LIST OF TABLES AND FIGURES IN THE CHAPTER 3

Table 3-2-1 Climatological Data (surin) for the Period 1951 - 1975

Temperature (°C)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean	24.4	26.8	29.2	30.0	29.3	28.4	28.0	27.7	27.4	26.9	25.4	24.0	27.3
Ext. Maximum	36.6	38.2	40.8	41.6	39.7	38.8	37.4	37.1	36.7	35.8	36.2	35.8	41.6
Ext. Minimum	6.4	11.0	11.0	15.2	20.0	19.8	19.6	20.0	19.0	16.3	11.9	8.2	6.4

Relative Humidity (%)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean	63.0	61.0	60.0	65.0	74.0	78.0	79.0	81.0	83.0	79.0	74.0	68.0	72.0
Mean Maximum	87.7	85.2	83.3	85.6	90.5	93.4	92.9	93.9	95.3	92.6	91.0	89.3	90.1
Mean Mimimum	43.4	43.2	41.8	45.7	55.2	62.0	63.1	65.5	68.2	66.3	57.9	49.6	55.2

Climatological Data (surin) for the Period 1951 - 1975

Evaporation (mm)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean - Piché	143.1	140.3	163.4	145.3	110.8	83.1	80.6	69.0	59.0	81.5	102.1	125.0	1303.2
Mean - Pan	202.6	194.4	229.6	218.0	207.9	178.1	188.1	164.8	145.3	179.4	188.2	192.9	2289.3

Rainfall (mm)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean	2.0	11.0	30.6	84.1	175.4	159.2	190.8	194.4	276.3	132.8	22.3	1.6	1280.5
Mean rainy days	0.7	2.1	4.3	8.4	14.3	17.6	17.8	19.6	20.8	11.6	3.5	0.6	121.3
Daily Maximum	12.8	57.7	40.1	108.9	106.3	114.4	97.6	94.5	104.5	132.1	39.6	19.5	132.1
Day/Year	26/54	12/70	24/64	12/68	25/51	12/70	18/61	6/58	28/73	6/60	14/66	26/66	6/60

Remark: Evaporation 1. Piché 1959 - 1975
2. Pan 1961 - 1975

Table 3-2-2 Monthly Diversion Water Requirements

1,000 m^3

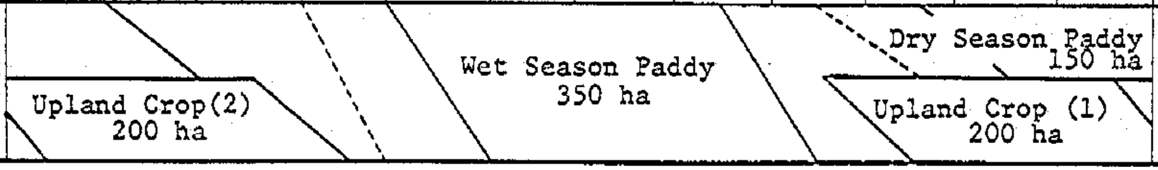
Month / Year	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Total	
													Annual	May ∿ Oct.
1968	305	181	33	34	1,154	33	676	745	247	759	1,102	866	6,135	2,111
1969	878	321	33	34	1,271	33	244	552	247	759	1,096	879	6,347	1,936
1970	838	41	33	34	1,137	33	127	766	247	759	1,062	1,100	6,177	1,405
1971	691	291	33	34	1,172	145	419	766	247	759	1,056	972	6,585	2,094
1972	678	472	33	57	1,207	33	297	105	247	759	1,057	879	5,824	2,099
1973	851	52	33	34	1,131	33	186	712	247	612	1,102	959	5,952	1,469
1974	806	52	51	34	1,189	33	239	379	247	759	1,102	932	5,823	1,598
1975	686	41	33	34	1,266	33	454	691	247	759	1,102	946	6,292	1,861
1976	518	34	33	34	886	33	34	712	247	759	1,102	1,020	5,412	1,054
1977	592	46	33	34	1,067	33	682	766	247	759	1,102	986	6,347	1,895
Average	684	153	35	36	1,148	44	336	619	247	744	1,088	954	6,088	1,752
Cropping Calender														

Table 3-2-3 Monthly Rainfall at the Project Site (PRASAT -- Ta Kao Dam)

in millimeter

Month / Year	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Total		
													Rainy Season	Dry Season	Annual
1968/69	138.3	102.7	335.4	211.9	152.1	375.9	24.5	3.4	0.0	0.0	0.0	46.1	1202.5	187.8	1390.3
1969/70	23.9	62.1	186.8	184.0	*99.7	273.0	122.9	48.2	0.0	0.0	1.5	44.4	928.5	118.0	1046.5
1970/71	31.4	158.8	283.9	285.2	158.6	275.7	148.8	0.0	0.0	0.0	8.7	0.0	1311.0	40.1	1351.1
1971/72	61.3	71.2	117.5	76.7	143.5	116.3	82.5	0.0	0.0	0.0	10.6	25.6	607.7	97.5	705.2
1972/73	64.1	18.0	150.6	43.7	127.6	382.9	110.2	148.9	0.0	0.0	5.2	43.7	833.0	261.9	1094.9
1973/74	29.2	148.3	101.0	110.5	159.4	231.0	135.9	12.3	0.0	53.2	0.0	27.5	886.1	122.2	1008.3
1974/75	38.6	144.6	91.0	175.4	136.4	164.0	123.8	87.4	0.0	0.0	0.0	33.8	835.2	159.8	995.0
1975/76	62.9	157.4	283.0	234.3	102.2	261.6	74.7	15.6	0.0	0.0	0.0	30.5	1113.2	109.0	1222.2
1976/77	96.0	184.2	143.7	189.7	276.4	380.6	321.9	12.4	0.0	0.0	0.0	15.6	1496.5	124.0	1620.5
1977/78	81.8	153.7	171.1	195.9	189.8	168.0	23.2	0.0	0.0	0.0	0.0	22.3	901.7	104.1	1005.8
Total	627.5	1201.0	1864.0	1707.3	1545.7	2629.0	1168.4	328.2	0.0	53.2	26.0	289.5			
N	10	10	10	10	10	10	10	10	10	10	10	10			
Average	62.8	120.1	186.4	170.7	154.6	262.9	116.8	32.8	0.0	5.3	2.6	29.0	1011.5	132.5	1144.0

Notes: Rainy Season (May ∿ Oct.) Dry Season (Nov. ∿ Apr.),

*(Data at Surin) x 0.623 + 58.679 (Rainy Season) or (Data at Surin) x 0.671 + 7.668 (Dry Season)

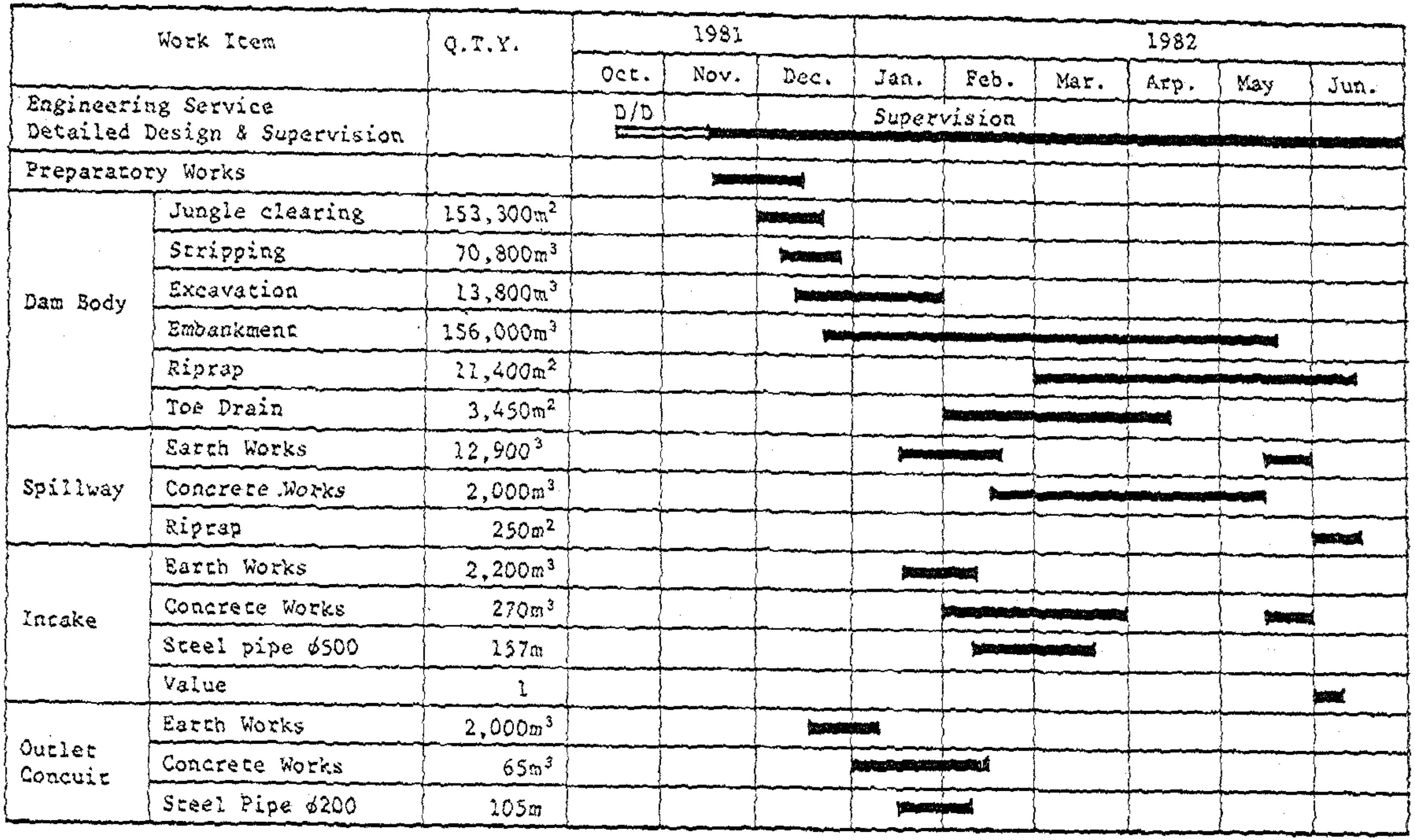
Table 3-2-4 Monthly Effective Rainfall

in millimeter

Month / Year	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Annual	
													Paddy	Upland
1968/69	104	77	(120) 200	(120) 159	114	(120) 200	18	3	0	0	0	35	910	711
1969/70	18	47	(120) 140	(120) 138	75	(120) 200	92	36	0	0	1	33	780	662
1970/71	24	119	(120) 200	(120) 200	119	(120) 200	112	0	0	0	7	0	981	741
1971/72	46	53	88	58	108	87	62	0	0	0	8	19	529	529
1972/73	48	14	113	33	96	(120) 200	83	112	0	0	4	33	736	656
1973/74	22	111	76	83	120	(120) 173	102	9	0	40	0	21	757	704
1974/75	29	108	68	(120) 132	102	(120) 123	93	66	0	0	0	25	746	731
1975/76	47	118	(120) 200	(120) 176	77	(120) 196	56	12	0	0	0	23	905	693
1976/77	72	(120) 138	108	(120) 142	(120) 200	(120) 200	(120) 200	9	0	0	0	12	1081	801
1977/78	61	115	(120) 128	(120) 147	(120) 142	(120) 126	17	0	0	0	0	17	753	690
Total	471	(882) 900	(1053) 1321	(1014) 1268	(1051) 1153	(1167) 1705	(755) 835	247	0	40	20	218		
Average	47	(88) 90	(105) 132	(101) 127	(105) 115	(117) 171	(76) 84	25	0	4	2	22	819	692

Note: ( ) -- Effective rainfall on upland field

Table 3-4-1 Construction Schedule (Ta Kao Dam)



Work Item		Q.T.Y.	1981			1982					
			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Arp.	May	Jun.
Engineering Service Detailed Design & Supervision			D/D			Supervision					
Preparatory Works											
Dam Body	Jungle clearing	153,300m^2									
	Stripping	70,800m^3									
	Excavation	13,800m^3									
	Embankment	156,000m^3									
	Riprap	21,400m^2									
	Toe Drain	3,450m^2									
Spillway	Earth Works	12,900^3									
	Concrete Works	2,000m^3									
	Riprap	250m^2									
Intake	Earth Works	2,200m^3									
	Concrete Works	270m^3									
	Steel pipe ϕ500	157m									
	Value	1									
Outlet Concuit	Earth Works	2,000m^3									
	Concrete Works	65m^3									
	Steel Pipe ϕ200	105m									

Fig. 3-2-1 Water Balance Study (Ta Kao Dam)

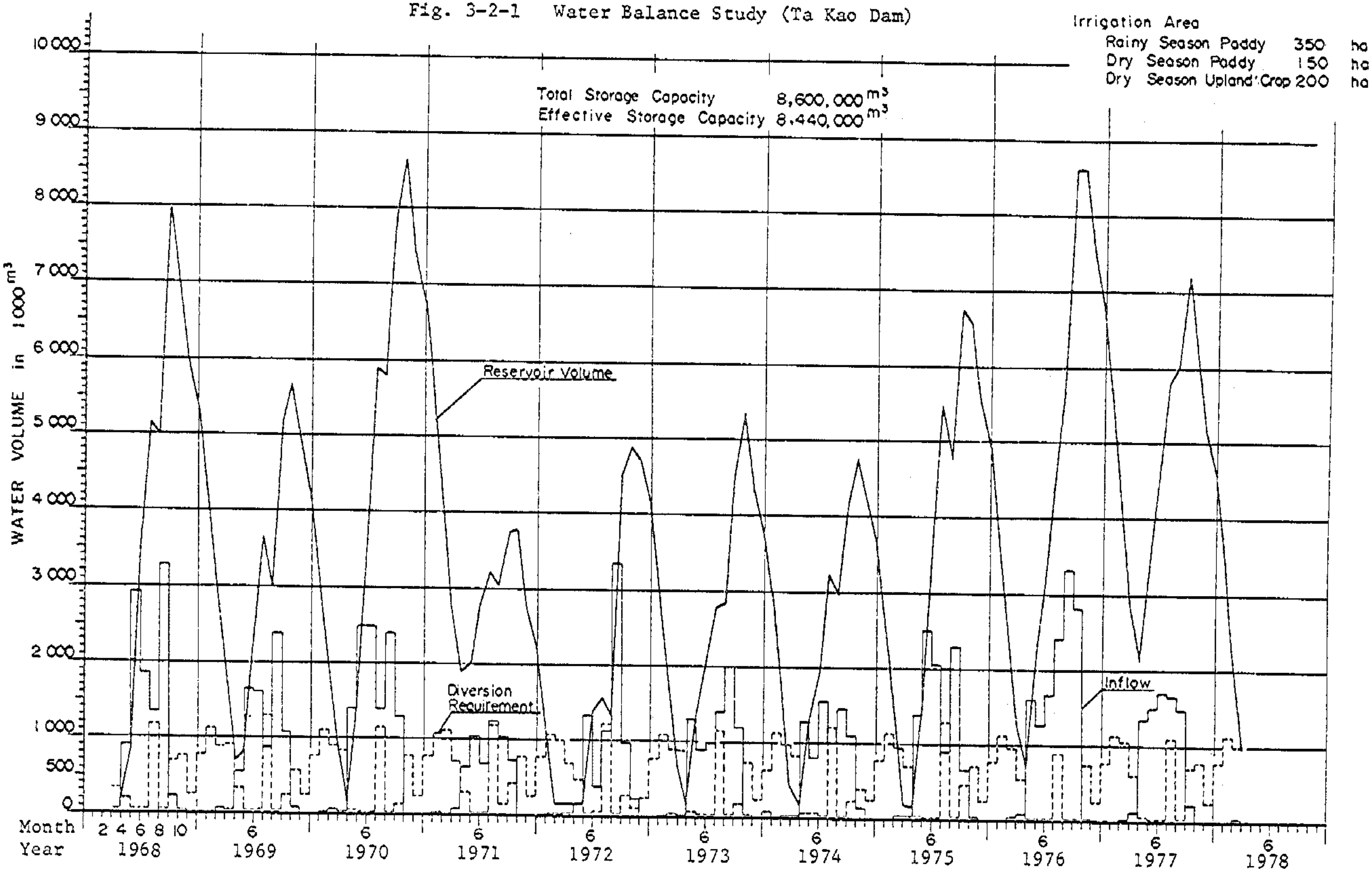
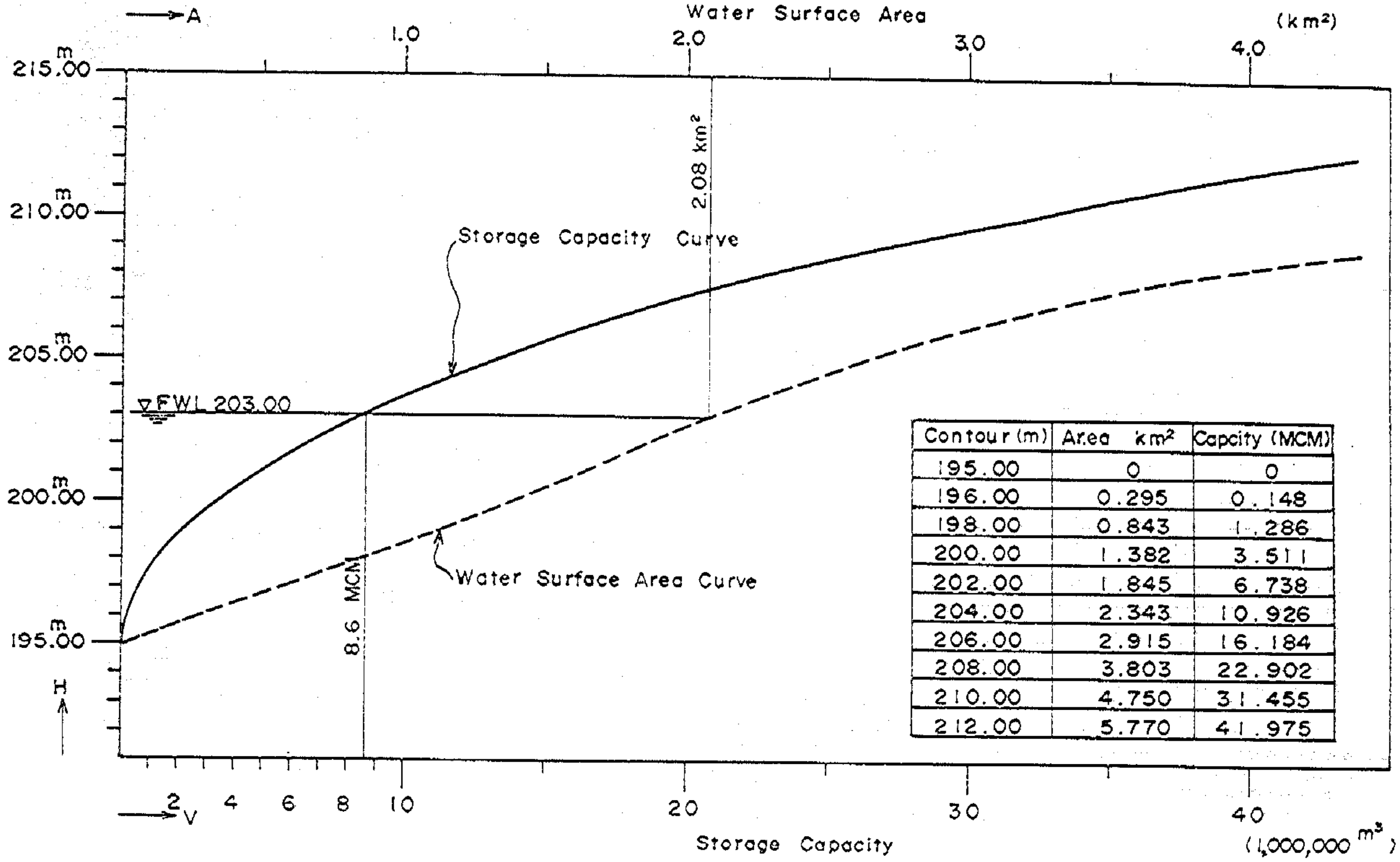


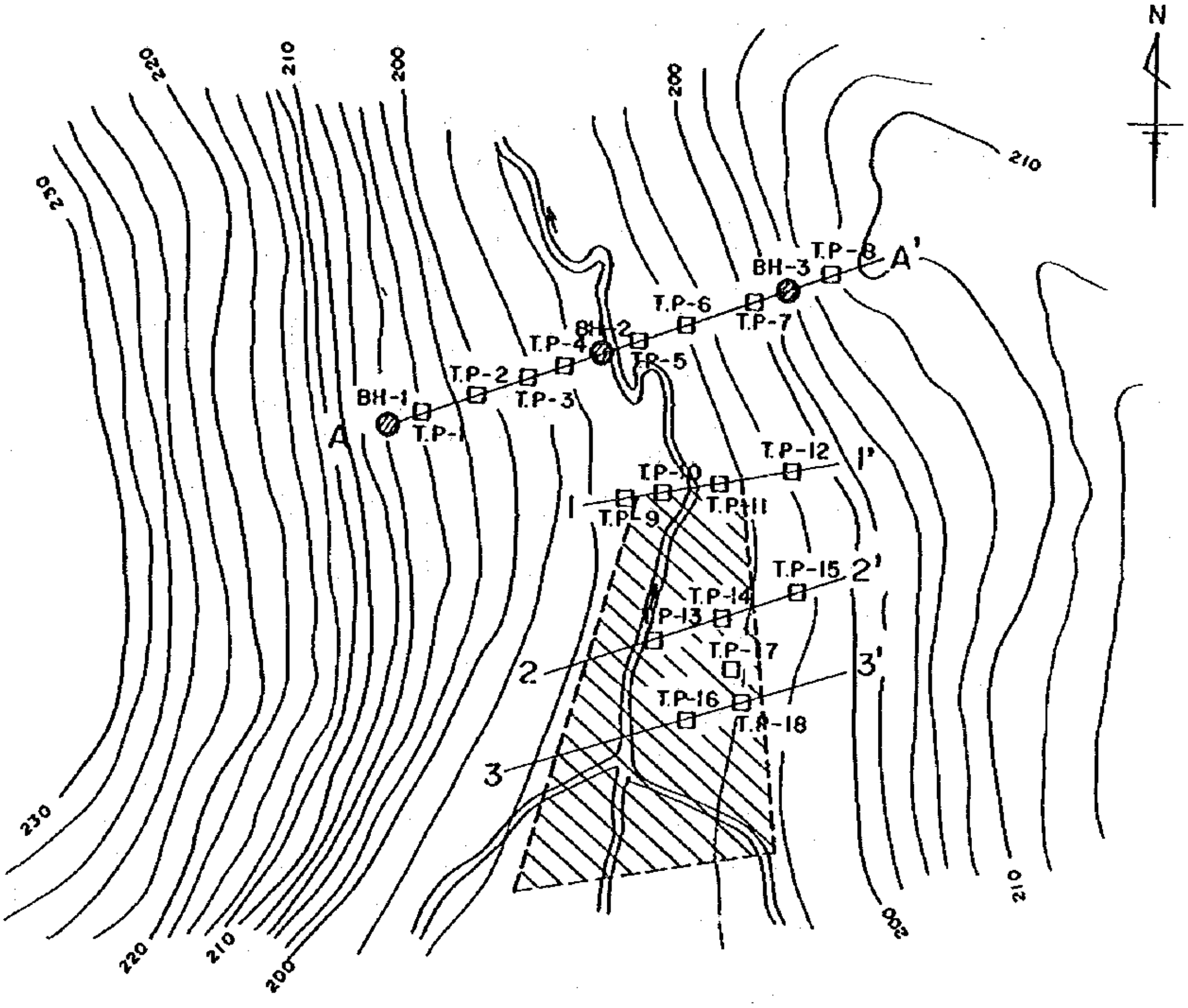
Fig. 3-3-1 H-V, H-A Curve (Ta Kao Dam)



Contour (m)	Area km²	Capcity (MCM)
195.00	0	0
196.00	0.295	0.148
198.00	0.843	1.286
200.00	1.382	3.511
202.00	1.845	6.738
204.00	2.343	10.926
206.00	2.915	16.184
208.00	3.803	22.902
210.00	4.750	31.455
212.00	5.770	41.975

Fig. 3-3-2 Location Map of Soil and Geological Investigation for Ta Kao Dam

Scale 1 : 10,000



LEGEND

⊘ Boring Point

□ Test Pit Point

▨ Borrow Area

Fig. 3-3-3 Geological Section of the Ta Kao Dam Axis

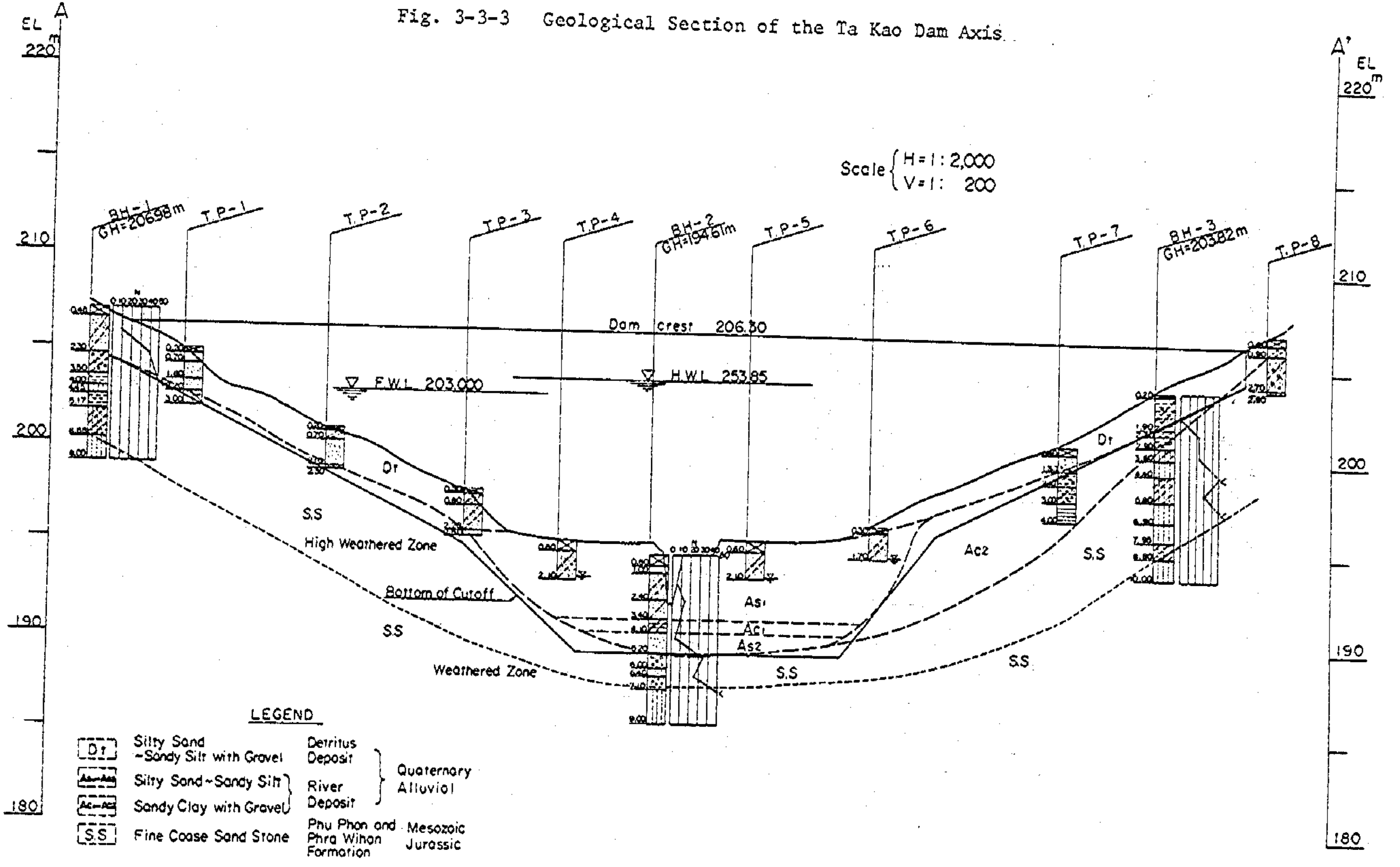


Fig. 3-3-4   Typical Section (Ta Kao Dam)

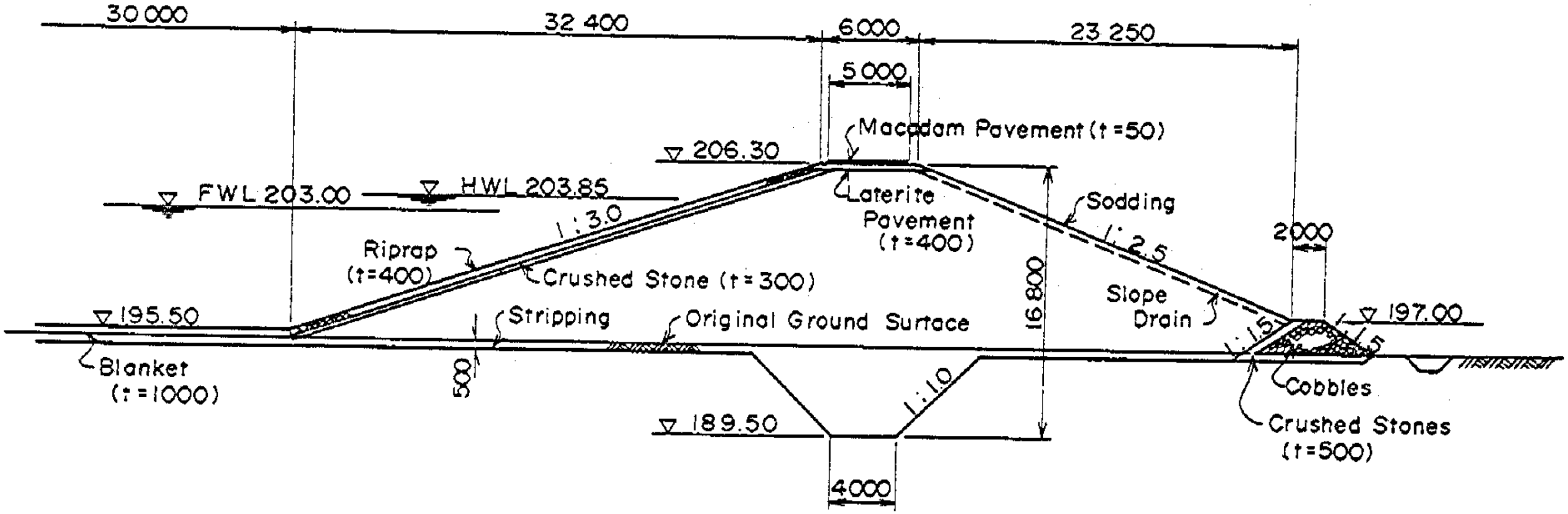


Fig. 3-3-5 Phreatic Surface

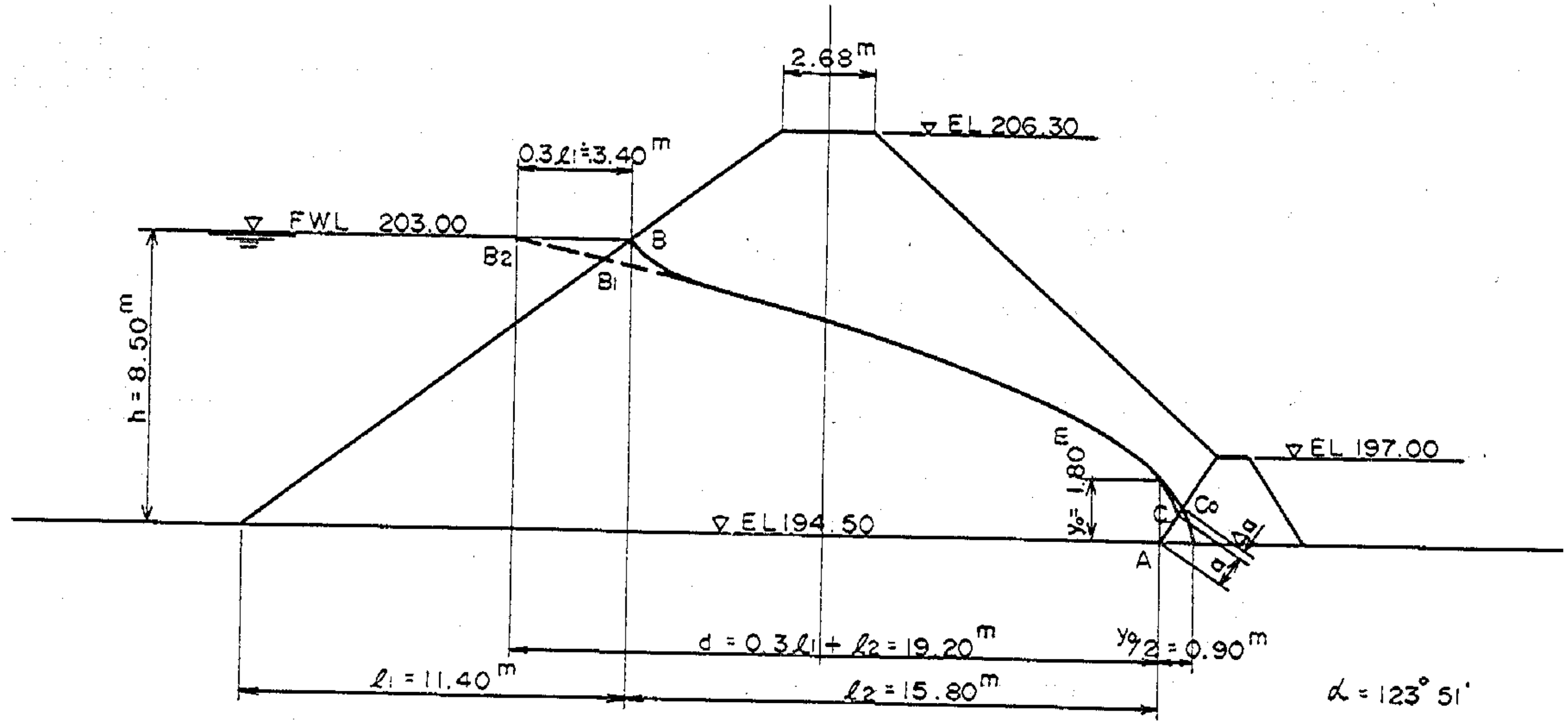


Fig. 3-3-6 Slope Stability Analysis (Ta Kao Dam)

