Where,

L is a length of the weir (m)

The amount of seepage at SW-1, SW-2 and SW-3 thus obtained is tabulated in Table A5.1. Also shown in the Table is the amount (mm) of rainfall recorded at the Angat dam site. It is indicated by the number of days before the measurement was undertaken as shown below:

R_0	:	Amount of rainfall on the same day when the measurement was undertaken.
R_1	:	Amount of rainfall on the day before the measurement day.
R_2	:	Amount of rainfall on two days before the measurement day.
R_3	:	Amount of rainfall on three days before the measurement day.
R_4	:	Amount of rainfall on four days before the measurement day.
R_5	:	Amount of rainfall on five days before the measurement day.
R_6-R_{10}	:	Amount of rainfall on six days before the measurement day through ten days before the measurement day.
R_{11}-R_{20}	:	Amount of rainfall on eleven days before the measurement day through 20 days before the measurement day.
R_{21}-R_{35}	:	Amount of rainfall on 21 days before the measurement day through 35 days before the measurement day.

5.2. Analysis of Measurement Data

5.2.1. Method of Analysis

The amount of seepage measured at SW-1, SW-2 and SW-3 is believed to be originated from the following three sources:

1) Surface runoff from rainfall or seepage through mountain masses.
2) Seepage through the dyke.
3) Seepage over a long period of time through the surrounding mountain masses.

Therefore, the measured amount of seepage has to be sorted out by the above components. The measured data were, therefore, analyzed using the method of least squares.

The measured value at each weir can be expressed by the following equation:

$$Qj = a_0R_0 + a_1R_1 + a_2R_2 + a_3R_3 + a_4R_4 + a_5R_5 + a_{6-10}R_{6-10} + a_{11-20}R_{11-20} + a_{21-35}R_{21-35} + C_1(RWL-S_1) + C_2(RWL-S_2) + C_3(RWL-S_3) + C_4(RWL-S_4) + b \quad ......(1)$$

Where,

$Q_{j(1,\ 2\ \text{and}\ 3)}$ = The measured value of seepage (ℓ/sec) at each weir of SW-1, SW-2 and SW-3.

a = Coefficient for calculation of the effect of rainfall.

R_i = Amount of rainfall (mm)

a_{0R0} $\alpha_{21-35}R_{21-35}$ = Effect of rainfall on the amount of seepage.

$a_{0a1}a_2$ ..... a_{21-35} = Effect on the amount of seepage of rainfall represented by the corresponding R value, R_0, R_1, R_2 .... R_{21-35}.

C = Coefficient for calculation of the effect of seepage through the dyke.

RWL= Reservoir water level (m)

Si = Elevations assumed to cause seepage through the dyke.

Ci(RWL-Si) = Amount of seepage through the dyke.

b = Constant, representing seepage over a long period of time through the adjoining mountain masses (seepage through the dyke in stable state or seepage through the surrounding mountain masses in stable state).

In the above equation, "S" was arbitrarily set at 180(S_1), 185(S_2), 190(S_3) and 195(S_4). For instance, in case when S_3 (190) is applied to the equation, and the value of C(RWL-S_3) produces any positive (plus) number, then it means that seepage should occur with the reservoir water level at higher than EL 190 m.

5.2.2. Result of Analysis

Tables 5.1 and 5.2 show the coefficients, "a_0 - a_{21-35}", "c" and the constant "b" obtained for each weir. In applying the method of least squares, calculation was first made using all coefficients given in the table. When the result of such calculation indicated any coefficients to be negative (minus), then calculation was made again with such coefficients assumed to be zero, because negative coefficients are inconsistent with the physical phenomena. The similar calculation was then repeated until all the coefficients become zero or positive (plus) number.

It is considered appropriate to reckon the measured amount of rainfall less evaporation or other losses as the net amount of runoff. An attempt was therefore made to obtain the corresponding coefficients in the equation (1) above with the

Table 5.1 Coefficients for Calculation of Effect of Rainfall on the Amount of Seepage Through the Dyke

Weir	Coefficients for Calculation of Effect of Rainfall								
	a_0	a_1	a_2	a_3	a_4	a_5	a_{6-10}	a_{11-20}	a_{21-35}
SW-1	0.018	0.013	0.018	0.016	0.002	0.007	0.003	0	0.001
SW-2	0.02	0.026	0.025	0.026	0.023	0.019	0.013	0.009	0.003
SW-3	0.033	0.019	0.029	0.016	0.016	0.014	0.017	0.003	0.009

Table 5.2 Coefficients for Calculation of Effect of Reservoir Water Level on the Amount of Seepage Through the Dyke

Weir	Coefficients for Calculation of Effect of Reservoir Water Level				Cons-tant
	C_1	C_2	C_3	C_4	b
SW-1	0	0	0	0.975	0.044
SW-2	0	0	0.424	0	0
SW-3	0	0	0	1.332	1.103

loss deductible from the daily rainfall varied from 0 mm to 1, 3 and 5 mm. Given below is the difference between the measured and calculated amount of seepage indicated in the standard deviation.

Weir	Loss of Rainfall			
	0 mm	1 mm	3 mm	5 mm
SW-1	1.517	1.517	1.513	1.510
SW-2	1.772	1.775	1.783	1.797
SW-3	3.225	3.233	3.251	3.276

As is seen in the above table, any changes in the loss of rainfall may produce little effect on the deviation. In other words, changes in the loss of rainfall may not produce any substantial effect on the accuracy of the calculated amount of seepage, though they may cause the corresponding coefficients to vary to some extent. (Refer Figs.5.4.1 through 5.4.3 with the loss of rainfall assumed to be 0, and Figs.5.5.1 through 5.5.3 with the loss of rainfall assumed to be 5 mm.) This report, therefore, adopted the results with the loss of rainfall assumed to be zero.

Based on the result of calculating coefficient "c", it appears that the seepage at SW-1, or through the left bank abutment of the dyke, is particularly noticeable when the reservoir water level is at the elevations higher than EL.195 m, while the seepage at SW-2, or through the center of the dyke, is so when the water level is at the elevations higher than EL.190 m. And, the seepage at SW-3 located downstream of SW-1 and SW-2 seems to be noticeable when the water level is at the elevations higher than EL.195 m. It can therefore be assumed that seepage through the dyke or the adjoining mountain masses would not pass through lower elevations, but predominantly through higher elevations than EL.190 m.

Fig. 5.4.1 Comparison Between Measured and Calculated Amounts of Seepage Through the Dyke (POINT :SW-1 RAINFALL ADJUST:-0 mm DEV= 2.3)

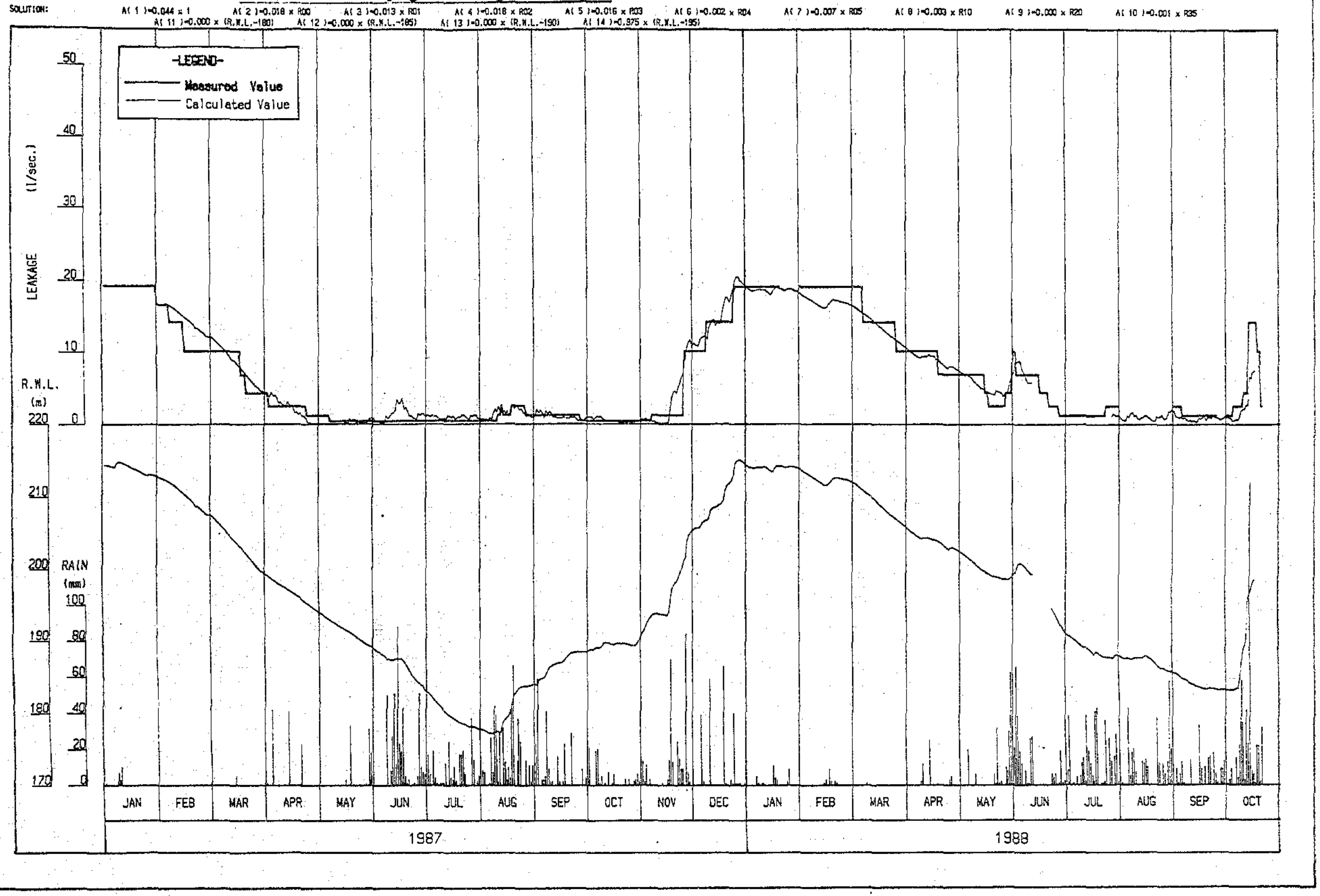


Fig. 5.4.2 Comparison Between Measured and Calculated Amounts of Seepage Through the Dyke (POINT :SW-2 RAINFALL ADJUST:-0 mm DEV= 3.14)

SOLUTION: A(1)=0.020 x R00 A(2)=0.026 x R01 A(3)=0.025 x R02 A(4)=0.026 x R03 A(5)=0.023 x R04 A(6)=0.019 x R05 A(7)=0.013 x R10 A(8)=0.009 x R20 A(9)=0.033 x R35
A(10)=0.000 x (R.W.L.-180) A(11)=0.000 x (R.W.L.-185) A(12)=0.424 x (R.W.L.-190) A(13)=0.000 x (R.W.L.-195)

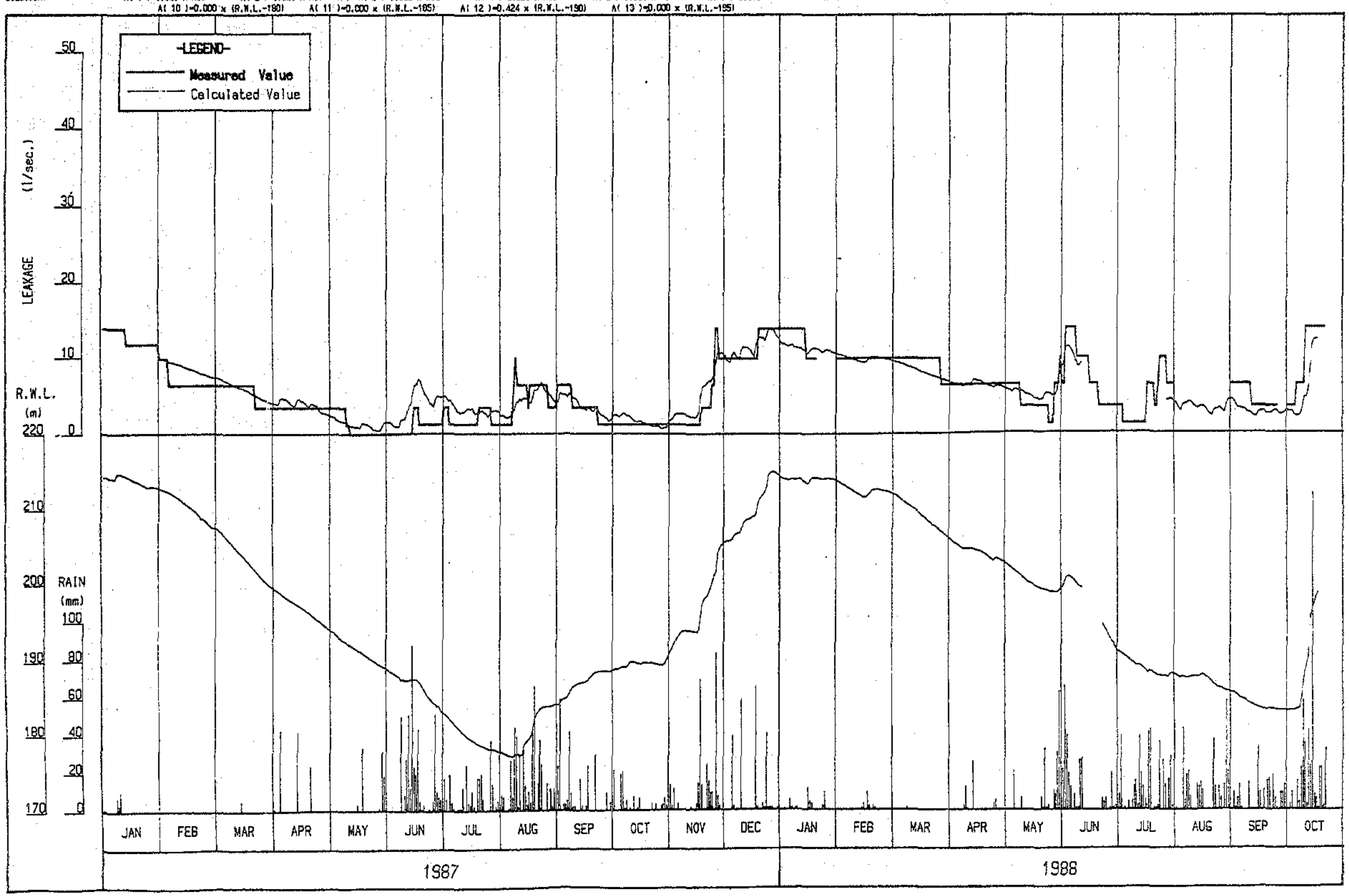


Fig. 5.4.3. Comparison Between Measured and Calculated Amounts of Seepage Through the Dyke (POINT :SW-3 RAINFALL ADJUST:-0 mm DEV= 10.4)

