acceleration of gravity = 9.8 (m/sec^2)

Pondage W.L. (m)	h_o Depth (m)	A Flow Area (m)	P Perimeter (m)	R Hydraulic Radius(m)	V Velocity (m/sec)	Q		
						Discharge of No. 1 Tunnel (m^3/sec)	Discharge of No. 2 Tunnel (m^3/sec)	Total Discharge (m^3/sec)
408.0	1.3	6.90	8.37	0.82	3.37	23.25	0	23.25
410.3	2.6	18.90	12.05	1.57	5.20	98.28	0	98.28
412.2	3.9	33.34	16.44	2.03	6.17	205.71	0	205.71
414.8	5.2	49.58	17.80	2.79	7.63	378.30	23.25	401.55
416.9	6.5	66.37	20.42	3.25	8.45	560.83	98.28	659.11
418.8	7.8	83.16	23.03	3.61	9.06	753.43	205.71	959.14
420.6	9.1	99.25	25.76	3.85	9.46	938.91	378.30	1,317.21
422.1	10.4	113.84	28.78	3.96	9.64	1,097.42	560.83	1,658.25
423.2	11.7	125.81	32.48	3.87	9.49	1,193.94	753.43	1,947.37
423.4	13.0	132.73	40.84	3.25	8.45	1,121.57	938.91	2,060.48

2) Discharge Capacity as Pressure Flow

(Remarks)

In case of pressure flow, discharge calculation is carried out using the formula mentioned below.

$$Q = \sqrt{2g}\ A\ \left(\frac{H + L.\mathrm{SIN}\,\theta - D}{1 + K_e + \frac{49.4n^2L}{r^{4/3}}}\right)^{\frac{1}{2}}$$

The above formula can be referred to "Design of Small Dams", published by USBR of U.S.A.

where, H : water head on the invert of tunnel inlet (m)

L : tunnel length (m)

θ : tunnel slope = 1:300 = 0°11'28"

K_e: head loss coefficient due to tunnel inlet with square edges = 0.2

Pondage W.L. (m)	H (m)	$\sqrt{2g}$ A	H+LSINθ-D (m)	$1+K_e+\frac{49.4n^2L}{r^{4/3}}$	Q		
					Discharge of No. 1 Tunnel (m^3/sec.)	Discharge of No. 2 Tunnel (m^3/sec.)	Total Discharge (m^3/sec.)
420.0	14.0	587.63	4.17	2.07	847.4	1,097.4	1,943.8
421.0	15.0	"	5.17	"	942.4	1,193.9	2,136.3
422.0	16.0	"	6.17	"	1,029.6	1,121.6	2,151.2
423.0	17.0	"	7.17	"	1,109.9	846.4	1,956.3
424.0	18.0	"	8.17	"	1,184.7	942.4	2,127.1
425.0	19.0	"	9.17	"	1,255.1	1,029.6	2,284.7
426.0	20.0	"	10.17	"	1,321.8	1,109.9	2,431.7
427.0	21.0	"	11.17	"	1,385.3	1,184.7	2,570.0
428.0	22.0	"	12.17	"	1,445.9	1,255.1	2,701.0
429.0	23.0	"	13.17	"	1,504.2	1,321.8	2,826.0
430.0	24.0	"	14.17	"	1,560.2	1,385.3	2,945.5
431.0	25.0	"	15.17	"	1,614.4	1,446.0	3,060.4
432.0	26.0	"	16.17	"	1,666.7	1,504.2	3,170.9
433.0	27.0	"	17.17	"	1,717.5	1,614.4	3,331.9
434.0	28.0	"	18.17	"	1,766.8	1,666.7	3,433.5

5.2.4 Cofferdams

The cofferdams required during the construction of the diversion tunnel and permanent works are as follows:

1) Upstream primary cofferdam for construction of diversion tunnel

2) Upstream primary cofferdam for construction of secondary cofferdam

3) Upstream secondary cofferdam for construction of permanent works

4) *Downstream cofferdam for construction of permanent works*

All the cofferdams are of rockfill type. The crest elevations are shown in Fig. 5-1.

5.3 Study on Discharge Capacity of Spillway

In case of free flow when all gates are opened

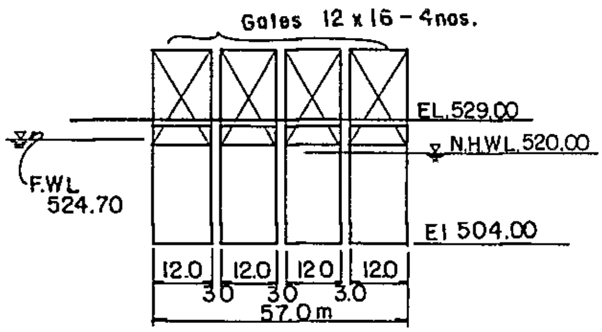
The relation among the discharge coefficient, overflow depth and the discharge capacity calculated by the following equations.

$$Q = nC' . B . H^{\frac{3}{2}}$$

$$C' = C \left\{ 1 - Md \left(\frac{H}{Hd}\right)^{\frac{3}{2}} \right\}$$

$$C = 1.704 \left\{ 1 + 0.704 \left(\frac{H}{Hd}\right)^{\frac{2}{2}} \right\}$$

$$Md = 0.0756 \left(\frac{Hd}{B}\right)^{\frac{1}{2}} \left\{ \frac{1}{n} + 1.465 \left(\frac{n-1}{n}\right)\left(\frac{b}{s}\right)^{1.7} \right\}$$