Case 1 Fill Completed Condition

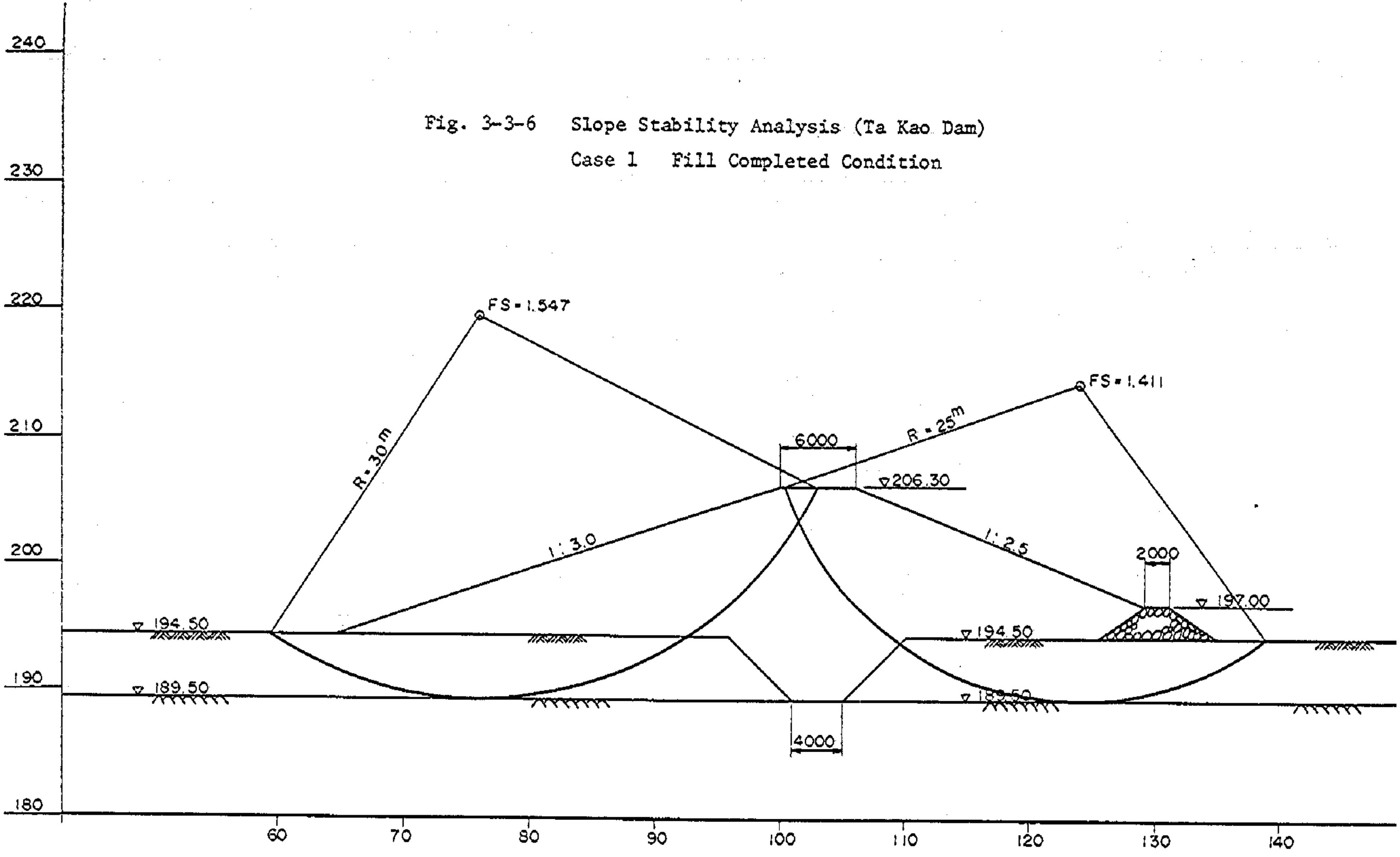


Fig. 3-3-7 Slope Stability Analysis (Ta Kao Dam)
Case 2 Full Reservoir Condition

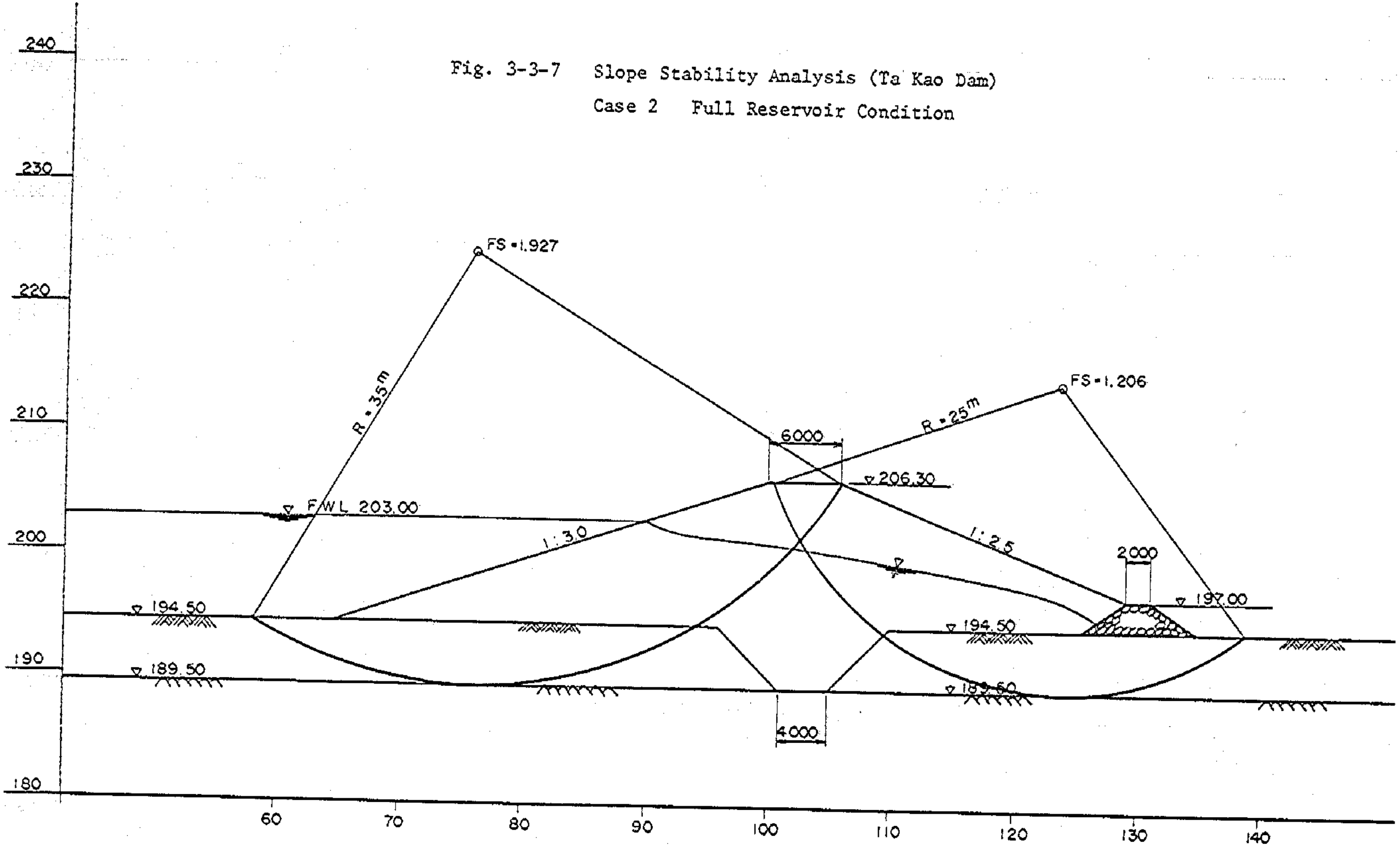


Fig. 3-3-8 Slope Stability Analysis (Ta Kao Dam)

Case 3 Rapid Drawdown Condition

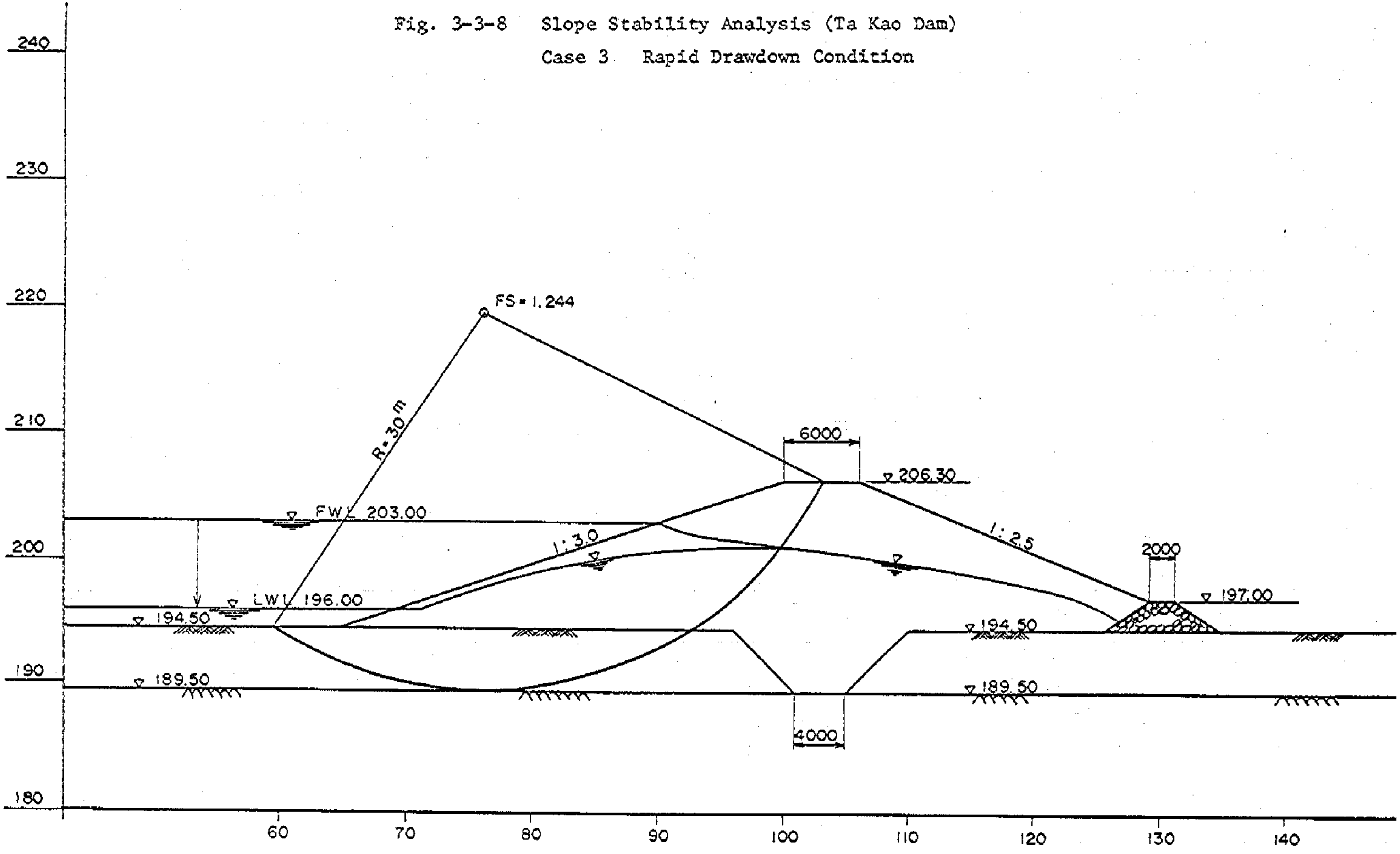
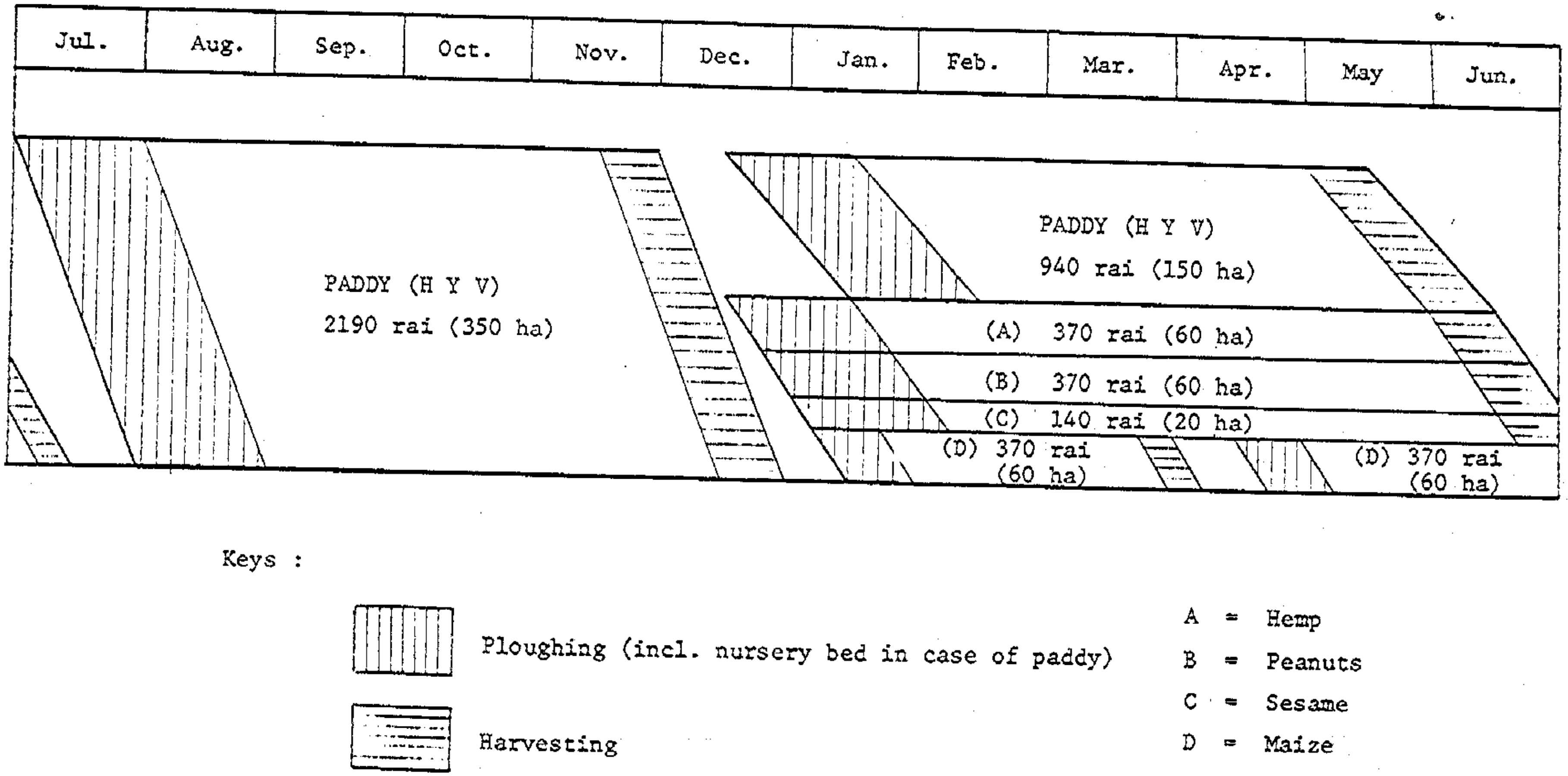
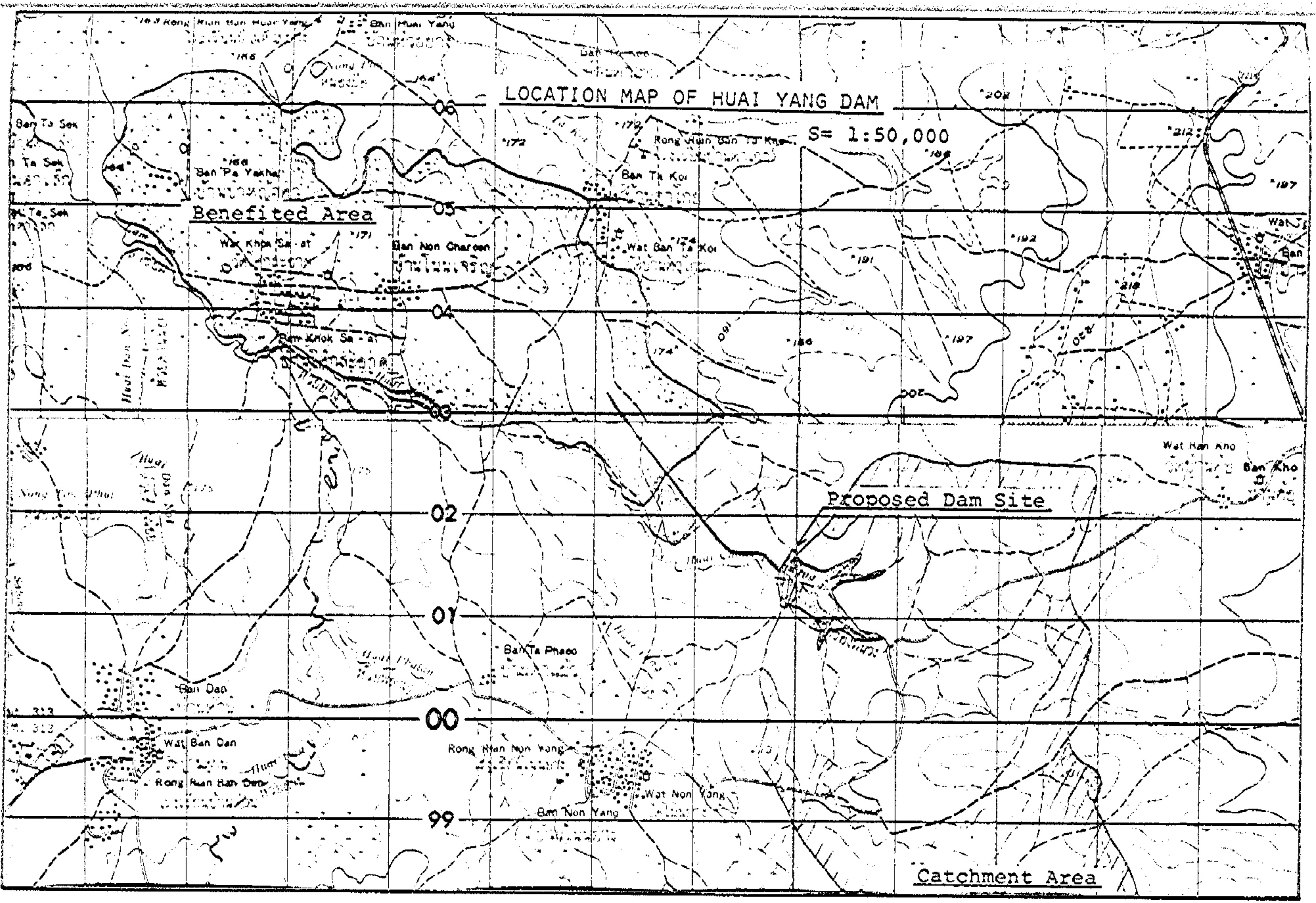


Fig. 3-6-1 Proposed Cropping Pattern





LOCATION MAP OF HUAI YANG DAM

S= 1:50,000

CHAPTER 4 HUAI YANG DAM IMPROVEMENT SCHEME

4.1 Project Area and Present Conditions

4.1.1 Location

The project area is located in Amphoe Nam Yun Province, Ubon Ratchathani District, in the eastern region of the Kingdom of Thailand. It is bordered by Provincial Highway No.2214, running at a distance of 6 km from the dam site in the east, and 10 km from Ban Nam Yun. The boundary for the catchment area of the dam is adjacent to the Cambodian border and is 10 km south of the site. In the neighbourhood of the boundary, a new village project is now in progress.

4.1.2 Topography

The catchment area has about 500 m high ridges in its boundary, forming the border between Cambodia and the eastern region of the country; and for the most part in the jungle zone with over 300 m elevation. The neighbouring area of the dam site is on a flat plain, and consists partially of upland crop fields and the remainder of copses. Paddy fields extend from about 1.0 km downstream. The existing dam is an earth-fill cofferdam built on the river, which can not function as a reservoir but only rather, as a divert.

The dam site is located on the Huai Chong Phanuang River which branches off from the Huai Yang River. The benefited area covers flat paddy and upland crop fields (170-180 m) extending along the right bank of the Huai Chong Phanuang River. The Huai Chong Phanuang River is believed to have a perennial flow throughout a year.

4.1.3 Geology and Soils

Here, no description is made for this project area because the area is governed by the same geological formation as for the Meh Kah Dam (see: 2.1.3).

4.1.4 Present Socio-Economic Conditions

(1) Direct and Indirect Benefit Area

With the completion of the project, the benefit area would involve five villages in total, i.e. three villages (Ban Khok Sa-at, Ban Nan Chareon and Ban Ta Kao in the Amphoe Nan Yun Province) directly and two villages (Ban Nan Yang and Ban Ta Kao) indirectly.

Area, Household, and Population under Benefited Area

Name of Villages	Total Area (Rai)	Total Household	Total Population
Ban Khok Sa-at	4,517	210	1,222
Ban Nan Chareon	1,756	76	476
Ban Ta Koi	2,326	98	496
Ban Nan Yang	3,405	181	1,128
Ban Ta Kao	2,375	49	306
Total	30,189	614	3,628

(2) Particulars of Direct Benefit Area

The particulars of the direct benefit area will be shown in the following.

Particulars of Direct Benefit Area

Name of Villages	Total Area (Rai)	Cultivated Land			Other Area	Buffalo
		Paddy	Upland Crops	Total		
Ban Khok Sa-at	4,517	4,100	200	4,300	217	432
Ban Nan Chareon	1,756	1,340	334	1,674	82	147
Ban Ta Koi	2,326	915	1,215	2,130	196	168
Total	8,599	6,355	1,749	8,104	495	747

Strenuous effort to develop this area has been made for many years and a new village (Ban Nan Chareon) has been recently set up due to the gradual population increase.

The benefit area extends on rather flat land and in two villages of Ban Sa-at and Ban Nan Chareon over 80% of land is being put under paddy cultivation. Paddy fields we used for rice production once a year during the rainy season and the upland fields are devoted exclusively for the cultivation of hemp and tapioca.

(3) Fundamental Structure of Agriculture

Enlargement of this area has been made through development efforts of the increased local inhabitants together with new settlers from other regions. They do not have landownership rights, but they do have cultivation rights.

The average farm-household has 20 rai of land. The rich (5%) has 80 rai, while the poor (10%), 7-8 rai.

(4) Agro-Economic Analysis

In the benefit area, average per rai yields of nuclear crops are: 300 kg of rice, 250 kg of hemp, and 2,000 kg of tapioca. While rice is primarily for the cultivator's own consumption with any surplus for marketing, upland crops are exclusively for marketing; the farm-gate price of rice is ฿2.9/kg, that of hemp, ฿5.0/kg, and that of tapioca, ฿0.4/kg. For rice production, the average farmers plough with their own buffaloes, and apply their own manure and little chemical fertilizer. They have no labour cost as they transplant and harvest through neighbourhood joint labour. The cash income per average farm-household would amount to about ฿16,000/year: ฿14,000 from paddy and ฿2,000 from upland crops.

In the long-range, however, it is reported that the cash income is actually below these estimates.

4.2 Water Utilization Plan

4.2.1 Water Resources

The source of water for this project is the Huai Chong Phanuang River which originates from the Huai Yang River. The dam proposed under the project would be constructed downstream of the confluence in which the Huai Chong Phanuang River is joined by the minor rivulets. The catchment area is 31.3 km^2.

Since there is no data on river discharge available for the Huai Chong Phanuang River, discharge would be calculated according to rainfall data.

4.2.2 Hydrology

The project area, which belongs to Ubon Ratchathani District, in the eastern region of the Kingdom of Thailand, is located 10 km from the border of Cambodia and extends on a semi-flat plain with approximately 190 m above sea level.

As meteorological data are not available within the project area, those recorded at the following two stations adjacent to the area have been adopted as the basic figures for the project.

Surin	170 km from the dam site
Ubon Ratchathani	80 km from the dam site

As much mean monthly and daily rainfall data as possible, recorded since 1952, have been collected. For reference, meteorological records at Ubon Ratchathani station are shown on the Table 4-2-1.

(1) Rainfall

The maximum probable daily rainfalls estimated by adopting the Gumbel Method at each station are as follows:

Maximum Probable Daily Rainfall (mm)

Probability	Station	
	Surin	Ubon Ratchathani
1/10	122	165
1/25	142	193
1/50	157	215
1/100	172	236
1/200	186	257

(2) Flood Discharge

The maximum flood discharge has been estimated at 184 m^3/sec by adopting the design rainfall of 193 mm/day and the 1/25 probable daily rainfall recorded at Ubon Ratchathani, situated nearby the dam site and having plentiful rainfall data available.

In designing the spillway, any temporary flood regulating capacity above the normal water level of a reservoir would not be taken into consideration due to its small capacity. Flood discharge calculation, based on the assumed overflow width and depth of the spillway as well as by adopting the maximum flood discharge valued at 184 m^3/sec, shows 1.20 m for overflow depth and 70.0 m for its width.

In view of the particularities of this dam, moreover in an emergency, the spillway would be used only for flood level above that with 1/25 probability.

4.2.3 Water Requirement

The amount of intake water of present estimated from the cross-sectional area and inclination of the channel is 0.1 - 0.5 m^3/sec. The amount of intake varies depending on the water level of the river, since the intake channel made by excavation without timbering has no facility for regulating the flow rate.

The existing channel without timbering measures 1.0 m wide at the bottom, 0.7 m in height and 2 km in total length. From the end of the channel the irrigation water flows from plot to plot.

About 2,000 rai of paddy field is irrigated at the high-water season of Hai Chong Phonuang. Water intake shall be made easier and the amount of intake greater on completion of a new dam. But there should be the accompanied construction of a new water channel or repairs on the existing ones in the lower streams. The present project being only for the dam, the amount of intake designed shall be the same as the present.

4.3 Dam Plan

4.3.1 Selection of Dam Axis

The existing dam body having a collapsed spillway portion is located at the dam site proposed under the project. The cofferdam was built for emergency purposes and now holds reservoir water. However, partial collapse on the slope of the existing dam body and seepage running towards the downstream slope (grassland) have been found. Either the drain was not operating effectively or there was no drain built. Its crest elevation is set at about 80 cm above the full water level. Therefore, additional embankment must be required due to freeboard shortage. A new dam axis has been selected at about 30 m downstream from the dam body, parrallel to it, as it would not be reliable enough alone. In the newly selected dam axis, construction would be done on the right bank by utilizing the existing dam body as a cofferdam and it would be done on the left bank without any cofferdam during the dry season.

Rough coffering would not be feasible as a temporary diversion channel. Under these circumstances, it is anticipated to have little influence on fish farming during the dry season.

4.3.2 Storage Capacity of Dam and Dam Type

Since the sounding of the existing reservoir could not be completed within the limited number of days, a water level - storage capacity curve shall not be made in this project.

The maximum depth at the high water level is 7 m and the designed high water level is the same as in the case of the existing reservoir. The estimated storage capacity of the dam in this project is approximately 300,000 m^3. At the proposed dam site, there is a small outcrop of rock on the left bank, but bedrock could not be identified in the present survey since drilling test was not performed. But the assumed position of the bedrock is relatively deep, and therefore, the construction of a concrete dam requiring deep excavation for providing a foundation of sufficient load bearing capacity, would not be economical.

In the case of an earth-fill dam, the scale of the dam body will be similar to that of the existing one and there is no problem regarding bearing capacity and the cost of construction will not be very high. Therefore the project dam will be an earth-fill dam.

4.3.3 Foundation and Borrow Areas

(1) Dam Foundation Bedrock

As a result of dam foundation study through 5 test pittings, the following have been concluded (see: Figure 4-3-1 and 4-3-2).

Geological Condition

Geological formation for the dam axis, as illustrated in Figure 2-1-1, consists of sandstone and conglomerate corresponding to

Phu Phan and Phra Wihan formations of the Mesozoic Jurassic period and, on top of it clay-sandy silt layers of the Quaternary Diluvial period and river and detritus deposits of the Alluvial period are being distributed.

The foundation bedrock has been confirmed at the depth of 2.0 m below the surface at TP-1 on the left bank of the dam axis and confirmation could not be made to any other points. The point of TP-1 is in an advanced stage of weathering and is in a very fragile condition.

On top of the bedrock, clay-sandy silt layers of the Diluvial period, which contain a small quantity of fine gravel and are stiff, extend to the depth of 0.6-4.6 m below the surface. The river and detritus deposits, which are being distributed over this Diluvium formation, are all unconsolidated and composed of silty sand, sandy clay, etc., containing cracked stumps and formicaries. Spreading to a depth of about 2 m below the surface, they are extremely loose. The completion of foundation treatment will be done primarily on the As_1-Ac_1 layer due to its weakness.

Permeability of Foundation Bedrock

Layer Coefficients of Permeability are as follows:

Name of Layer	Coefficient of Permeability (cm/sec)
As_1, As_2 and Dt Layers	$K = 10^{-4} - 10^{-5}$
D C Layer	$K = 10^{-5} - 10^{-6}$

Judging from the above results, the dam foundation particularly for As_1, As_2, and Dt layers has been found to have higher permeability, giving it a piping tendency. The completion of foundation treatment will, therefore, be by cutoff, etc.

(2) Borrow Areas

Borrow areas were selected through field investigation at two places: each bank of about 100-300 m upstream from the proposed dam axis (see: Figure 4-3-1). Their selection was made in consideration of topographic, geological and soil conditions and also by taking into due account the present condition of the existing dam along with new dam construction planning.

Selected borrow area study has been made through 5 test pittings, sampling and soil tests. The results of the study are shown below

Soil Condition

Borrow Area	Soil Quality	Depth	Colour
Left Bank	Surface Soil and Silty Sand	Surface-0.4 m	Yellowish-Gray Dark Brown
	Sandy Silt-sandy Clay	0.4 m and Deeper	Yellowish-Gray
	Surface Soil and Silty Sand	Surface-1.3 m	Yellowish Brown-Dark Brown
Right Bank	Sandy Clay	1.3 m and Deeper	Light Gray

Sandy silt sandy clay, which extends to the depth of 0.4-1.3 m below the surface, can be used as embankment material. Availability of this sandy silt sandy clay is as follows:

	Left Bank	Right Bank	Total
Borrow Area :	18,750 m^2	30,000 m^2	
Excavation Depth:	3.0 m	3.0 m	
Volume :	56,250 m^3	90,000 m^3	146,250 m^3

The entire requirement for the embankment material is about 100,000 m^3 which is quantitatively sufficient.