SOLUTION:
A(1)=1.103 x 1 A(2)=0.033 x R00 A(3)=0.019 x R01 A(4)=0.029 x R02 A(5)=0.016 x R03 A(6)=0.016 x R04 A(7)=0.014 x R05 A(8)=0.017 x R10 A(9)=0.003 x R20 A(10)=0.009 x R35
A(11)=0.000 x (R.W.L.-180) A(12)=0.000 x (R.W.L.-185) A(13)=0.000 x (R.W.L.-190) A(14)=1.332 x (R.W.L.-195)

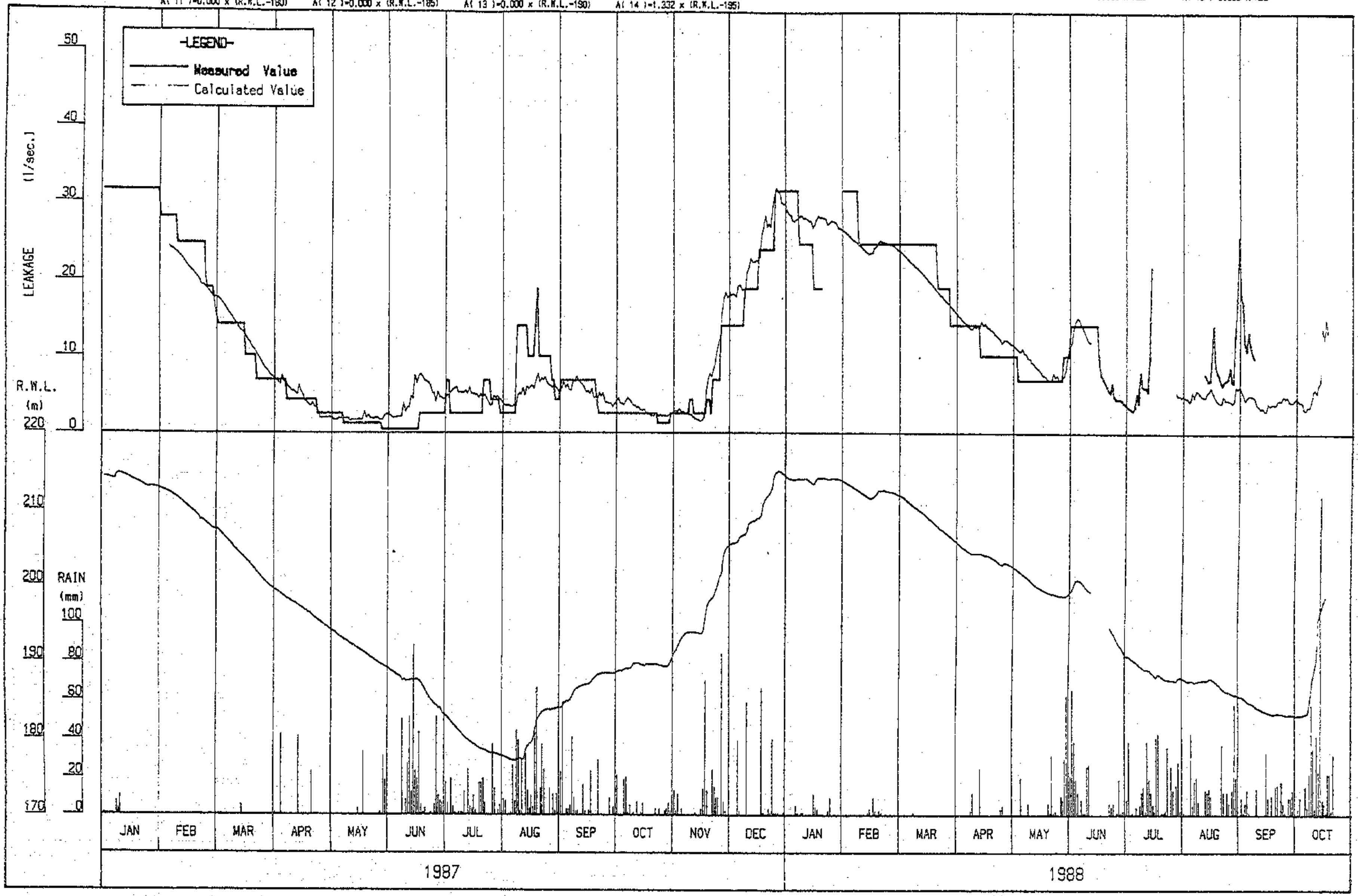


Fig. 5.5.1 Comparison Between Measured and Calculated Amounts of Seepage Through the Dyke (POINT :SW-1 RAINFALL ADJUST:-5 mm DEV= 2.28)

SOLUTION: A(1)=0.160 x 1 A(2)=0.022 x R00 A(3)=0.016 x R01 A(4)=0.021 x R02 A(5)=0.019 x R03 A(6)=0.003 x R04 A(7)=0.008 x R05 A(8)=0.004 x R10 A(9)=-0.001 x R20 A(10)=0.002 x R35
A(11)=0.000 x (R.W.L.-180) A(12)=0.000 x (R.W.L.-185) A(13)=0.000 x (R.W.L.-190) A(14)=0.969 x (R.W.L.-195)

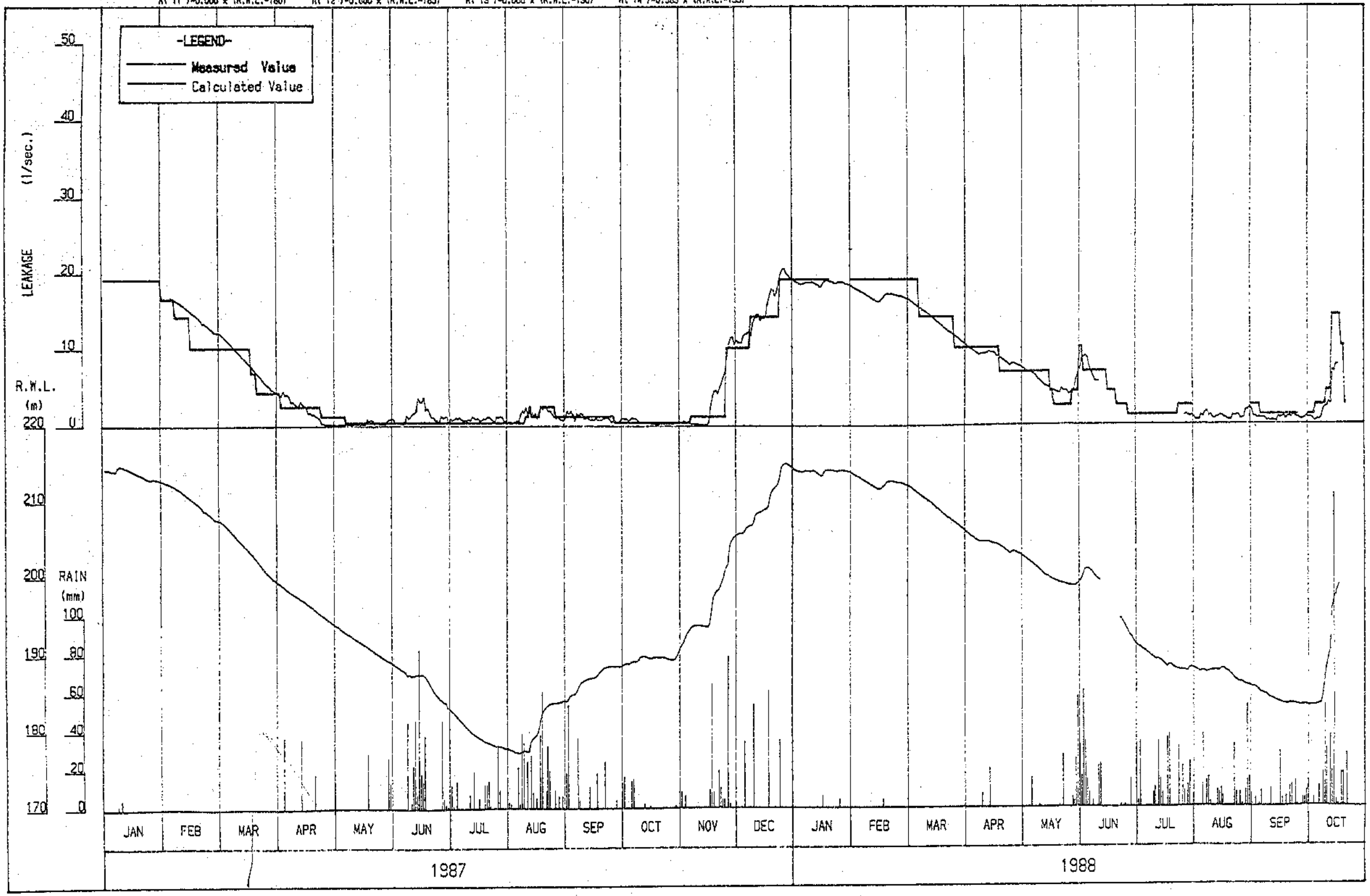


Fig. 5.5.2 Comparison Between Measured and Calculated Amounts of Seepage Through the Dyke

(POINT :SW-2 RAINFALL ADJUST:-5 mm DEV= 3.23)

SOLUTION:

A(1)=0.026 x R00 A(2)=0.031 x R01 A(3)=0.030 x R02 A(4)=0.030 x R03 A(5)=0.028 x R04 A(6)=0.023 x R05 A(7)=0.016 x R10 A(8)=0.011 x R20 A(9)=0.006 x R35

A(10)=0.000 x (R.W.L.-180) A(11)=0.000 x (R.W.L.-185) A(12)=0.425 x (R.W.L.-190) A(13)=0.000 x (R.W.L.-195)

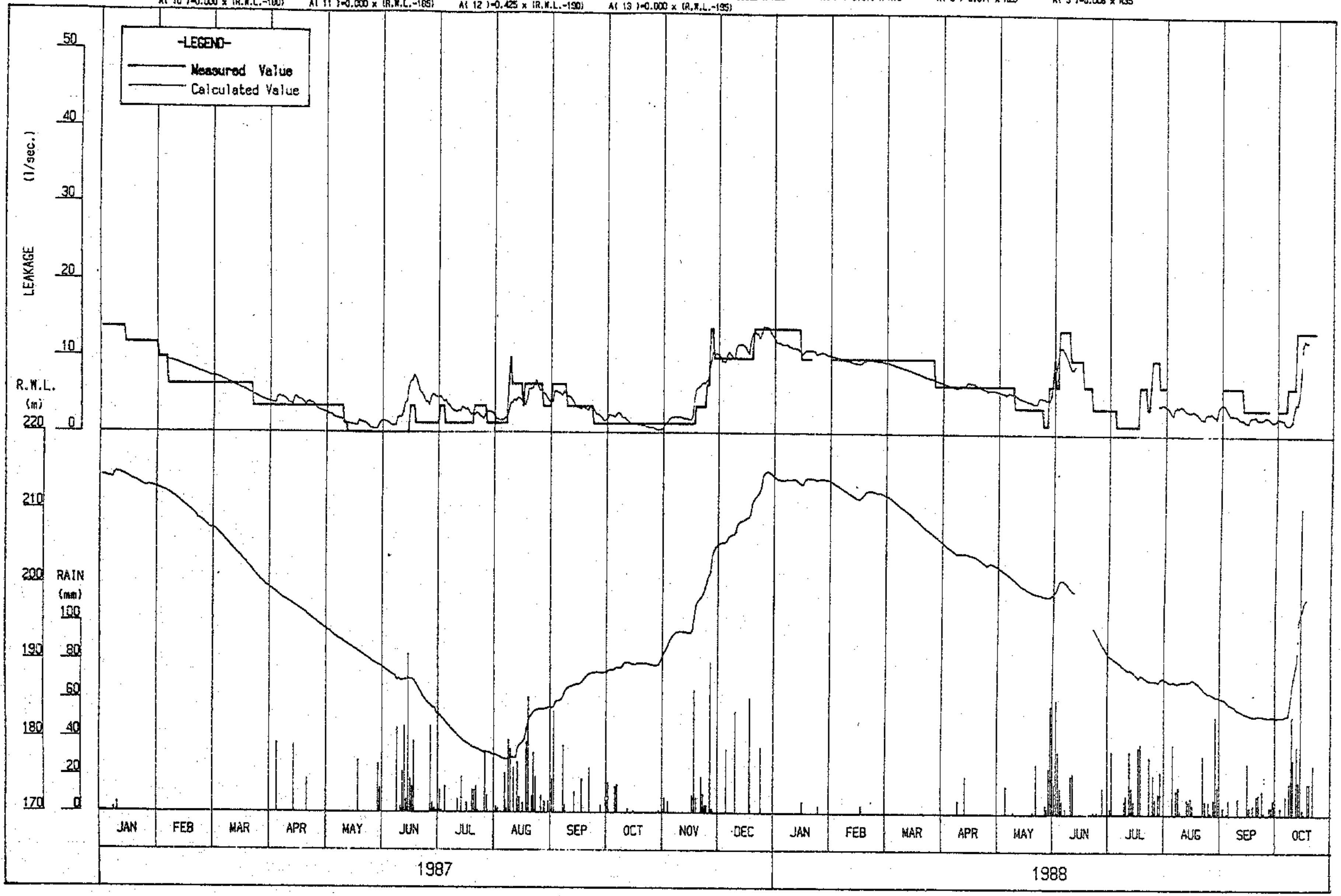
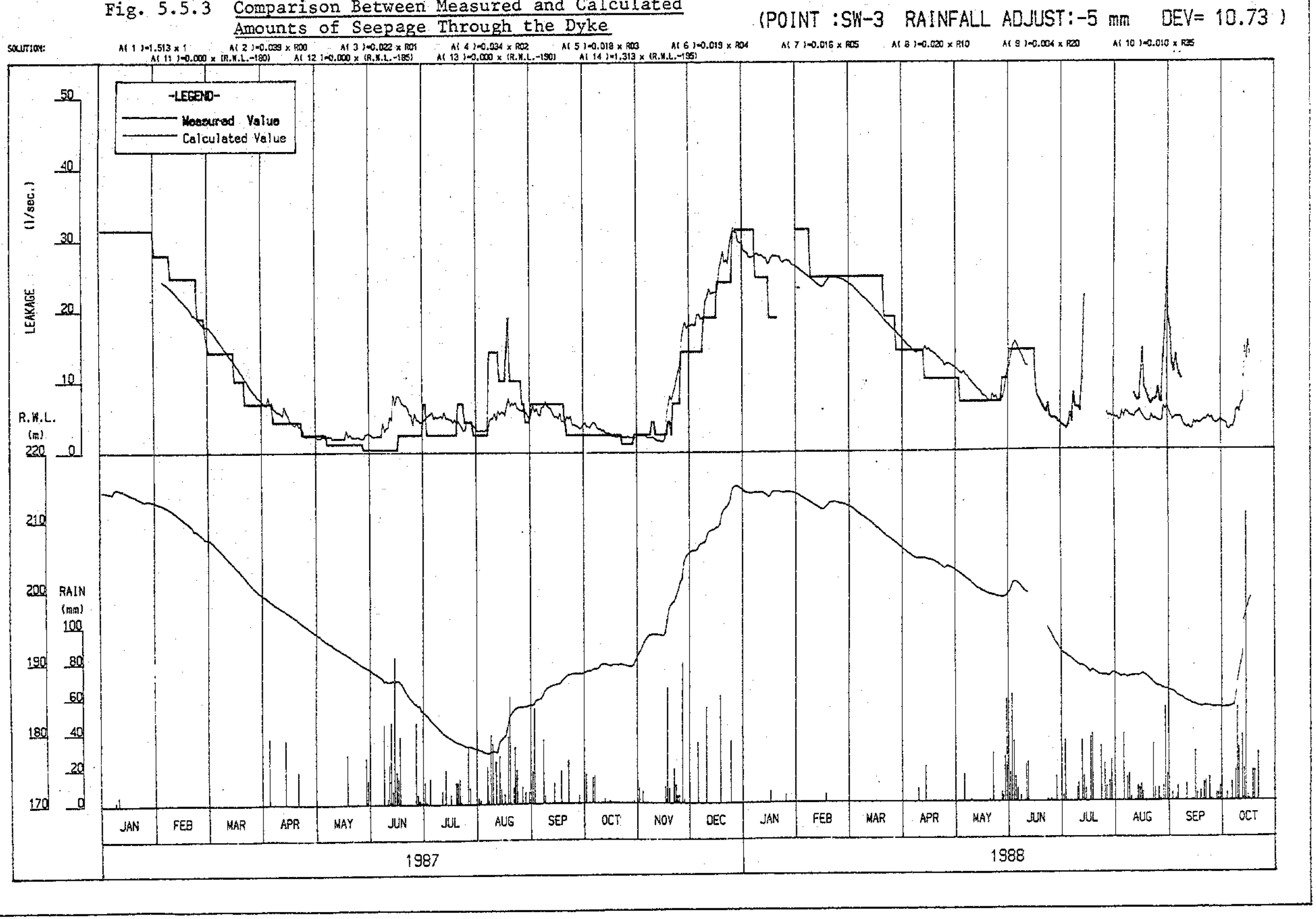