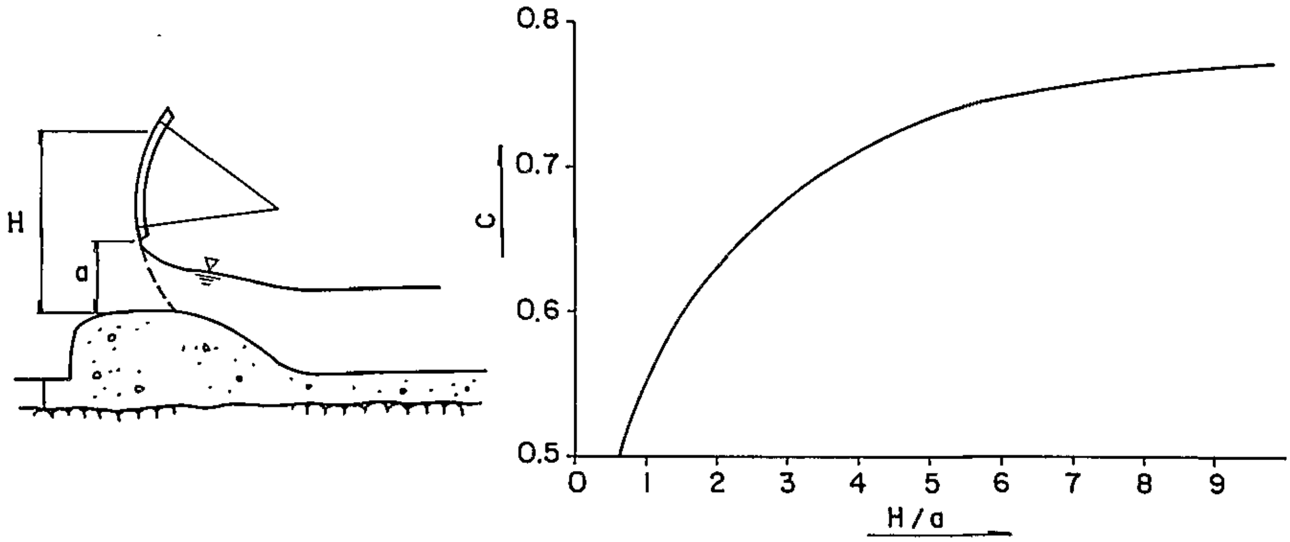
where, Q : discharge (m^3/sec)

- n : number of gates
- B : gate width per one gate (m)
- H : overflow depth (m)
- Hd: design head on overflow crest = 20.7 m
- C : discharge coefficient
- C': adjusted discharge coefficient under the influence of piers and abutments etc.
- Md: adjustment factor of C due to pier and abutment
- b : pier width = 3.0 m
- s : horizontal length from overflow crest to edge = 30.0 m

From the above equation, relations among the three variables are abtained as listed in Table 5-1.

And, when the design discharge 10,300 m^3/sec flows into the reservoir, water level is kept under F.W.L. of El. 524.7 m.

In case of pressure flow when all gates are partially opened



As shown in the above figure, when a F.W.L. is fixed to be 522.00 m, the relation among the opening depth, discharge coefficient and discharge capacity are calculated by the following equations.

$$Q = n.C\ a\ B \sqrt{2g\ (H-a/2)}$$

where, Q : discharge (m^3/sec)

n : number of gates

C : discharge coefficient

a : gate opening (m)

B : gate width per one gate = 12.0 m

g : acceleration of gravity = 9.8 m/sec^2

H : water head on overflow crest = 20.7 m

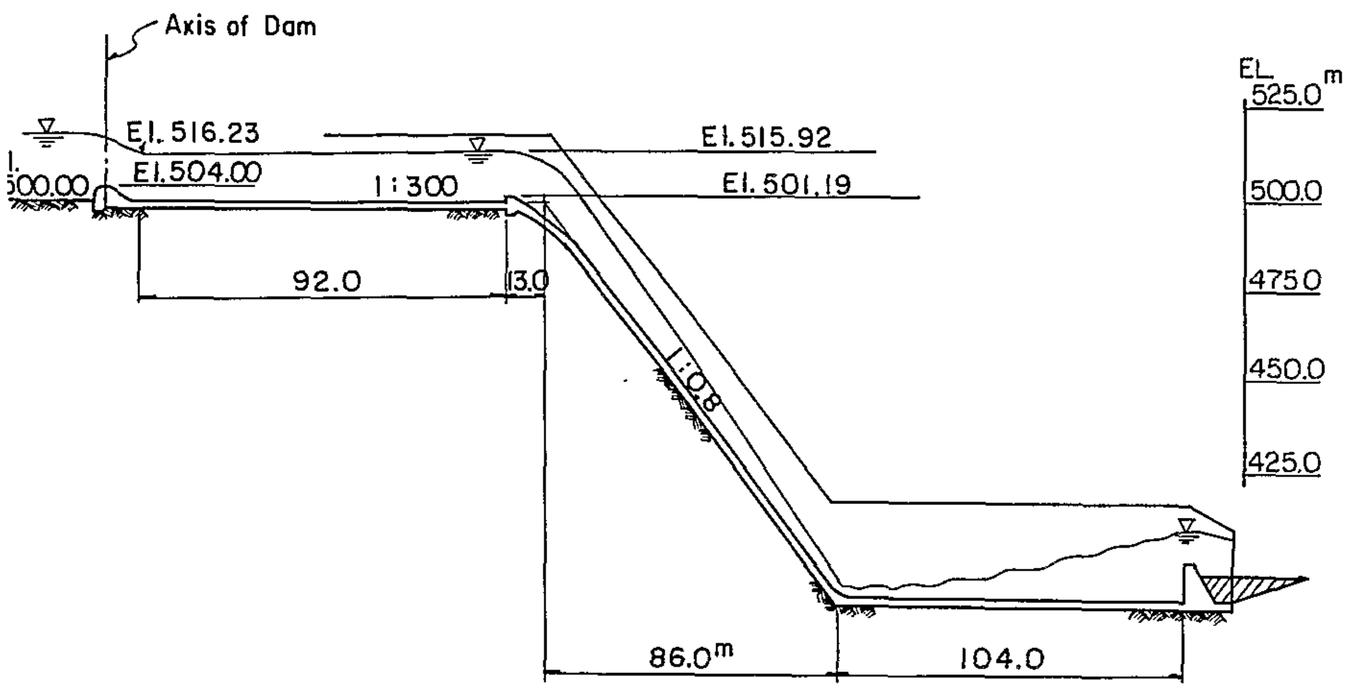
The relation among the three valiables is calculated as listed in Table 5-2.

On the other hand, results of study on relation between the gate operation speed and the discharge capacity is shown in Fig. 5-3.

And also, as shown in Fig. 5-2 the planned spillway size is satisfactory for the following discharge conditions.