Design Parameters for Embankment Material

Sandy silt sandy clay, usable as embankment material is shown the following design parameters:

	Left Bank	Right Bank
Specific Gravity	Gs = 2.657	Gs = 2.652
Design Moisture Ratio	W = 13.0 ± 1.5%	W = 16.5 ± 1.5%
Design Dry Density	$\gamma d = 1.85\ t/m^3$	$\gamma d = 1.71\ t/m^3$
Design Wet Density	$\gamma t = 2.09\ t/m^3$	$\gamma t = 1.99\ t/m^3$
Saturated Density	$\gamma sat = 2.15\ t/m^3$	$\gamma sat = 2.07\ t/m^3$
Coefficient of Permeability	K20 = 5.0 x 10^{-6}cm/sec.	K20 = 4.0 x 10^{-6}cm/sec.
Cohesion	Cu = 1.0 t/m^2	Cu = 2.2 t/m^2
Angle of Internal Friction	$\phi u = 24$	$\phi u = 17$

4.3.4 Dam Design

(1) Selection of Dam Type

As a result of the test pittings and field investigation conducted in the proposed borrow area, silty sand, sandy silt and sandy clay were found to be obtainable from the neighbouring area of the dam site. As was judged that the left bank is primarily composed of earth containing a large percentage of sand, which would be considered usable as embankment material due to its low active head. The material, which contain a relatively large percentage of clay, can be used for the right bank located towards the current river course on the collapsed portion of the spillway. The volume of the embankment material available on the left foot would be about 56,000 m^3, while that on the right foot, about 90,000 m^3, ensuring a liberal supply from both proposed borrow areas. Judging from the condition of the embankment material, the homogeneous dam has been chosen over other types.

Toe, horizontal, and chimney drains would be deemed suitable for use on the homogeneous dam. Among those, the chimney drain would be dependable and is easily constructed. The chimney and outlet require a large volume of sand, however, the permeability of the required sand would not be high. A sufficient volume of filter material would be obtainable by removing fine-grained portions through sluicing of the sandy soils available in the downstream of the dam body. The toe drain has been adopted for the Meh Kah and Ta Khao dams. However, for this dam, whose water utilization is such that it will remain at full-water level, the more stable chimney drain will be used.

(2) Determination of Dam Height

This reservoir can be used for fish farming and irrigation purposes. The design full water level would be determined at 50 cm higher than the level at which the existing intake work is being located (EL. 188.80). Assuming that the overflow depth corresponding to 25-year probable design flood discharge is 1.20 m and that corresponding to 200-year probable design flood discharge 1.50 m, the proposed spillway would be based on

Design Full Water Level : F.W.L. 189.30
Design Flood Level : H.W.L. 190.50
Design Maximum Water Level : Max. W.L. 190.80

Design freeboard is equal to the computed run-up from a specific wave height plus the wind setup would be as low as 0.01 m on the assumptions of 20 m/sec wind velocity, 400 m fetch distance, and 4.5 m average water depth. As a result of wave run-up calculation

gether with 1:2.5 upstream slope, it shows 0.21 m for dumped riprap and 0.56 m for hard-placed riprap. The dumped riprap is most popular and also effectively usable as slope protection; however, the hard-placed riprap would be chosen as riprap material, because riprap material have been purchased. Accordingly, the wave run-up has been determined at 0.56 m.

As a result of the above, crest elevation will be computed by the following formula:

Design Crest Elevation = Design Maximum Water Level
+ Wind Runup + Wind Setup
= Max. W.L. 190.80 + 0.01 + 0.56
= EL. 191.37 = EL. 191.40

Dam Height = Design Crest Elevation - Excavated Riverbed Level
= EL. 191.40 - EL. 181.00
= 10.40 m

(3) Design of Typical Cross Section

The homogeneous dam has been chosen. As the existing dam body located in the upstream of the proposed dam would not be reliable enough, it would not be combined with a newly designed dam but used as a cofferdam. Embankment for the old river course will be made through excavation of its toe of downstream slope and the slope would be made so as to retain its stability during the construction period. Filter will be provided to prevent piping. It is necessary to consider seepage through the excavation surface but the height of a filter would be determined at any level above an intersection point with a new slope.

Since on the foundation of this dam a lot of formicaries could be found in between the surface and 2 m below it, foundation treatment will be done through a cutoff trench which will have to be 2 m or more deep and reach the Diluvium deposits. The cutoff trench will have to be conducted upto the level of EL. 180.00 because of the unknown depth of younger river deposits on the old river course but necessary measures would be taken up for this purpose through confirmation of the thickness of the younger river deposits during the construction period. The bottom width of the cutoff trench has been decided at 4 m to facilitate the operation of compaction machines.

The slope will be 1:2.5 on the upstream side and 1:2.0 on the downstream side. The crest width has been estimated at 4 m from the view of construction ease. To prevent the slope from gully erosion by shortening the surface sloping distance of rainfall water, beam and drainage channel will be provided on the downstream slope (EL. 187.00). The chimney drain would be adoptable. Usable materials would be sandy soils which extend on the downstream portion of the dam axis, but will need to be washed. The chimney drain will have a 1 m width. Adoptable construction method for this project is to backfill the trench, with sand which was excavated by the use of a backhoe loader, after three-layer bankings each. The crest elevation of the chimney drain would be set at the level, which is identical to that of the maximum water level, i.e. EL. 189.30, due to the existance of high probability for horizontal seepage through unisotropy of compacted earth.

The upstream slope will be protected by riprap, while the downstream slope below the berm, which is easily affected by high downstream water level at flood time, will be also protected by riprap but upward it will be covered by grass turf.

As a result of the above discussion, the typical cross section of the dam body is illustrated in Figure 4-3-3.

Design of Slope Protection

As riprap materials are purchased, soil cement would be usable as substitute for riprap. However, the upstream slope will be protected by riprap because the grain distribution of the embankment materials would require fine grains which do not meet the applicability of soil cement. The grain size of the riprap materials ranges between 60 kg at a maximum and 2 kg at a minimum, averaging at 14.8 kg and has been determined by the computed waveheight. The usable riprap material should have a specific gravity of over 2.63 and over 500 kg/cm^2 compressive strength. Beneath the downstream berm, the thickness of riprap has been decided at 20 cm due to little wave effectiveness. A sand and gravel blanket will be provided with the same thickness below the riprap of the upstream and downstream slopes. In order to prevent the blanket leaking through riprap voids, the blanket would be required to have coars grains. Filter fabric would be used in between the embankment an the blanket.

Filter Design

In order to prevent piping, a chimney drain needs to have such grain size as usable as a filter. Embankment materials involve sandy soils for which soil test was not made but filter design would be done for fine-grained and cohesive soils. The maximum grain size of the filter would be 4.76 mm or lower. The grain distribution of cohesive soils and that of filter are illustrated in Figure 4-3-4 and the latter would be determined by filter criteria.

(4) Seepage Analysis

This embankment will be compacted by a sheepsfoot roller. In determining the flow net, the ratio of the vertical permeability coefficient to the horizontal is assumed to be 1:5. A phreatic surface will be estimated by adopting the Cassgrande's method. The flow net, which has been made by taking into consideration the computed phreatic surface and boundary conditions, is shown in Figure 4-3-5.

Seepage through the embankment has been calculated in terms of the determined flow net. The coefficient of permeability has been computed at 1×10^{-6} cm/sec through a laboratory permeability test but it will be replaced by 2×10^{-6}cm/sec. by taking into account dry-season construction. Assuming that the embankment would be 400 m long corresponding to the total head valued at 7.3 m, the leakage amount through the embankment has been calculated as below.

Leakage Amount Through Embankment: $Q = 11.3\ m^3/day$

The seepage water, which runs round the embankment, would be deemed negligible because cohesive soil layers extend at the right abutments, while the left abutments are flat providing higher groundwater level.

Under these circumstances, the leakage amount would be found to be within the allowable range. In order to safety keep the above seepage water out, the filter will have to have over 7 x 10^{-4} cm/sec. coefficient of permeability.

(5) Stability Analysis

Determination of Design Parameters

Embankment construction will be done through density control. It will have the following conditions: the moisture content would be within ± 3% of its optimum and the dry density be 95% of its maximum. The results of compaction tests are shown in Figure 4-3-6. The higher the density, the lower the safety factor will be, if and only if other conditions remain unchanged. Accordingly, if the design density is determined from TP-6 which gives a high density, the results are as follows:

Wet Density $\gamma t = 2.17\ t/m^3$
Saturated Density $\gamma sat = 2.22\ t/m^3$

Direct shear test has been made on the basis of the above moisture content and dry density and the results are illustrated in Figure 4-3-7. Design strength has been determined through Data TP-6 which proves to be low within the range of stress on sliding surface. Assuming that the design strength would be 70% of the empirical value, the results are as follows:

Cohesion :. $C = 1.0\ t/m^2$
Angle of Internal Friction : $\phi = 24$

In stability analysis, moreover, the above-mentioned design parameters would be used regardless of any distinction between the riprap and the chimney drain.

Stability Analysis

In this dam, the effective water level for irrigation purposes ranges to the depth 50 cm below the full water leve.
A rapid drawdown of the water level to below the minimum

effective water level would be deemed impossible. Accordingly, stability analysis would be made for the upstream and downstream slopes with two cases: the one for fill completed condition and the other for at full reservoir condition. Construction pore prssure is taken into account in the case of the former, and the pore pressure of a steady seepage in the case of the later. As judging from a 1 x 10^{-6} cm/sec. coefficient of permeability obtained from laboratory tests, pore pressure dissipation would be deemed low. It would be assumed that 50% of the uplift would be residual construction pore pressure. It is illustrated in Figure 4-3-8 by taking into account the above assumption and the boundary conditions. The flow net at full reservoir has been determined through seepage analysis.

Stability analysis has been made on the basis of the slice method and allowable safety factors for all cases has been assumed to be 1.2. The results of stability analysis are illustrated in Figures 4-3-9 and 4-3-10 and the computed safety factor is as follows:

	Upstream Face	Downstream Face
Fill Completed Condition	1.577	1.360
Full Reservoir Condition	1.404	1.67

Accordingly, this dam would be under any circumstances considered to be safe against sliding.

(6) Extra-Banking Design

Extra-banking would be provided for settlement of the embankment and the foundation which occurs upon completion of the embankment. Consolidation analysis will not be made for the foundation, which is composed of cohesive soils, because the occurrence of the foundation has not been found to have settled.

The extra-banking is determined through the results of consolidation test as a rule, but in this project this test has not been made. Accordingly, determination of the extra-banking has been made by using the shear modulus and the elastic modulus estimated from the shear strain-shear stress curve through a direct shear test. The curve is illustrated in Figure 4-3-11.

Shear Modulus : $G = 110\ t/m^2$

Elastic Modulus : $E = 319\ t/m^2$

Assuming that the residual rate of pore pressure upon completion of the embankment work is set at 50%, identical to that in stability analysis, the estimated extra-banking height would be 0.22 m but in this project it has been decided at 25 cm. Its longitudinal distribution is shown in a profile.

4.3.5 Spillway Design

(1) Selection of Location

The spillway has been located at Point No.2 + 50.0 on the left bank. This decision is based on topographical and geological considerations, as well as safety and economy.

The bedrock spread to the depth of about 2 m below the surface providing a good foundation in the neighbourhood of Point No.2 + 50.0. Furthermore, on the right bank there is not enough space between the existing canal and natural ground to construct the proposed spillway. As Point No.2 + 50.0 is flat from the topographical viewpoint and located on the left bank of the current river course, the spillway end can be easily connected to the river course. Accordingly, Point No.2 + 50.0 would be deemed most appropriate for the location of the proposed spillway.

(2) Determination of Type

Overflow spillway (non-regulating type) equipped with a straight open channel has been adopted from the topographical, geological and O & M considerations. Hydraulic, economic and the structural stability were also considered.

(3) Determination of Spillway Scale

Intercepted plain river water can be used for irrigation purposes and for fish farming purposes in the reservoir upstream. Due to the limited scale of the dam, however, the capacity for flood regulation would be negligible. Therefore, spillway will be designed without considering flood control potential.

In designing a spillway, any combination of overflow depth and width should be calculated; however, the overflow width and the overflow discharge have been estimated at 70 m and 184 m^3/sec, respectively, on the assumption that the overflow depth is set at 1.2 m in consideration of financial limitations, as no restrictions should be made from the topographical point of view. In this project, the overflow depth, the overflow width, and the overflow discharge have been determined at 1.2 m, 70 m, and 184 m^3/sec., respectively, by taking into account the following: (1) this scale is fairly large in comparison to those given to other dams, but its capacity for flood regulation would be negligible; (2) the amount of rainfall water within this area is larger than that in the catchment areas covered under other dams; and (3) the overflow width was set at 60 m according to the plan worked out by the Ministry of Interior.

(4) Diversion Channel

A diversion channel will be provided at the bed height of the existing river with 1/1.5 slope and would be designed as a concrete structure type because of its sharp fall.

(5) Scour Protection Stilling Basin

In order to drain chute water to the current river by reducing the water pressure, a scour protection stilling basin will be provided on the diversion channel end. A horizontal stilling basin type III has been adopted with 6 m length.

(6) Emergency Spillway

An emergency spillway will be provided by taking into consideration the flood discharge determined in terms of the Japanese design standards for fill dam. In connection with its scale, it has been designed to have a 35 m width, 1/300 excavation-without-timbering slope, and 37 m^3/sec. falling discharge

4.3.6 Design of Intake Work

(1) Selection of Location

At present, the irrigation water supply canal in which irrigation water is being kept and the channel for which excavation work is being suspended are located on the right bank. Under these circumstances, location of the proposed intake work has been determined in the neighbourhood of the existing irrigation canal. Point No.7 + 15.0 has been selected as the proposed location of the intake work by taking into account the topography and geology of the vicinity of the proposed point.

(2) Determination of Type

In this project, the intake work will be provided at a level identical to the full water level and thereby making negligible the effect of water pressure on it. The ferro-concrete sluice type, having a water control gate, has been adopted and will be provided on the reservoir side.

(3) Amount of Intake Water

As a result of local interviews, the amount of intake water could not be computed because the benefit area remain unclear. Therefore, a determination of the scale of the proposed channel has been made more or less identical to that of the existing one.

4.4 Construction Plan

4.4.1 Outline

The quantities of construction work for the dam are shown in Table 4-4-1. Construction work can be done in the dry season only and all work shall be completed in one year.

The execution plan shall be made in order that the existing dam can be used as a coffer dam at the same time paying full attention to minimizing the influence of the work on the fish in the fish farm.

Riprap and gravel will be included in the materials for dam construction to be purchased.

The filter material to be collected from the lower stream shall be stock-piled after washing in the abundant river water of the rainy season.

In order to made use of the existing dam as a coffer dam, the embankment work shall be divided into three sections, namely the right bank, the left bank and the fractured part of the existing spillway.

It is at the time of embanking of the existing fractured part that a full-scale coffer dam shall be constructed, and the intake works on the right bank will be used as a temporary drain at that time. The conceived temporary drain is considered to be sufficient in the dry season for the short construction period.

The concrete work shall be done using a concrete mixer at the site.

4.4.2 Construction Schedule

Roads for construction work shall be built after opening an access road of about 2.0 km.

Even though one way of making it possible to across the river course at the existing fractured part is building a wooden bridge there, embanking on top of corrugated metal pipes is suited if the structure is to be removed on completion of the project.

The main construction schedule following the temporary works is as follows:

1) Intake works on the right bank excavation and concreting after rough coffering.

2) Removal of the top soil from the right bank embankment, excavation of a cut-off trench and embanking to be performed in sequence. The removel of top soil is to be done prior to embanking. The seepage water in the cut-off trench is to be drained by drain ditch.

   Spillway excavation and concreting to be done along with embanking of the right bank. No temporary cofferdam upstream, but one to be made the donwstream.

3) Embanking on the left bank - removal, first of the top soil, and the excavation of a cut-off trench. Removal of the top soil of the borrowpit on the left bank is to be done prior to embanking emergency spillway excavated woil other than the top soil being used for embanking the emergency spillway to be constructed at the time of embanking.

4) Temporary coffering making use of the intake channel on the right bank as a temporary drain channel. Temporary coffers to be made upstream and downstream of the existing fractured part.

   Embanking of the fractured part for maintaining a smooth sequence of work, with embanking on the right banking the soil from the borrow pit on the right bank.

5) Removal of the Existing Dam and Coffer Dams.

The plan indicating the construction schedule and the flow-chart of the works to be shown Fig. 4-4-1 and Table 4-4-2.

4.4.3 *Construction Machinery*

The following machinery shall be selected considering the topographical features of the dam site and the properties of embanking soil.

For excavation: Since the distance for conveying embanking materials is only about 400 m, tractor shovels shall be used considering the small scale of the dam to be built and the limited space of work.

For conveyance: Dump trucks corresponding in capacity to the tractor shovels for loading.

For spreading: Bull dozers shall used. Spreading of the filter materials shall be done manually considering the working space in the excavated trench of a chimney drain.

For compaction: A sheepsfoot roller shall be used since the majority of the embanking materials is cohesive soil. The scale of the sheepsfoot roller shall be heavy since the embanking materials has a low moisture content. A small-scale oscillating roller shall be used for compaction of the chimney drain since its width is only one meter. The soil of horizontal part of the drain shall be spread and compacted by bulldozer.

4.5 Project Cost

4.5.1 General Description

The total cost of this project is estimated at 277,000,000 Yen (฿26,670.00). The project cost includes the construction cost and the consultancy cost. The construction cost consists of both dam embankment and related works (intake facility and spillway) only. The consulting services comprise the detailed design. The proposed site of this project is identified to be government-owned land; partially light jungle and partially uncultivated fields, therefore there is no need to compensate for the land.

As stated above, only the dam embankment and related works shall be constructed in this project, however construction of new irrigation channels or repairing of existing ones shall be carried out by the Government of Thailand.

4.5.2 Construction Cost

In estimation of construction cost, unit costs for the Meh Kah Dam were used as a reference.

4.5.3 Detailed Design and Construction Supervision

Since the proposed site of construction is located in a remote frontier without no town in the neighbourhood having a suitable accommodation facilities and also because the scale of construction work is small, the cost of construction supervision is not included in the consultancy cost. Supervising engineers stationed at the Ta Kao Dam shall visit the site about once a week for construction supervision. Only the detailed design cost is included in the consultancy cost.

4.5.4 Project Cost

A. Construction Cost

		¥	฿
1.	Dam Body	94,577,000	9,093,842
2.	Spillway	111,920,000	10,761,511
3.	Intake Works	3,714,000	357,143
4.	Temporary Works	5,660,000	544,240
5.	Indirect Construction Cost	12,950,000	1,245,264
	Sub total	228,821,000	22,002,000
6.	Miscellaneous Expenses	43,576,000	4,190,000
	Total	272,397,000	26,192,000
		≒ 272,000,000	≒ 26,190,000

B.	Detailed Design and Construction Supervision	5,000,000	480,000
	Grand Total	277,000,000	26,670,000

(No contingency is included in the above)

Exchange rates (as of 15 August, 1981)
1 U.S.$ = 22.6 ฿
1 U.S.$ = 235 ¥
1 ฿ = 10.4 ¥

4.6 Evaluation

4.6.1 Outline

The dam proposed under this project is actually a diversion weir rather than a dam from the engineering point of view. The river water will flow into the channel located on the river bank by stopping the current river flow and thereby raising the river water level. The aim of the project is to utilize the river water for irrigation and miscellaneous purposes. However, the reservoir water (its estimated storage capacity: 300,000 m^3), which would be stored as a natural result of stopping the river flow, will be utilized for fish farming purposes.

As the project effect is for securing the present efficiency of the existing dam by preventing its collapse, the anticipated amount of damage incurable in case of collapse would be computed as a project benefit. This states that the present benefits accruable from the existing dam would be identified as a benefit for this project. However, the facilities proposed under the project would be provided with stable and perennial water supply and thereby being more beneficial than the existing dam.

4.6.2 Project Benefits

At present, little flowing water is available even during the dry season. It has been considered adequate for miscellaneous usage, but inadequate for irrigation. Therefore, dry season irrigation water would not be counted as a project benefit. The present amount of intake water has been estimated at about 0.1 - 0.15 m^3/sec through a rough estimation of the scale of the irrigation canal. The irrigable paddy area during the rainy season would amoutn to 2,000 rai (430 ha).

As mentioned above, project benefit calculation has been made based on the premise of the existing dam's collapse and the cost of subsequent replacement by a newly designed one. It is reported that paddy yield per rai is 300 kg based on only rainfall water; however, it would actually only be about 250 kg in the ling-run due to the large variability of annual rainfall according to rainfall records which include some drought years.

With river water irrigation and rainfall water, however the paddy field provide stable harvests and 350 kg paddy yield per rai can be assured.

(350 kg - 250 kg) x 2,000 rai = 200 tons

200 tons x ฿2,900/ton = ฿ 580,000

After the completion of the project, the amount of intake water would be expected to be large through the widening and lining of the irrigation canal and the irrigable area would be broadly enlarged.

Under these circumstances, the project benefits would be expected to be three times the amount of present benefits, as shown below.

฿ 580,000 x 3 = ฿ 1,740,000

In this mountain village adjacent to the border, a very important part of the protein diet of the villagers is supplied by fish. The important strategy adopted by this project is fish farming. Benefit calculation for fish farms is included. With a 6-month hatching period of fish and 50 kg annual fish production household, the rough benefits accruable have been computed as follows:

300 Housholds x 50 kg = 15,00 kg

15,00 kg x ฿ 25 = ฿ 375,000

Hence, the total annual rough benefits will be:

	Present	Future
Rainy-season Paddy Production:	฿ 580,000	฿1,740,000
Fish Production:	฿ 375,000	฿ 375,000
Total	฿955,000	฿2,115,000

4.7 Urgent Matters

4.7.1 Matters to be Settled Prior to the Commencement of Construction

The Government of Thailand is requested to take full consideration in settling the following matters promptly, since it would be impossible to start the projected construction, even after the completion of the official international procedures between Thailand and Japan, if they are left unsettled.

(1) Approval for cutting trees at the dam site and the area of future submersion.

(2) Cutting trees in the area of future submersion.

(3) Compensation for farm crops and all other kinds of compensations for the dam site and the area of future submersion.

(4) Offering the land required for field offices, laborers houses, a machinery and material yard and other facilities for construction.

(5) Receiving agreement by the inhabitants concerned regarding the road reconstruction work for an access road.

4.7.2 Note for Construction Work

There may be some changes in the schedule and specification of the construction work at the time of execution, since the short period of survey necessarily limited the number of test drilling and test pits for data collection.

LIST OF TABLES AND FIGURES IN THE CHAPTER 4

TABLES

FIGURES

Table 4-2-1 Climatological Data (Ubon Ratchathani) for the Period 1951 - 1975

Temperature (°C)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean	24.0	26.3	28.8	29.9	29.2	28.2	27.9	27.4	27.2	26.7	25.3	23.8	27.0
Ext. Maximum	37.0	38.4	40.2	41.8	40.2	38.2	36.2	37.8	34.6	34.8	35.8	34.9	41.8
Ext. Minimum	7.6	11.7	13.5	15.9	19.9	21.2	20.8	20.7	20.6	16.4	12.5	8.5	7.6

Relative Humidity (%)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean	64.0	62.0	61.0	65.0	74.0	79.0	79.0	82.0	82.0	77.0	71.0	67.0	72.0
Mean Maximum	87.5	85.5	83.0	84.4	90.6	92.8	92.9	93.9	94.3	90.6	88.1	87.7	89.3
Mean Minimum	42.8	41.4	42.0	46.4	56.1	63.0	64.5	66.9	67.5	61.5	53.8	46.9	54.4

Climatological Data (Ubon Ratchathani) for the Period 1951 - 1975

Evaporation (mm)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean -- Piché	120.5	119.8	139.9	124.8	99.4	79.1	81.5	71.8	59.7	84.8	102.1	117.1	1200.5
Mean -- Pan	189.2	185.9	227.3	227.0	195.4	168.1	175.0	158.7	144.6	175.6	188.8	190.1	2225.7

Rainfall (mm)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean	0.8	10.0	49.7	73.2	212.3	240.4	291.3	313.8	266.9	96.9	20.4	1.8	1577.5
Mean rainy days	0.4	1.2	3.8	7.4	15.3	18.4	19.6	22.4	20.7	10.6	3.4	0.9	124.1
Daily Maximum	6.4	44.7	124.1	82.1	138.5	189.4	203.9	182.8	130.3	113.4	69.5	8.2	203.6
Day/Year	25/54	23/72	14/60	1/56	18/56	4/72	7/70	8/51	5/68	9/67	5/64	15/66	7/70

Remark: Evaporation 1. Piché 1954 - 1975
2. Pan 1961 - 1975

Table 4-4-1 Summary of Quantities for Huai Yang Dam

	Item	Unit	Left Bank	Riverbed	Right Bank	Total
Embankment	Jungle clearing	m^2	2,602.0	2,347.0	14,835.0	19,784.0
	Stripping	m^3	1,132.6	1,647.0	5,004.7	7,784.3
	Cutoff & Excavation	"	1,454.1	3,133.0	11,105.0	15,692.1
	Embankment	"	2,877.0	8,408.8	33,372.5	44,658.3
	Chimney drain & Outlet	"	51.5	1,026.7	4,631.9	5,710.1
	U/S Riprap	"	207.4	416.0	1,313.3	1,936.7
	U/S Sand and gravel blanket	"	210.2	389.1	1,249.0	1,848.3
	D/S Riprap	"	0	18.0	289.2	307.2
	D/S Sand and gravel blanket	"	4.7	53.4	311.6	369.7
	Filter sand	"	0	0	131.3	131.3
	Grass turf	"	489.9	829.7	2,733.4	4,053.0
	Excavation of Existing Embankment	"	0	0	6,499.5	6,499.5
Spillway	Jungle clearing	m^2	4,500.0			
	Excavation	m^3	73,764.6			
	Back filling	"	609.4			
	Plain concrete	"	1,450.446			
	Reinforced concrete	"	71.402			
Emergency Spillways	Jungle clearing	m^2	15,030.0			
	Excavation	m^3	18,406.3			
	Embankment	"	22.8			

Table 4-4-2 Construction Schedule (Huai Yang Dam)

Work Item		Q.T.Y.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.
Engineering Service Detailed Design			▬	▬							
Preparatory Works				▬	▬						
Dam Body	Jungle Clearing	19,800m²			▬						
	Stripping	11,800m³			▬						
	Excavation	15,700m³			▬	▬					
	Embankment	44,700m³				▬	▬	▬	▬		
	Drain and Outlet	5,700m³				▬	▬	▬			
	Riprap	1,900m³						▬	▬	▬	
	Grass turf	4,000m²							▬	▬	▬
Spillway	Earth Works	74,400m³				▬			▬	▬	▬
	Concrete Works	1,500m³				▬	▬	▬	▬	▬	
	Masonry	1,200m²							▬	▬	
	Water Stop	300m²					▬	▬	▬		
Intake	Earth Works	600m³			▬	▬					▬
	Concrete Works	50m³				▬					
	Masonry	22m²				▬	▬				
	Steel Gate	1									▬