Fig. 5.5.3 Comparison Between Measured and Calculated Amounts of Seepage Through the Dyke



On this basis, the amount of seepage measured at the three weirs was sorted out by three different sources; runoff from rainfall, seepage through the dyke and seepage over a long period of time through the adjoining mountain masses, and the effect of each source on the amount of seepage was analyzed.

5.2.3. Amount of Seepage

Tabulated in Tables 5.1 and 5.2 are the coefficients, a_0, a_1, a_2, a_3, a_4, a_5, a_{6-10}, a_{11-20}, a_{21-35}, c_1, c_2, c_3, c_4, and the constant b, obtained by using the equation (1). These were obtained by varying a value of "S" in the equation, so as to make the difference between the calculated and measured amounts minimum.

The amount of seepage by different sources; runoff from rainfall and seepage through the dyke, was calculated by using the above coefficients. The result of the calculation is given in Table A5.2.

Figs. 5.4.1. through 5.4.3 show the comparisons between the measured values and the calculated values obtained using the equation (1) by applying the coefficients given in Tables 5.1 and 5.2. As illustrated in these Figures, the calculated values are relatively identical with the measured values except part of those at the SW-3 weir. In these Figures, the value S (elevations assumed to cause seepage through the dyke) was set at EL.195, 190 and 195 for SW-1, SW-2 and SW-3, respectively.

The result of the calculation is as shown below:

(1) Table 5.3 shows a comparison between the calculated amount and the measured amount of leakage by month based on the data taken during months in 1987 and 1988.

Table 5.3 Comparison of Measured and Calculated Amount of Seepage Through the Dyke

(Data Taken During Months in 1987 and 1988)

(m^3)

	SW-1				SW-2				SW-3			
	Measured	Calculated			Measured	Calculated			Measured	Calculated		
		Total	Influenced by			Total	Influenced by			Total	Influenced by	
			Rain	RWL			Rain	RWL			Rain	RWL
JAN	51,586	50,361	588	49,773	33,818	29,735	2,477	27,258	68,913	74,230	3,440	70,790
FEB	31,445	35,750	18	35,732	16,870	20,673	64	20,609	60,557	51,498	168	51,330
MAR	20,990	21,974	41	21,933	14,485	15,290	111	15,179	29,637	32,930	106	32,824
APR	6,128	5,576	878	4,698	8,891	9,899	2,517	7,382	11,445	11,547	2,426	9,121
MAY	1,663	691	584	107	2,873	3,388	1,636	1,752	3,747	4,993	2,047	2,946
JUN	1,218	3,508	3,404	104	2,226	9,680	9,680	0	3,722	12,023	9,172	2,851
JUL	1,259	2,569	2,462	107	5,123	8,427	8,427	0	9,887	13,341	10,395	2,946
AUG	3,677	3,961	3,854	107	12,675	11,523	11,523	0	23,764	14,478	11,532	2,946
SEP	2,970	2,669	2,565	104	9,361	9,236	9,236	0	13,952	13,595	10,744	2,851
OCT	1,259	1,313	1,206	107	3,187	4,236	4,213	23	6,023	8,076	5,130	2,946
NOV	6,711	8,810	2,099	6,711	9,562	12,771	5,423	7,348	14,155	17,617	5,749	11,868
DEC	38,481	41,377	2,286	39,091	30,482	30,817	8,201	22,616	58,049	65,173	8,986	56,187
Total	167,387	178,559	19,985	158,574	149,553	165,675	63,508	102,167	303,851	319,501	69,895	249,606

The maximum amount of seepage through the dyke during the period (January 1987 to October 1988) in which the measurement data are available is estimated as follows:

SW-1 : 1.14 m^3/min. ≑ 19 ℓ/sec.
SW-2 : 0.66 m^3/min. ≑ 11 ℓ/sec.
SW-3 : 1.80 m^3/min. ≑ 30 ℓ/sec.

(2) Table 5.4 shows the average amount of seepage by different season (wet and dry) and sources of influence (rainfall and the reservoir water level).

Table 5.5 shows the maximum amount of seepage during the period in which the measurement data are available.

Table 5.4 Average Amount of Seepage through the Dyke

(m^3/min.)

Season	Weir	Seepage probably from the reservoir	Runoff probably from rainfall	Total
Dry (Dec. thru May)	SW-1	0.5759 (97)	0.0167 (3)	0.5926
	SW-2	0.3608 (86)	0.0571 (14)	0.4179
	SW-3	0.8493 (92)	0.0653 (8)	0.9146
Wet (June thru Nov.	SW-1	0.0275 (32)	0.0593 (68)	0.0868
	SW-2	0.0280 (13)	0.1846 (87)	0.2126
	SW-3	0.1005 (33)	0.2006 (67)	0.3011
Total	SW-1	0.3017 (89)	0.0380 (11)	0.3397
	SW-2	0.1944 (62)	0.1208 (38)	0.3152
	SW-3	0.4749 (78)	0.1330 (22)	0.6079

Note: The parenthesized shows percentage of the total.

Table 5.5 Maximum Amount of Seepage Through the Dyke During the Period in which the Measurement Data are Available

Weir	Seepage from the reservoir (D)	Sum of seepage from the reservoir and runoff from rainfall (T)
	m^3/min	m^3/min
SW-1	1.14	1.24
SW-2	0.66	0.83
SW-3	1.80	1.91

Note: The above figures are non-coincidental.

(3) In the dry season, the seepage at SW-1 and SW-3 is predominantly influenced by the reservoir water level. The seepage at SW-2, too, is influenced by the water level, but at a lesser degree (86%). In general, the seepage is influenced by the water level in the dry season regardless of the measurement weir, as shown in the following table:

Weir	Seepage in the Dry Season: Influenced by the reservoir water level (as % of Total)	Influenced by rainfall (as % of total)
SW-1	97	3
SW-2	86	14
SW-3	92	8

(4) While in the wet season, the seepage is mostly influenced by rainfall as shown in the following table:

Weir	Seepage in the Wet Season: Influenced by the the reservoir water level (as % of Total)	Seepage in the Wet Season: Influenced by rainfall (as % of total)
SW-1	32	68
SW-2	13	87
SW-3	33	67

(5) The influence on the seepage of the reservoir water level and rainfall throughout the year is as shown below:

Weir	Influenced by the reservoir water level (as % of Total)	Influenced by rainfall (as % of total)
SW-1	89	11
SW-2	62	38
SW-3	78	22

(6) Table 5.4 shows that the sum of the amount of seepage at SW-1 and SW-2 (0.3397 + 0.3152 = 0.6549 m^3/min.) is almost identical with the amount of seepage at SW-3 (0.6079 m^3/min.). SW-3 being located downstream of SW-1 and SW-2, the measured amount of seepage at the weir should represent the seepage from the left bank (SW-1) and the center (SW-2) of the dyke and other sources. Since the sum of the amount of seepage at SW-1 and SW-2 is almost identical with the amount of seepage at SW-3, it can be assumed that practically no sources other than SW-1 and SW-2 would contribute to seepage through the dyke.

(7) Annual loss of water by seepage through the dyke measured at SW-1 and SW-2 is estimated as follows:

SW-1 : 159,000 m^3/year
SW-2 : 102,000 m^3/year
Total : 261,000 m^3/year

This amount of loss corresponds to only about 56,100 kWh in terms of annual generation.

(8) The amount of seepage influenced by the reservoir water level can be expressed, using coefficients given in Table 5.2, as follows:

Seepage at SW-1:

$0.975 \times (RWL - 195) + 0.044$ ℓ/sec.

$= 58.5\ (RWL - 195) + 0.003\ m^3/min.$

Seepage at SW-2:

$0.424 \times (RWL - 190)$ ℓ/sec.

$= 25.44\ (RWL - 190)\ m^3/min.$

Seepage at SW-3:

$1.332 \times (RWL - 195) + 1.103$ ℓ/sec.

$= 79.92\ (RWL - 195) + 0.066\ m^3/min.$

Where,

RWL = Reservoir water level

190, 195 = S value, or elevations assumed to cause seepage through the dyke.

Table 5.6 shows the amount of seepage assumed at different elevations of the reservoir water by measurement weir. In the Table, the sum of the amount at SW-1 and SW-2 is also shown to compare with the amount at SW-3 as discussed in (6) above.

Table 5.6 Amount of Seepage Assumed at Different RWLs.

(ℓ/sec.)

RWL	SW-1	SW-2	SW-1+SW-2	SW-3
190	0	0	0	0
195	0.04	2.12	2.16	1.10
200	4.92	4.24	9.16	7.76
205	9.79	6.36	16.15	14.42
210	14.67	8.48	23.15	21.08
215	19.54	10.60	30.14	27.74
220	24.42	12.72	37.14	34.40

5.2.4. Studies of Seepage from Measured Water Level in the Boreholes Provided Downstream of the Dyke

Investigation of ground water level was conducted using seven boreholes provided downstream of the dyke for this Study.

Tabulated in Table 5.7 is the result of measurement of water levels in the boreholes made in September and October, 1988.

In September 1988, the reservoir water level was generally stable throughout the month, and so was the water level in the boreholes. In October 1988, the reservoir water level rose by 10.22 m from EL 183.65 to EL.193.87 during six days from October 8 to 14. The water level in the boreholes, too, rose in keeping pace with the increase in the reservoir water level.

The locations of boreholes are as shown in Fig.5.3. No.1 through No.4 boreholes are provided closer to the left bank of the dyke, No.5 and No.6 boreholes closer to the center of the dyke, and No.7 borehole closer to the right bank of the dyke. The measurement records are, however, available only for the water level in Nos. 2, 4, 5 and 6 boreholes.

Table 5.7 Water Level in Boreholes Downstream of the Dyke

(EL m)

Date of Measure-ment	Borehole						RWL
	DDH -1	DDH -2	DDH -3	DDH -4	DDH -5	DDH -6	
	(190.641)	(184.351)	(177.465)	(149.067)	(136.379)	(134.682)	
Sept. 1988							
2	—	164.451	—	146.497	—	—	186.64
6	—	164.561	—	146.647	—	—	184.87
8	—	164.571	—	146.087	—	—	184.66
10	—	164.621	—	146.097	—	—	184.21
12	—	164.531	—	146.037	—	—	183.98
14	—	164.381	—	145.927	—	—	183.75
23	—	163.751	—	145.267	119.879	—	183.43
25	—	163.731	—	145.322	120.104	118.928	183.30
27	—	163.724	—	145.279	120.094	120.620	183.29
Oct. 1988							
3	—	163.351	—	145.224	120.085	118.467	183.28
4	—	163.364	—	145.265	120.100	118.483	183.26
5	—	163.370	—	145.274	120.151	118.491	183.23
6	—	163.379	—	145.281	120.168	118.494	183.40
7	—	163.381	—	145.282	120.178	118.503	183.45
8	—	163.384	—	145.286	120.186	118.509	183.65
9	—	163.39	—	145.288	120.207	118.510	185.39
10	—	163.394	—	145.298	120.191	118.508	187.49
11	—	164.118	—	145.345	120.200	118.631	188.39
12	—	165.303	—	145.688	120.227	119.027	189.00
13	—	166.018	—	145.969	120.765	119.063	189.46
14	—	167.456	—	148.067	120.477	119.234	193.87
15	—	—	—	—			

Note: The parenthesized figures represent elevations at the borehole mouth.

Some studies were made on the leakage problem, using the result of measurement of the water level in these four boreholes.