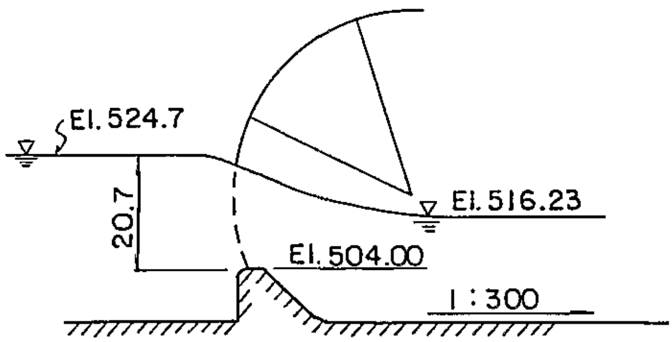
R.W.L. (m)	Discharge (m^3/sec)	Probability (Yen)
W.L. 522.00	7,600.0	1,000 (Approx.)
F.W.L. 524.70	10,300.0	10,000

5.4 Hydraulic Study on Spillway and Stilling Basin



The spillway is of open chute type made of reinforced concrete. The spillway is composed of four parts, i.e., i) gated overflow weir at the beginning near the dam axis, ii) a flat open channel with a gentle slope of 1/300, iii) a steep chuteway with a slope of 1:0.8 and iv) a flat and deep stilling basin ended by a sub-dam. The main dimensions are illustrated in the above drawing.

1. Calculation of Discharge Coefficient at Spillway Weir



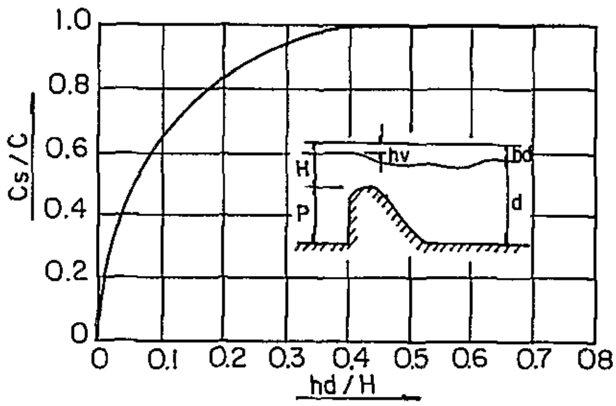
By the hydraulic analysis of non-uniform flow, the water level at immediately downstream of spillway weir is obtained to be El. 513.25 m as mentioned in the water surface studies.

Hence, the reasonable discharge coefficient shall be calculated after the water level of upstream side is assumed.

When the design discharge of 10,300 m^3/sec flows, the overflow depth at spillway weir is assumed to be 20.7 m approximately.

In case of the standard overflow weir with downstream free nappe curve, the discharge coefficient C = 2.28 can be used as shown in Table 5-1.

As discharge coefficient is affected by the downstream water level, discharge coefficient is calculated by the following figure.



where,

Cs: discharge coefficient in case of submerged overflow

C : discharge coefficient in case of unsubmerged overflow

H : overflow depth at weir (m)

d : water depth downstream (m)

The coefficient Cs is obtained to be 2.21 by the following equations.

$$\frac{hd}{H} = \frac{11.40}{20.70} = 0.55, \qquad \frac{Cs}{C} = 1.00$$

$$Cs = C \times 1.00 = 2.28$$

Accordingly, water head is calculated by using the equation $Q = CBH^{\frac{3}{2}}$ as follows:

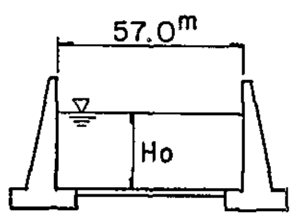
$$H = \left(\frac{Q}{CsB}\right)^{\frac{2}{3}} = \left(\frac{10,300.0}{2.28 \times 48.0}\right)^{\frac{2}{3}} = 20.70 \text{ m}$$

The water level at spillway weir is calculated to be 524.70 m (= 504.00 + 20.70).

Accordingly, the flood high water level is selected to be 524.7 m.

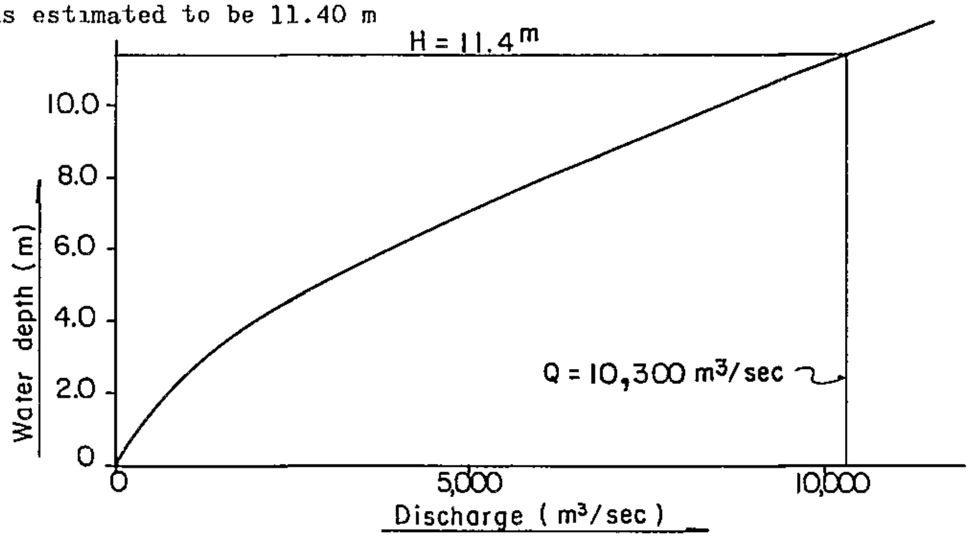
2. Hydraulic analysis on non-uniform flow in approach channel

The dimensions of open channel are selected as follows.



channel slope: 1/300, channel width: 57.0 m roughness coefficient, n: 0.015, sill height: 1.5 m

The relation curve between the water depth and the discharge in the open channel is calculated by using the Manning's formula as shown below. From this curve, for design discharge 10,300 m3/sec, a uniform flow depth is estimated to be 11.40 m



1) Discharge per unit width q in open channel

$$a = bHo = 1.0 \times 11.4 = 11.40 \ m^2$$

$$V = \frac{1}{n} \cdot R^{\frac{2}{3}} I^{\frac{1}{2}} = \frac{1}{0.015} \times \left(\frac{649.8}{79.8} \right)^{\frac{2}{3}} \times \left(\frac{1}{300} \right)^{\frac{1}{2}} = 15.59 \ m/sec$$