Fig. 4-3-1 LOCATION MAP OF SOIL AND GEOLOGICAL INVESTIGATION FOR HUAI YANG DAM

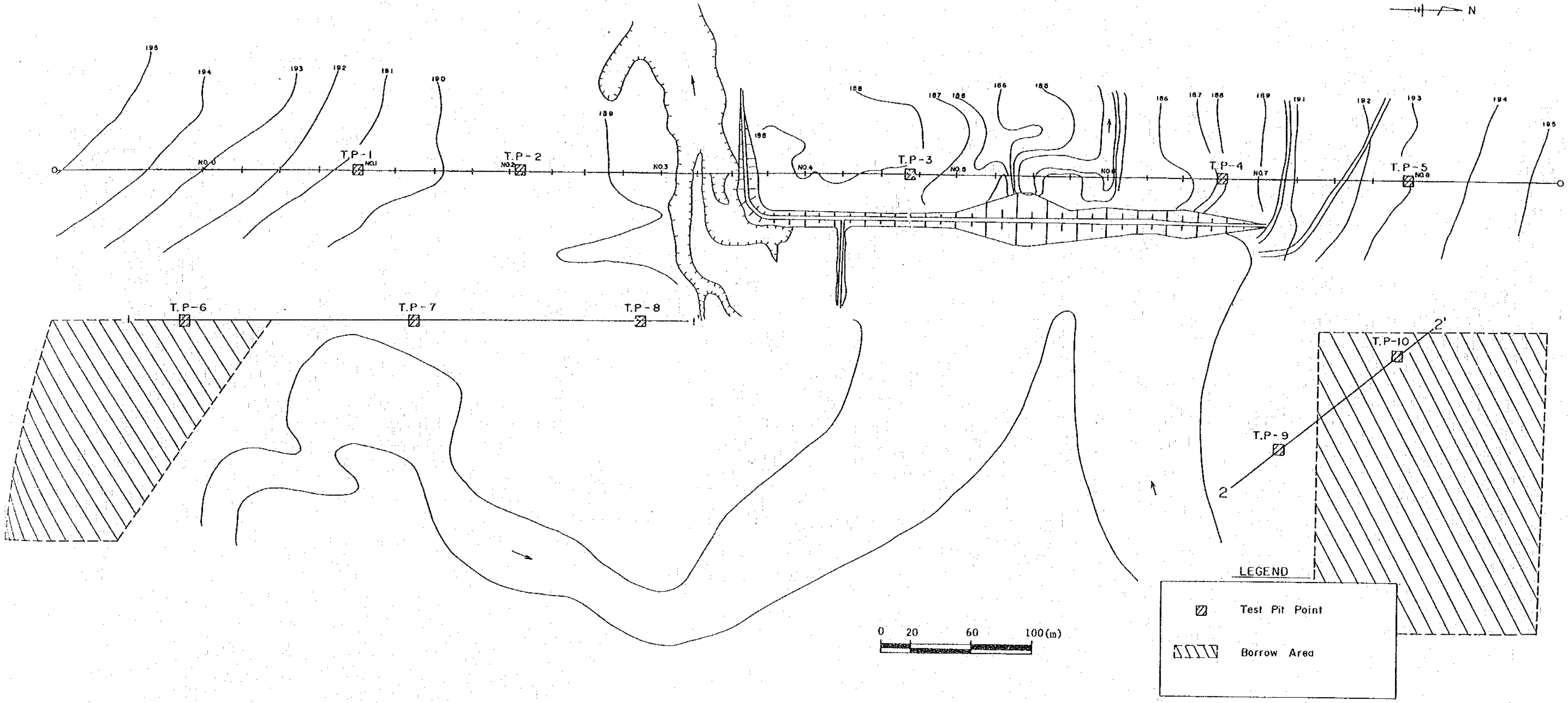


Fig. 4-3-2 Geological Section of Huai Yang Dam Axis

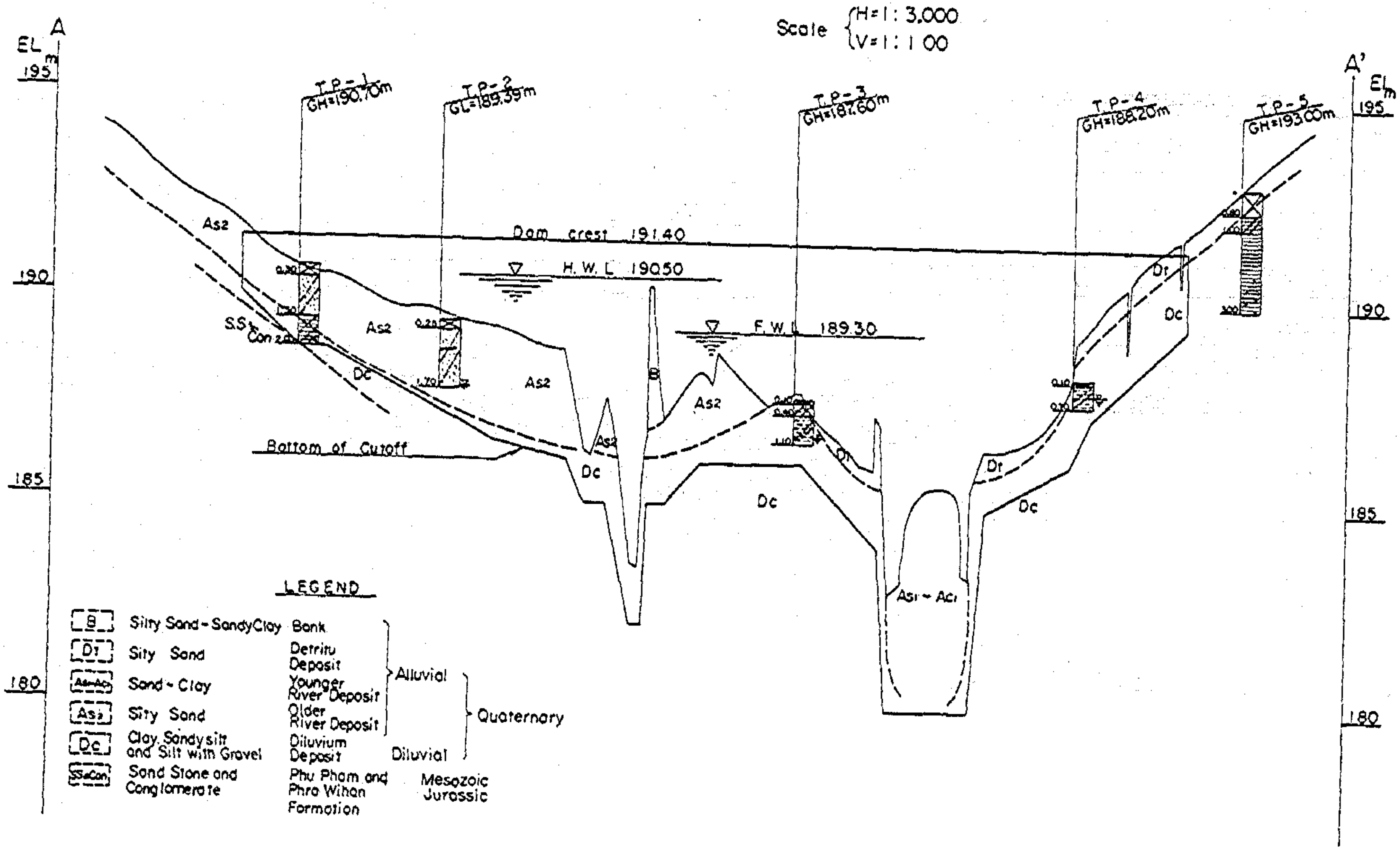


Fig. 4-3-3 Typical Cross Section

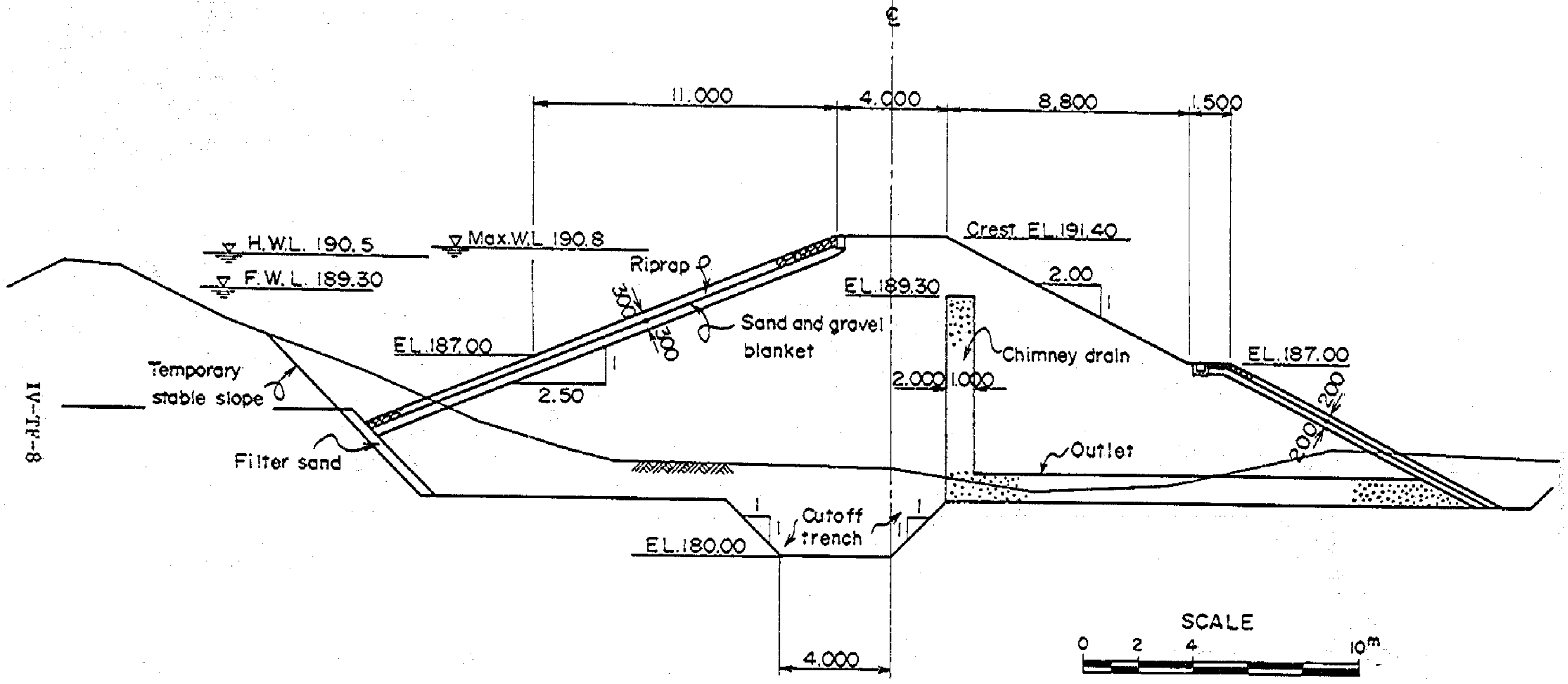


Fig. 4-3-4 Filter Gradation

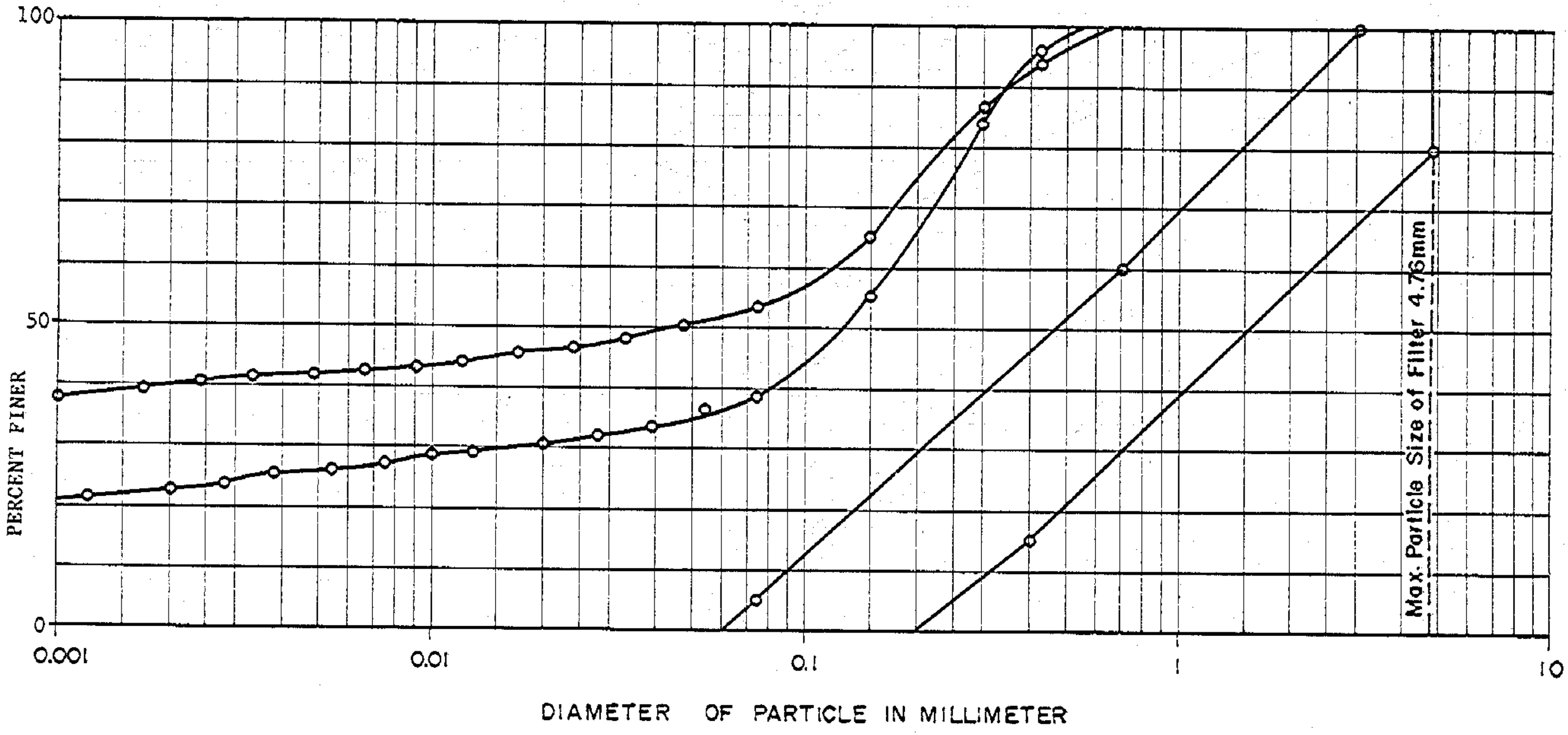


Fig. 4-3-5 Flow Net

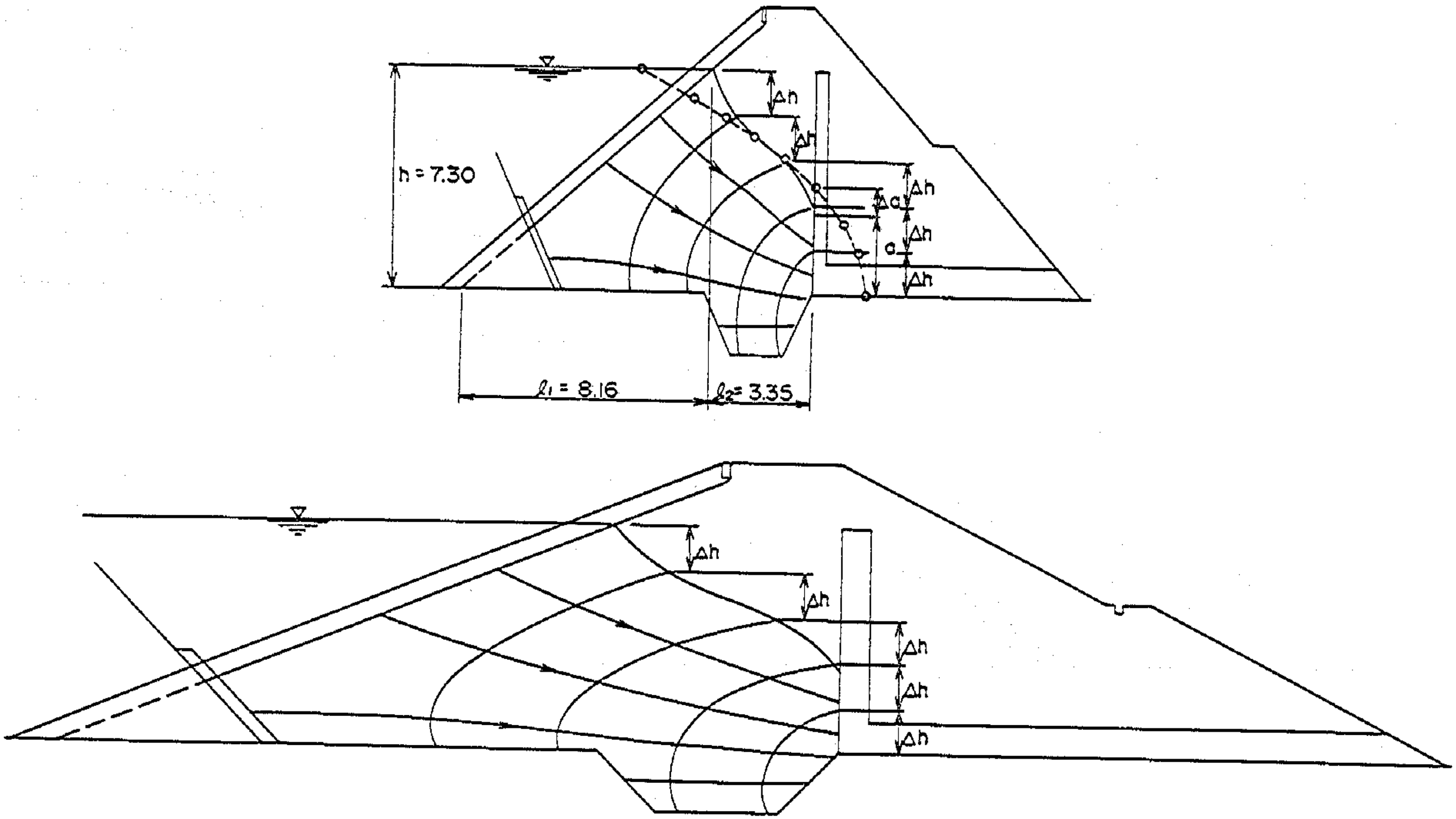


Fig. 4-3-6 Compaction Test

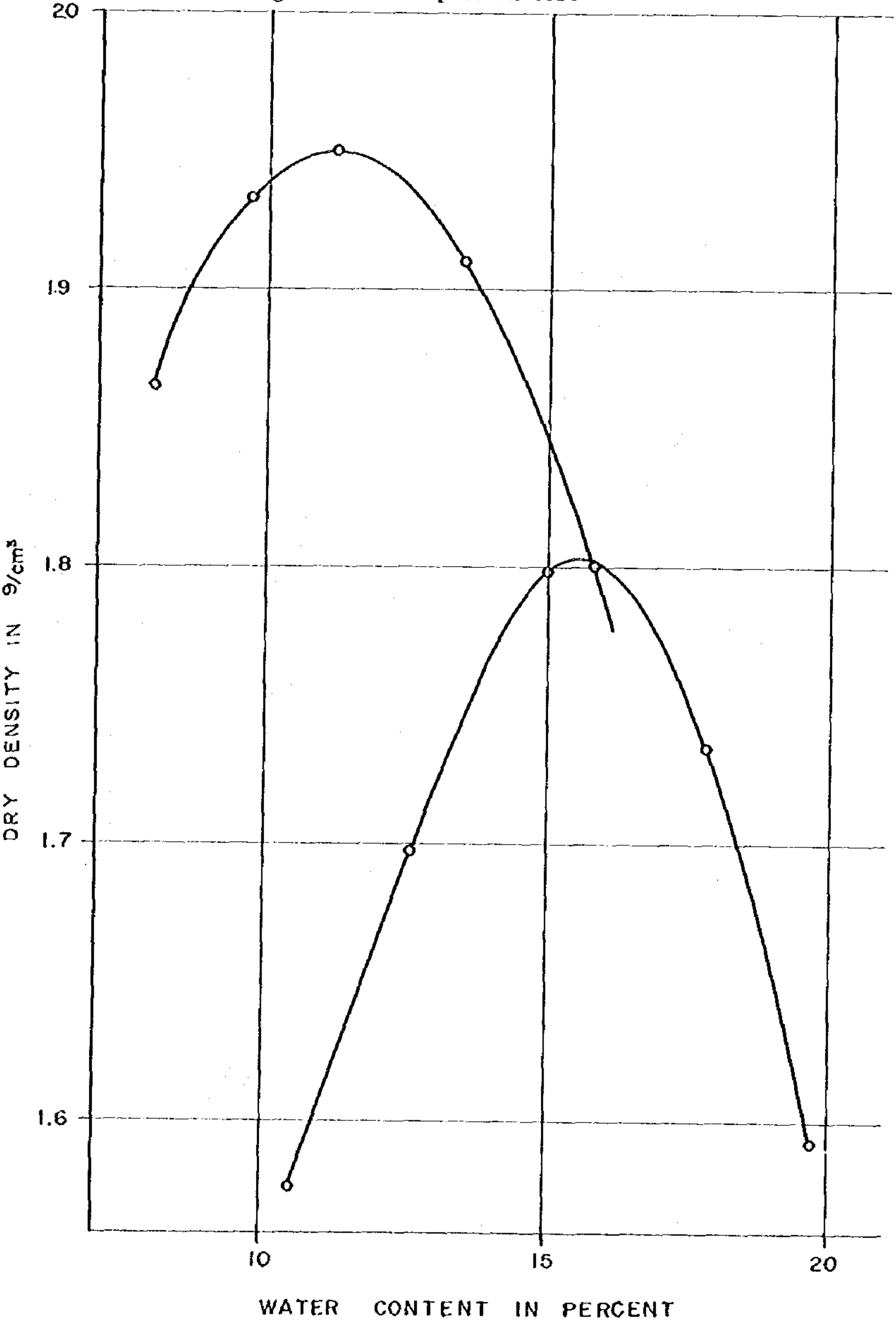


Fig. 4-3-7 Direct Shear Test

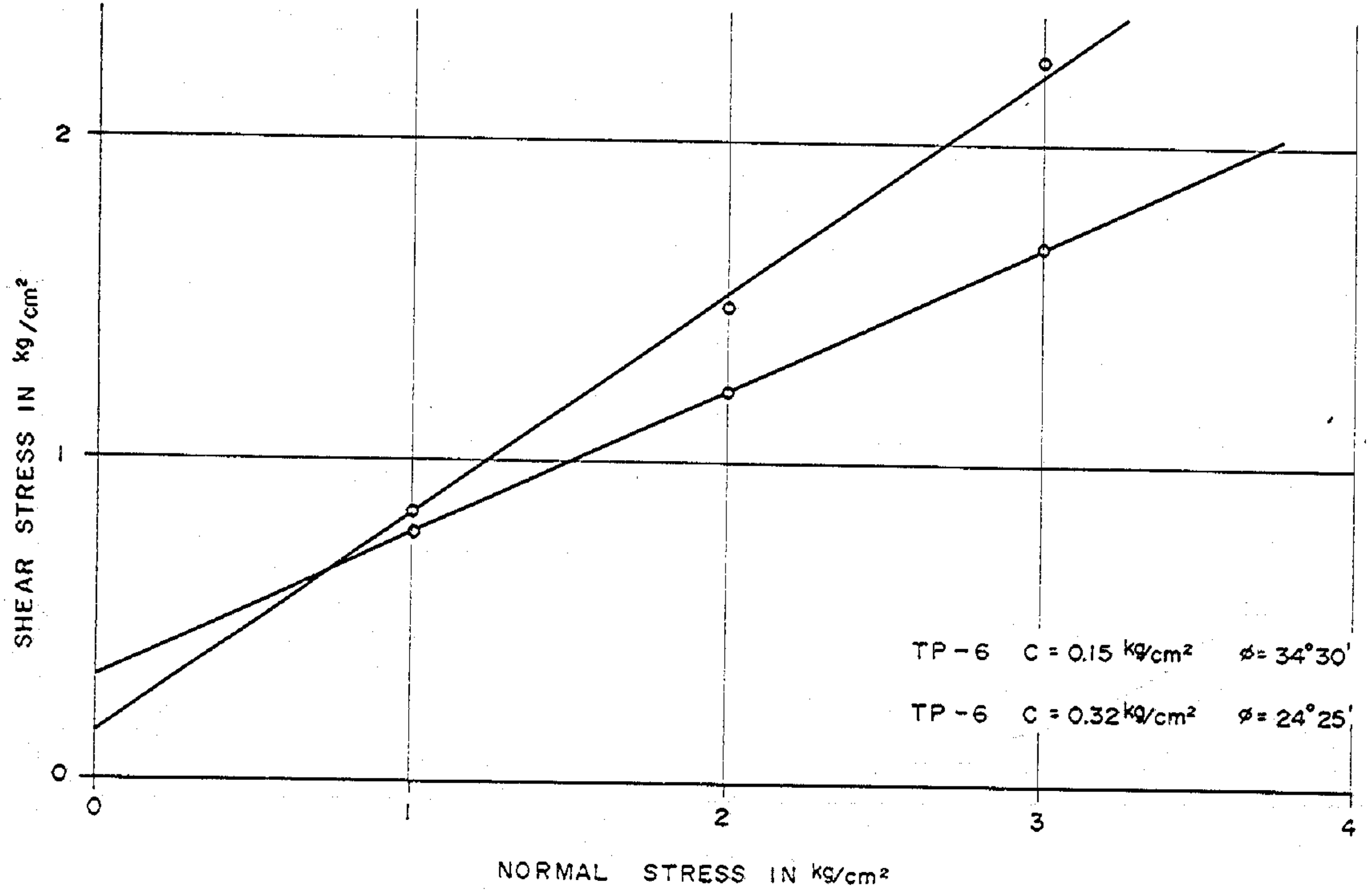


Fig. 4-3-8 Construction Pore Pressure

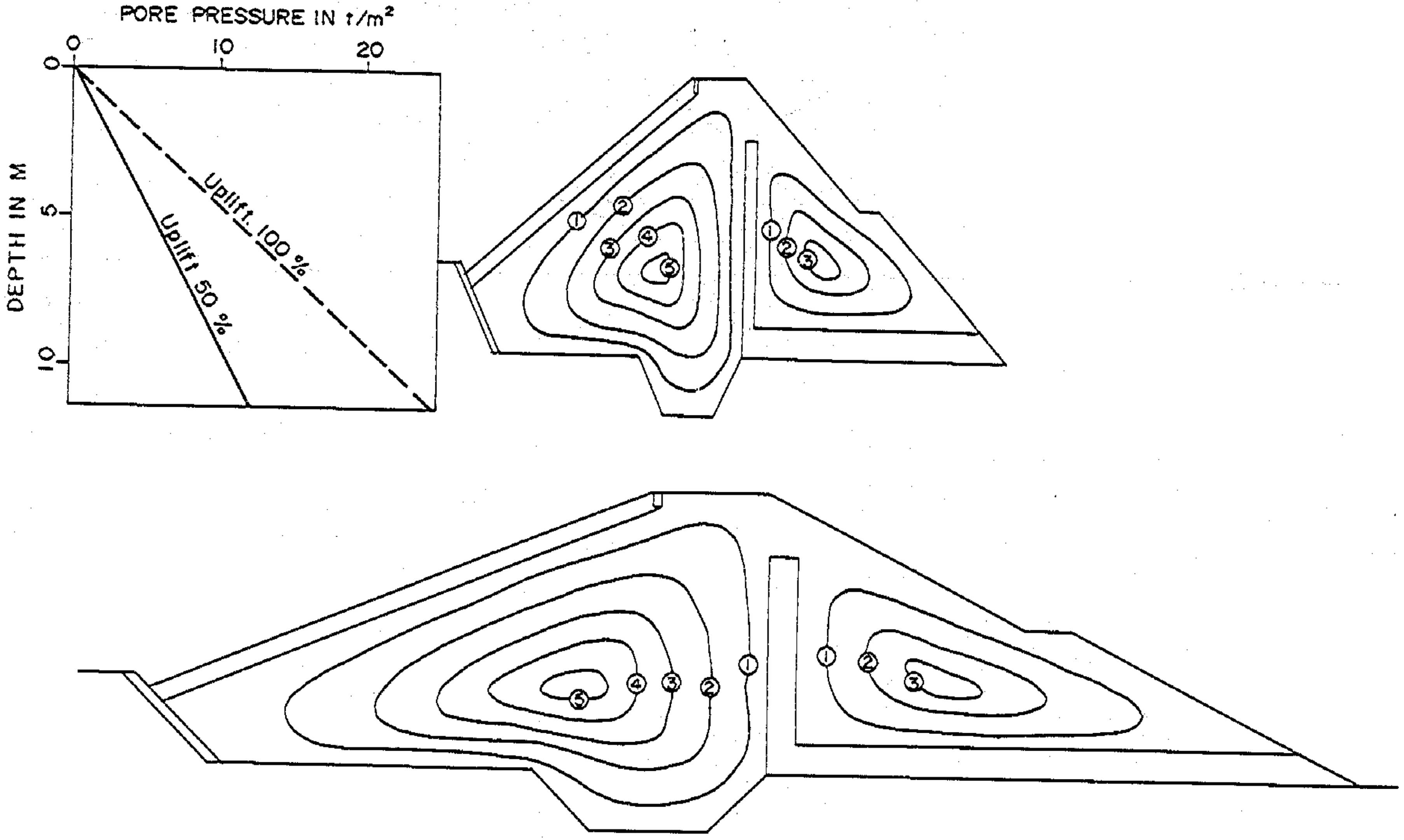


Fig. 4-3-9 Slope Stability Analysis for Fill Completed Condition

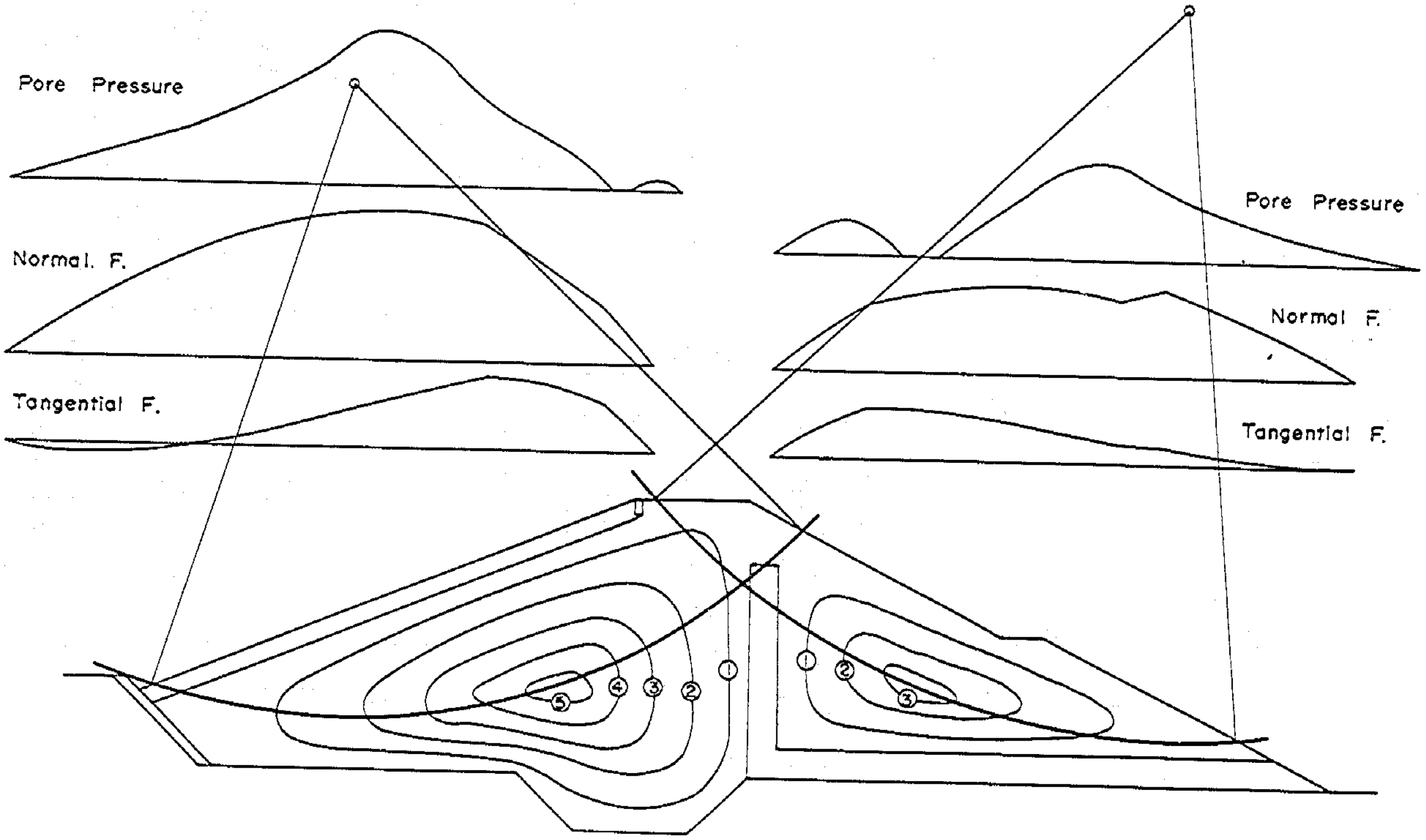


Fig. 4-3-10a Slope Stability Analysis for
Full Reservoir Condition Upstream Face

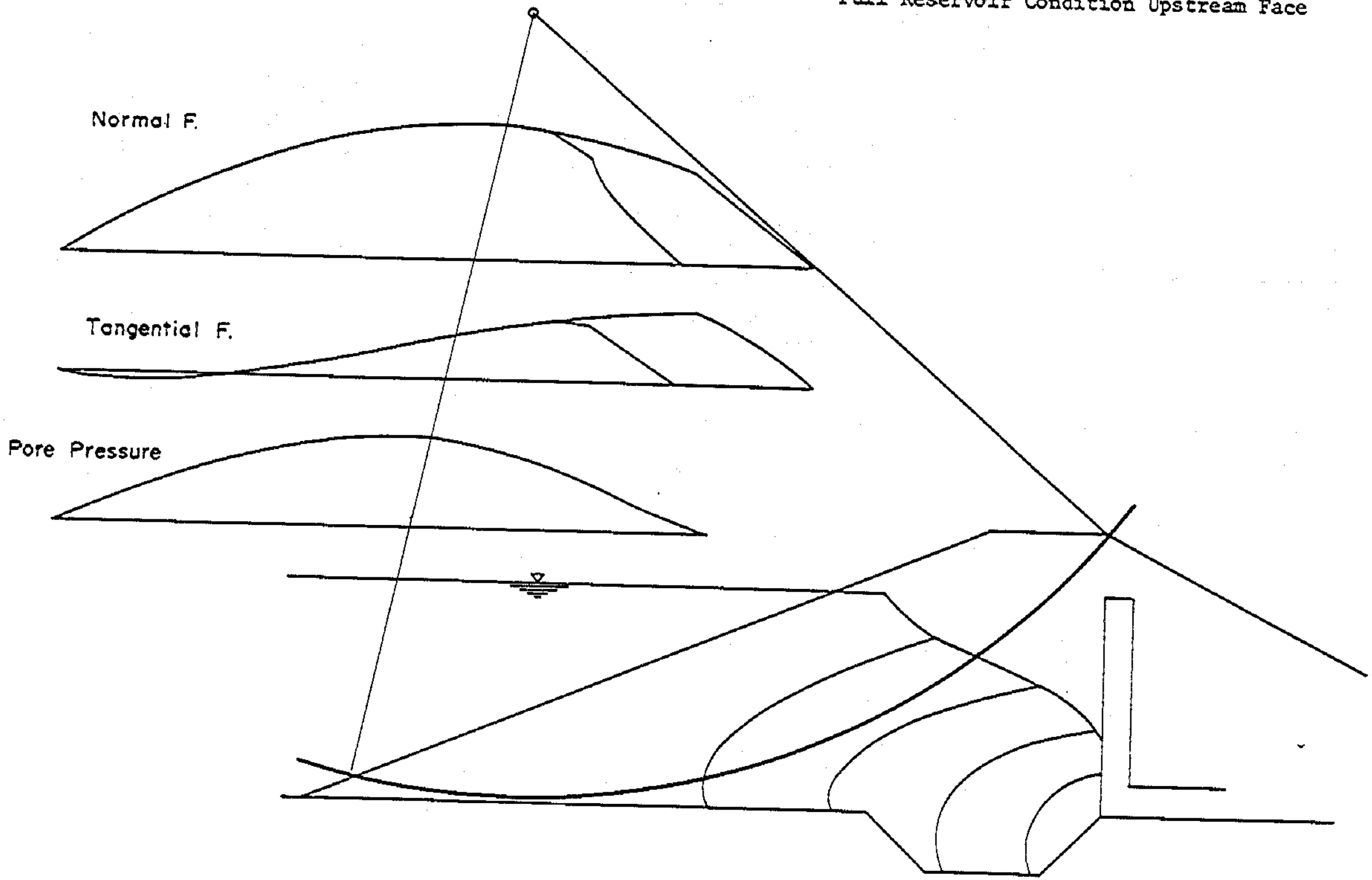


Fig. 4-3-10b Slope Stability Analysis for Full Reservoir Condition Downstream Face