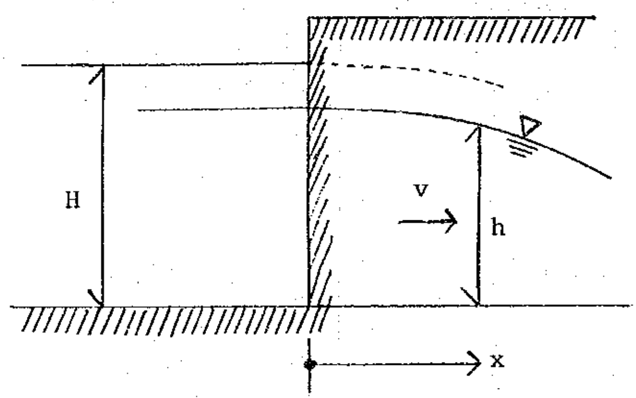
In analysing the result of measurement, the following basic equations were used:

$$\lambda \frac{\partial h}{\partial t} + \frac{\partial hv}{\partial x} = 0 \quad \ldots\ldots \text{(Equation of continuity)}$$

$$v = -k\frac{\partial h}{\partial x} \quad \ldots\ldots\ldots\ldots \text{(Darcy's equation of motion)}$$

Where,

h = Water level at an arbitrary time at an arbitrary point "x" on the x-axis running through the dam (mountain masses) to the downstream direction as illustrated in the Figure shown below:



v = Water velocity at the point "x".

λ = Porosity in the mountain masses (%)

k = Coefficient of permeability

When assuming "h" has no substantial difference from "H" under condition of t=0, and if neglecting second order, then the above two equations can be replaced with the following single equation:

$$\frac{\partial h}{\partial t} \doteqdot \frac{kH}{\lambda} \cdot \frac{\partial^2 h}{\partial x^2}$$

Now, assuming that the reservoir water level would rise at the rate of $Ae^{\mu t}$ with time when $x=0$, and "h" would come to zero when $x=\infty$, then a particular solution of the above equation can be obtained as below:

$$h = Ae^{\mu t} e^{-\sqrt{\frac{\lambda\mu}{kH}}\,x} \quad \cdots\cdots\cdots\cdots \text{(2)}$$

The problem of seepage through the dyke can be dealt with by using the above equation.

Given below is the change in the water level in the reservoir and the boreholes with a lapse of time (from t=0 to t=6 days):

Change in Water Level in the Reservoir and Boreholes

(EL m)

Water Level	Time	DDH-2	DDH-4	DDH-5	DDH-6
Reservoir	t=0 (Oct.8)	.... 183.65			
	t=6 days (Oct.14)	.... 193.87			
Boreholes	t=0 (Oct.8)	163.384	145.286	120.186	118.509
	t=6 days (Oct.14)	167.456	148.067	120.477	119.234

When the reservoir water level at t=0 and t=6 days is indicated in R_0 and R_6, and the borehole water level at t=0 and t=6 days in W_0 and W_6, the unknown figure of water depth (Y) from R_0 to the impermeable layer face under conditions of x=0 and t=0 can be expressed as given below:

$$Y = A$$

$$Y + (R_6 - R_{10}) = Ae^{6\mu}$$

$$Y - (R_0 - W_0) = Ae^{-L\sqrt{\frac{\lambda\mu}{kH}}}$$

$$Y - (R_0 - W_6) = Ae^{6\mu}e^{-L\sqrt{\frac{\lambda\mu}{kH}}}$$

Where,

L = Distance from x=0 to the boreholes.

From the above, values of $e^{6\mu}$, $e^{-L\sqrt{\frac{\lambda\mu}{kH}}}$ and "A" can be obtained as follows:

Borehole	$e^{6\mu}$	$e^{-L\sqrt{\frac{\lambda\mu}{kH}}}$	A
DDH-2	1.3163	0.3906	32.9
DDH-4	1.2015	0.2619	51.7
DDH-5	1.1600	0.0285	65.1
DDH-6	1.1497	0.0669	69.6

Given that L in $e^{-L\sqrt{\frac{\lambda\mu}{kH}}}$ is uniformly 150 m (since the four boreholes are located at nearly the same distance from the dyke), the ratio of k/λ (coefficient of permeability/porosity of mountain masses) at each borehole can be obtained as given below:

Borehole	k/λ
DDH-2	0.041 cm/sec.
DDH-4	0.0086 cm/sec.
DDH-5	0.00079 cm/sec.
DDH-6	0.00119 cm/sec.

Note: Unit of k is converted from m/day to cm/sec.

If the value λ is regarded constant, the value k (coefficient of permeability) should have direct relationship with the amount of seepage. It can, therefore, be construed from the above table that the seepage through the dyke is most in the direction to DDH-2, and least in the direction to DDH-5 and DDH-6, while in between in the direction to DDH-4.

As shown in Fig.5.3, the DDH-2 borehole is located close to the left bank abutment of the dyke, and the DDH-4 borehole is distant from DDH-2 toward the center of the tyke, but both are on the left side of the seepage measurement weir SW-1. The DDH-5 and DDH-6 boreholes are closer to the center of the dyke, and nearby the SW-2 weir.

From the above standpoints, it is considered pertinent to conclude that the source of seepage would be mostly through the mountain masses on the left bank abutment of the dyke, or on the left side of the SW-1 weir.

5.2.5. Relations between Borehole Water Level and Reservoir Water Level and Rainfall

Similarly to the studies made in Subsection 5.2.1, the water level in the boreholes located downstream of the dyke can be calculated using data of rainfall and the reservoir water level.

The borehole water level can be calculated by the following equation:

$$W = a_0R_0 + a_1R_1 + a_2R_2 + a_3R_3 + a_4R_4 + a_5R_5 + a_{6-10}R_{6-10} + a_{11-20}R_{11-20} + a_{21-35}R_{21-35} + C(RWL-BEL) + b \quad \ldots\ldots\ldots\ldots \quad (3)$$

Where,

W = Borehole water level (EL m)

BEL = Elevation of the bottom of the borehole (EL m)

a,R,C, RWL and b = Same as given in Eq.(1) in Subsection 5.2.1.

The values of W, R, RWL and BEL are known parameters available from the measurement. Using such known figures, it is possible to obtain coefficients in Equation (3) in similar manner to the case of Equation (1). Shown in Table 5.8 are the coefficients thus obtained. Figs. 5.6.1 through 5.6.4. show the comparison between the measured and the calculated values of the borehole water level.

5.2.6. Recommendation for Rehabilitation Work for Seepage through the Dyke

Judging from the results of analyses in Subsection 5.2.3.(1) through (8) and in Subsection 5.2.4, it is considered adequate to limit the grouting work, if implemented, to a section around the left bank abutment toward the center of the dyke at the elevations higher than EL.190-195 m.

However, the loss of water by seepage through the dyke is so small that the advantage of grouting work to stop the seepage would not be appreciable from the economical point of view.

Should there be any indication that the amount of seepage will increase in the future, it is necessary to deal with the seepage problem not from the economical standpoint, but from the standpoint of securing the stability of the dyke.

Table 5.8 <u>Coefficients for Calculation of Water Levels in the Boreholes Downstream of the Dyke</u>

Bore-hole	Coefficients for Calculation of Effect of Rainfall								
	a_0	a_1	a_2	a_3	a_4	a_5	a_{6-10}	a_{11-20}	a_{21-35}
DDH-2	0.011	0.002	0.012	0.020	0.019	0.025	0.018	0.020	0.008
DDH-4	0	0.002	0	0.003	0.012	0.035	0.018	0.014	0.004
DDH-5	0.002	0	0.001	0	0	0	0	0	0
DDH-6	0.001	0	0.003	0.001	0.011	0.001	0	0	0

Bore-hole	Coefficient for Calculation of Effect of Reservoir Water Level		BEL
	C	b_m	m
DDH-2	0	159.729	154.351
DDH-4	0	142.828	119.067
DDH-5	0	120.102	106.379
DDH-6	0	118.424	104.682

Fig. 5.6.1 Comparison Between Measured and Calculated Values of Borehole Water Levels

(BO.NO:DYK-2)

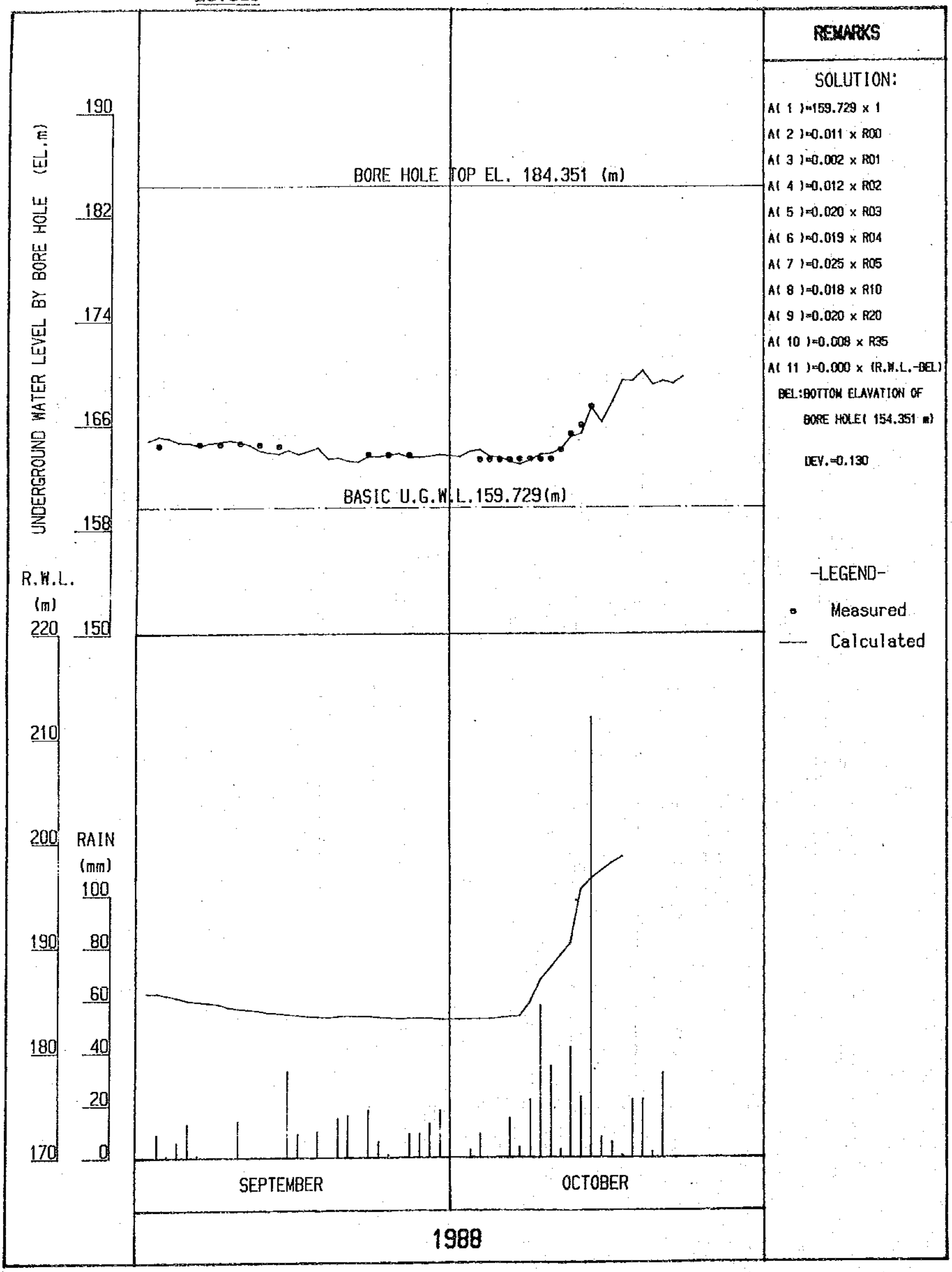


Fig. 5.6.2 Comparison Between Measured and Calculated Values of Borehole Water Levels (BO.NO:DYK-4)

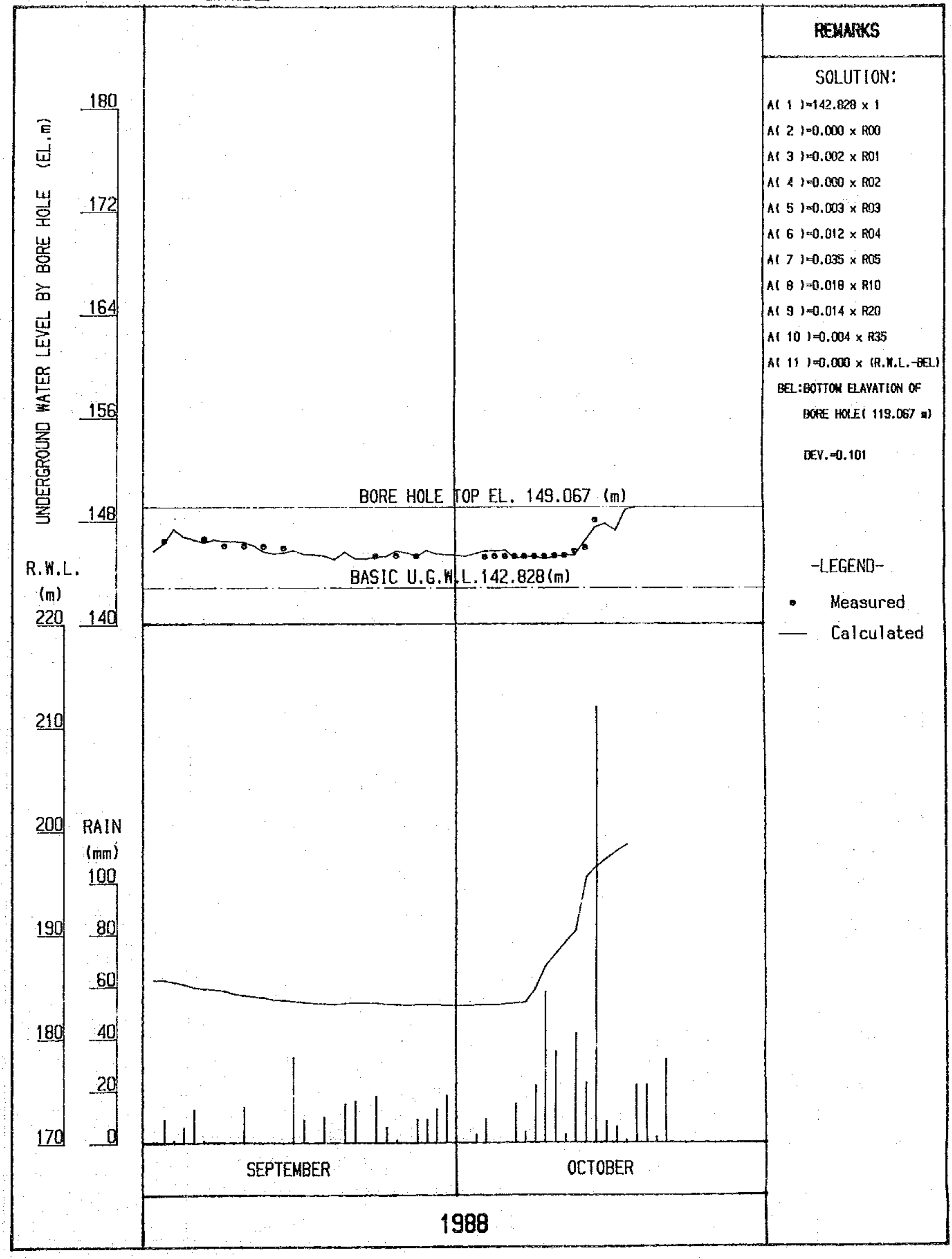


Fig. 5.6.3 Comparison Between Measured and Calculated Values of Borehole Water Levels (BO.NO:DYK-5)