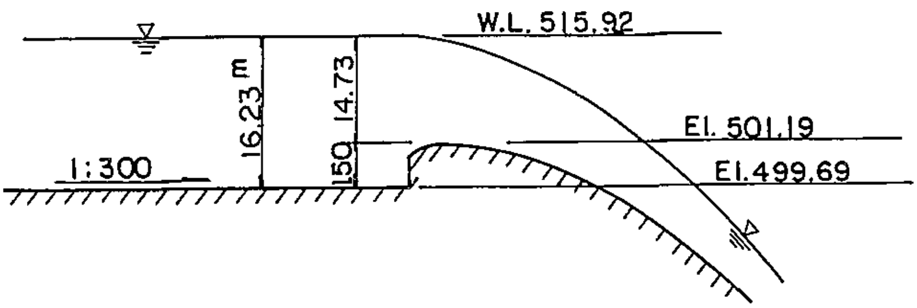
$$q = av = 11.40 \times 15.59 = 177.73$$

$$Q = 177.7 \times 57 = 10,131 \doteqdot 10,300 \ m^3/sec$$

2) Overflow depth at sill

$$h_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{177.7^2}{9.8}} = 14.73 \ m$$

therefore, the water level on the sill is calculated as illustrated below,



3) Water surface profile in open channel

As shown in the previous study, the critical depth at the sill is higher than the uniform flow in the open channel. So that the water surface profile in the open channel is controlled as super-critical frow from the upstream's water surface.

The water surface profile is shown in the previous figure.

3. Determination of design discharge for stilling basin

It is mentioned in the Structual Standard of Dam of JAPAN that the larger value specified below should be taken as the design discharge for stilling basin.

Case-1. The maximum discharge at the surcharge water level for stilling basin with straight apron. (Ref. Fig. 1-32)

Case-2. 80% value of the design discharge in the case of concrete type dam.

The discharges for the case-1 and the case-2 are calculated to be 6,800 m^3/sec and 5,048 m^3/sec respectively, so that the design discharge is selected to be 6,800 m^3/sec.

1) Determination of the dimensions of the stilling basin

Determination of the dimensions of the stilling basin are made by using the following equations.

$$h_1 = \frac{Q}{0.95B\sqrt{2gH}} \quad , \; L = 4.0\ hj$$

$$h_j = \frac{1}{2} h_1 \left(\sqrt{1 + 8F_1{}^2} - 1 \right), \; F_1 = \frac{V_1}{\sqrt{gh_1}}$$

$$\frac{W}{h_1} = \frac{(1+2F_1{}^2)\sqrt{1+8F_1{}^2} - 1 - 5F_1{}^2}{1+4F_1{}^2 - \sqrt{1+8F_1{}^2}} - \frac{3}{2} F_1{}^{\frac{2}{3}}$$

where, h_1: water depth at the beginning part of stilling basin (m)

h_j: conjugate depth of the hydraulic jump (m)

H : total water head (m)

F_1: Froude number at the beginning part

L : length of stilling basin (m)

W : height of sub dam (m)

The results of the calculation are shown in the previous figure.

4. Study on roller bucket type stilling basin

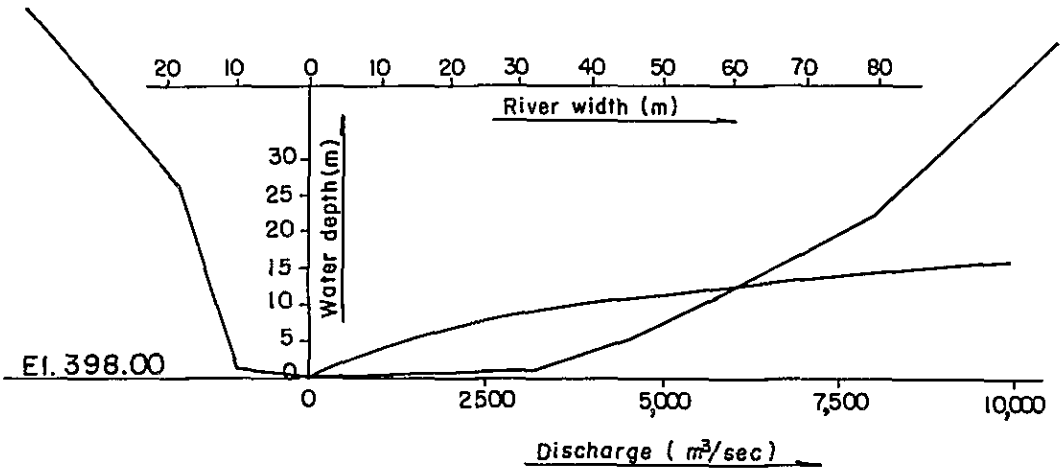
As decided by the previous study on horizontal apron type stilling basin, when the design discharge flows, the water level corresponding to the conjugated depth of 25.5 m is higher than the uniform flow depth of 13.0 m when the design discharge flows in the existing river, so that the roller bucket type turns out to be not appropriate from the reason below. To connect both of the water levels of conjugated depth and uniform flow depth smoothly involves a big amount of excavation for the bottom part of the roller bucket, this results in higher construction cost.

Accordingly, the previous design study on the roller bucket type stilling basin is abandoned.

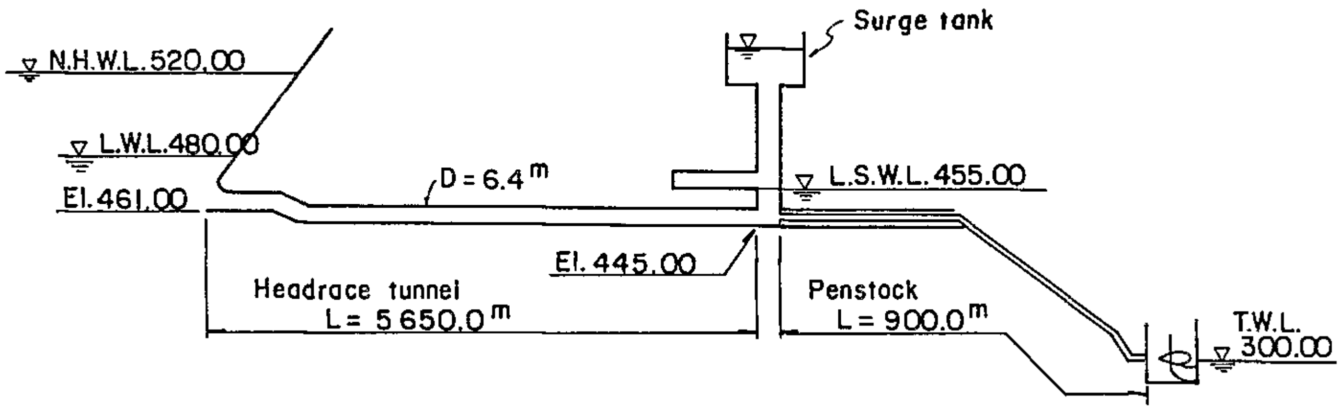
Note: /1 the study on the uniform flow depth in the existing river is made in section 5.