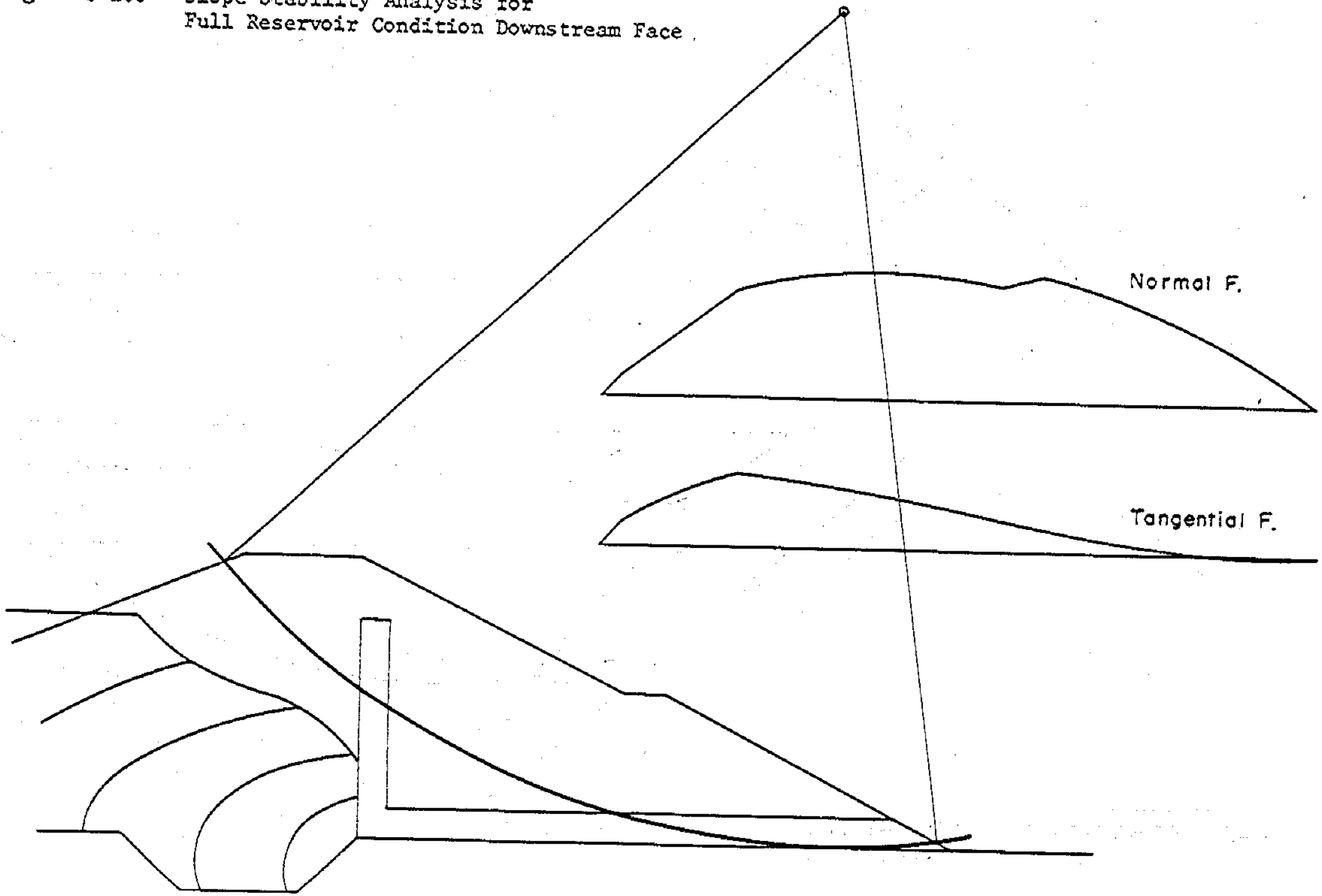


Fig. 4-3-11 Stress-Strain Curve

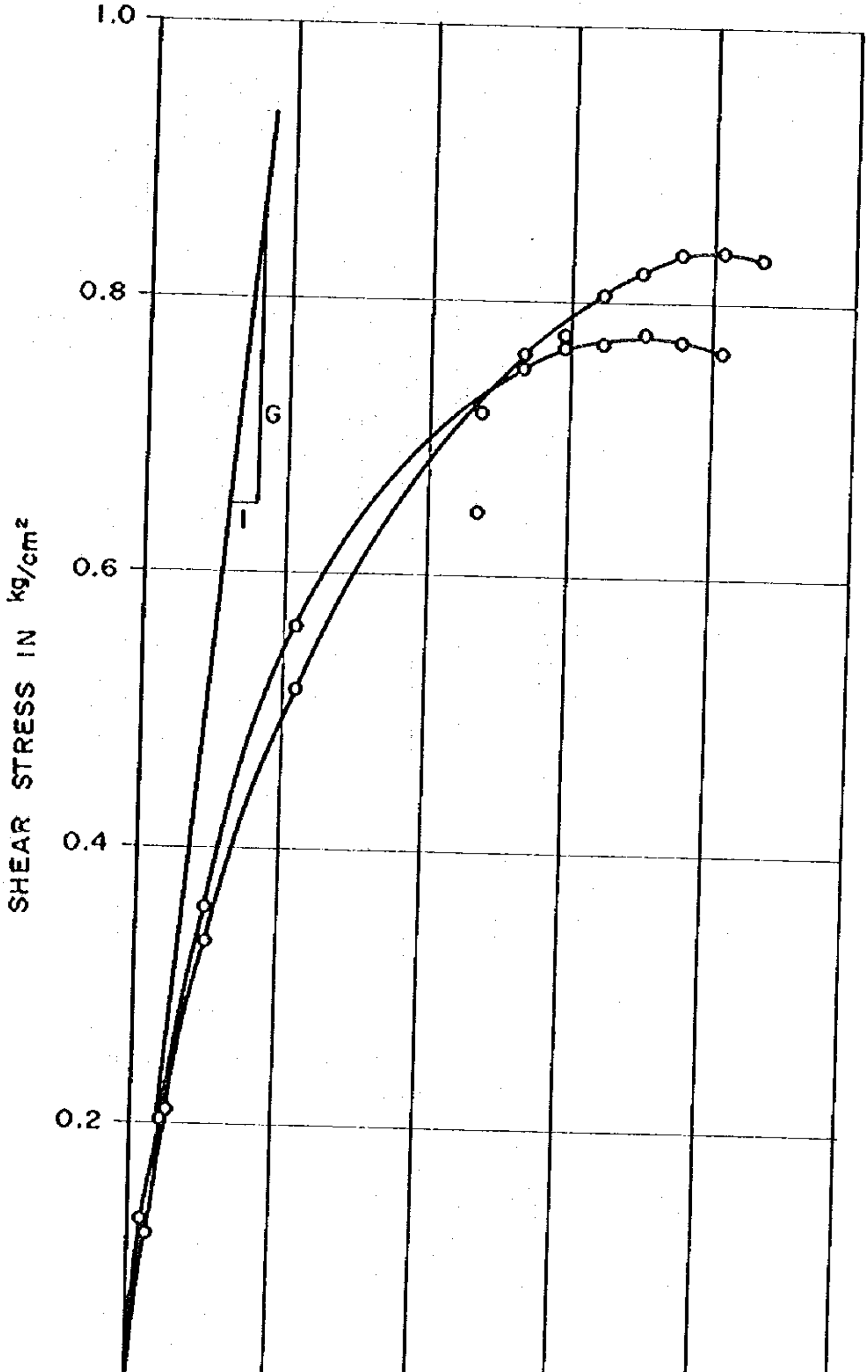
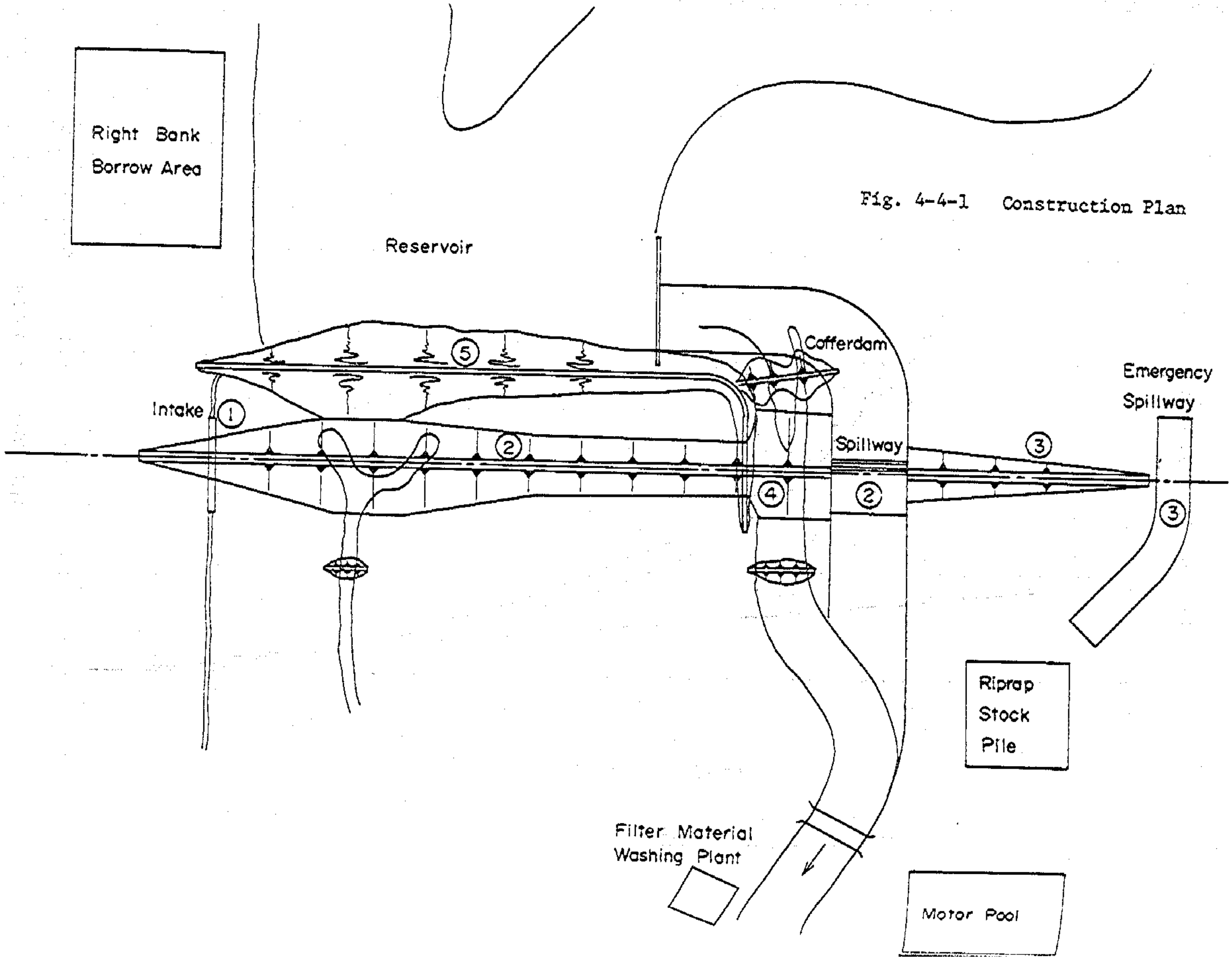


Fig. 4-4-1 Construction Plan



CHAPTER 5 IMPROVEMENT PLAN OF EXISTING WELLS

5.1 Present Conditions of Ubon Refugee Camp

5.1.1 Location and Geological Features of the Refugee Camp.

The refugee camp, established at the site of a former U.S. military camp, is located about 5.5 km to the north of the city of Ubon Ratchathani.

Huai Chara Mae, a tributary to Lam Nam Mun, flows in the vicinity passing the point of about 1.0 km to the west of the refugee camp.

The location map of the refugee camp is shown in Fig. 5-1-1.

Even though there are four existing deep wells in the refugee camp, exact geological information of the area cannot be obtained for the time being, since the drilling data of them were not available in any record.

But the following assumption should be dependably close to the reality.

The Ubon refugee camp rests on the surface of the medium high terrace formed on the left valley of Lam Nam Mun, the elevation of the terrace being 120 m - 124 m.

This river terrace is formed by gravel, sand, clay and silt which were collected here by floods.

The estimated depth of such deposit is about 10 m.

That part of the terrace deposit consisted of gravel and sand, being unconsolidate, has sufficiently high permeability and accordingly good water collecting potential for making a ground-water aquifer.

Immediately underneath the terrace deposit are bed rocks.

Since layers of rock salt are present among the bed rocks, the fissure water seeping into the deep wells drilled passing them may have salinity.

The depth of the Shallow wells currently in use in the refugee camp varies from 7 m to 10 m.

The water stored in these wells is the ground water suspended in the gravelly and sandy zones of the terrace deposit which found its way into them seeping through their walls and bottoms.

This type of shallow well is known as seepage spring.

The topography of the area of the camp and its vicinity being chiefly characterised by flatness, it is quite certain that the aquifers formed there directly owe to the rainfall of the area seeping fairly in a downward manner, which, therefore, means that the water table can go quite low when the supply lessens in the dry season.

It should be pointed out, in this connection, that the Underground flows of creeks and swamps located near the refugee camp suspected not to be in such a state as catering to the aquifers of the terrace deposit.

A hydrogeological map of Ubon is shown in Fig. 5-1-2.

5.1.2 Water Supplying Conditions

What is called Ubon Refugee Camp consists of "CAMP" and "HOLDING CENTER".

The "Camp" is occupied by those who have received identification cards from the Government of Thailand and the "Holding Center" by those who have not.

Those in the "Holding Center" may be transferred to the "Camp" when identified by the Government.

The total population of the refugee camp at present is 18,262 of which 17,962 is in the "Camp" and 300 in The "Holding Center". The population is on the decrease. The refugee population in the past is as shown in following Table.

Year	Population as of January
1975	1,000
1976	3,000
1977	5,222
1978	10,083
1979	38,408
1980	34,371
1981	19,070

Some of the shallow wells in the regugee camp are public and others private, the former having been constructed by MOI and the latter by some co-investing refugee families.

The number of public wells is 163 and the number of private wells 37.

The four deep wells from which salty water is pumped contributes to miscellaneous purposes except for drinking.

The shallow wells well satisfy the need of water with abundant fountain in the rainy season, while falling short of the duty in the dry season, some even being too dry to lift and others barely supplying little water.

The deep wells servicing on a grudging basis, are operated for about 10 minutes in the morning and another 10 in the afternoon rendering a very small lifted volume.

5.1.3 Water Supply Method

The water from the shallow wells at present is lifted in pails by hand alone or with pulleys.

Some of the private wells, however, are equipped with engine-driven pumps.

People carry the containers of water from the well to the plase of usage in the hand or on the shoulder.

5.2 Well Improvement Plan

5.2.1 Outline of the Survey on the Wells

A birds-eye view of the refugee camp was prepared, and then wells were plotted in it with different series of numbers for different usages of water.

Making use of this well location map, field interviews were conducted for taking note of volume of water used, number of users, type of usage, etc. for each well.

36 of the wells thus surveyed were selected for a further research as to their forms, water level and water quality analysis.

The well location map is shown in Fig. 5-2-1 and the survey data in Table 5-2-1.

The total number of the shallow wells is 163, of which 98 serve for drinking water.

The dimensions of them vary from 3.5 m to 10.5 m in depth, and from 0.8 to 1.1 m in diameter, but most of the wells measure 1.2 m in diameter.

The lining of the majority of the wells is made with concrete pipes of 1.2 m in diameter (see Fig. 5-2-2 for what the well looks like).

The water of some of the wells is deteriorated in quality owing to the inflow of miscellaneous drainage water through mismatched joints of the concrete base surrounding the well body on the ground.

And all the wells are without covers, foreign matters dropping into them any moment.

Each well is utilized by 20 to 60 families, and they boil the water before using it for drinking.

3 wells equipped with water storage tanks were selected for pumping test, which is to minimize the possible loss of the valuable resource even in the test. Proper volume of water to be lifted was arrived at on the basis of the findings from the test.

5.2.2 Classification of Drinking Water and Miscellaneous Water

Since the data such as PH values, residual chlorine, electric conductivity specific resistence, etc. obtained by the water quality analysis were not sufficient for judging if fit to drink or not, so additional evidence was collected by interview with refugees before arriving at the conclusion.

The classification of the well water by recommendable usages is shown in Fig. 5-2-1.

For reference, standards of water quality from the Ministry of Public Health of Thailand, WHO, and Japan are shown in Table 5-2-2, 5-2-3, 5-2-4 and 5-2-5.

5.2.3 Plan for Improvement of Wells

What should be taken into consideration in planning the improvement of wells include the volume of water to be supplied the volume of water to be lifted, the cost of construction works for water processing units, and the cost of management and maintenance.

The volume of Water to be supplied was determined on the basis of the present refugee population. The unit of water to be supplied shall be the value currently in general use as a minimum supply for refugee camps in Thailand.

Water supply unit per person per day ≒ 20 ℓ

Since the present refugee population is 18,262, the volume of water to be supplied shall be as follows.

Volume of water supplied = 18,262 persons x 20 ℓ/person/day
= 365.24 m/day = 366 m/day

Next, the volume of water to be lifted shall be considered.

The data of water level recovery in the earlier mentioned water pumping test at the 3 selected wells are shown in Table 5-2-6.

The amount of the recovered water, here, is assumed as equal to the volume of water to be lifted. The volume of water to be lifted from the tested wells is as shown in following Table.

Well	H'=h't - h'o	t'	V	Q
No	(m)	(min)	(m^3)	(ℓ/min)
13 - A	0.38	60	0.361	6.02
27 - B	0.52	50	0.494	9.88
28 - C	0.895	40	0.851	21.3
			Average	12.4

Where, ht': Water level recovered from the time of stopping pumping

ho': Water level at the time of stopping pumping

t' : Time after stopping pumping until the recovery of water level

V : Volume of recovered water

Q : Volume of pumped water

Now, the rate of lifting water (by refugees) is estimated at 12.4ℓ/min.

Next, the water lifting rate shall be estimated from the volume of water actually used by refugees.

The number of refugee families using each well varies from 20 to 60, but the average is 30. Assuming the average number of family members is 17, the average number of individuals for a well is as follows,

The number of individuals = 30 families x 7 members/family
= 210 individuals

The volume of water used, assuming the unit as 20ℓ/person/day, is as follows.

Volume of water used for a well = 20ℓ/person/day x 210 = 4.2 m^3

Consequently, it is safe to assume that the volume of water used for a well is equal to the volume of lifted water.

On the basis of the above, the number of the wells to be improved shall be determined.

Assuming 366 m^3/day of water is supplied operating 5 hours a day, 2 hours in the morning and another 2 hours in the afternoon and 1 hour about noon, the volume of hourly supply will be as follows.

Volume of hourly supply = 366 m^3/day ÷ 5 hours = 73.2 m^3/hour

The number of wells to be improved arrived at by way of the volume of water lifted known from the pumping test is as follows.

The number of wells to be improved = 73.2 m^3/hour ÷ 12.4ℓ/min
= 99 wells

The number of wells to be improved arrived at from the volume of water used by refugees is as follows.

The number of wells to be improved = 366 m^3/day ÷ 4.2 m^3/day
= 88 wells

Even though it is ideal to lift as much water as the volume of water level recovery for the sake of maintaining a constantly high water level of the wells it does not seem to be a strict requirement in the light of the observed practice of water usage by refugees.

So, the concluded number of wells to be improved shall be the one based on the volume of water used by refugees.

Even though the arithmetically obtained number of wells to be improved is 88, it is proposed to improve the 98 wells now in service for drinking water taking into account the situation of the dry season.

The following considerations shall be held in drafting the drawings of the well improvement project.

a) A concrete base shall be provided for each well as a protection against the see page of water drained from laundering, bathing and so on.

b) A cover shall be provided for each well to prevent the entry of foreign matters.

c) A hand pump shall be installed at each well for convenience in lifting water

The hand pump to be selected shall be the one of good quality commonly in the market.

d) The proposed water processing is only sterilization.

Chemical for this purpose shall be directly dropped into the well, for which an inlet hole shall be made into each cover.

The plan of the improvement is shown in Fig. 5-2-3.

The sterilizing chemical shall be chloride of time dozed by 2ppm (mg/l) averagely.

For instance, in the case of a well supplying 4.2 m^3/day of water, 4.2 m^3/day x 2 mg/l = 8.4 g/day.

But if it is assumed that the effective portion of chloride of time is 30% of the total volume, 8.4 kg/day ÷ 0.3 = 28 g

Therefore, the total monthly dosage is, 28 g/day x 30 days/month = 840 g/month.

5.2.4 Computation of Construction Cost and Period of Construction

It is judged that the construction work can be executed by contractors in Ubon (Ratchathani), and the construction materials required can also be procured there.

The unit costs of the main materials and works for the project in the area were surveyed prior to the computation of the total estimated cost of the construction work.

The unit costs herein used are the values as of July, 1981, whose fluctuation afterwards are not taken into account.

In execution of the construction work, flexible approaches should be taken from well to well depending on the variation in location, form, depth and soon.

It should also be pointed out that repairing of the concrete bases and joints of the concrete pipes is an important requirement.

The overall construction cost is shown in following Table.

construction cost for well	number	unit	total construction cost (฿)
9,800	98	well	960,400

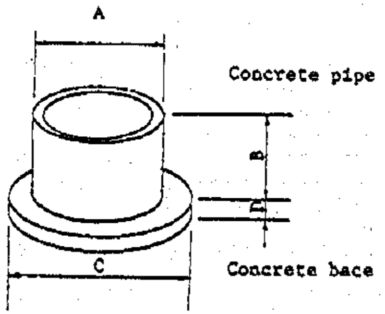
A representative breakdown of the cost for a well is shown in the Table 5-2-7.

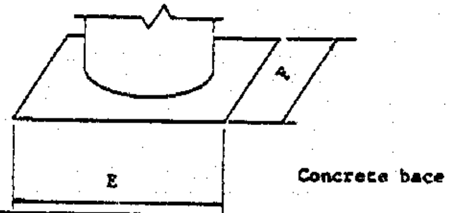
LIST OF TABLES AND FIGURES IN THE CHAPTER 5

TABLES

FIGURES

Table 5-2-1 Data Sheet of Existing Well





Well No.	Depth of well (m)	Water Level (m)	A (m)	B (m)	C (E x F) (m)	D (m)	View	pH	$C\ell_2$ (mg/ℓ)	E.C. (μS/cm) (25°C)	S.R. (KΩ·cm) (25°C)	Remark
①-A	8.5	6.0	1.2	0.58	1.92	-	Colorless	About 6	< 0.1	300.3	3.3	Two taps deep well are about 3 meters from this well.
③-A	6.5	5.0	1.2	0.47	2.64 x 2.7	-	Colorless	About 6	< 0.1	2457	0.407	
⑤-A	9.0	7.8	1.2	0.76	3.96	-	Colorless	About 6	< 0.1	1092	8.8	
⑥-A	9.0	8.0	1.2	0.7	2.8	0.2	Colorless	About 6	< 0.1	855.4	1.188	
⑥-B	9.0	8.5	1.2	0.92	4.0	0.25	Colorless	-	-	1183	0.825	
⑥-C	9.5	8.5	1.2	0.86	4.5	0.2	Colorless	About 6	< 0.1	1638	0.625	
⑦-A	6.5	5.1	1.2	1.2	3.6	0.2	Colorless	-	-	136.5	71.5	
⑧-A	8.5	5.5	1.2	1.0	2.64	-	A little light brown color	-	-	582.4	1.705	The lower well diameter is smaller than the upper.
⑧-F	6.5	5.2	1.2	0.7	3.6	-	Colorless	-	-	172.9	5.83	
⑩-A	7.0	5.5	1.2	0.36	4.4	-	Colorless	About 6	< 0.1	209.3	4.84	Sewage water stay about 2 meters from this well.
⑬-A	9.0	6.5	1.2	0.47	-	-	Colorless	About 6	< 0.1	309.4	3.19	
⑭-B	8.8	7.0	1.2	0.86	3.4	0.15	Colorless	About 6	< 0.1	1001	0.99	
⑭-A	10.5	9.0	1.0	0.87	2.5 x 2.9	-	-	-	-	-	-	This well is about 3 meters between houses.
⑮-B	7.0	6.0	1.2	0.68	3.26	-	A little light brown color	About 6	< 0.1	-	-	The lower well diameter is smaller than upper.

Data Sheet of Existing Well

Well No.	Depth of well (m)	Water Level (m)	A (m)	B (m)	C (E x F) (m)	D (m)	View	pH	Cl_2 (mg/ℓ)	E.C. (μS/cm) (25°C)	S.R. (KΩ·cm) (25°C)	Remark
⑮ -C	8.5	6.3	1.2	0.6	2.3	-	A little light brown color	About 6	< 0.1	391.3	2.53	Dry up in the dry season
⑯ -A	8.2	7.1	1.2	0.32	3.2	-	Colorless	About 6	< 0.1	373.1	2.695	Enough in the dry season
⑰ -A	7.5	6.0	0.8	0.93	2.3 x 2.6	-	Colorless	About 6	< 0.1	509.6	1.98	
⑱ -A	8.5	5.5	0.8	0.5	1.1 x 2.8	-	Colorless, Contain a little dust	About 6	< 0.1	409.5	2.42	This well is about 0.8 meters from house.
⑳ -A	7.5	5.0	1.2	0.56	3.72	-	Colorless	About 6	< 0.1	345.8	2.86	
⑳ -I	8.2	5.5	1.2	0.58	4.0	0.12	Colorless	About 6	< 0.1	700.7	1.43	Sewage water stay in the drain
㉒ -D	7.0	5.5	1.2	0.66	3.0	-	Colorless	-	-	700.7	1.43	
㉔ -A	7.1	5.8	1.2	0.76	3.6	-	Colorless	-	-	163.8	6.05	
㉕ -D	6.6	5.2	1.2	0.82	3.14	-	Colorless	-	-	600.6	1.65	
㉗ -B	6.0	4.39	1.2	0.67	2.8	-	A little light brown color	-	-	382.2	2.64	
㉗ -D	6.5	4.5	1.2	0.9	-	-	Colorless	-	-	3.64	2.75	
㉘ -A	5.5	3.5	1.2	0.9	2.4	-	A little light brown color Mosquito larvaes are found.	About 6	< 0.1	445.9	2.145	
㉘ -C	5.0	3.1	1.2	0.5	4.8	0.3	Colorless, Contain a little dust	-	-	409.5	2.475	
㉘ -I	4.5	3.5	1.2	0.7	3.2	0.3	Colorless	About 6	< 0.1	-	-	
㉙ -A	3.5	2.9	1.2	0.63	3.14	0.4	A little light brown color	-	-	263.9	3.85	The mud accumulate.
㉙ -B	5.1	2.8	1.2	0.7	-	-	Light brown color	-	-	609.7	1.65	
㉚ -A	4.5	3.5	1.2	0.6	3.2	-	Colorless	-	-	200.2	4.95	
㉚ -C	5.2	3.2	1.2	1.1	-	-	Colorless	-	-	354.9	2.86	
㉚ -D	5.0	3.1	1.2	0.9	-	-	Colorless	-	-	318.5	3.08	A duck pen is about 3 meters from this well.
㉜ -A	6.5	4.5	1.2	0.72	3.2	-	A little light brown color	-	-	127.4	7.7	
⑭ -E	7.1	5.1	1.2	0.96	-	-	A little light brown color	-	-	172.9	5.83	
⑮ -I	5.5	4.0	1.0	0.67	2.5	0.2	A little light brown color	About 6	< 0.1	846.3	1.54	

Table 5-2-2 Standard of Drinking Water

The Notification of the Ministry of Public Health No.20 (1979)

1. Physical Properties

Colour	not more than 20
Oder	no other order (not include chloring)
Turbidity	not more than 5
PH value	between 6.5 - 8.5

2. Chemical Properties

Total solids	not more than 1,000 mg/kg
Total hardness	not more than 300 mg/kg
Chloride (expressed as chlorine)	not more than 250 mg/kg
Fluoride (expressed as fluorine)	not more than 1.5 mg/kg
Albuminoid ammonia (expressed as ammonia)	not more than 0.1 mg/kg
Free ammonia (expressed as ammonia)	not more than 0.1 mg/kg
Nitrates (expressed as nitrogen)	not more than 4.0 mg/kg
Nitrite (expressed as nitrogen)	not more than 0.1 mg/kg
Iron	not more than 0.5 mg/kg
Lead	not more than 0.1 mg/kg
Arsenic	not more than 0.05 mg/kg

3. Bacterial Properties

Standard plate Count at 35 - 37°C, 24 hours, not exceeding 500 colonies per 1 mℓ.

Most Probable Number of Coliform Organism per 100 mℓ. (M.P.N.) less than 2.2

Free from E. coli type 1 (Escherichia coli)

Table 5-2-3 WHO Water Quality Standards

Substance		Concentration in parts per million (ie mg per litre)		
Total Solid		500		(1,500)
Color		5 Unit	I	(50 unit)
Turbidity		5 Unit	II	(25 unit)
Taste Unobjectionable				
Odour Unobjectionable				
Iron	(Fe)	0.3		(1.0)
Manganese	(Mn)	0.1		(0.5)
Copper	(Cu)	2.0		(1.5)
Zinc	(Zn)	5.0		(15.0)
Calcium	(Ca)	75.0		(200.0)
Magnesium	(Mg)	50.0		(150.0)
Sulphate	(SO4)	200		(400)
Chloride	(Cl)	200		(600)
Magnesium + Sodium Sulphate		500		(1,000)
Phenolic Substance (Such as Phenol)		0.001		(0.002)
PH		7.0 - 8.5		(6.5 - 9.2)
Fluoride		1.0		(1.5)
Nitrates (as NO3)		50		
Oxygen absorbed from permanganate		2		
Albuminoid ammonia		0.1		
Free and saline ammonia		0.05		
Nitrites		a trace		
Lead	(Pb)	0.1		
Selenium	(Se)	0.05		
Arsenic	(As)	0.2		
Chromium	(Cr hexavalent)	0.05		
Cyanide	(C)	0.01		
Coliform group bacteria		Less than 10 ppm Through out a year		

( ) - Excessive.

I - Platinum cobalt scale

II - Turbiduty Units

Table 5-2-4 Japanese Water Quality Standards

Not to be affected by any pathogenic organism nor to contain any organism or substance which gives ground for suspicion of being affected by pathognic organism	Nitrite nitrogin and Nitrate nitrogen	Max. 10 mg/ℓ
	Chloride ion	Max. 200 mg/ℓ
	Organic substances	Max. 10 mg/ℓ
	Total colonies	Max. 100
	(as potassium permanganate consumption)	(colony counts per mℓ)
	Coliform group	Not to be detected
Not to contain cyanide, mercury and other poisonous substances	Cyanide ion	Not to be detected
	Mercury	Not to be detected
	Organic phosphate	Not to be detected
Not to contain copper, iron, fluorine, phenols and other substances in excess of their allowable quantities.	Copper	Max. 1.0 mg/ℓ
	Iron	Max. 0.3 mg/ℓ
	Manganese	Max. 0.3 mg/ℓ
	Zinc	Max. 1.0 mg/ℓ
	Lead	Max. 0.1 mg/ℓ
	Chromium (haxavalent)	Max. 0.05 mg/ℓ
	Cadmium	Max. 0.01 mg/ℓ
	Arsenic	Max. 0.05 mg/ℓ
	Fluoride	Max. 0.8 mg/ℓ
	Calium, Magnesium (hardness)	Max. 300 mg/ℓ
	Total resudue	Max. 500 mg/ℓ
	Phenols	Max. 0.005 mg/ℓ
	Surface-active agents (anionic)	Max. 0.5 mg/ℓ
Not to assume abnormal acidity or alkality	PH	From Max. 8.6 to Min. 5.8 as PH val.
Not to give an offensive smell except the smell caused by sterillization	Odor	Not to be abnormal
	Taste	Not to be abnormal
To be almost colorless and transparent in appearance.	Color	Max. 5 degree
	Turbidity	Max. 2 degree

Remarks:

In addition to these water quality standards, the Ministry of Health & welfare takes such administrative measures as follows:

(1) As for mangnese, there are some instances where increase in color and blace suspended matters due to manganese were observed. Manganese removeal equipment shall there fore be provided for water susceptible to the influence of manganese with a view to reducing the manganese content 0.05 ppm or less.

(2) Cadmium content shall not exceed 0.01 ppm as a provisional standard.

(3) Atomic absorption spectorophotometry shall be used for the inspection of mercury content. Mercury content shall not exceed 0.001 ppm on a total mercury basis.