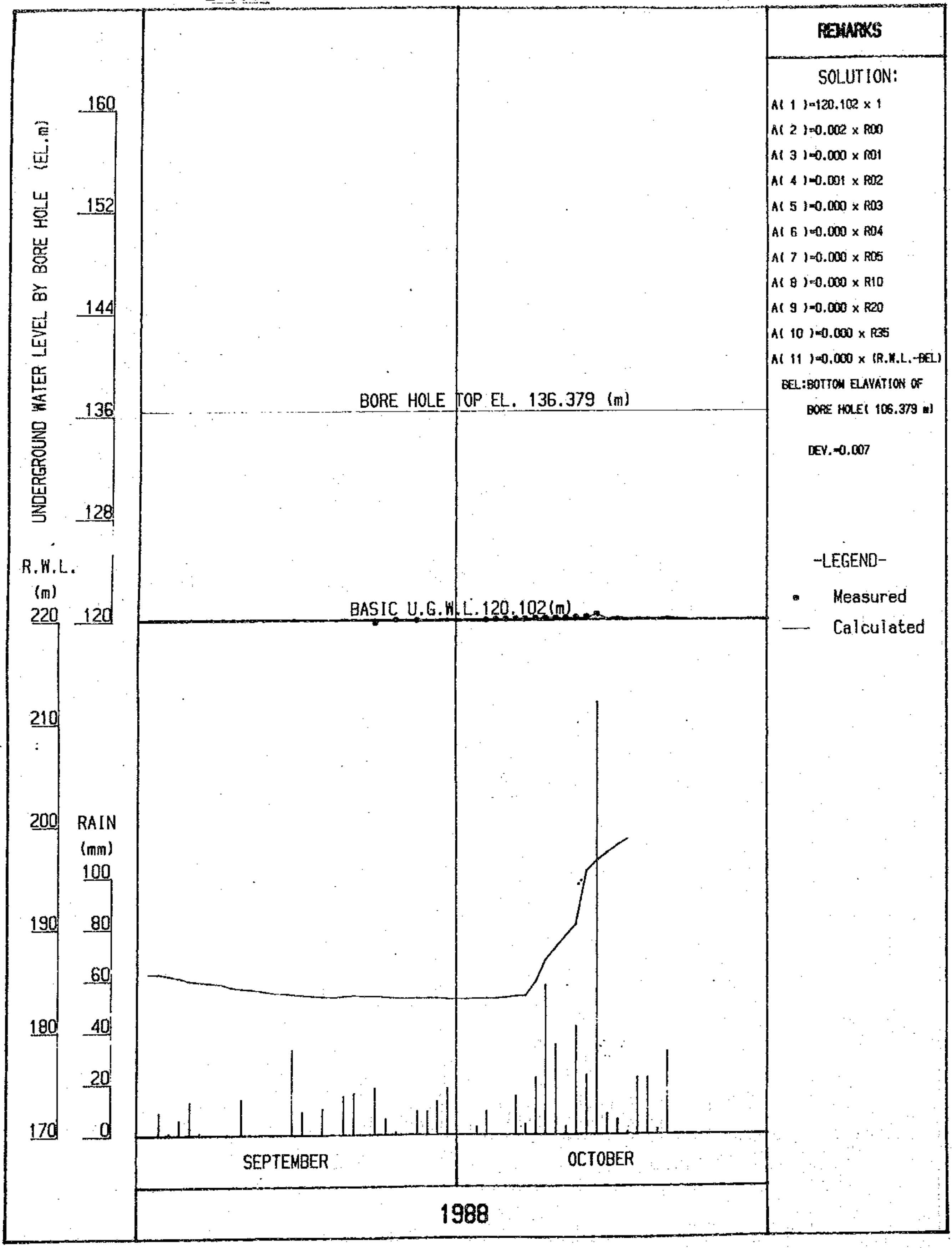
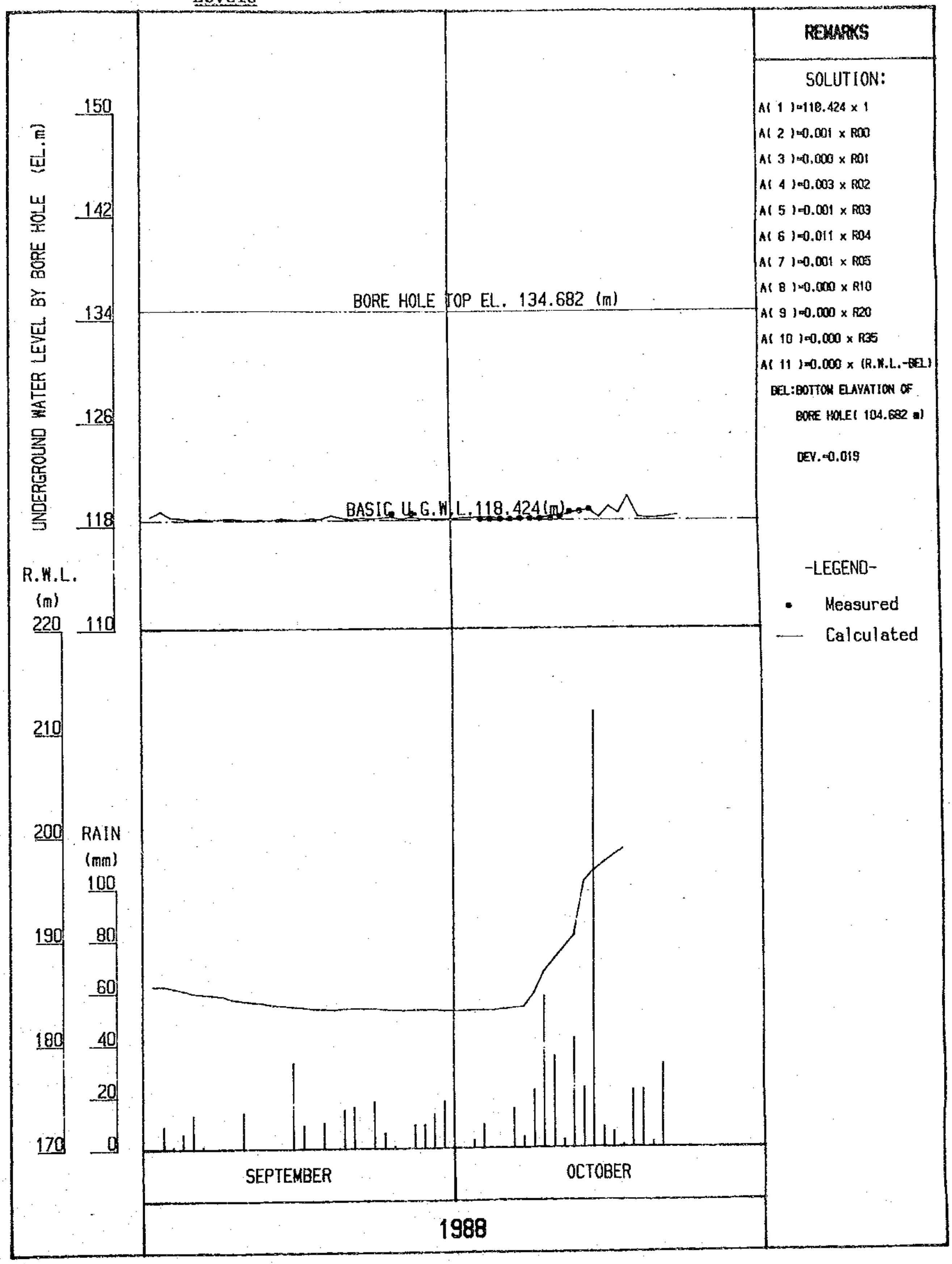


Fig. 5.6.4 Comparison Between Measured and Calculated Values of Borehole Water Levels (BO.NO:DYK-6)



Under the present condition, it appears that seepage is in the stable state with no prospect of augmentation with time in the future. In fact, no more seepage is being observed at the right bank abutment which was once leaking immediately after the dyke was built.

However, there is a possibility that the amount of seepage will increase in the future, should there occur a large intensity earthquake or an unusually large natural phenomenon that would break the present state of equilibrium.

Therefore, it is very essential to continue monitoring the seepage on a long-term basis so as to have at all times an accurate knowledge of any changes in the present condition, though it is not necessary to take an immediate action for repair.

6. REHABILITATION PLANS

6. Rehabilitation Plans

At the inception, the Study was directed to carry out the following work to determine the scope of rehabilitation project:

(1) Investigation to check if the spillway capacity is adequate.

(2) Investigation to check if the dam and the dyke is safe enough against sliding.

(3) Investigation to check if it is necessary to implement any measure to stop seepage of water through the dyke.

(4) Investigation to check if it is necessary to implement any protective measure against possible landsliding at the ex-batcher plant site.

(5) Investigation to check if it is necessary to implement measures to stop leakage of water from the penstock.

As to the item (1) above, it was concluded that, under the present circumstances, no immediate rehabilitation plan is necessary to be worked out as discussed in detail in Section 2, because the spillway capacity, though not sufficient enough, satisfies the requirement even of the respective Japanese standards.

As to the item (2), the stability analysis as discussed in detail in Section 3 indicates that there may be cases where the safety factor of the dam against sliding could be smaller than 1.0 in the event when an earthquake of 150 gal equivalent to the design condition would occur, but such zone as with a safety factor smaller than 1.0 may be limited only to the portion close to the surface, and may not have any serious effect on the dam stability. The conclusion from the analysis is that it is not necessary to work out any rehabilitation plan until there occurs such a substantially large kh earthquake that may cause any surface layer sliding.

As to the item (5), it is believed mandatory to make repair as discussed in Section 10, but the investigation had to be excluded from the scope of work because circumstances did not permit any internal inspections with the penstock being dewatered. However, the rehabilitation plan must be established anyhow, and it should be worked out whenever the internal inspections are made possible.

In this section, therefore, the items (3) and (4) only are discussed to work out the rehabilitation plans.

6.1. Seepage through the Dyke

6.1.1. Rehabilitation Plans

The amount of seepage currently through the dyke is estimated to be about 261,000 m^3 a year, and the would-be resultant loss of power production is estimated to be only about 56,100 kWh a year. Besides, it appears that the seepage is in the stable condition with no prospect of augmentation in the immediate future.

Solutions to be taken as measures for this seepage problem are therefore either of the two mentioned below:

1) To remain the things uncorrected, but to continue monitoring the seepage until there arises a sign of any increase in the amount of seepage, and take action for an appropriate measure when such time has come.

2) To reduce the amount of seepage by providing grouting work in a section around the left bank abutment toward the center of the dyke at the elevations higher than EL.190-195 m. The grouting should be provided not on the embankment but on the mountain masses.

The grouting work may prove more effective and economical, if it is performed by using the existing drain tunnel provided on the left bank side of the dyke at EL.175 m.

It should be noted, however, that it is very difficult to reduce substantially such amount of seepage as currently estimated (based on the permeability coefficient of mountain masses of 10^{-4}/sec.), by means of grouting in an ordinary manner, and it cannot be avoidable to expend a great amount of money in attaining the satisfactory results.

It is, therefore, considered recommendable under the present circumstances to adopt the first solution, that is, to continue monitoring of seepage through measurement of quantity and to test the water quality until any conspicuous change is confirmed, and to take action for an appropriate measure when such time has come.

6.1.2. Rehabilitation by Grouting

As mentioned above, it is not considered critical to leave the seepage problem uncorrected, but in order for the owner to take future action for the second solution, an attempt was made to make a cost estimate of the grouting work as given below:

(1) Grouting is to be made at intervals of 3 m in a section around the left bank abutment toward the center of the dyke as shown in Fig. 6.1.

(2) Grouting is to be done, by using the existing dyke drain tunnel, up to the elevation of 218 m.

(3) Grouting hole is to be of a 46 mm diameter. Grouting work is to be done by staged method with each stage to a depth of 5 m in principle.

(4) The maximum grouting pressure of 5.0 kg/cm^2 is to be used for injection of materials of cement milk with concentration ratio of 1 (cement) : 2 (water) for starting time and 1 : 1 for finishing time.

The cost estimate of the grouting work is shown in Table 6.2.

Fig. 6.1 Grouting for Prevention of Seepage Through the Dyke (Left Bank Abutment)

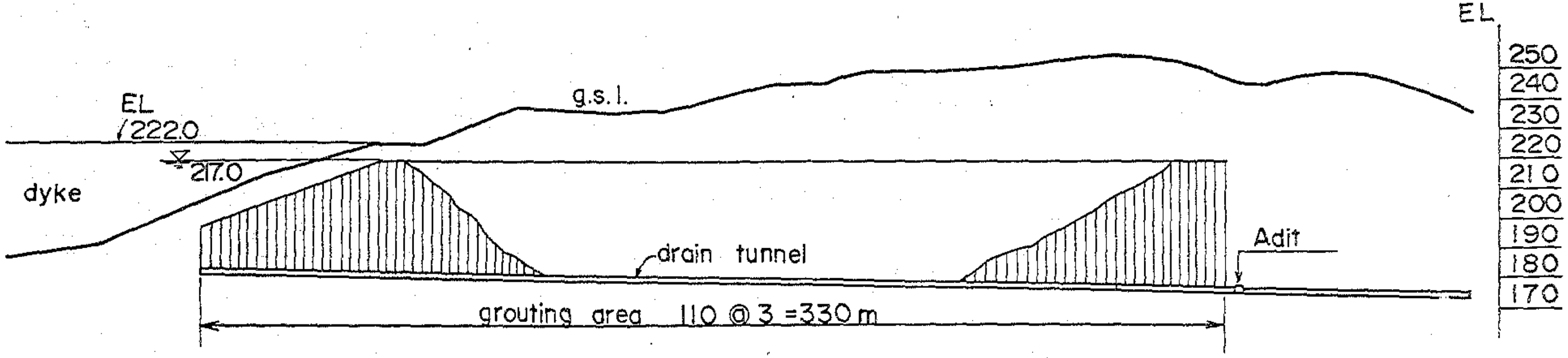


Table 6.1 Cost Estimate of the Grouting Work

Description	Specification	Unit	Quantity	Unit Price (Peso)		Amount (Peso)		
				F . C	L . C	F . C	L . C	Total
1. Boring (Non-core)	ϕ56	m	2,770	924	726	2,559,480	2,011,020	4,570,500
2. 〃 (Core recovery)	ϕ56	m	900	1,188	792	1,069,200	712,800	1,782,000
3. Cement Grouting		ton	560	350	2,898	196,000	1,622,880	1,818,880
4. Mortal Grouting		m^3	50	674	2,739	33,700	136,950	170,650
Total					Peso	3,858,380	4,483,650	8,342,030
					US$(Equiv)	175,381	203,802	379,183

6.2. Landsliding at the Ex-Batcher Plant Site

6.2.1. Rehabilitation Plans

There is a possibility of recurrence of landsliding at the ex-batcher plant site if a heavy rainfall hits the area again, and any recurrence of large-scale landsliding may produce a serious effect on the water supply to Metro Manila. Therefore, it is a must to provide any appropriate protective measure against possible landsliding.