5. Study on water depth of downstream

As seen in the topographical map around the dam site, the design discharge of the stilling basin flows into the river channel downstream. The flow condition in the river channel downstream is assumed to be uniform, reach of 500 m approx. in length with not much difference in river width. Accordingly, the discharge rating curve is obtained by the Manning's formula as shown below.



5.5 Hydraulic Study on Pressure Tunnel, Surge Tank and Penstock Tunnel



The waterway system are illustrated above.

1. Design of a chamber type surge tank

The principal dimensions adopted are as follows:

- Maximum discharge: 112.2 m^3/sec (37.4 m^3/sec x $\frac{24}{8}$ = 112.2)
- Pressure tunnel: L = 5,650.0 m having two bends of 50.0 degrees in internal angle

  Dia = 6.4 m
- High Water Level: 520.0 m
- Low Water Level: 480.0 m
- Tail Water Level: 300.0 m
- Highest Surging Water Level due to full load rejection = 540.0 m
- Gross Head: 220.0 m
- Roughness Coefficient: (For load rejection), = 0.012
- " : (For load increasement), = 0.015

a. Headloss Calculation of Pressure Tunnel

(1) Friction loss head from intake to surge tank (h_f)
(Headloss due to friction of headrace tunnel)

$$h_f = f\frac{L}{D} \cdot \frac{V^2}{2g}, \quad f = \frac{124.5\ n^2}{D^{1/3}}$$

where, h_f: friction loss head, L: tunnel length = 5,650.0 m

D : tunnel diameter = 6.4 m

f : (Darcy) resistance coefficient for circular tunnel

V : velocity in tunnel = 3.5 m/sec

In case of n = 0.012

$$f = \frac{124.5 \times 0.012^2}{6.4^{1/3}} = \frac{0.01793}{1.857} = 0.009654$$

$$h_f = 0.009663 \times \frac{5{,}650.0}{6.4} \times \frac{3.5^2}{19.6} = 5.332 \text{ m}$$

In case of n = 0.015

$$f = \frac{124.5 \times 0.015^2}{6.4^{1/3}} = 0.015085$$

$$h_f = 0.015097 \times \frac{5{,}650.0}{6.4} \times \frac{3.5^2}{19.6} = 8.323 \text{ m}$$

(2) Head loss at tunnel entrance (he)

$$He = fe \cdot \frac{V^2}{2g}$$

where, fe: entrance loss coefficient = 0.2

$$he = 0.2 \times \frac{3.5^2}{19.6} = 0.125 \text{ m}$$

(3) Head loss due to bend (hbe)

$$hbe = fbe \cdot \frac{V^2}{2g}$$

where, $fbe = 0.946 \sin^2 \frac{\alpha}{2} + 2.05 \sin^4 \frac{\alpha}{2}$

the internal angle of 2 bends is decided to be 50 degrees, and the value of fbc = 0.2343

$$hbe = 2 \times 0.2343 \times \frac{3.5^2}{19.6} = 0.293 \text{ m}$$

(4) Velocity head at the bottom of surge tank

$$hv = \frac{Vo^2}{2g} = \frac{3.5^2}{19.6} = 0.625 \text{ m}$$

(5) Total head loss from intake to surge tank

<u>in case of n = 0.012,</u> $ho = h_f + he + h_{bc} + h_v = 6.370$ m

coefficient of total head loss, $C = \frac{ho}{Vo^2} = \frac{6.332}{3.5^2} = 0.520$

<u>in case of n = 0.015,</u> $ho = 9.366$ m

$C = 0.765$

(6) Total head loss of waterway

If the maximum velocity in penstock is 11.0 m/sec, the velocity head becomes

$$hv = \frac{V^2}{2g} = \frac{11.0^2}{19.6} = 6.17 \text{ m}$$

therefore, the maximum value of the total head loss from the intake to the turbine for the maximum discharge is calculated to be 15.50 m approximately.

b. <u>Study on Dimensions of Surge Tank</u>

In determination of dimensions of surgetank, a study is carried out by using equations below.

- Thoma-Schuller's equation

$$ho < \frac{Hg}{3} \sim \frac{Hg}{6}$$

$$Fs > \frac{Lf}{2cgHo} \qquad \text{(for static stability)}$$

$$Ds = \sqrt{\frac{4Fs}{\pi}}$$

$$Z = Vo\sqrt{\frac{Lf}{gF}}$$

$$F_s > \left(1 + 0.482 \frac{Z_*}{H_o}\right) \frac{L_f}{2cgH_o} \text{ (for dynamic stability)}$$