Table 5-2-5 Environmental Standard Concerning the Protection of Human Helth

Item	Standard content
Cadmium	0.001 ppm or less.
Cyanogen	Shall not be detected.
Organophosphoric compounds	Shall not be detected.
Lead	0.1 ppm or less.
Chromium (VI)	0.05 ppm or less.
Arsenic	0.05 ppm or less.
Total mercury	0.0005 ppm or less.
Alkyl mercury	Shall not be detected.
PCB	Shall not be detected.

Remarks:

1. The standard content shall be the maximum value.
   However, the standard content for total mercury shall be a mean value throughout a year.

2. The term "organo-phosphoric compounds" means parathion, methylparathion, methyldimeton and EPN.

3. Only in the case where river water is apparently contaminated by mercury due to any natural cause, the standard content for total mercury shall be 0.001 ppm or less.

4. The expression "Shall not be detected" means that the content detected by the predetermined measuring method is below the threshold value of detection.
   (The threshold value of detection is 0.1 ppm for cyanogen, 0.1 ppm for organo-phosphoric compounds. 0.0005 ppm for alkyl mercury and 0.0005 ppm for PCB, respectively).

Table 5-2-6

⑬ -A

Well diameter : 1.2 m (internal diameter: 1.1 m)
Depth of well : 9.0 m
Water level (H): 6.5 m
Time past from start of pumping (t): 20 min.
Volume of pumping : 1.6 m^3

Time t' (min)	Water Level h' (m)	H - h' = s' (m)	t/t'	Time t' (min)	Water Level h' (m)	H - h' = s' (m)	t/t'
0	8.18	1.68		32	7.94	1.44	0.625
2	8.16	1.66	10	34	7.93	1.43	0.588
4	8.14	1.64	5	36	7.925	1.425	0.556
6	8.12	1.62	3.33	38	7.92	1.42	0.526
8	8.11	1.61	2.5	40	7.91	1.41	0.5
10	8.09	1.59	2	42	7.9	1.4	0.476
12	8.075	1.575	1.67	44	7.89	1.39	0.455
14	8.06	1.56	1.43	46	7.88	1.38	0.435
16	8.045	1.545	1.25	48	7.865	1.365	0.42
18	8.03	1.53	1.11	50	7.855	1.355	0.4
20	8.015	1.515	1	52	7.842	1.342	0.385
22	8.0	1.5	0.909	54	7.835	1.335	0.37
24	7.985	1.485	0.833	56	7.82	1.32	0.357
26	7.975	1.475	0.769	58	7.81	1.31	0.345
28	7.97	1.47	0.714	60	7.80	1.3	0.333
30	7.95	1.45	0.667				

㉗ -B

Well diameter : 1.2 m (internal diameter: 1.1 m)

Depth of well : 6.0 m

Water level (H): 4.2 m

Time past from start of pumping (t): 12 min.

Volume of pumping : 1.6 m^3

Time t' (min)	Water Level h' (m)	H - h' = s' (m)	t/t'	Time t' (min)	Water Level h' (m)	H - h' = s' (m)	t/t'
0	5.79	1.59		32	5.435	1.235	0.375
2	5.775	1.555	6	34	5.415	1.215	0.353
4	5.73	1.53	3	36	5.40	1.20	0.333
6	5.70	1.5	2	38	5.38	1.18	0.316
8	5.68	1.48	1.5	40	5.36	1.16	0.3
10	5.665	1.465	1.2	42	5.345	1.145	0.286
12	5.64	1.44	1	44	5.325	1.125	0.273
14	5.615	1.415	0.857	46	5.31	1.11	0.261
16	5.60	1.4	0.75	48	5.29	1.09	0.25
18	5.57	1.37	0.667	50	5.27	1.07	0.24
20	5.55	1.35	0.6	52			
22	5.53	1.33	0.545	54			
24	5.515	1.315	0.5	56			
26	5.49	1.29	0.462	58			
28	5.475	1.275	0.429	60			
30	5.455	1.255	0.4				

㉘ -C

Well diameter : 1.2 m (internal diameter: 1.1 m)
Depth of well : 5.0 m
Water level (H): 3.1 m
Time past from start of pumping (t): 11 min
Volume of pumping : 1.6 m

Time t' (min)	Water Level h' (m)	H - h' = s' (m)	t/t'	Time t' (min)	Water Level h' (m)	H - h' = s' (m)	t/t'
0	4.8	1.7		32	4.04	0.94	0.344
2	4.7	1.6	5.5	34	4.0	0.9	0.324
4	4.65	1.55	2.75	36	3.97	0.87	0.306
6	4.6	1.5	1.83	38	3.938	0.838	0.289
8	4.545	1.445	1.375	40	3.905	0.805	0.275
10	4.505	1.405	1.1	42			
12	4.45	1.35	0.917	44			
14	4.405	1.305	0.786	46			
16	4.365	1.265	0.688	48			
18	4.32	1.22	0.611	50			
20	4.275	1.175	0.55	52			
22	4.235	1.135	0.5	54			
24	4.195	1.095	0.458	56			
26	4.15	1.05	0.423	58			
28	4.11	1.01	0.393	60			
30	4.08	0.98	0.367				

Table 5-2-7 Cost Estimate

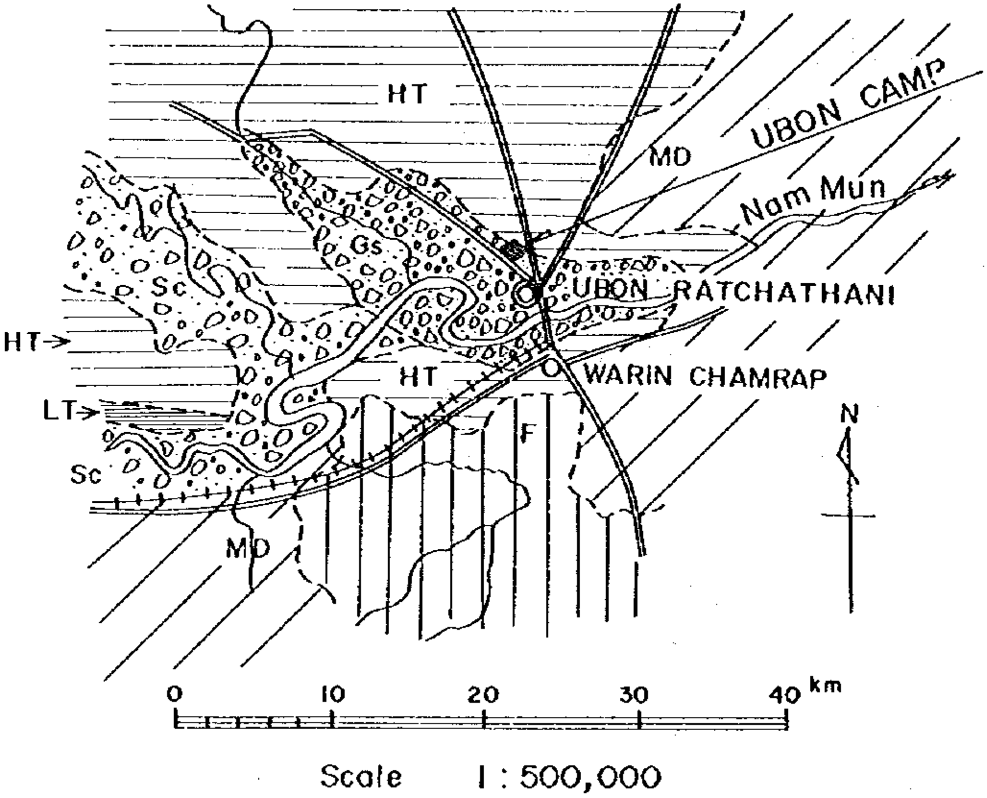
Description	Unit	Quantities	Rate ฿	Total Amount ฿
Item 1 Well foundation				
Excavation	m^3	4.7	25	118
Cobble stones	m^3	2.7	300	810
Concrete	m^3	2.6	1,400	3,640
Reinforcement	kg	80.7	11	888
Forms	m^2	7.5	150	1,125
Other Works	sum	1		179
Sub total				6,760
Item 2 Concrete Cover				
Concrete	m^3	0.06	1,400	84
Reinforcement	kg	1.9	11	21
Forms	m^2	0.8	150	120
Other Works	sum	1		25
Sub total				250
Item 3 Pump Works				
Hand pump	sum	1		1,110
Laber	sum	1		340
Gravel	m^3	0.2	300	60
Sub total				1,510
Item 4 Overhead Exprnses				
(Item 1 - 3) x 15%	sum	1		1,280
Total				9,800

Fig. 5-1-1 Location Map of Ubon Camp

Fig. 5-1-2 Hydrogeological Map of Ubon

By CHAROEN PHIANCHAROEN

DEPARTMENT OF MINERAL RESOURCES
MINISTRY OF INDUSTRY, THAILAND 1973



Gs	Unconsolidated aquifer(alluvium, Holocene) Gravelly or sandy deposits.
Sc	Unconsolidated aquifer (alluvium, Holocene) Sandy and clayey deposits within flat bedrock terraces.
HT	Medium high terraces.
LT	Low terrace
MD	Mound and depression-type topography with remnances of erosional surface.
F	Flat land or poddy field, flooded in rainy season.

Fig. 5-2-1 Location Map of Shallow Well and Deep Well

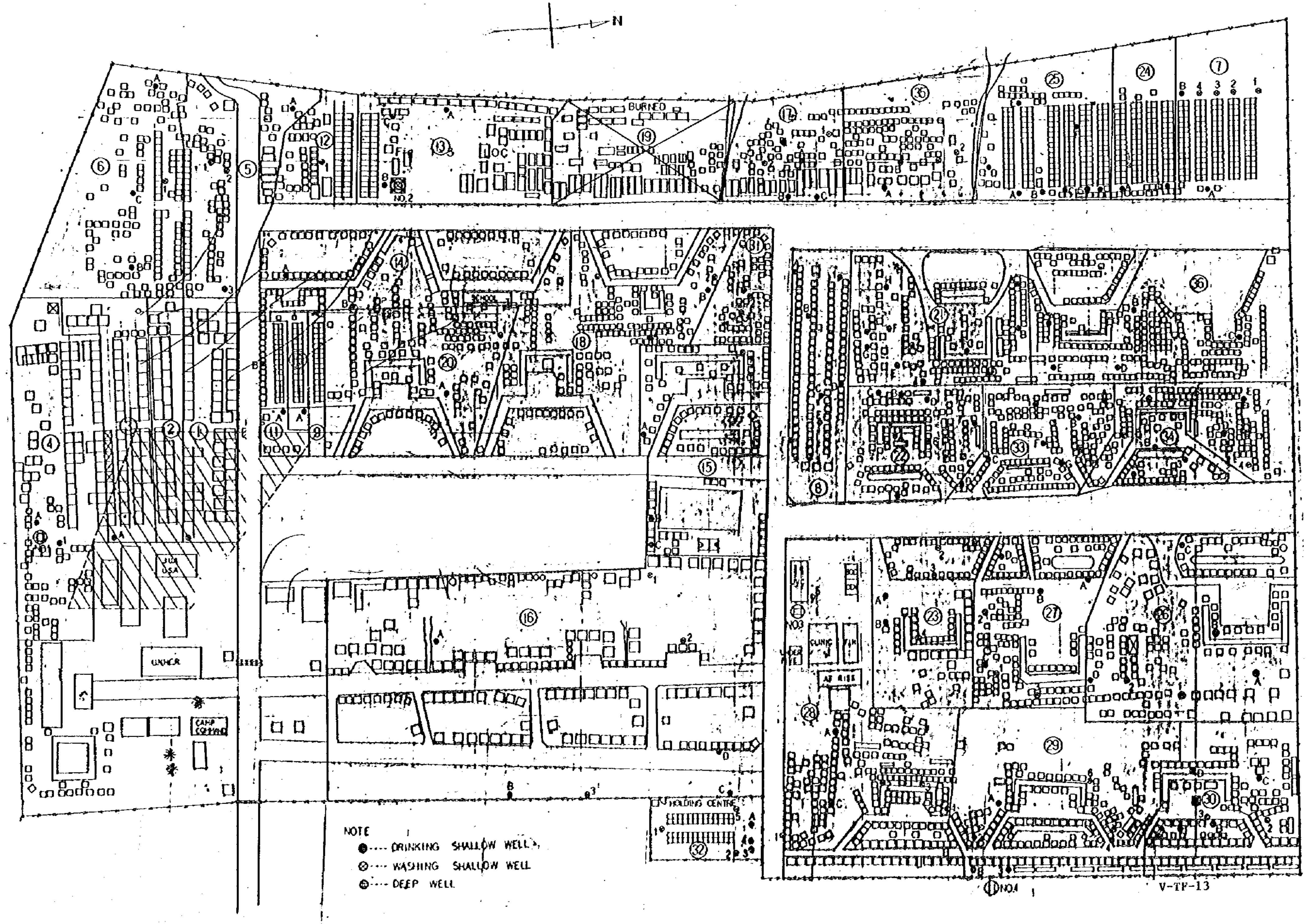
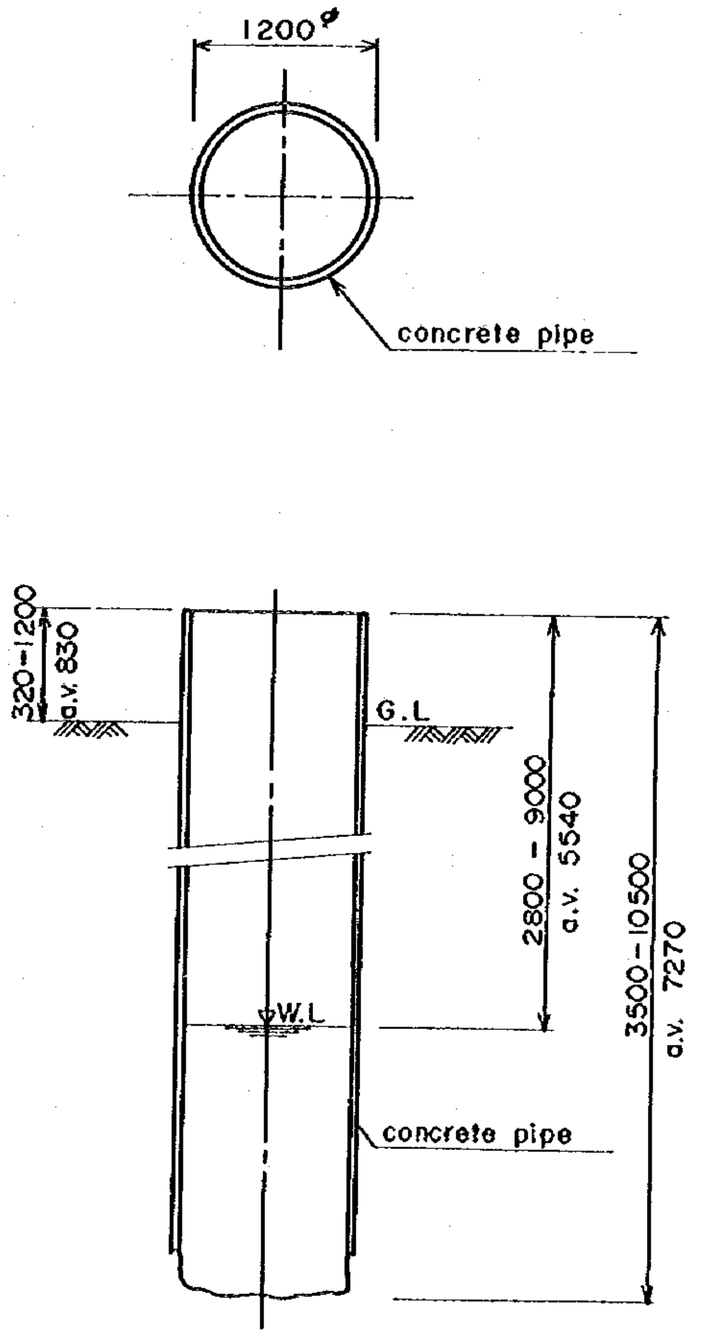


Fig. 5-2-2 Existing Well Type

TYPE-I



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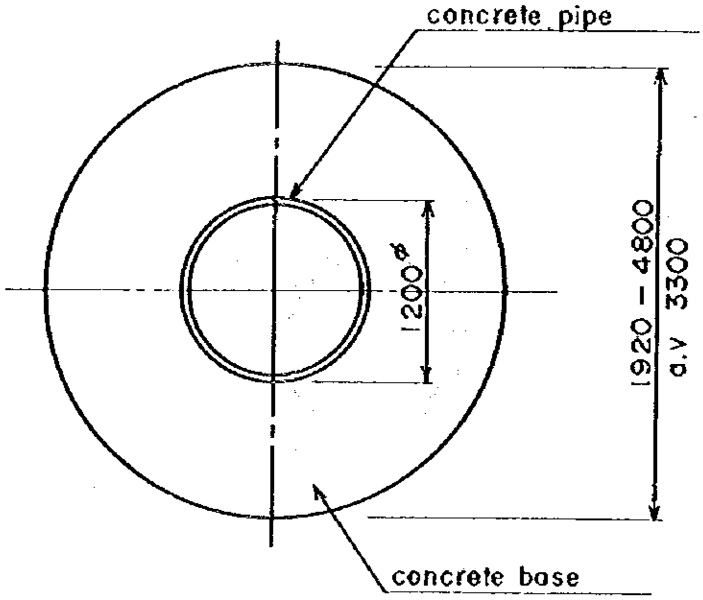

concrete pipe
1200ø
1920 - 4800
a.v 3300
concrete base

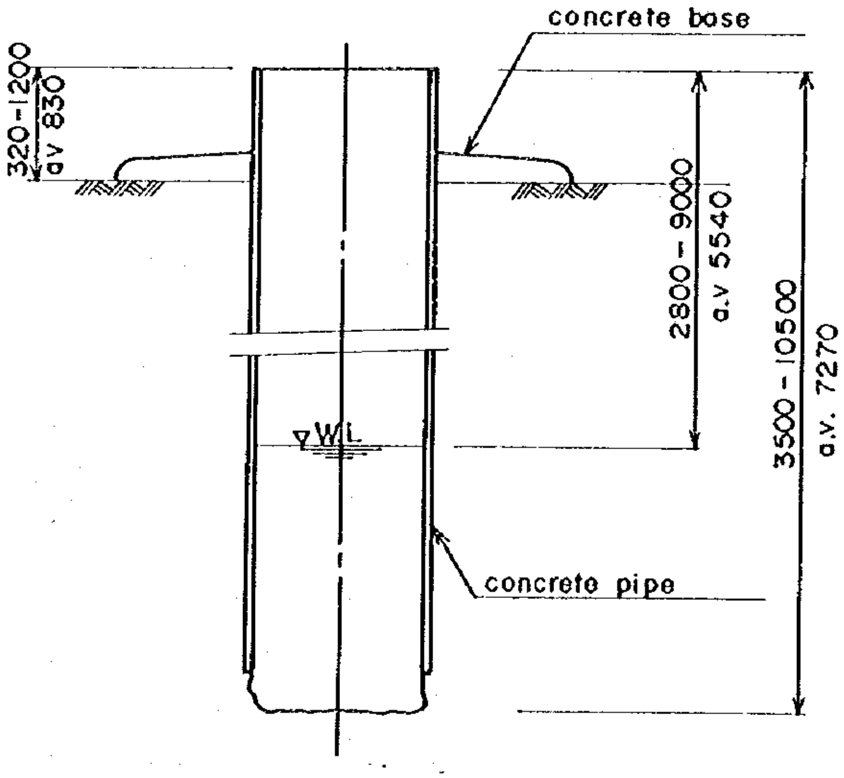
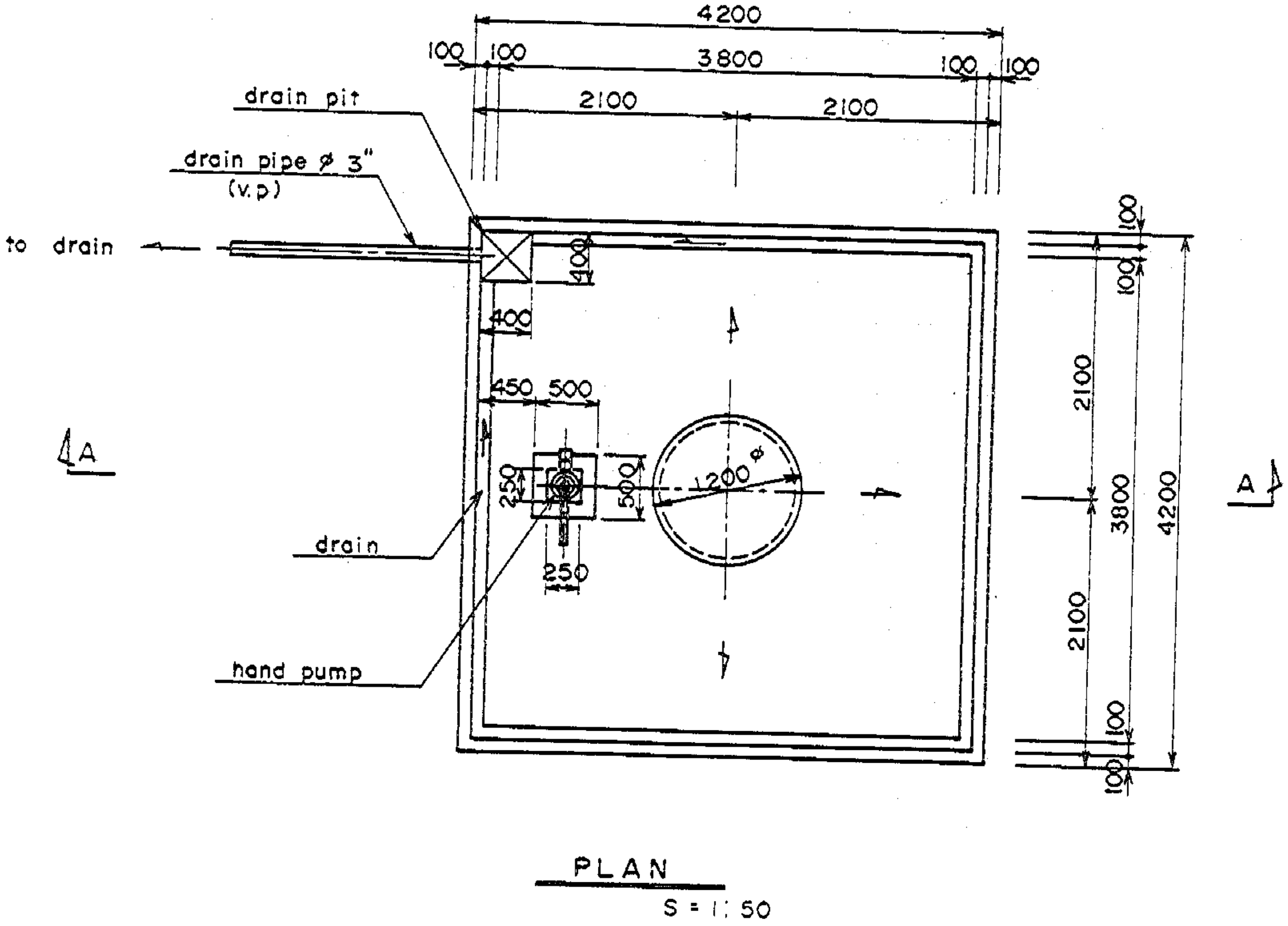
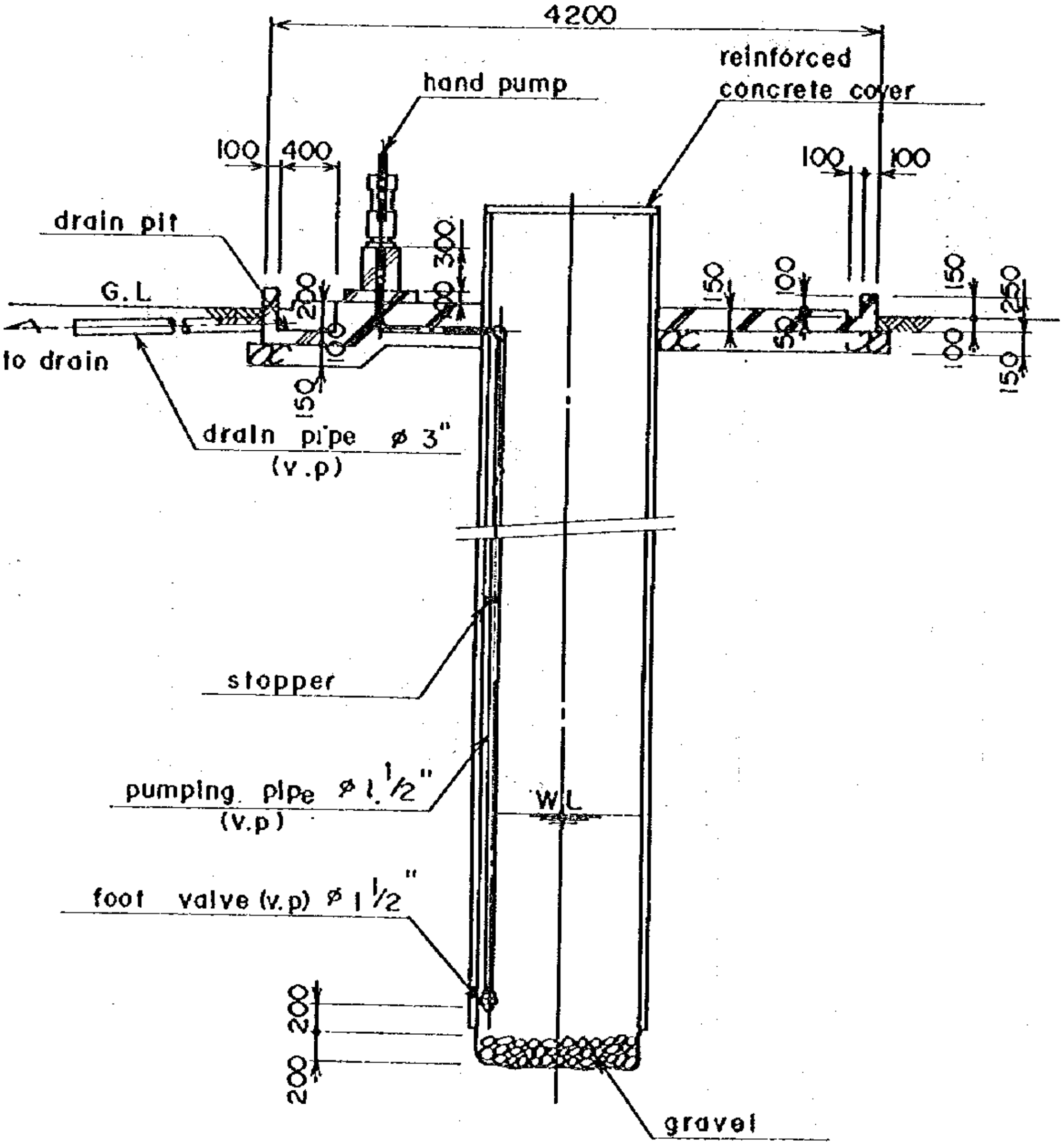

concrete base
320-1200
av 830
2800 - 9000
a.v 5540
3500 - 10500
a.V. 7270
W.L
concrete pipe

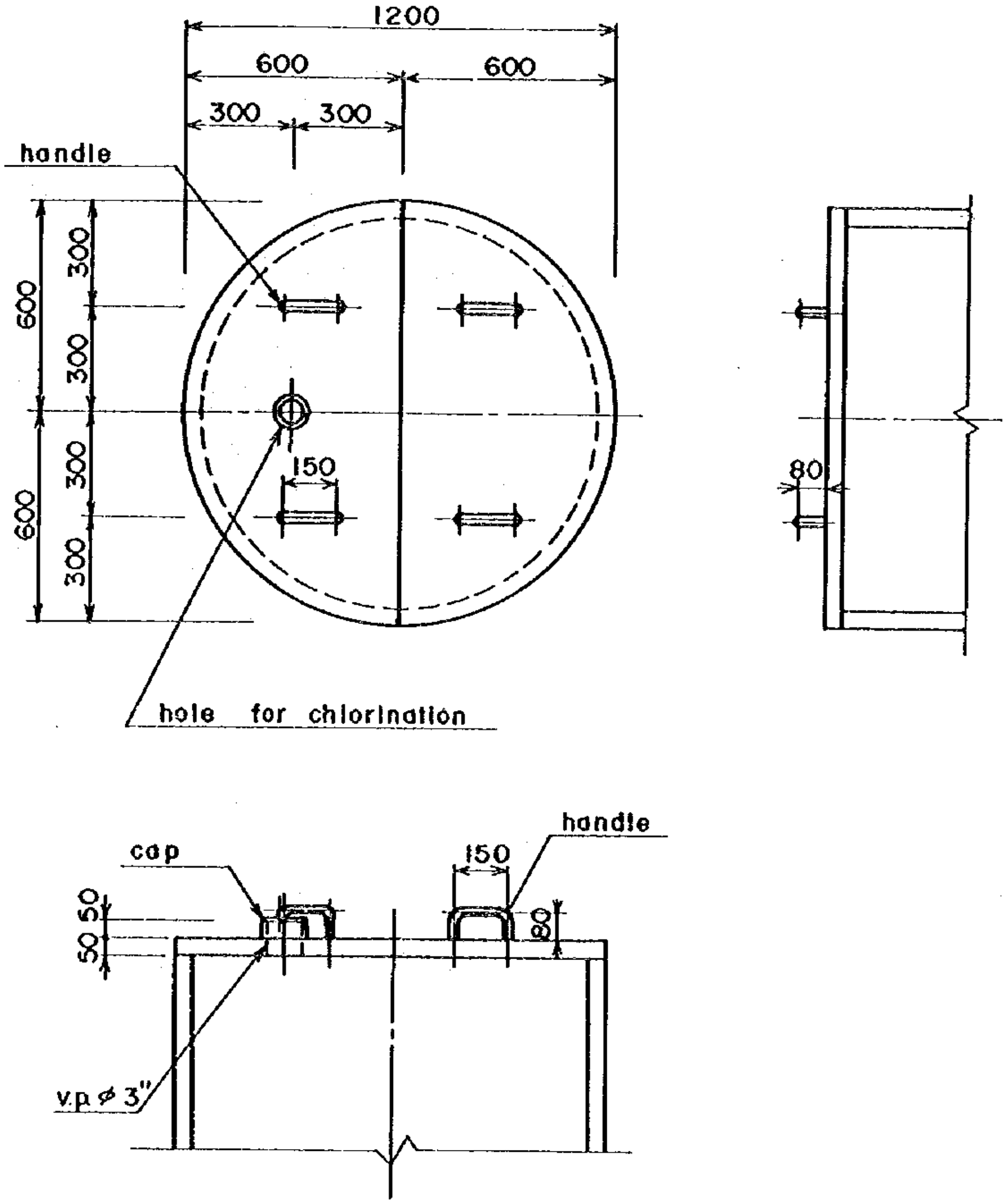
Fig. 5-2-3 Model Plan





SECTION A - A

S = 1: 50



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CHAPTER 6 SURVEY AND PROPOSED CONSTRUCTION WORKS OF GROUND WATER DEVELOPMENT

6.1 Present Condition of Kab Cherng Holding Center

The total land area of the Holding Center is approximately 152,800 m^2, and about 6,500 at Kampuchean refugees are accommodated in it, who can be classified as shown in the table below.

Boys	Girls	Men	Women	Total
1,189	1,203	1,228	2,262	6,482

(as of July 1, 1981)

6.1.1 Location and Geological Features of the Holding Center

The Holding Center, located about 44 km roadway to the South of the City of Surin at the point where Highway 2,121 meets Highway 2,145, can be reached from the City by Highway 2,145 via Prasat (see Fig. 6-1-1).

The place where the Holding Center is located used to be a woody area with a small stream flowing through it but partly under cultivation of upland crops.

The premises of the Holding Center was constructed mostly by filling, embanking and grading into the present flatness, on which the houses for refugees are being built.

The elevation of this place is about 185 m.

The top soil is sandy clay containing fine square pieces of sandstone, conglomerate and shale deposited for the thickness of about 5 m. The part of the premises where Well No. 1 is located is presumably in the old river course and the thickness of deposit in it is nearly 30 m.

6 deep wells have been made in the Holding Center so far. The record of drilling data could be collected for each of them, but core drilling was not performed for none of them.

Since the main part of the recorded data consists of results of analysis of the slime (finely pulverized rocks) drained out at the time of drilling of the well for identification of rocks, there are individual differences among observers in judgement.