Considered as the protective measures against possible landsliding are (1) to provide a retaining structure by driving piles into the ground, and (2) to reform the slope by removing the shoulder portion and filling up the bottom portion. The measure (2) is to reduce a load of earth to release a sliding force on the shoulder portion while to increase a shearing resistance of earth on the bottom portion by reforming the slope.

Of these, the measure (2) is considered more effective and economical. In reforming the slope, it is considered recommendable to excavate the portion at the elevations higher than EL 215 m and fill up the portion at the elevations lower than EL 215 m with the excavated earth to make the slope gentler.

6.2.2. Rehabilitation by Slope Reformation

Shown in Figs. 4.15.1 and 4.15.2 are longitudinal profiles of the reformed slope to be considered most stable in terms of the safety factor against sliding (1.2 or larger in case of the earthquake with kh of 0.15 g or the ground water in existence up to the ground surface, and 1.0 or larger even in case of both conditions in existence at the same time).

Shown in Fig. 4.14 is a plan after the slope reformation is implemented at the ex-batcher plant site. In this reformation, the steeper slope at the elevations higher than EL.215 m will be cut out and the earth dug out will be filled up on the ground at the lower elevations to make the slope gentler.

The volume of the earthwork necessary to reform the slope is estimated as follows:

Excavation: 25,200 m^3
Fill-up : 19,200 m^3

The reformed land will be complete with drainage facilities. Perforated concrete pipings will be laid down on the existing slope surface in an attempt to keep down the ground water level after the fill-up. The toe of the fill-up slope will be provided with gabions to keep the fill-up earth from sliding down. Drain ditches will be provided on the surface of the reformed land to drain surface runoff from rainfall. The fill-up ground will be encircled by drain ditches to protect it from inflow of surface runoff away from the adjoining mountain slopes. Drain holes will be provided at the elevations of EL.215 m and EL.225 m in an attempt to lower the ground water level after the slope reformation.

Drain water through conduit pipings, drain holes and ditches is designed to flock to a duct to be provided at the bottom toe of the fill-up ground and further run down through the duct. This may protect the ground surface from erosion by rainfall. Figs. 6.2 and 6.3 show sections of the arrangement of conduit pipings, drain ditches and holes and drainage duct.

The cost estimate of the slope reformation work complete with drainage facilities is shown in Table 6.2.

It is recommended that the slope reformation work be undertaken in a single term, but in an event that the work be done in lots, it is advisable to divide the work schedule to the following three stages:

(1) Reformation of the slope, including preparation work, clearing and grubbing, excavation and fillup, and installation of conduit pipings.

Cost estimate : US$199,000

(2) Provision of drain holes and gabions

Cost estimate : US$110,000

(3) Installation of surface drain ditches and drainage ducts

Cost estimate : US$128,152

Fig. 6.2 Typical Cross Section of Surface Drain Ditch and Subdrain

Surface drain ditch | Subdrain

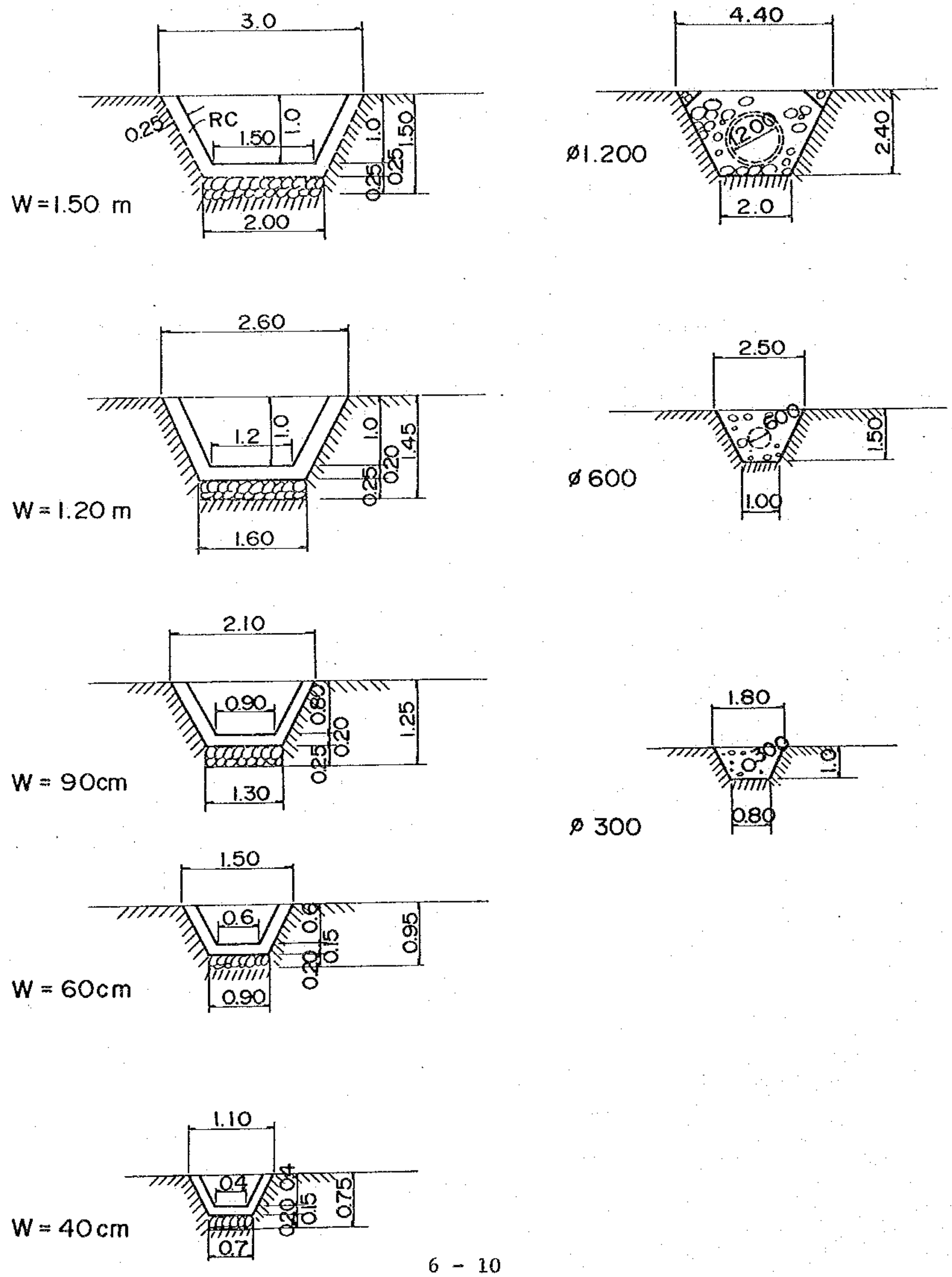
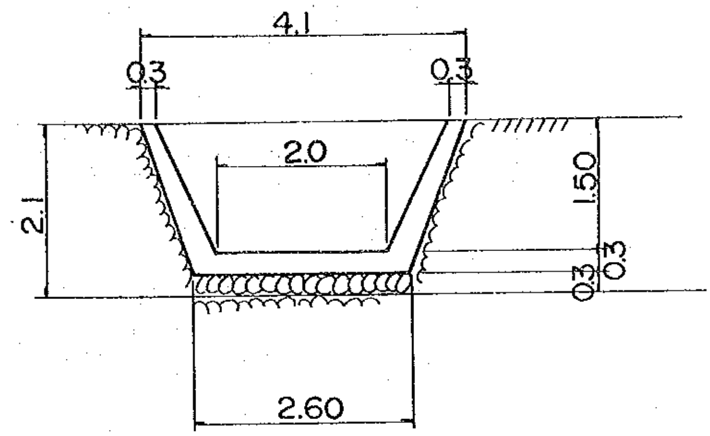


Fig. 6.3 Typical Cross Section of Canal and Gabion

Canal



Gabion

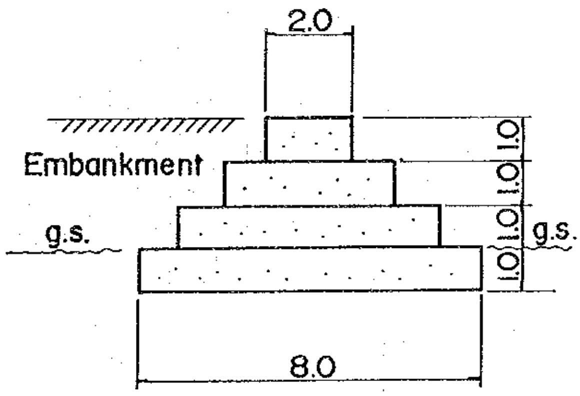
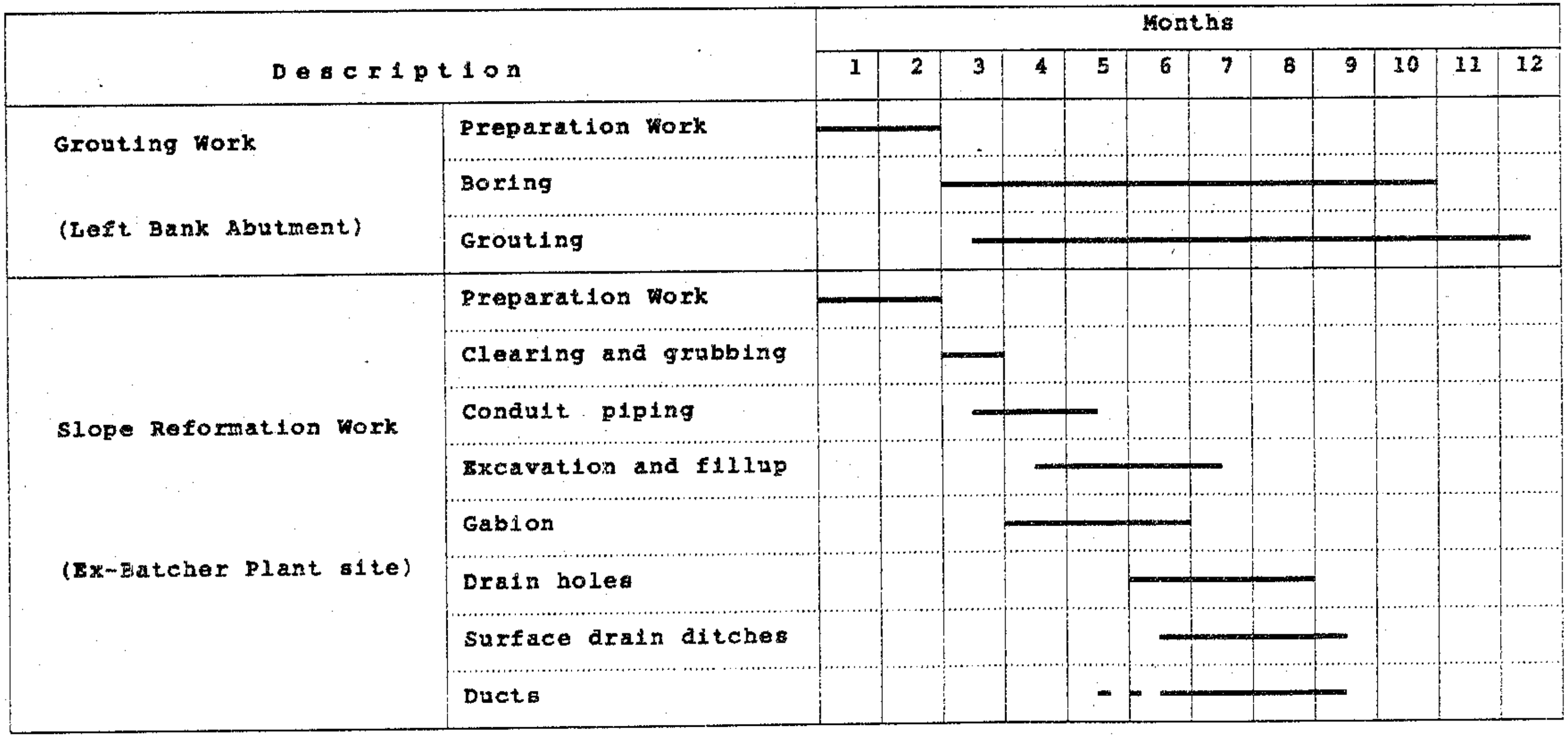


Table 6.2 Cost Estimate of the Slope Reformation Work

Description	Specification	Unit	Quantity	Unit Price (Peso)		Amount (Peso)		
				F . C	L . C	F . C	L . C	Total
1. Clearing and grubbing		m^2	16,000	32	34	512,000	544,000	1,056,000
2. Excavation - 1		m^3	17,600	38	34	668,800	598,400	1,267,200
3. Excavation - 2		m^3	2,700	64	63	172,800	170,100	342,900
4. Embankment		m^3	19,200	18	11	345,600	211,200	556,800
5. Trench excavation		m^3	4,900	60	61	294,000	298,900	592,900
6. Gabion		m^3	1,545	68	570	105,060	880,650	985,710
7. Cobble stone		m^3	1,950	68	559	132,600	1,090,050	1,222,650
8. Concrete		m^3	710	196	1,629	139,160	1,156,590	1,295,750
9. Reinforcement		ton	33	521	24,500	17,193	808,500	825,693
10. Concrete piping	ϕ 1,200	m	150	200	604	30,000	90,600	120,600
11. — do —	ϕ 600	m	150	72	216	10,800	32,400	43,200
12. — do —	ϕ 300	m	60	50	154	3,000	9,240	12,240
13. Boring	ϕ 75	m	700	1,000	800	700,000	560,000	1,260,000
14. P.V.C Piping	ϕ 50	m	700	8	43	5,600	30,100	35,700
Total					Peso	3,136,613	6,480,730	9,617,343
					US$(Equiv)	142,573	294,579	437,152

Fig. 6.4 Work Schedule of the Angat Dam Rehabilitation Project



7. ECONOMIC ANALYSES

7. Economic Analyses

7.1. The Importance of the Angat Dam and Power Plant in the Luzon Island

The principal objectives of the Angat dam and power plant are to produce electricity to serve peak loads in the Luzon grid, and to supply water to National Irrigation Administration (NIA) and Metropolitan Waterworks and Sewerage System (MWSS).