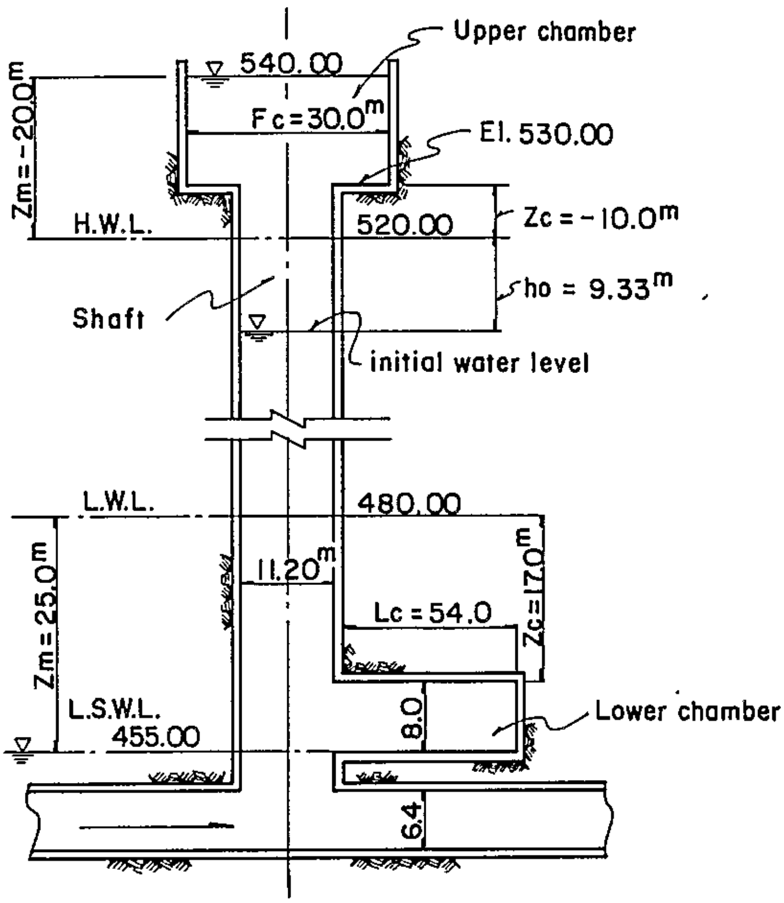
where, Hg : gross head = 220.0 m

Fs : sectional area of surge tank shaft (m^2)

Ds : diameter of surge tank shaft (m)

Z : surging height (m)

As a result, the dimentions of surge tank are decided as illustrated below.



2. Design of Penstock Tunnel

a. General

According to the "Engineering Standard for the Gates and Steel Pipes", the maximum value of possible combination of static head, pressure rise due to water hammer and the maximum water level due to surging should be taken as the maximum internal pressure for design.

And, the combination of load depending on type of surge tank is carefully studied.

b. Diameter and thickness of Penstock

The penstock diameter is tentatively decided as mentioned below.

(1) The diameter of penstock is planned to be reduced from 5.5 m at the beginning by 0.1 m in every 100.0 meters to 4.7 m at the end.

(2) The diameter of manifold to the turbine is planned to be 2.8 meters.

c. Calculation of Required Pipe Thickness

The velocity of pressure wave and the allowable tensile stress through penstock are expressed by the following equations.

$$a = \frac{1}{\frac{w}{g}\left(\frac{1}{K} \cdot \frac{1}{E} \cdot \frac{D}{t}\right)} :$$

$$\sigma a = \frac{HD}{2t \times 0.90}$$

where, a : velocity of pressure wave

D : Diameter of pesntock = 5.10 m in average

t : thickness of pipes

w : unit weight of water = 1 t/m^3

g : the acceleration of gravity = 9.8 m/sec^2

K : elastic modulus of water = 2×10^5 t/m^2

E : elastic modulus of pipe material = 2×10^7 t/m^2

σa: allowable tensile stress of pipe material = 1,600 kg/cm^2 for SM50 and 2,300 kg/cm^2 for SM58

H : maximum internal pressure (kg/cm^2)

As a result, the values "a" and "σa" becomes to be 680.0 m and 1,600 ~ 2,300 kg/cm^2.

As the lower parts of penstock and casing are used to be kind of material SM58, so that, both thickness are calculated as follows:

<u>Lower part of penstock</u>

$$t = \frac{H.D}{2 \quad a \times 0.9} = \frac{31.0 \times 470}{2 \times 2{,}300 \times 0.9} = 3.519 = 35.2 \text{ mm}$$

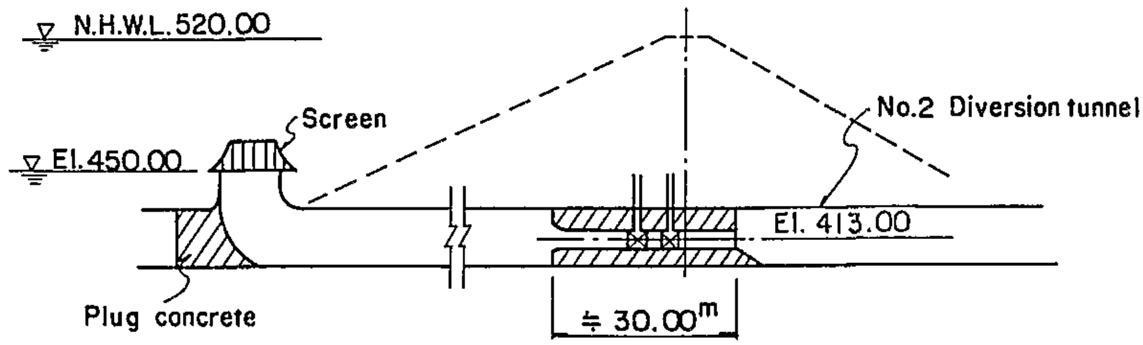
<u>Casing part</u>

$$t = \frac{H.D}{2 \quad a \times 0.9} = \frac{31.0 \times 280}{2 \times 2{,}300 \times 0.9} = 2.097 = 21.0 \text{ mm}$$

The relation between diameter and thickness is estimated as tabulated below, and the thickness is added to by an allowance for corrosion of 1.5 mm to the above value.

Diameter (m)	Thickness (mm)	
5.50	25.0	
5.40	27.0	SM50
5.30	28.0	
5.20	30.0	
5.10	31.0	
5.00	33.0	
4.90	35.0	SM58
4.80	37.0	
4.70	38.0	
2.80	24.0	

5.6 Study on Discharge Calculation of River Outlet



No.2 diversion tunnel will be reused as a river outlet as illustrated above after completion of the dam.

The discharge control for the river outlet will be made by jet flow gates and the discharge calculation is carried out by using the following equation.

$$Q = C \cdot A\sqrt{2gH}$$

where, Q : discharge (m^3/sec.)

A : flow area of 2 jet flow gates of 1,700 mm in diameter $= \pi D^2/4 \times 2 = 4.54\ m^2$

C : discharge coefficient ≑ 0.7

H : head on the center axis of outlet valve

Hydraulic Study of River Outlet

A facility to make a drawdown of water level of the reservoir should be installed for the safety, maintenance and improvement of the Dam.