However, it is possible to infer the geological features roughly from the recorded data.

The layer under the top soil is a rock bed which consists of sandstone, shale and siltstone partly alternated with each other.

The water stored in the deep wells of the Holding Center, therefore, is spring water from the joints of the rock bed, or in other words, it is the ground water called either fissure water or juvenile water.

The newly made deep wells J-No. 1 and J-No. 2 are located about 175 m and 265 m, respectively, to the south of the southern most one of the existing deep wells made by UNHCR.

So it was judged that the geological features there are nearly same.

And "HYDROGEOLOGICAL MAP OF NORTH EASTERN THAILAND" (Scale 1 : 500,000) surveyed and published by Department of Mineral Resources, Ministry of Industry, Thailand, also shows the main yield of groundwater in this area is located where there is spring water from rock beds. (Fig. 6.1.2)

6.1.2 Water Supply Conditions and Methods in the Holding Center

Of the 6 existing wells in the Holding Center, 1 (UN Well No. 4) completely dry, and another (UN Well No. 1) equipped with a hand pump is almost in disuse.

The 4 existing deep wells in service were UN Wells No. 2,3,5 and 6.

The previous pumping condition of the 4 wells were as shown in the table below.

UN-Well	Pumped Capacity (/m)	Pumped Capacity (ton/day) (10 hrs)
No. 2	37	22.2
No. 3	25	15.0
No. 5	18	10.8
No. 6	90	54.0
Total		102.0

(March, 1981)

The total pumpage from the 4 wells showed a decrease in the dry season, about the middle of July 1981.

Even though there were no recorded data of accurately measured pumpage, water volume was recorded in terms of number of storage tanks used as shown in the table below.

UN-Well	Number of Water Tanks	Pumped Capacity Per Day (ton)
No. 2	22	33.44
No. 3	7	10.64
No. 5	7	10.64
No. 6	31	38.75
Total	67	93.47

Water Tank : 1.15 m x 1.15 m x 1.15 m ≒ 1.520 m^3 (water 1.52 ton)

(as of June 17, 1981)

But the daily pumpage of UN-Well No. 2 is on the increase.

The time of pumping from each of them is about 10 hours a day.

UN-Well No. 2 had been under continuous misfunction of the pump since April, 1981 and UN-Well No. 6 went unpumpable about the end of June, 1981.

Later in July, 1981, UN-Well No. 2 also went unpumpable probably due to a trouble in the pump.

So the UN-Wells No. 3 and No. 5 were left in service from which only about 21 tons of water was lifted a day for storage in 14 tanks (1.52 m^3)

For additional supply, there were about 40 trips of tank-trucks a day from a reservoir located about 40 km away from the Holding Center carrying about 56 tons of water. The water from the reservoir was also used for drinking purpose.

Now that it is rainy season, the rich storage of the rain water is available mainly for miscellaneous purposes (for bathing, washing tableware, clothes and farm products)

So the total about of water supplied from the 2 wells and from the reservoir by truck was about 76 tons a day and it is equivalent to per capita daily supply of 11.7 ℓ.

Later, about July 20th, UN-Well No. 3 also failed leaving only one, UN-Well No. 5 in operation, consequently causing a drop in the supply of well water to 10 tons a day.

The probable cause of the failure of pumpage from the wells is the trouble in the submerged pumps, but not the lowering of the level of the ground water or well interference.

UN-Well No. 5 which is closest to Well-J No. 1 of all the existing wells (distance = 175 m) is in normal function.

There is no evidence of mutual well interference between the two.

6.2 Outline of the Survey

At the time of the preliminary survey it was learned that UN-Well No. 1 was equipped with a hand pump for lifting water from the depth of about 100 ft and other UN-Wells, depth varying from 270 ft to 300 ft, were lined with casing pipes with submerged pumps installed near the bottom.

This information, at first lead the term members to suspect that there could be ground-water aquifers near the depth of 30 m and 90 m, and the casing pipes were inserted to prevent the well walls from falling.

The assumption in the preparation of equipment and materials for well drilling therefore, was that the thickness of alluvial stratum should be over 100 m.

But, according to the fractional data obtained from UNHCR as a result of the efforts to collect information concerning the existing wells in the Holding Center in the present survey, the thickness of the top soil is about 5 m and the foundation bedrock under it is consisted of sandstone, shale, and siltstone.

So, it was judged that ground water aquifers are absent there, but the water supplied to the wells is fissure water yielded through cavities in the rocks.

Later, with the cooperation of UNHCR officers, it was made possible for the team members to inspect the documented data for the 6 existing UN-Wells.

The data, all written in a form of letter, lacked geological well logs and other graphic presentations.

So the team members translated the information of the documents into figures and tables for the convenience and efficiency of further studies. (Appendix 1-6)

6.3 Selection of the Sites of Test Drilling

Since the identification numbers and location of the wells contained in the data available at first were the record made at the time of planning or during execution, efforts were made to actually locate the wells and to obtain data relevant to each of them.

The existing wells are located roughly on the north-west of the Holding Centre. By comparative analysis of the geological logs of these wells, it was possible to roughly grasp the geological formation of the area, but it was not possible to determine the presence or absence of joints or fissures.

The position of a strainer of a casing pipe was recorded only for UN-Well No. 2.

Electrical exploration was made at two points (E-1, E-2) in the Holding Centre by Wenne Electrode Array (four electrode array), and more specifically vertical resistivity exploration with the electrode separation of a = 0.5 to 100 m. was conducted. In other words the apparent resistivity of the depth from 0.5 to 100 m. was measured.

The exploration did not show any evidence indicating outstanding geological and other features as known from the resistivity curve (Figure 6-3-1 and Figure 6-3-2).

The locations of test drilling are only within the Camp premises. And in selecting suitable places, there were various limiting factors like the layout of refugees houses, width of the road for conveying drilling equipment, presence of electric poles and cables and so on. But one ideal point was finally selected which is on the side of a main road, with sufficient space, partly cultivated and surrounded by houses and fairly far from the existing wells. It was named J-No. 1 well.

And another point selected in the northern corner of the premises was named J-No. 2 well. Both are fairly far from the sites of previous test drilling.

The location of the new J-No. 1 and J-No. 2 wells and of the existing 6 UN-Wells are as shown in Figure 6-3-3.

6.4 Result of Test Drilling

Since test drilling was done with drilling bits only, drilling cores were not obtained, but geological identification of rocks was based on slim rocks finely pulverized by the bit which is carried in the mud water drained out in drilling.

The time taken for drilling was checked for drilling depth of each 0.50 m and was recorded in the geological well log for the drilling depth of each one meter.

Soon after drilling, electrical logging was conducted, in which the most outstanding recording was observed when the interval between the potential electrode and energizing electrode was 50 cm as indicated in the geological well log. Figure 6-4-1 shows the geological well log of J-No. 1 well, the first one tried in this project, and Figure 6-4-2 is for J-No. 2 well.

The sandstone encountered in the drilling was characterized by extremely strong and close texture in the category of grey wacke, for penetrating which maximum time taken was 255 min. per 1 m.

A-B geological section with geological well logs lined up, as in Figure 6-4-3,shows the geological features of the Holding Centre.

Pumping test conducted after the completion of the drilling test for J-No. 1 well showed the yield of 2,115 liters in 24 minutes.

The ground water level in it was 10.7 m from the ground surface at first, but went down to 85 m by the time test pumping was over. So the drawdown was 74.3 m. The time of water level recovery was 3 hours and 30 minutes.

This implies the need of intermittent pumping. In the case of J-No. 2 well the yield of springing fissure water was abundant and water level recovery was quick, so that it was possible to do continuous pumping at the rate of 54 liters/min.

6.5 Description of Well Drilling

A casing pipe of 4 inches was inserted to each of the J-No. 1 and J-No. 2 wells. A submersible motor pump of 1-1/2 inches was used for pumping.

A PVC pipe with the inner diameter of 17 mm was attached to the pumping pipe for gaging by a simple flood meter.

The electric power for driving the pump was generated by diesel engine-driven generator. These structures on the surface were housed in a simple shed.

Equipment Installed

Submersible Motor Pump
Well J-No.1 .. Barkley Pump Company (California USA) Single Phase 230 Volts 13.2 Amps. 2 HP.
Well J-No.2 .. Barkley Pump Company (California USA) Single Phase 230 Volts 13.2 Amps. 2 HP.

Generator - 2 (Well J-No. 1 and No. 2)
Yammar Model TA 160-L 13 HP. (16 HP.) Diesel Powered 8 KVA 220 Volts Dynamo (Thailand)

6.6 Prospected Water Supply

For the present refugee population of 6,500, the daily water requirements at the rationing rate of 20 liters/capita /day is,

6,500 x 20 = 130,000 liters/day = 130 m^3/day

Whereas the amount supplied by UN Well - No. 2,3,5 and 6 during the dry season was 93.5 m^3/day. And the pumpage from the new J-No. 1 and No. 2 wells is as follows:

J-No. 1 ... Intermittent pumping 4 times a day
each time yielding 2,115 liters
2,115 x 4 = 8,460 liters/day = 8.5 m^3/day.

J-No. 2 ... Continuous pumping is possible at the
rate of 54 liters/min. So if operated
10 hours a day,
54 x 10 x 60 = 32,400 liters/day = 32.4 m^3/day

Total 8.5 + 32.4 = 40.9 m^3/day

Pumping test of J-No. 1 and J-No. 2 wells was conducted in the rainy season, so the value of pumpage can be modified for the dry season after the tendency of the existing UN-Wells as follows:

$$40.9 \times 93.47/102.0 = 37.5 \text{ m}^3/\text{day}$$

So the total yield from the 4 UN-Wells and 2 J-Wells is,

$$93.5 + 37.5 = 131 \text{ m}^3/\text{day}$$

and this is equal to the daily requirements.

But the problems to solve for satisfactory water supply operation hereafter is the repair of the 3Of the 4 UN-Wells which are currently out of service. The necessary repair done, subsistence water of sufficient amount and cleanliness shall be supplied to the refugees.

However, the underground water thus available in the Holding Centre, being originated as fissure water from the bedrocks, can fluctuate wildly in yield. Therefore it is suggested that, instead of depending too much on the ground water, consideration is taken for making of same other sources such as reservoirs to store rain water directly or small-scale dams in streams.

LIST OF TABLES AND FIGURES IN THE CHAPTER 6

FIGURES

Fig. 6-1-1 Location of Kab Cherng Holding Center

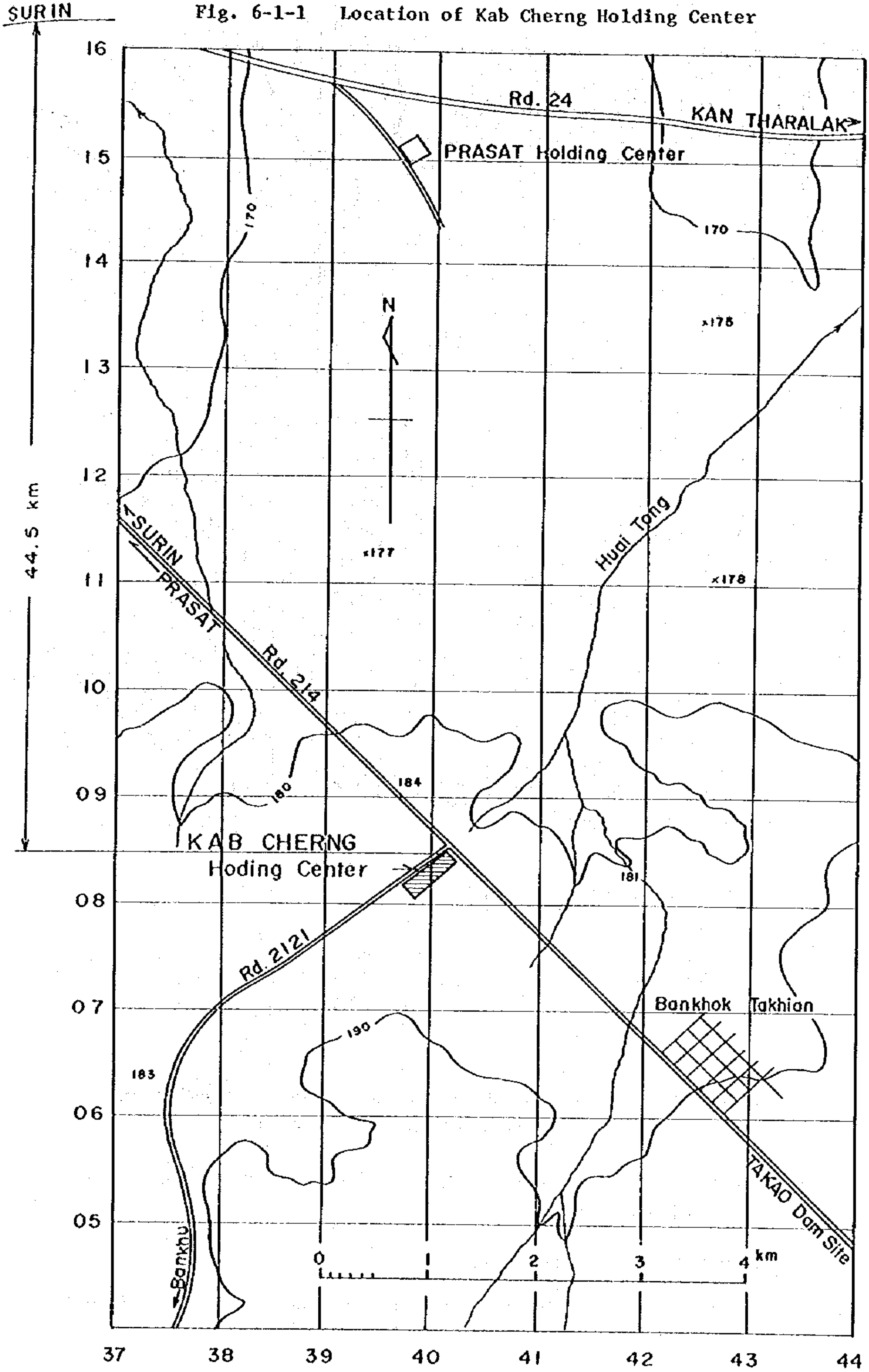
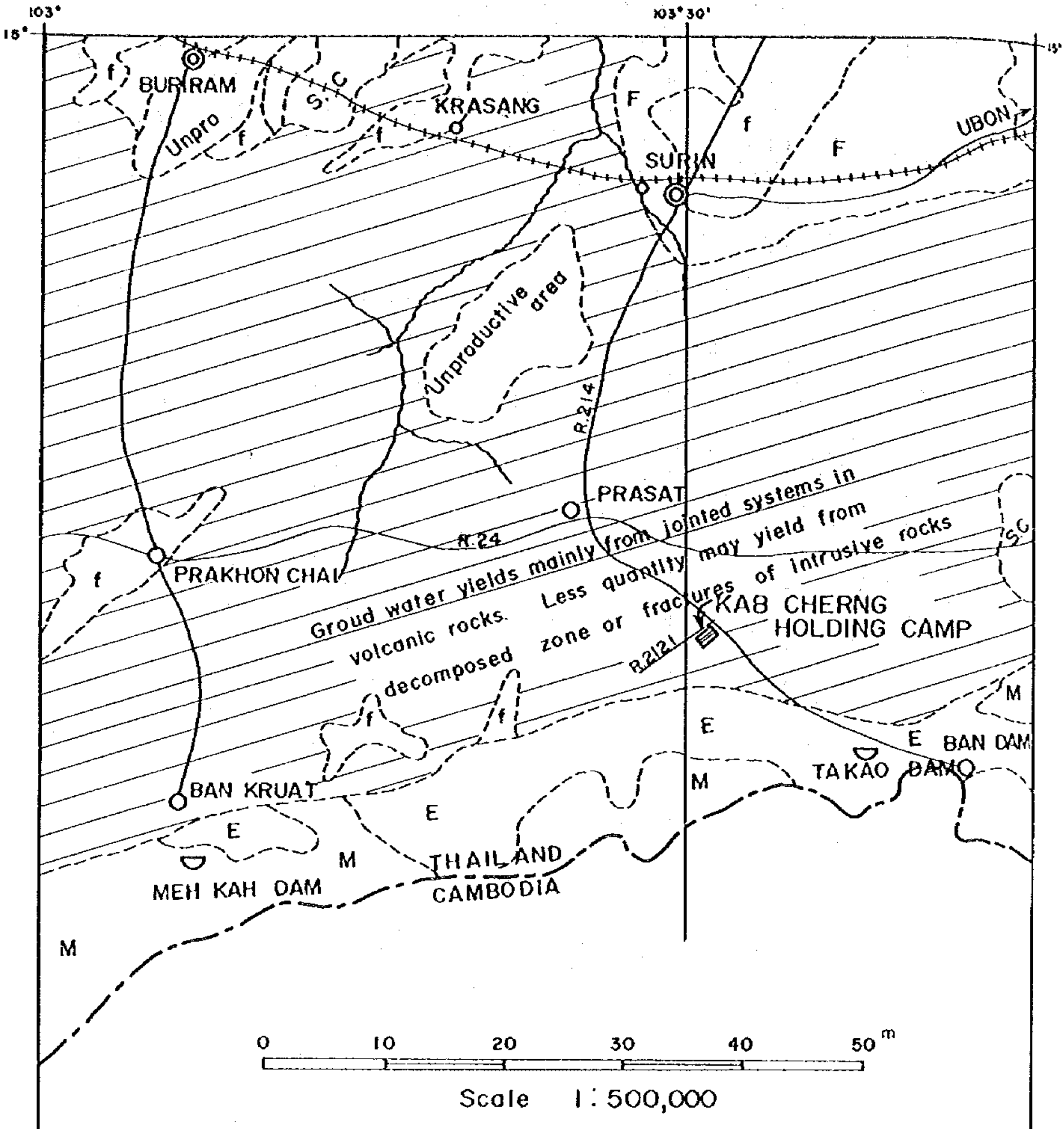


Fig. 6-1-2 Hydrogeological Map of Kab Cherng Area



f : Flood plain and isolated, depressin within areas of higher relif, efflorescence salt commonly occurs on ground surfaces.

E : Eroded surfaces of former cuestas or dip slopes where high artesian head is observed.

M : Middle khorat aquifers (Jurassic).

Sandstone, conglomerate, quartzose sandstone, shale, and siltstone

Fig. 6-3-1 E-1 Resistivity (ρ-a) Curve

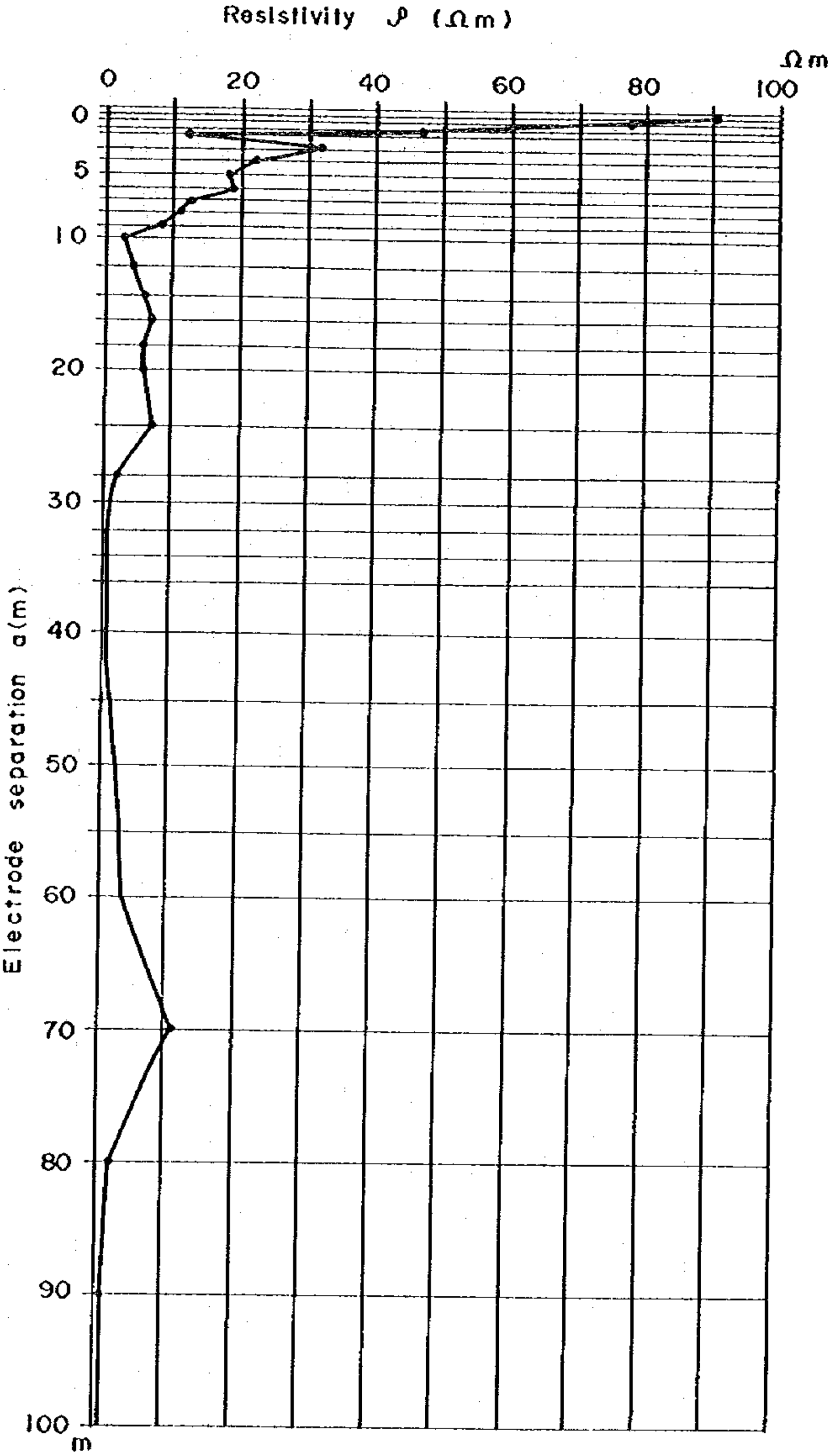


Fig. 6-3-2 E-2 Resistivity (ρ-a) Curve

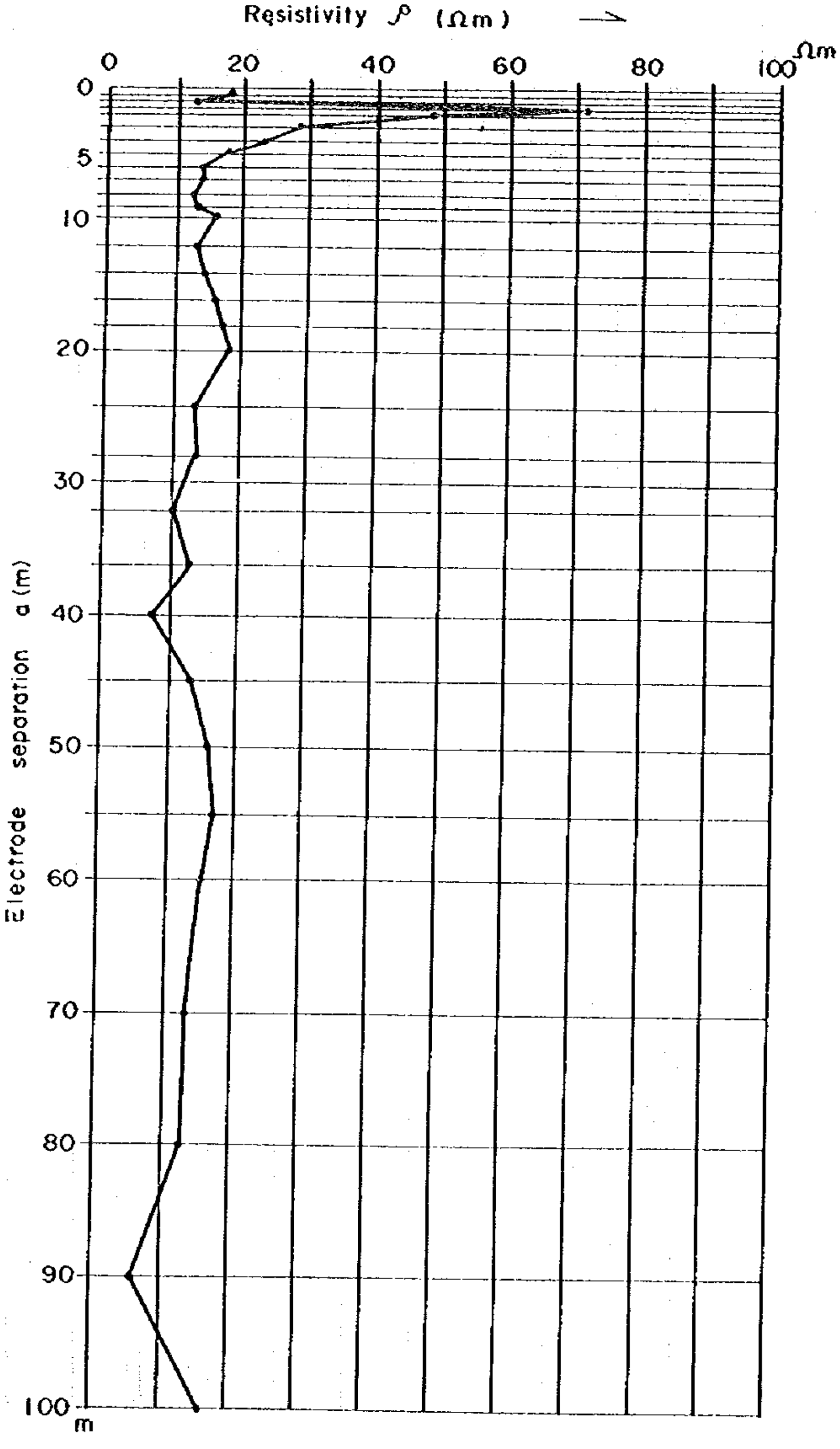


Fig. 6-3-3 Location of the Drilled Wells

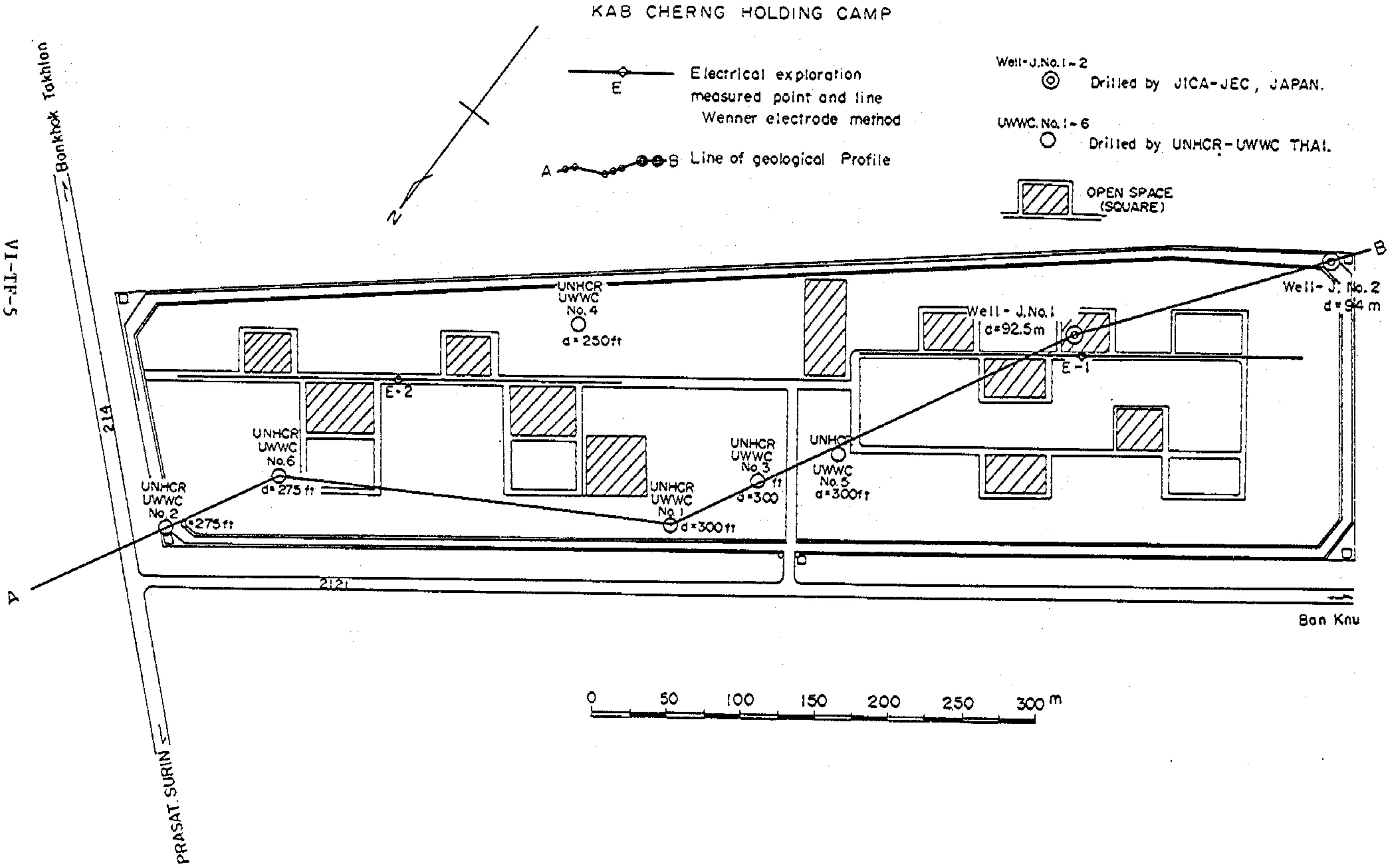


Fig. 6-4-1 GEOLOGICAL RECORD OF BORING

WELL HOLE No. J NO. 1

PROJECT	Survey of Water supply to the refugees		LOCATION	KAB CHERNG HOLDING CENTER	
GROUND ELEVATION	185.00 m	DEPTH OF HOLE	92.50 m	ANGLE FROM VERTICAL	
DIAMETER OF HOLE	6 3/4 inch	MACHINE	Y Large Hole Drill THS-70	DATE OF DRILLING	18 June ~ 17 July 1981
CORE RECOVERY	Bits	DEPTH TO GROUND WATER LEVEL IN HOLE		10.70 m	
By Slime observation		DRILLED BY SUZUKI, MIYAJIMA		LOGGED BY T. HAGIWARA	1981

ELEVATION (m)	DEPTH (m)	THICKNESS (m)	SOIL OR ROCK CLASSIFICATION	COLOUR	FORMATION DESCRIPTION
Ground	Water Level 1976/'81 1.30 ▽				
180.40	4.60	4.60	compact	brown ∫ gray	Sandy Clay with Detritus
176.50	8.50	3.90	fairly hard	reddish brown	Sandstone
163.50	21.50	13.00	soft	reddish brown	W.L. 10.70m ▽ 13/7/'81 Shale
157.30	27.70	6.20	hard	dark gray	Shale and Sandstone alternation
153.50	31.50	3.80	very hard	dark gray	Sandstone (Greywacke)
150.50	34.50	3.00	hard	dark gray	Sandstone and Shale alternation
149.30	35.70	1.20	soft	reddish brown	Shale
141.90	43.10	7.90	very hard	dark gray	Sandstone (Greywacke)
139.75	45.25	7.15	soft	dark gray	Siltstone
119.50	65.50	20.25	very hard	dark gray	Sandstone (Greywacke)
102.40	82.60	17.10	soft	reddish brown	Shale
92.50	92.50		very hard	dark gray	Sandstone (Greywacke)