7.1.1. Characteristics of the Angat Dam and Power Plant

The Angat power plant has a total installed capacity of 228 MW, consisting of 200 MW (50 MW x 4) for the main plant and 28 MW (6 MW x 3 and 10 MW x 1) for the auxiliary plant.

A combined annual energy output of the plant in 1987 is 380,689 MWh, which breaks down to 195,988 MWh by the main plant and 184,701 MWh by the auxiliary plant. Despite the installed capacity, the auxiliary plant is producing virtually the same amount of electricity as the main plant. The plant factor of the auxiliary plant is, therefore, as high as 75%, while that of the main plant is as low as 11 %. Table 7.1 shows the monthly generation for the year 1987 by plant.

Also shown in Table 7.2 are the monthly water requirements for MWSS and NIA for the year 1987. As is seen in the Table, the annual requirements for MWSS were 862.66 million m^3, while those for irrigation were 641.18 million m^3. The former was the release from the auxiliary plant and the latter from the main plant. As shown, more than a half of water requirements for the Angat dam is used for water supply to Metro Manila. This indicates the Angat dam has played a critically important part in water supply to Metro Manila.

Table 7.1 Monthly Generation of Angat Hydroelectric Power Plant for the Year 1987

Month	Auxiliary Plant (Water Supplied to MWSS)	Main Plant (Water Supplied to NIA)
January	17,229 MWh	33,794 MWh
February	14,815	29,549
March	15,376	24,260
April	13,802	14,710
May	13,743	-
June	12,564	13,258
July	13,529	4,056
August	14,959	-
September	15,068	7,426
October	17,201	4,069
November	17,630	31,170
December	18,785	33,696
Total	184,701	195,988

Table 7.2 Monthly Water Requirements for Angat Hydroelectric Power Plant for the Year 1987

Month	Auxiliary Plant	Main Plant
January	69.47 $10^6 m^3$	104.46 $10^6 m^3$
February	62.75	94.35
March	69.58	80.35
April	67.23	51.84
May	70.42	0
June	70.11	51.84
July	76.90	16.07
August	77.25	0
September	72.63	25.92
October	76.72	13.39
November	73.52	98.50
December	76.08	104.46
Total	862.66	641.18

Table 7.3 Installed Capacity and Annual Energy Output of the Luzon Power System (1986)

Type of Power Source	Installed Capacity		Annual Energy Output	
	(MW)	(%)	(GWh)	(%)
Hydro, total	1,226	29.8	2,956	20.0
- Pumped storage	300	7.3	211	1.4
- Reservoir	895	21.8	2,643	17.9
- Run-of-river	31	0.7	102	0.7
Oilfired	1,925	46.8	6,328	42.9
Coalfired	300	7.3	1,572	10.7
Geothermal	660	16.1	3,900	26.4
Grand Total	4,111	100.0	14,756	100.0

Peak Demand : 2,435 MW

7.1.2. The Importance of the Angat Power Plant in the Luzon Grid

Shown in Table 7.3 is the installed capacity and annual energy output of the Luzon power system by type of power sources for the year 1986.

With the installed capacity of 228 MW and energy production of 561.85 GWh in the same year, the Angat power plant shared the total installed capacity and energy output of the Luzon power system by 5.5 % and 3.8 %, respectively.

The corresponding shares of the plant in the total capacity and energy output of the Luzon hydro power system in the same year were 18.6 % and 19.0 %, respectively.

The importance of the Angat power plant in the Luzon system can be compared with the Binga power plant, which shared the total installed capacity and energy output of the Luzon power system in 1986 by 2.4 % and 3.4 %, respectively.

Thus, the Angat power plant, as a peak load supplier, has been contributing, in a greater degree than the Binga plant, to a smoother operation of the system to maintain the demand and supply balance, in particular to a steady operation of the intermediate load suppliers such as oil-fired power plants, and likewise to a higher plant factor operation of the base load suppliers such as coal-fired and geothermal power plants.

As mentioned, the Angat power plant not merely plays an important part in contributing to a smoother operation of the system to keep the demand and supply balance in the most economical manner, but plays a very important role in maintaining water supply to MWSS without any interruption.

7.2. Principles of Economic Appraisal of the Rehabilitation Plans

In appraising the rehabilitation plans, it is essential to determine the justification for implementation on the basis of the following criteria for judgement:

(1) Rehabilitation is by all means necessary.

(2) Rehabilitation is not necessarily required, but it is advantageous for the owner to do so.

(3) Rehabilitation is not necessary.

A case to which the criterion (1) is applied is in an extremely critical condition, particularly from structural point of view. In this case, the critical condition of structures gets worse with time, and may cause disruption or collapse when there occurs a calamity such as flood or earthquake even with a relatively short return period. This case is such that the justification for implementation should be determined from the safety securing point of view, rather than from the economic point of view.

A case to which the criterion (2) is applied is in a condition that the defects of structures seem to remain unchanged, though they are accompanied with some disadvantages or losses, such as leakage of water. In this case, an appraisal of the rehabilitation plan should be made on the economical basis by comparing the cost of measures to be taken to eliminate such defects with the resultant benefits. Included in this case is the one in which the plant can be upgraded in the capability and energy output by implementing rehabilitation plans, though there are seemingly no defects on structures.

A case to which the criterion (3) is self-explanatory.

This Study was, at the inception, directed to carry out the following work:

(i) Investigation of seepage through the dyke built on the left bank of the main dam.

(ii) Investigation of landsliding at and around the ex-batcher plant site.

(iii) Investigation of leakage from the penstock.

Besides, the following work was added to this Study:

(iv) Examination of the adequacy of the spillway capacity.

(v) Analyses of the stability of the dam.

With regard to (i) above, seepage through the dyke seems to remain unchanged with no further serious development, and accordingly, the justification for implementation of the rehabilitation plan should be determined on the basis of the criterion (2).

Economic appraisal of the rehabilitation plan in this case should be made by comparison of the cost of the rehabilitation plan and the benefits in terms of energy output from the recovery of loss of water. The benefits should be estimated on the basis of the would-be loss in revenues from the decrease in energy output.

The seepage through the dyke, however, produces no adverse effect on the water supply to MWSS since such leaked water flows into the river upstream of the Ipo dam, where the intake facilities for NWSS are provided.

With regard to (ii) above, it should be noted that the Angat dam is extremely important for supplying water to Metro Manila, and any recurrence of landsliding may cause a blockage of stream flow into the Ipo dam, which may produce a very serious effect on the water supply to Manila. There might be a case where the Ipo reservoir is ultimately filled up with deposits of slidden materials. It is, therefore, a must to provide any appropriate protective measure against possible landsliding. In fact, there is a possibility of recurrence of landsliding at the ex-batcher plant site if a heavy rainfall hits the area again. The justification for implementation of the rehabilitation plan should, therefore, be determined on the basis of the criterion (1).

With regard to (iii) above, the investigation had to be held in abeyance because circumstances did not permit field inspections with the penstock being dewatered.

As it is very likely that there are some defects on the penstock line because leakage is still in progress, the justification for implementation of the rehabilitation plan should be determined on the basis of the criterion (1).

With regard to (iv) above, the spillway capacity is considered adequate as discussed in detail in Section 2, and it is believed unnecessary to provide any rehabilitation measures under the present circumstances.

7.3. Cost of Measures Worth Expending to Eliminate Seepage through the Dyke

7.3.1. Cost Appraisal

As discussed in detail in Section 5, it is estimated that the total amount of seepage through the dyke would be 317,000 m^3 a year. Of this, the amount of seepage from the reservoir is estimated to be 261,000 m^3, and the resultant loss of power production to be 56,100 kWh a year.

The present worth value of monetary loss equivalent to cumulative would-be losses of power production over a period of the remaining useful life can be expressed as:

$$a[1 + \frac{1}{(1+i)} + \frac{1}{(1+i)^2} + \ldots\ldots] = a \cdot \frac{1 + i}{i}$$

Where,

a : Monetary loss of power production a year (US$/year)
i : Discount rate

The monetary loss is represented by the loss of would-be revenues from sales. For the loss of power production to be represented ,as the loss of revenues, it is necessary to take into account transmission loss, power production costs such as operation, maintenance, repair and miscellaneous expenses and relevant transmission, transformation and distribution costs.

Given that the transmission loss is 5%, and power production, transmission, transformation and distribution costs is 2% (according to the Japanese standard), "a" value in the above equation can be expressed as:

$$\begin{aligned} a &= (1 - 0.05)(1 - 0.02) \cdot W \cdot \lambda \\ &= 0.93\ W\ \lambda (\text{US\$/year}) \end{aligned}$$

Where,

W = Loss of power production due to leakage of water passing through the dyke (KWh/year)
λ = Average billing rate per unit (US$/KWh)

Now that it is extremely difficult to stop seepage completely by means of grouting work, and supposing that only α% of the amount of seepage may be stopped by implementing grouting work, then, cost worth expending for the measure should be evaluated in the following manner:

$$S = 0.93 \frac{1 + i}{i} W \cdot \lambda \cdot \alpha$$

Now, given that the discount rate is 14%, then the above equation can be replaced with:

$$S = 7.6 W \cdot \lambda \cdot \alpha \text{ (US\$)}$$

7.3.2. Cost Analysis

Assuming that the would-be loss of power production due to leakage of water passing through the dyke would be 56,100 kWh a year, and given that the average billing rate per kWh is one peso, the "S" value in the equation given in 7.3.1 should be:

$$S = 0.93 \frac{1 + i}{i} W \cdot \frac{1}{22} \cdot \alpha \text{ (US\$ converted at P22/US\$)}$$

On the other hand, the cost for the grouting work is estimated to be US$ 380,000 as discussed in Section 6. Therefore, the benefit-cost (B/C) ratio for the economic analysis should be:

$$B/C = 0.93 \frac{1 + i}{1} W \cdot \frac{1}{22} \cdot \alpha/380.000$$

Assuming that the grouting work could be fully effective and the value "α" (amount, as % of total, to be stopped from seeping by the grouting work) would be equal to 1, then B/C value should be

0.05, when calculated at the discount rate (i) of 14%. This implies that the measure may in no case be economical. When calculating the discount rate under this condition to make B/C equal to 1, then the value "i" comes to only 0.63%.

It is therefore obvious that the grouting work, if implemented only for the purpose of reducing seepage of water, should not be justifiable from the pure economic point of view.

However, if it is confirmed from the continuous measurement that seepage of water through the dyke will increase in the future, it is necessary to implement the seepage prevention measure without regard to the economics.

8. SAFETY CONTROL STANDARDS OF THE DAM AND ASSOCIATED STRUCTURES

8. Safety Control Standards of the Dam and Associated Structures

The safety control of a hydroelectric power plant is multi-disciplinary. It involves the safety against damages of structures, such as dam, intake, pressure tunnel, headrace channel, powerhouse, generating equipment, tailrace channel and other ancillary structures. It also involves the measures against progress of reservoir sedimentation and instability of adjacent mountain slopes.

With regard to the Angat Power Plant, it is confirmed from the investigation made by the JICA Study Team in October 1987 that there exist problems on its dam, dyke, penstock and ex-batcher plant site, and accordingly, a greater emphasis in maintenance control should be directed to the dam and its associated structures.

8.1. Monitoring

Monitoring for the safety control should be done continuously on a regular basis. The items are as follows:

8.1.1. Collimation Survey at the Dam and Dyke

It is necessary to keep monitoring the behaviors of the dam and dyke. In order to do so, measurement of deformations of the dam and dyke should be conducted on a regular basis using the control points installed during the 1987 investigation, and the measured values should be checked periodically to confirm if there are any significant changes in displacements due to reservoir water pressure.

Collimation survey should be conducted at the frequency of once a month. It is essential to have at all times an accurate knowledge on the relation between displacements and reservoir water levels, and to check how displacements vary against the same reservoir water level.

In the event that there occured an earthquake with the acceleration of greater than 70 gal registered in the seismometers installed at the dam site, possibly accompanied with a decrease in the safety factor against sliding of the slope surfaces of the dam and dyke to smaller than 1.0, it is mandatory to make an additional collimation survey at the dam and dyke, and a slope survey of both upstream and downstream faces, thereby checking if these slopes have undergone any changes and damages. If damaged, appropriate corrective actions should be taken immediately.

8.1.2. Monitoring of Seepage through the Dyke

It appears that the seepage through the dyke is in the stable condition, but it is still necessary to keep monitoring in order to check if seepage tends to increase or decrease.

Monitoring items are as follows:

i. Measurement of outflow from the three seepage measurement weirs, SW-1, SW-2 and SW-3.

ii. Measurement of rainfall at the dam site.

The measured outflow from the weirs is the sum of the amount of runoff from rainfall and the seepage through the dyke. Therefore, it is necessary to segregate the amount of rainfall from the sum.

Table 5.2 shows the coefficients applied for calculation to obtain the amount of seepage through the dyke out of the measured outflow from the weirs.

$$C = \frac{Q_0 - [a_0R_0 + a_1R_1 + a_2R_2 + a_3R_3 + a_4R_4 + a_5R_5 + a_{6-10}R_{6-10} \cdots a_{21-35}R_{21-35} + b]}{(RWL - 195)}$$

Where,

a_0, a_1, a_2 .... is the coefficient for calculation of the effect of rainfall as given in Table 5.2.

R_0 is the amount of rainfall on the same day when the measurement is done.

R_1 is the amount of rainfall on the day before the measurement day.

R_2 is the amount of rainfall on two days before the measurement day.

R_3 is the amount of rainfall on three days before the measurement day.

R_4 is the amount of rainfall on four days before the measurement day.

R_5 is the amount of rainfall on five days before the measurement day.

R_{6-10} is the total amount of rainfall on six days before the measurement day through ten days before the measurement day.

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R_{21-35} is the total amount of rainfall on 21 days before the measurement day through 35 days before the measurement day.

The above equation is effective only when the reservoir water level is at higher than EL 195 m.

It is essential to check how the value "C" changes with a lapse of time, by using the equation and the table as given above.

The calculation should be made periodically at the frequency of at least once a month thereby to make it possible to conduct a quantitative analysis.

In the event when an earthquake occurred, it is necessary to shorten the interval of monitoring so as to closely check how the seepage is affected by the earthquake.