Accordingly, a river outlet facility is planned to be provided for a drawdown of the water level and for satisfying the irrigation water discharging and the river maintenance flow in case when the hydropower generation is stopped.

The relationship between the reservoir water level and the discharge is tabulated below.

R.W.L. (m)	H (m)	Q (m^3/sec)	Remarks
520.00	108.00	145.5	N.H.W.L. is EL.520.00
515.00	103.00	142.1	
510.00	98.00	138.6	
504.00	92.00	134.2	EL.504.00 is bottom elevation of spillway gate
500.00	88.00	131.2	
495.00	83.00	127.4	
490.00	78.00	123.5	
485.00	73.00	119.4	
480.00	68.00	115.2	L.W.L. is EL.480.00
475.00	63.00	110.8	
470.00	58.00	106.2	
465.00	53.00	101.5	
460.00	48.00	96.5	
455.00	43.00	91.2	
450.00	38.00	0	Inlet invert level is EL.450.00

On the other hand, natural inflow depending on the season should be considered in the study on drawdown of water level. According to the past records of hydrological data for 20 years, the mean monthly discharge for the dry/1 season and wet/2 season are 24.2 m^3/sec and 50.5 m^3/sec respectively.

Note: /1 from January to June
/2 from July to December

As a matter of fact, drawdown for the water level above El.504.00 is made mainly by operation of the spillway gate.

Therefore, drawdown below El.504.00 should be made by using the river outlet facility.

The Study results of the duration for drainage considering the inflow discharge are tabulated below.

Dry Season

Reservoir Water Level R.W.L. (m)	Discharge $Q(m^3/sec)$	Inflow $I(m^3/sec)$	Reservoir Storage $V(x10^6 m^3)$	Drawdown days $D=\frac{V}{86,400(\frac{Q_1+Q_2}{2}-I)}$	Accumulated days
504.00	135.0	24.2			
500.00	132.0	"	14.0	1.5	1.5
495.00	128.2	"	11.0	1.3	2.8
490.00	124.2	"	9.0	1.0	3.8
485.00	120.2	"	8.0	0.9	4.7
480.00	116.0	"	7.0	0.9	5.6
475.00	111.7	"	6.5	0.8	6.4
470.00	107.2	"	5.5	0.7	7.1
465.00	102.4	"	4.5	0.6	7.7
460.00	97.5	"	3.5	0.5	8.2
455.00	92.3	"	3.0	0.5	8.7
450.00	0	"	2.5	0.9	9.6

Wet Season

Reservoir Water Level R.W.L. (m)	Discharge $Q(m^3/sec)$	Inflow $I(m^3/sec)$	Reservoir Storage $V(x10^6 m^3)$	Drawdown days $D=\frac{V}{86,400(\frac{Q_1+Q_2}{2}-I)}$	Accumulated days
504.00	135.0	50.5			
500.00	132.0	"	14.0	2.0	2.0
495.00	128.2	"	11.0	1.6	3.6
490.00	124.2	"	9.0	1.4	5.0
485.00	120.2	"	8.0	1.3	6.3
480.00	116.0	"	7.0	1.2	7.5
475.00	111.7	"	6.5	1.2	8.7
470.00	107.2	"	5.5	1.1	9.8
465.00	102.4	"	4.5	1.0	10.8
460.00	97.5	"	3.5	0.8	11.6
455.00	92.3	"	3.0	0.7	12.3
450.00	0	"	2.5	5.9	18.2

As seen in both of the tables, the duration for drainage down to El.450.00 are estimated to be 9.6 days in dry season and 18.2 days in wet season respectively.

According to the Structural Standard of Dams of JAPAN, the drawdown to Low Water Level on reservoir operation for a fill type dam is desirably designed to be less than 10 days. The above discharge capacity of the river outlet is deemed adequate.

5.7 Stability Analysis of Rockfill Dam

5.7.1 Design value for stability analysis of embankment

Considering the results of material tests during the investigation at site for Matuno Project, the data of the dam embankment in Japan, and the design and control data of the Magat High dam in the Philippines, the engineering properties of materials for the dam design are proposed tentatively as follows;

(1) Core materials (GM, GC material)

. Density (Unit weight)

Dry density	$\gamma_d = 1.85\ t/m^3$
Water content	$w = 13\%$
Specific gravity	$Gc = 2.50$ (coarse)
	$G_f = 2.66$ (fine)

modified specific gravity (Gs)

$$Gs = 2.50 \times 0.3 + 2.66 \times 0.7 = 2.61$$

Void ratio

$$e = \frac{Gs \cdot w}{d} - 1 = \frac{2.61 \times 1.0}{1.85} - 1 = 0.41$$

Saturated density

$$\gamma_{sat} = \frac{Gs + e}{1 + e} \cdot w = \frac{2.61 + 0.41}{1 + 0.41} \times 1.0$$

$$= 2.14\ t/m^3$$

Submerged density

$$\gamma_{sub} = \gamma_{sat} - \gamma_w = 2.14 - 1.0$$

$$= 1.14\ t/m^3$$

. Shearing strength $\quad c' = 0\ kg/cm^2$

$\phi' = 28^o$ ($\tan \phi' = 0.532$)

. Coefficient of permeability

$$K = 1 \times 10^{-6}\ cm/sec.$$

(2) Filter materials (Sand & Gravel)

. Density (Unit weight)

Dry density	$\gamma_d = 2.10\ t/m^3$
Specific gravity	$G_f = 2.62$ (fine)
	$Gc = 2.76$ (coarse)

modified specific gravity (Gs)

$Gs = 2.62 \times 0.35 + 2.76 \times 0.65 = 2.71$

Void ratio $e = 0.29$

Saturated density $\gamma_{sat} = 2.33\ t/m^3$

Submerged density $\gamma_{sub} = 1.33\ t/m^3$

. Shearing strength $c' = 0\ kg/cm^2$

$\phi' = 38^o$ ($\tan \phi' = 0.781$)

(3) <u>Rock material</u>

a. Inner shell rock (Fresh - moderately weathered Limestone or Conglomerate)

. Density (unit weight)