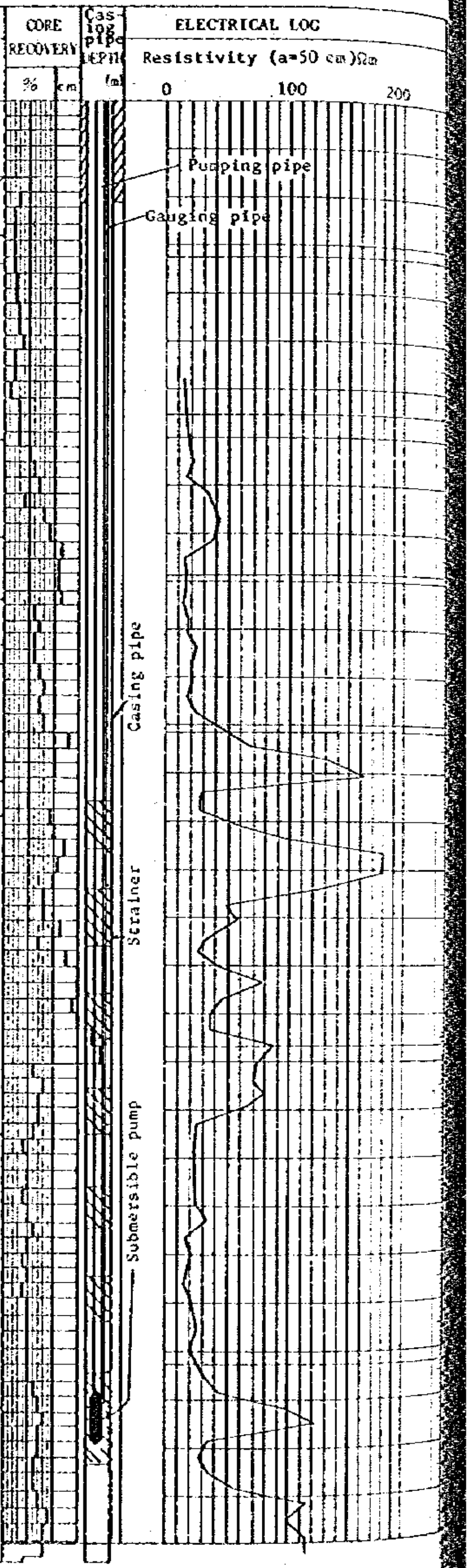


Fig. 6-4-2 GEOLOGICAL RECORD OF BORING

WELL HOLE No. J NO. 2

PROJECT: Survey of Water Supply to the refugees
LOCATION: KAB CHERNG HOLDING CENTER
GROUND ELEVATION: 185.00 mm
DEPTH OF HOLE: 94.00 m
ANGLE FROM VERTICAL:
DIAMETER OF HOLE: 6 3/4 inch
MACHINE: T THS-70
DATE OF DRILLING: 21 July ~ 1 Aug. 1981
CORE RECOVERY: Bits
By Slime observation
DEPTH TO GROUND WATER LEVEL IN HOLE: 10.20 m
DRILLED BY SUZUKI, MIYAJIMA
LOGGED BY T. HAGIWARA 1981

ELEVATION (m)	DEPTH (m)	THICKNESS (m)	FIELD OBSERVATION			
			COLUMN SECTION	SOIL OR ROCK CLASSIFICATION	COLOUR	FORMATION DESCRIPTION
179.10	5.90	5.90		compact	brown ~ gray	Sandy Clay with Detritus
164.50	20.50	14.60		soft	reddish brown	Shale
160.30	24.70	4.20		very hard	dark gray	Sandstone (Greywacke)
155.60	29.40	4.70		soft	reddish brown	Shale
144.50	40.50	11.10		very hard	dark gray	Sandstone (Greywacke)
138.90	46.10	5.60		soft	dark blue ~ gray	Siltstone
135.30	49.70	3.60		hard	dark gray	Sandstone and Siltstone alternation
120.20	64.80	15.10		very hard	dark gray	Sandstone (Greywacke)
111.30	73.70	8.90		soft	reddish brown	Shale
103.20	81.30	8.10		hard	dark gray	Sandstone
101.50	83.50	1.70		hard	reddish brown	Shale
91.00	94.00			hard	dark gray	Sandstone (Greywacke)

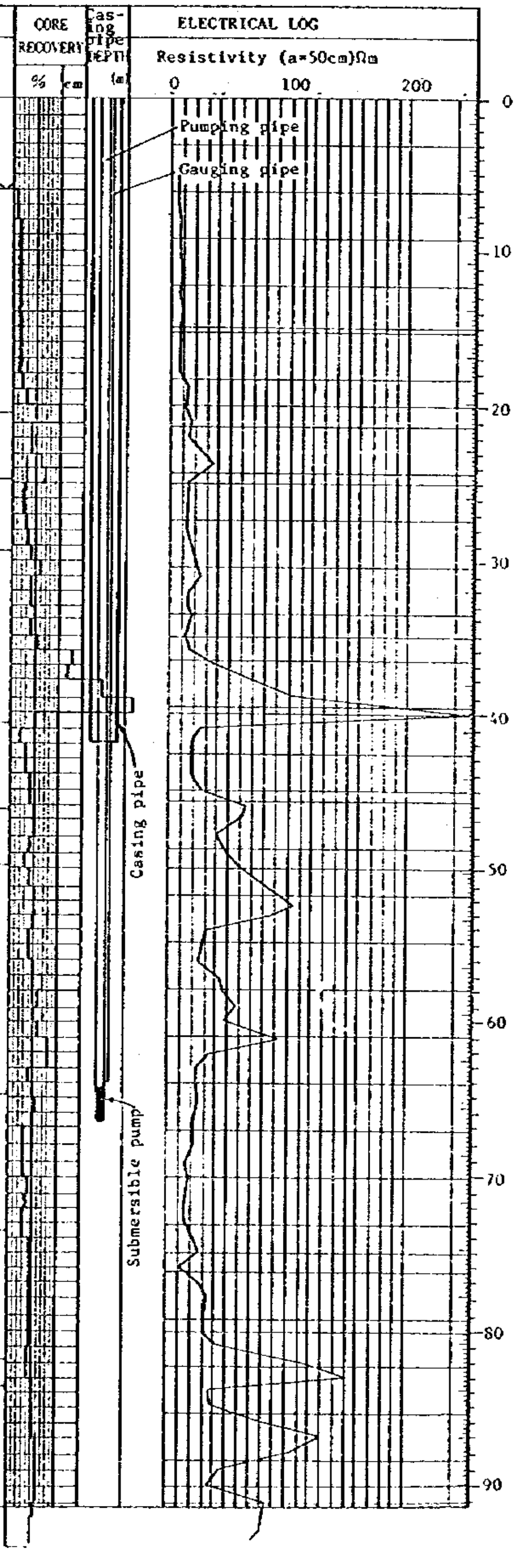
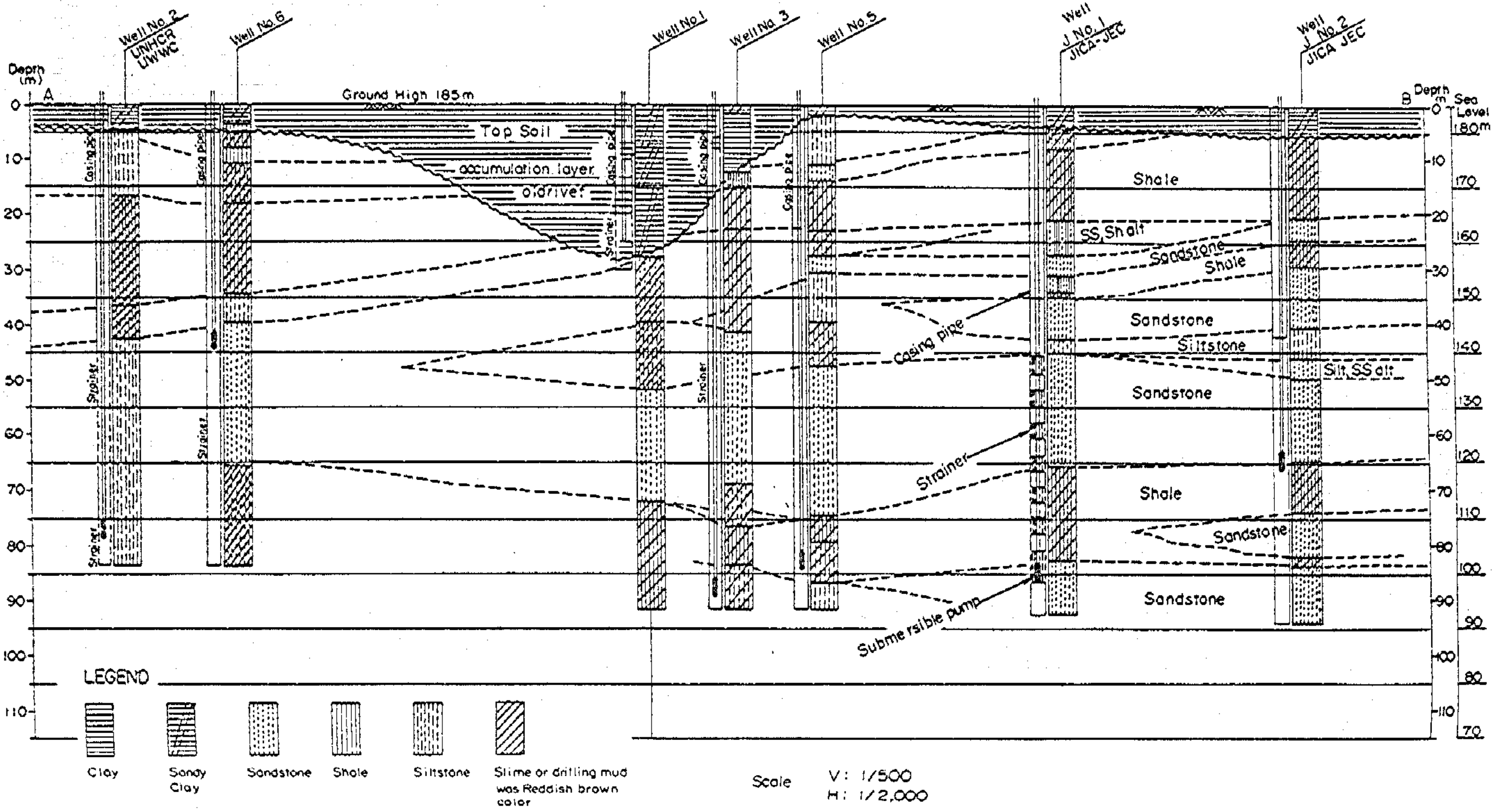


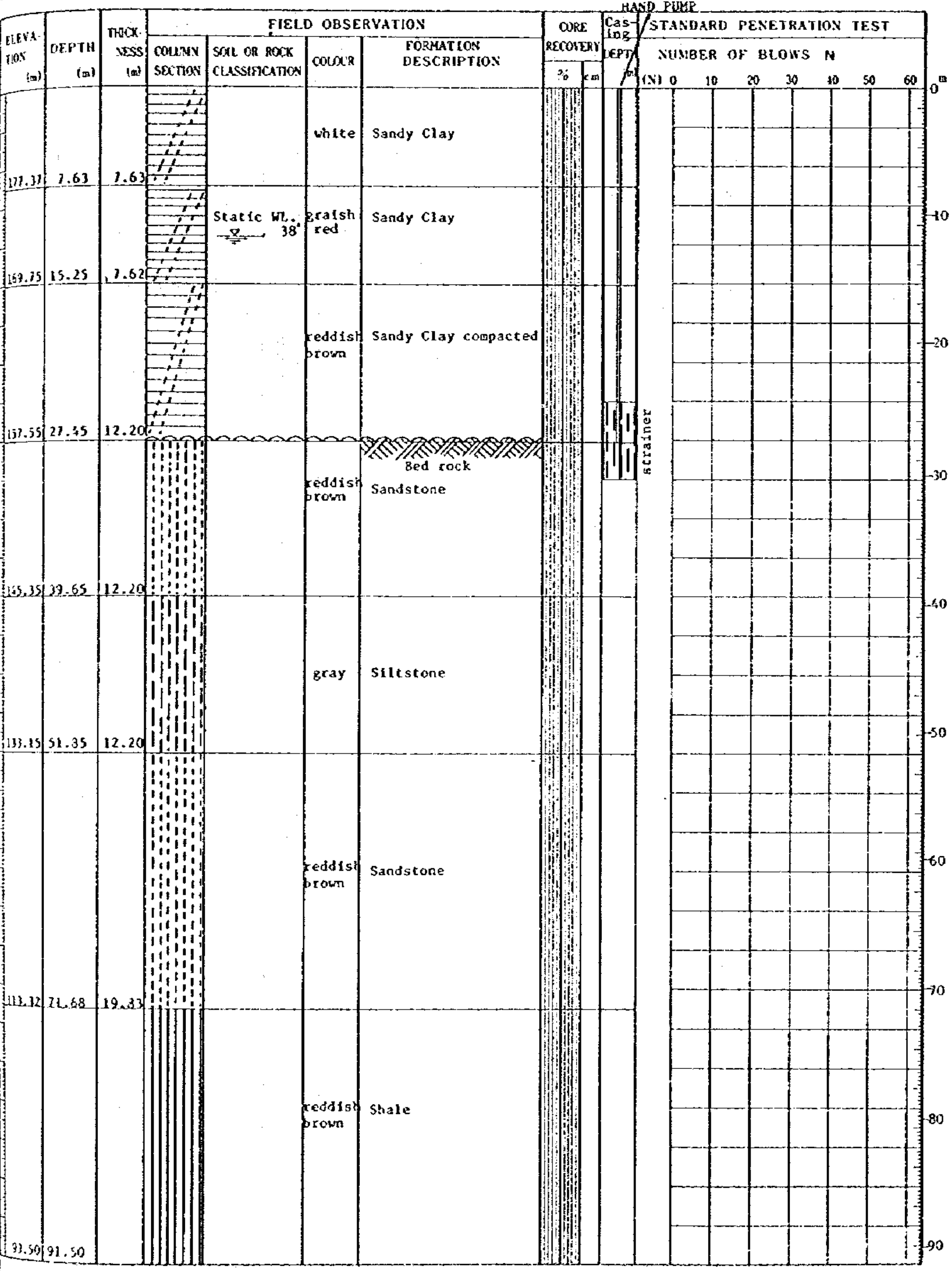
Fig. 6-4-3 A-B Geological Profile of Kab Cherng Holding Center

Geologica logs by Drilling-Slime observation



Appendix-1 **GEOLOGICAL RECORD OF BORING** UNHCR UWWC HOLE No. 1

PROJECT		LOCATION	Kab Cherng Holding Center, SURIN		
GROUND ELEVATION	185 m	DEPTH OF HOLE	300 feet	ANGLE FROM VERTICAL	
DIAMETER OF HOLE	6 3/4"	MACHINE	Bit-drilling Mach.	DATE OF DRILLING	18 May ~ 5 June 1980
CORE RECOVERY		DEPTH TO GROUND WATER LEVEL IN HOLE			
By Slime observation		DRILLED BY	United Water Well Construct.	LOGGED BY	



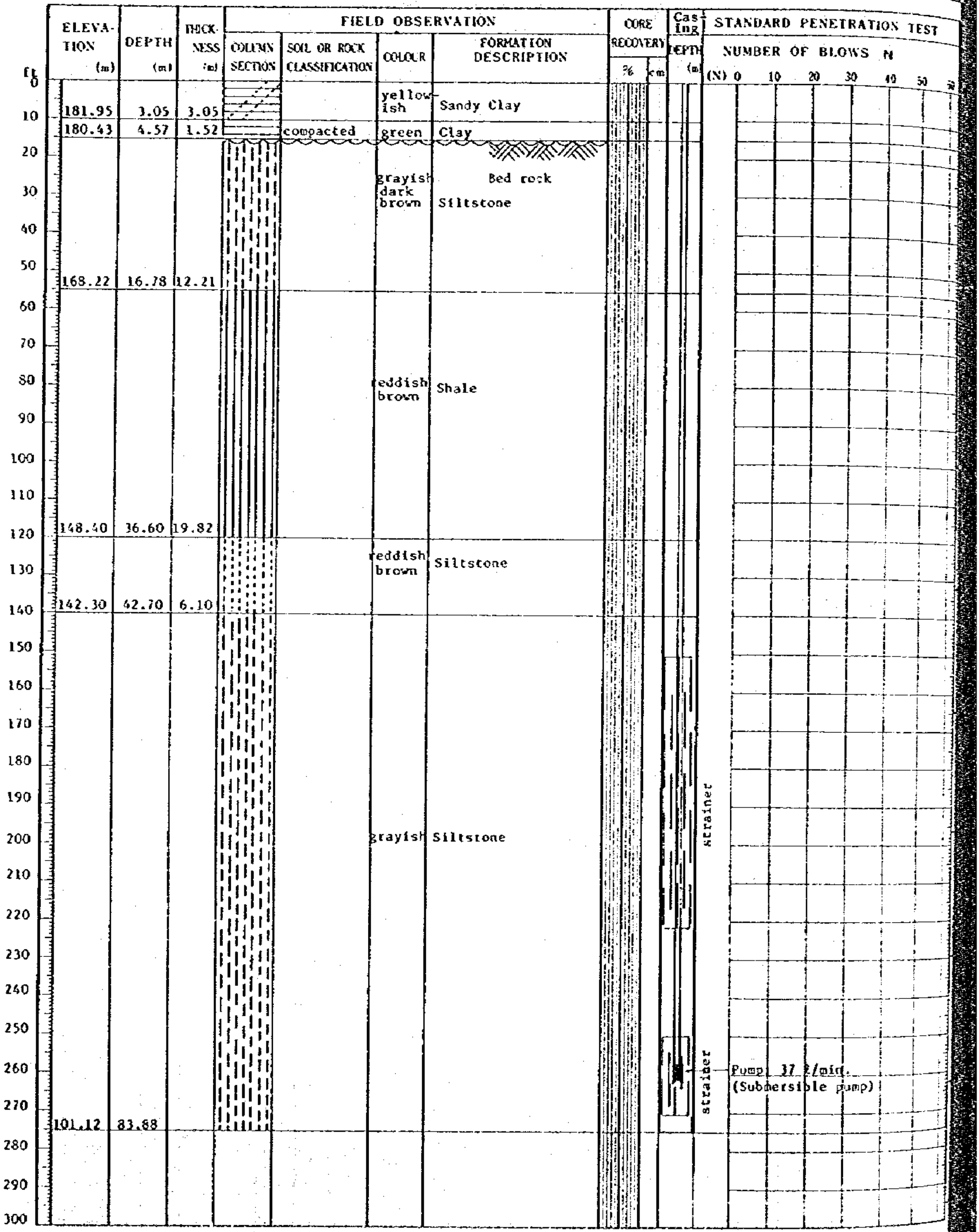
ELEVATION (m)	DEPTH (m)	THICKNESS (m)	SOIL OR ROCK CLASSIFICATION	COLOUR	FORMATION DESCRIPTION	CORE RECOVERY %	CORE RECOVERY cm	Casing DEPTH m
177.37	7.63	7.63		white	Sandy Clay			
169.75	15.25	7.62	Static WL. 38'	graish red	Sandy Clay			
157.55	27.45	12.20		reddish brown	Sandy Clay compacted			
145.35	39.65	12.20		reddish brown	Sandstone			
133.15	51.35	12.20		gray	Siltstone			
113.32	71.68	19.33		reddish brown	Sandstone			
93.50	91.50			reddish brown	Shale			

Appendix-2 GEOLOGICAL RECORD OF BORING

UNHCR
UWWC

HOLE No. 2

PROJECT		LOCATION	Kab Cherng Holding Center, SURIN		
GROUND ELEVATION	185 m	DEPTH OF HOLE	275 ft.	ANGLE FROM VERTICAL	
DIAMETER OF HOLE	6 3/4"	MACHINE	Bit-drilling Mash	DATE OF DRILLING	20 June ~ 15 July 1980
CORE RECOVERY		DEPTH TO GROUND WATER LEVEL IN HOLE			
By Slime observation		DRILLED BY	United Water Well Construction	LOGGED BY	



ELEVATION (m)	DEPTH (m)	THICKNESS (m)	SOIL OR ROCK CLASSIFICATION	COLOUR	FORMATION DESCRIPTION
181.95	3.05	3.05		yellowish	Sandy Clay
180.43	4.57	1.52	compacted	green	Clay
168.22	16.78	12.21		grayish dark brown	Bed rock Siltstone
148.40	36.60	19.82		reddish brown	Shale
142.30	42.70	6.10		reddish brown	Siltstone
101.12	83.88			grayish	Siltstone

Appendix-3 GEOLOGICAL RECORD OF BORING UNHCR UWWC

HOLE No. 3

PROJECT				LOCATION	Kab Cherng Holding Center, SURIN
GROUND ELEVATION	185 m	DEPTH OF HOLE	300 feet	ANGLE FROM VERTICAL	
DIAMETER OF HOLE		MACHINE	Bit-drilling Mach	DATE OF DRILLING	13 ~ 21 Aug. 1980
CORE RECOVERY		DEPTH TO GROUND WATER LEVEL IN HOLE			
By Slime observation		DRILLED BY United Water Well Construct.		LOGGED BY	

ELEVATION (m)	DEPTH (m)	THICKNESS (m)	FIELD OBSERVATION				CORE RECOVERY		Casing DEPTH (m)	STANDARD PENETRATION TEST NUMBER OF BLOWS N
			COLUMN SECTION	SOIL OR ROCK CLASSIFICATION	COLOUR	FORMATION DESCRIPTION	%	cm		(N) 0 10 20 30 40 50 60
183.47	1.53	1.53			white to yellow	Sandy Clay compacted				
172.80	12.20	10.67			grayish dark brown	Clay compacted				
169.75	15.25	3.05		Bed rock	grayish dark brown	Shale with some sandstone				
162.12	22.83	7.63		well soft	reddish brown	Shale				
143.82	41.18	18.30		compacted	reddish brown	Shale				
116.37	68.63	27.45			gray dark brown	Sandstone				
108.75	76.25	7.62			reddish brown	Sandstone				
101.65	83.35	7.10		well soft	reddish brown	Shale				
93.50	91.50				grayish dark brown & reddish brown	Siltstone with some sandstone				

Obscure situation of installed strainer

25 ℓ/min.

Pump (N/A Contructor's pump)

0m 10 20 30 40 50 60 70 80 90

Appendix-4 GEOLOGICAL RECORD OF BORING

HOLE No. 4

PROJECT		LOCATION	Kab Cherng Holding Center, SURIN		
GROUND ELEVATION	185 m	DEPTH OF HOLE	250 feet	ANGLE FROM VERTICAL	
DIAMETER OF HOLE		MACHINE		DATE OF DRILLING	13 ~ 29 Sep. 1980
CORE RECOVERY		DEPTH TO GROUND WATER LEVEL IN HOLE			
By Slime observation		DRILLED BY United Water Well Construct.		LOGGED BY	

ELEVATION (m)	DEPTH (m)	THICKNESS (m)	FIELD OBSERVATION: COLUMNS SECTION	FIELD OBSERVATION: SOIL OR ROCK CLASSIFICATION	FIELD OBSERVATION: COLOUR	FIELD OBSERVATION: FORMATION DESCRIPTION	CORE RECOVERY %	CORE RECOVERY cm	STANDARD PENETRATION TEST: DEPTH (m)	STANDARD PENETRATION TEST: NUMBER OF BLOWS N (N) 0 10 20 30 40 50
183.47	1.53	1.53			white to yellow	Sandy Clay				
181.95	3.05	1.52			white & brown	Clay				
180.42	4.58	1.53			light green and brown	Clay				
174.32	10.68	6.10		compacted	dark gray	Clay				
						Bed rock				
169.75	15.25	4.57			dark gray	Sandstone				
156.02	28.98	13.73			reddish brown	Shale				
152.97	32.03	3.05			green & darkgray	Siltstone with some clay				
145.35	39.65	7.62			reddish brown	Shale				
139.25	45.75	6.10			dark gray	Siltstone with some shale				
137.72	47.28	1.53			light gray light green	Shale				
134.67	50.33	3.05			dark gray	Siltstone				
131.42	53.38	3.05			dark gray and light green	Siltstone with some shale				
130.10	54.90	1.52			dark gray	Siltstone				
128.57	56.43	1.53			dark gray	Siltstone with shale				
122.47	62.53	6.10			dark gray	Siltstone				
120.95	64.05	1.52			dark gray reddish brown	Shale				
116.30	68.63	4.58			reddish brown	Sandstone				
108.75	76.25	7.62			reddish brown	Shale				dry well

Appendix-5 GEOLOGICAL RECORD OF BORING

UNHCR
UWWC

HOLE No. 5

PROJECT		LOCATION	Kab Cherng Holding Center, SURIN		
GROUND ELEVATION	185 m	DEPTH OF HOLE	300 feet	ANGLE FROM VERTICAL	
DIAMETER OF HOLE		MACHINE	Bit-drilling mach.	DATE OF DRILLING	13 ~ 31 Oct. 1980
CORE RECOVERY		DEPTH TO GROUND WATER LEVEL IN HOLE			
By Slime observation		DRILLED BY	United Water Well Construct.	LOGGED BY	

ELEVATION (m)	DEPTH (m)	THICKNESS (m)	FIELD OBSERVATION COLUMN SECTION	SOIL OR ROCK CLASSIFICATION	COLOUR	FORMATION DESCRIPTION	CORE RECOVERY %	CORE RECOVERY cm	Casing DEPTH (m)	STANDARD PENETRATION TEST NUMBER OF BLOWS N
183.47	1.53	1.53			yellowish brown	Sandy Clay				
174.32	10.68	9.15		Water Level 30'	grayish brown	Bed rock Siltstone				
171.27	13.73	3.05			dark gray	Sandstone				
162.12	22.88	9.15			reddish brown	Shale				
157.55	27.45	4.57			reddish brown	Siltstone				
154.50	30.50	3.05			dark gray	Sandstone				
145.35	39.65	9.15			reddish brown	Siltstone with some shale				
137.62	47.38	7.73			dark and light	Siltstone				
110.27	74.73	27.35			light gray	Sandstone				
105.70	79.30	4.57		very soft	reddish brown	Shale				
99.07	86.93	7.63		very soft	reddish brown	Siltstone with some shale				
93.50	91.50				dark gray	Siltstone				

(N) 0 10 20 30 40 50 60

0m 10 20 30 40 50 60 70 80 90

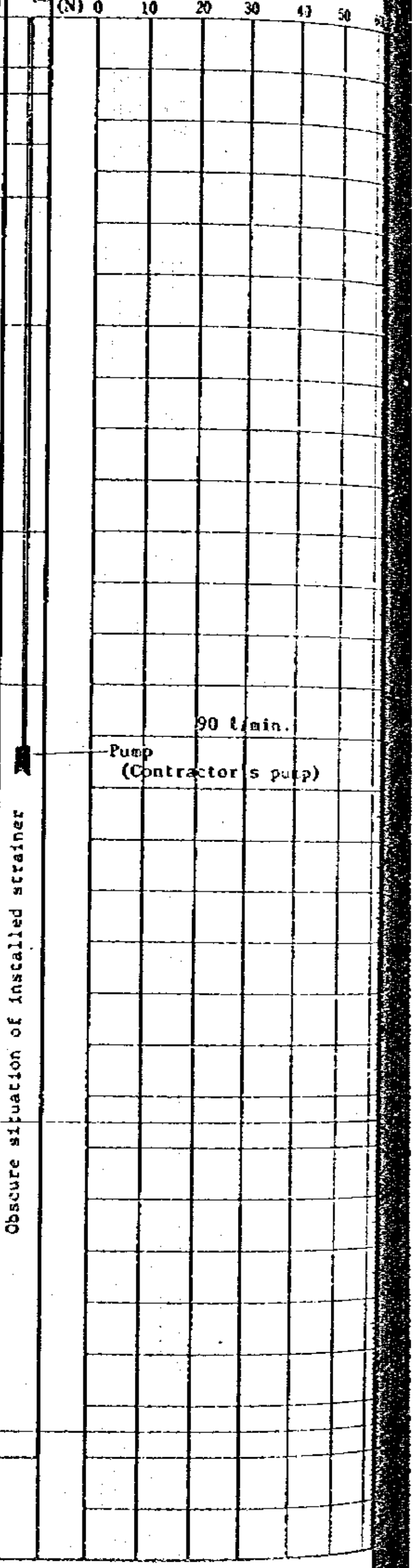
Pump 18 ℓ/min.
(Submersible pump)

Appendix-6 GEOLOGICAL RECORD OF BORING UNHCR UWWC

HOLE No. 6

PROJECT				LOCATION	Kab Cherng Holding Center, SURIN
GROUND ELEVATION	185 m	DEPTH OF HOLE	275 feet	ANGLE FROM VERTICAL	
DIAMETER OF HOLE		MACHINE	Bit-drilling mash.	DATE OF DRILLING	23 Nov. ~ 9 Dec. 1980
CORE RECOVERY		DEPTH TO GROUND WATER LEVEL IN HOLE			
By Slime observation		DRILLED BY	United Water Well Construct.	LOGGED BY	

ELEVATION (m)	DEPTH (m)	THICKNESS (m)	FIELD OBSERVATION: SOIL OR ROCK CLASSIFICATION	COLOUR	FORMATION DESCRIPTION
181.95	3.05	3.05	compacted	white to yellow	Sandy Clay
180.43	4.57	1.52		grayish brown	Shale Bed rock
177.37	7.63	3.06		reddish brown	Shale
174.32	10.68	3.05		light gray	Shale
166.70	18.30	7.62		reddish brown	Siltstone
154.52	30.48	12.18		reddish brown	Shale with some siltstone
145.35	39.65	9.17		light gray	Siltstone with some shale
119.42	65.58	25.93		grayish brown	Sandstone
101.12	83.88			reddish brown	Sandstone with some shale



CHAPTER 7 GROUND WATER DEVELOPMENT SURVEY FOR KAO-I-DANG HOLDING CENTRE AND PHANAT NIKHOM HOLDING CENTRE

7.1 Completion of the Well in Kao-I-Dang Holding Centre

The deep well made by a group of Japanese Volunteers in the premises of Kao-I-Dang Holding Centre was left unfinished after drilling.

The well was drilled with drilling bits of 8-1/2 inches from the ground surface to the depth of 12 m and then with bits of 6-1/2 inches through the bedrock on to the depth of 88 m.

A submersible motor pump was placed and operated at the depth of about 81 m in the well without timbering or other reinforcement. If the well would be left uncased the top soil layer of sandy clay of about 12 m might fall off, burying the pump in the soil.

Another problem was that run-off of rain and other types of water sometimes contaminated, the well. In order to cope with these problems, it was found necessary to insert a casing pipe into the well and to cast concrete in the gap between the casing and the top soil portion of the well wall for complete insulation.

So the well finishing work was done in the following sequence.

1) Installing a drilling machine and the supporting tripod on the well

2) Lifting the existing submersible motor pump

3) Insertion of a casing pipe of 7 inches down to the bedrock at 12 m from the ground surface with a projection of 50 cm above the ground surface.

4) Casting concrete into the gap between the well wall soil and the casing pipe for complete insulation.

5) Cleaning the well wall draining slime by air lift with a pipe connected to an air compressor inserted into the well.

6) Re-installing the submersible motor pump after re-connecting it to the pumping pipe at the original position of 81 m from the ground surface.

7) Connection of the electric wires.

Pumping was resumed on completion of the above procedures.

7.2 Ground Water Development Plan for Phanat Nikhom Holding Centre

According to an extensive survey conducted in the area from the City of Chon Buri over to Phanat Nikhom, it is true that some of the ground water tapped is salty and unfit to drink. "A Study on Water Supply Potential for PHANAT NIKHOM REFUGEE (1981)" reports that out of 20 wells surveyed 11lwells yielded water with salinity.

The location and geological classification of these deep wells are shown in Figure 7.2.1.

From the town of Phanat Nikhom toward north-west, deep wells from which saline water was sampled are distributed. The depth of the wells range from 21 m to 184 m.

The altitude of this zone is lower than 10 m above the sea level and the geological feature is the alluvial deposit consisting of clay, silt sand and gravel. The sandy and gravelly layers in it serve as a good ground water aquifers.

It is suspected that the infiltration of the sea water from the Gulf of Thailand into some of the aquifers gives salinity to well water. Or, it is possible to infer that the present plain used to be under the sea once in the course of the depositing of the alluvial layer.

The area comprising the City of Chon Buri, Town of Phanat Nikhom and the Holding Centre has a slight feature of plateau. This plateau can be supposed to be diluvium, and the water yielded from the wells made in this diluvium plateau does not contain salinity.

It is supposed that layer under this diluvium is bedrock mainly consisting in metamorphic rocks.

The said geological features for the section between the seaside City of Chon Buri and Phanat Nikhom Holding Centre passing through the Town of Phanat Nikhom, can be expressed in the conjectual geological profile as shown in Figure 7-2-2.

On the basis of the above observation the following well development plan can be made.

Shallow wells can be made in the Holding Centre and its neighbourhood to obtain the water from the aquifers in diluvium layer or from the aquifers presumed to be present at the interface between the diluvium and the eroded top of the bedrock.

And deep wells can be made expecting the yield of fissure water coming from the joints or fissures of bedrock.

The location of conceivable wells to be made is as shown in Figure 7-2-3.

The geological profile there is as shown in Figure 7-2-2.