Dry density $\gamma_d = 2.10\ t/m^3$

Specific gravity $Gs = 2.65$

Void ratio $e = 0.262$

Saturated density $\gamma_{sat} = 2.31\ t/m^3$

Submerged density $\gamma_{sub} = 1.31\ t/m^3$

. Shearing strength $c' = 0\ kg/cm^2$

$\phi' = 38^o$ ($\tan \phi = 0.781$)

b. Outer shell and riprap (Fresh Limestone or Conglomerate)

. Density (unit weight)

Dry density $\gamma_d = 2.00\ t/m^3$

Specific gravity $Gs = 2.67$

Void ratio $e = 0.34$

Saturated density $\gamma_{sat} = 2.25\ t/m^3$

Submerged density $\gamma_{sub} = 1.25\ t/m^3$

. Shearing strength $c' = 0\ kg/cm^2$

$\phi' = 42^o$ ($\tan \phi = 0.900$)

The lift thickness and compaction of the above zones are proposed as follows;

	Material (zone)	Type of Roller	Number of coverages	Lift thickness (m)
(1)	Core	Pneumatic or Tamping	6 times	0.25
(2)	Filter	Vibratory	4 times	0.30
(3)	Rock (Outer Inner)	Vibratory	6 times	1.00

5.7.2 Slope stability analysis by surface sliding method

The critical safety slope in order to keep the safety factor of Fse = 1.2 is calculated by the following equations.

(1) In case of the reservoir being full (upstream side)

$$Fse = \frac{m + k \cdot \gamma'}{1 + k \cdot \gamma' \cdot m} \cdot \tan \phi$$

where, m : slope

k : coefficient of earthquake = 0.20

': $\gamma_{sat}/\gamma_{sub}$ = 2.25/1.28 = 1.80

tan ϕ: coefficient of internal friction of rock material = 0.781

hence,

$$m = \frac{Fse + k \cdot \gamma' \cdot \tan \phi}{\tan \phi - k \cdot \gamma' \cdot Fse} = \frac{1.20 + 0.20 \times 1.80 \times 0.90}{0.90 - 0.20 \times 1.80 \times 1.20} = \frac{1.524}{0.468} = 3.26$$

therefore, the critical safety slope is 1:3.26, the adopted slope of the upstream riprap is 1:3.30 in consideration of the dam maintenance and the past records in Japan.

(2) In case of the reservoir being empty (upstream and downstream side)

$$m = \frac{Fse + k \cdot \tan \phi}{\tan \phi - k \cdot Fse} = \frac{1.20 + 0.20 \times 0.90}{0.90 - 0.20 \times 1.20} = \frac{1.380}{0.66} = 2.09$$

therefore, the critical safety slope is 1:2.091, the slope of the downstream riprap adopted is 1:2.10 in consideration of the dam maintenance and the past records as well as the result of analysis in case of the reservoir being full.

5.8 Study on Lining Thickness of Pressure Tunnel

1. Acting load against tunnel

For reference, the rock loads proposed by Terzaghi are shown as follows;

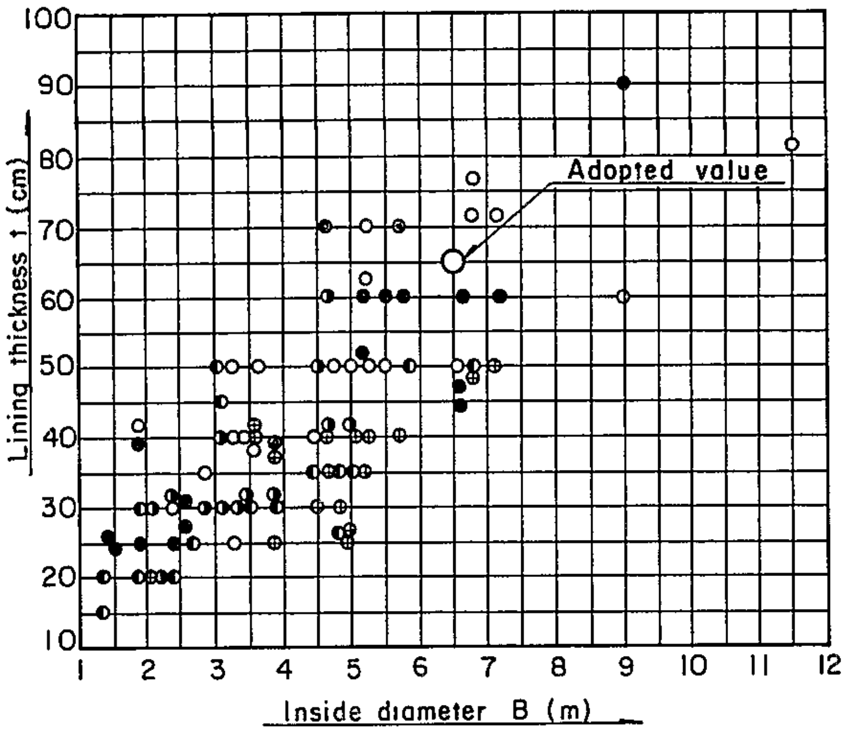
Rock Condition	Rock Load H_p in meter	Remarks
a. Hard and intact	zero	Light lining, required only if spalling or popping occurs.
b. Hard stratified or schistose	0 to 0.5 B	Light support.
c. Massive, moderately jointed	0 to 0.25 B	Load may change erratically from point to point.
d. Moderately blocky and seamy	0.25 B to 0.35 (B + H_t)	No side pressure.
e. Very blocky and seamy	(0.35 to 1.10) (B + H_t)	Little or no side pressure.
f. Completely crushed but chemically intact	1.10 (B + H_t)	Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs.

According to the results of the site investigation, the geological conditions along the pressure tunnel concerned seem to be separated into many layers such as the limestone, tuff and the andesite etc., which are formed having a fresh part and weathered part. Accordingly, the rock condition is considered to be the category of "d" and rock load is calculated to be 6.0 m approximately.

Note: B = tunnel width (m)
H_t = tunnel height (m)

2. Lining thickness

As the determination of lining thickness is not easy, the past records in Japan in the following figure are referred to. And, the lining thickness is tentatively adopted to be 10% of inside diameter of the tunnel.



Pressure tunnel for Hydropower

Remarks; ⊕ Excellent hard rock

◐ Hard rock having small cracks

◑ Hard rock having large cracks

● Fresh weathered rock or sandstone layer

○ Weathered soft rock or sand silt layer

◎ Expansible rock or clay in alteration zone