8.1.3. Monitoring Against Possible Landsliding at the Ex-Batcher Plant Site

It is expected that the land reformation measure against possible recurrence of landsliding at the ex-batcher plant site will be taken in due course of time, but even after completion of the measure, it is recommended to continue measurements with inclinometers and drill holes to check the possibility of landsliding and to determine ground water behaviors. This should be continued until it is confirmed that the land has become totally stabilized.

The measurements are needed to be conducted at the frequency of once a month by the time of completion of the land reformation measure, and once every two months over a period of two years after the completion of the measure. It is further recommended that monitoring be done since then at the frequency of once every four months.

8.1.4. Monitoring of Leakage from the Penstock

It was not possible in this Report to make studies on the leakage of water from the penstock and evaluate their results to the fullest extent possible.

Rehabilitation of the penstock is indispensable, and it will be conducted sooner or later. In order to determine the implementation schedule of the rehabilitation program, it is necessary to keep monitoring of changes in the amount of leakage from the penstock.

Monitoring can be made at the two weirs installed close to the powerhouse. It is very essential to have at all times accurate knowledges of the relation between the measured amount of leakage and the reservoir water level, and of the changes in the measured amount of leakage against the same reservoir water level.

The measurement is needed to be done at the frequency of once a week.

8.1.5. Monitoring of Water Quality

It is as a matter of course necessary to have a knowledge of the status of the seepage and determine its changes in the magnitude on a quantitative basis, but also essential to look into the changes in water quality.

Measurement of water quality should be made at the following points:

i. In the proximity of the intake.

ii. Dyke seepage measurement weirs, SW-1, SW-2 and SW-3.

iii. Penstock leakage measurement weirs (two).

The result of measured water quality should be retained for the analysis of changes in water quality. It is considered appropriate to make the measurement at the frequency of once a month.

8.2. Detailed Inspection and Repair

In case when conditions adverse to the safety of structures were found in existence as the result of monitoring as mentioned above, or due to unusual natural phenomena such as heavy rain, large earthquake or flood, it is necessary to make an additional inspection immediately after the incident. In case when the inspection revealed any abnormality on the structures, it is necessary to conduct further detailed inspection, and if such detailed inspection disclosed significant conditions adverse to the safety of the structures or serious damages thereon, it is mandatory to take an immediate action for repair or rehabilitation.

9. SAFETY CONTROL SYSTEM OF THE DAM STRUCTURE

9. Safety Control System of the Dam Structure

It is very essential to continue regular inspection and constant monitoring to maintain dam structure and the associated facilities built in a huge amount of capital in a good condition over a long period of time. This enables those facilities to function in the most stabilized and efficient condition, so that the capital investment for such facilities may prove effective to the maximum extent possible.

At the inception of this Study, the JICA Study Team made an investigation of the present state of maintenance practices of hydroelectric power plants and the associated facilities. As a result, the team members had an impression that the organizational structures and personnel placement are adequate in maintaining electro-mechanical plant and facilities that are directly in connection with the operation of power plants, but not in maintaining civil structures that are also essential for the operation of power plants.

There are marked differences between the Philippines and Japan in many respects, such as in weather conditions, scale and composition of power plants and the associated facilities, and consuming modes of consumers. It is, nevertheless, believed that there is enough room for NAPOCOR for improvement of plant utilization. This can be achieved by improving and strengthening the organizational structures for maintenance control and increasing capital outlay for preventive maintenance.

It is more important for the Angat Power Plant to establish the maintenance control system of its dam and associated facilities in that it plays not merely an important part in generating electricity for the Luzon grid, but a critically important role in maintaining water supply to Metro Manila without any interruption.

Electricity requirements in the Philippines, particularly in the Luzon Grid, have been increasing at a high rate to keep pace with the recent robust expansion of the nation's economy and are expected to grow in even a higher pace in the future. Water requirements in Metro Manila, too, have been increasing year after year.

It is therefore very essential to improve and strengthen the maintenance control system and structures of power plants and the associated facilities for sound operation of the electric utility business and of the water supply business as well.

9.1. NAPOCOR Head Office

Hydroelectric power sources owned and operated by NAPOCOR include aged and newly commissioned power plants, and, accordingly, operation and maintenance practices of these power sources including their civil structures differ from plant to plant.

In order to implement smoother operation of these plants and structures, it is highly essential to establish comprehensive maintenance programs on a year-to-year basis as well as on a long-term basis, based on the results of the review and analyses of records and data on maintenance and inspection conducted on a regular basis. Equally important is to formulate year-to-year maintenance budgets and to carry out maintenance and repair works in accordance with the maintenance programs.

Besides, it is necessary to keep monitoring behaviors of critically important civil structures in large scale, and to review and analyze periodically the records and data on such monitoring work.

For efficient and smooth execution of the respective functional assignments, it is recommended that a maintenance control group or division be newly established in the Hydro Power Projects Dept., composed of several civil engineers who shall exclusively deal with the maintenance control assignments.

9.2. Northern Luzon Regional Center

The Northern Luzon Regional Center is responsible for operation and maintenance of five hydro power plants with a combined installed capacity of 863 MW, including Angat, Binga and Ambuklao power plants, and plays a leading role in power generation in the Luzon Grid.

The Center is also responsible for operation and maintenance of substations and transmission lines under its four area offices of jurisdiction. NAPOCOR staff working for the Center totals about 1,600, mostly of electrical and mechanical engineers. The number of personnel engaged in operation and maintenance of five hydro power plants, both in the Center and power plants, is about 1,120 (455 for operation and 655 for maintenance).

At the Angat Power Plant, two hydrologists are posted, but no such staffs as qualified for inspection and monitoring of its dam and the associated structures are posted. This implies that such inspection and monitoring is not being conducted by the qualified civil engineering staffs.

In actuality, there are cases in which civil structures, if left unattended with periodical maintenance programs for long, have undergone a serious trouble which needs a considerable amount of money for corrective actions.

It is, therefore, recommended that a civil structure monitoring group or section be newly established in the Regional Center, composed of at least three civil engineers and three assistants, who shall engage in particular in inspection and monitoring of the civil structures of Angat and other power plants under its jurisdiction.

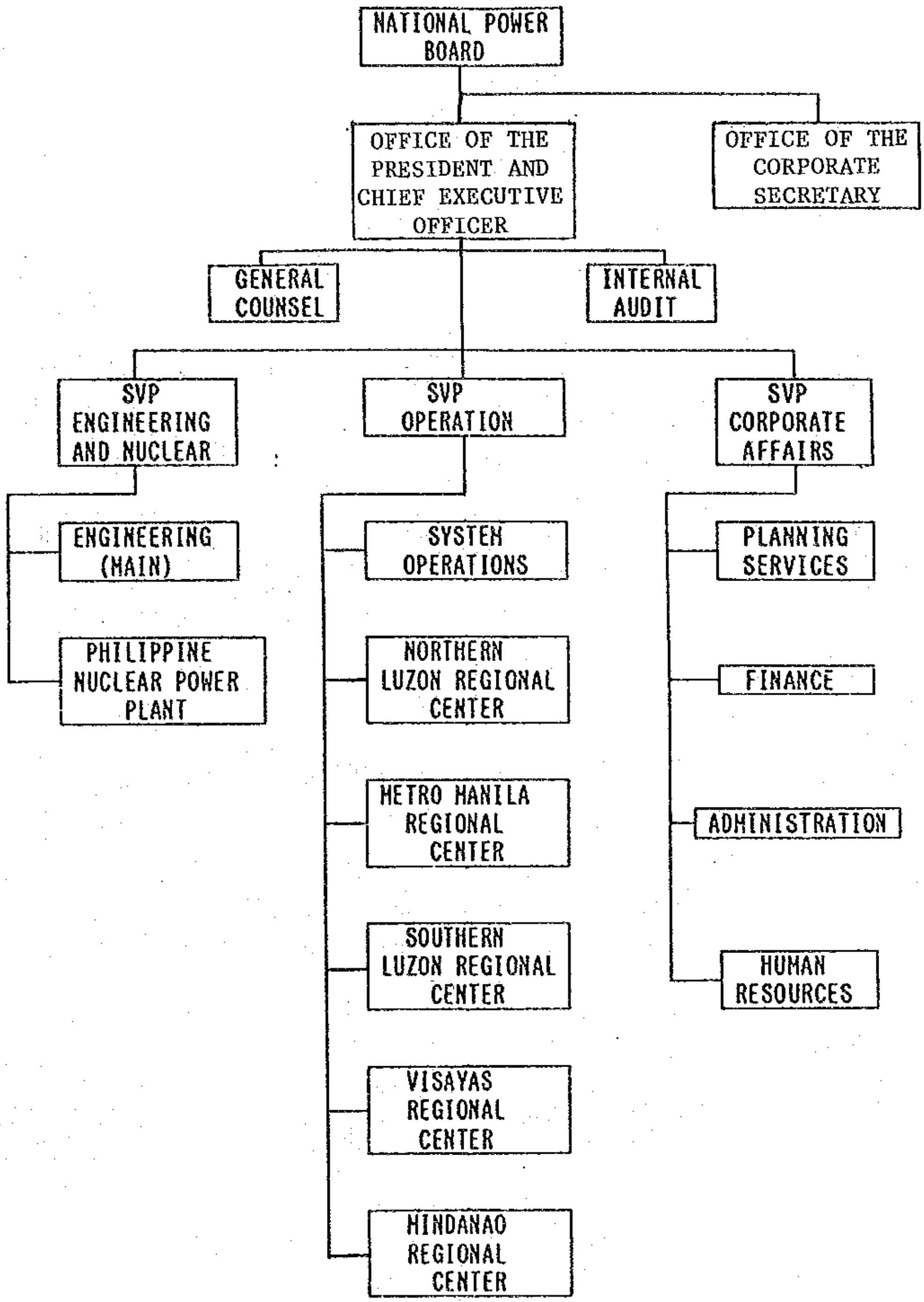
9.3. Angat Power Plant

In view of the urgency and importance of the Angat Dam Rehabilitation Project, there are things to be done by the power plant according to priorities. The first thing is to have an accurate knowledge of the present status of the plant facilities particularly of civil structures, through review of all relevant documents, drawings and data since the start of the construction and also through conduct of actual surveys and measurements. These documents get scattered and lost with a lapse of time and it becomes more and more difficult to have an accurate knowledge of the status of structures. The second thing is to conduct regular patrolling inspection of civil structures as a routine activity.

Since it is believed difficult for the Northern Luzon Regional Center to cover its entire Northern Luzon Grid area, encompassing large scale and important civil structures, in conducting patrolling inspection, it is considered necessary to post at least one civil engineering staff at each power plant for maintenance and inspection assignments. In addition to these assignments, he shall be responsible for many routine duties, including operation of spillway gates and monitoring of the reservoir water levels. For the Angat Power Plant which is located relatively distant from the Regional Center, it is particularly desirable to post such a staff as capable of taking emergency steps at his own discretion as need arises.

Fig. 9.1 ORGANIZATION CHART

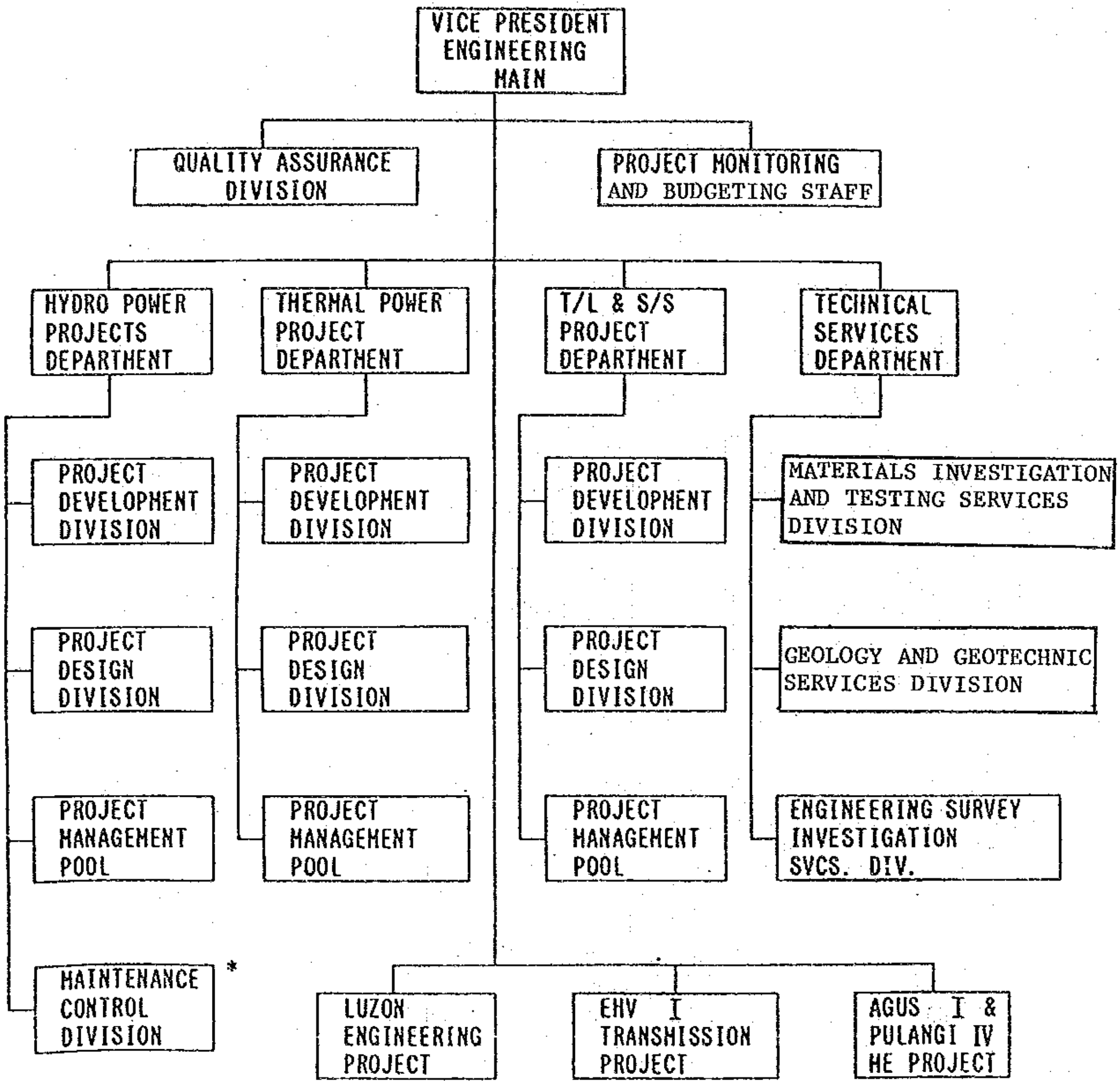
NAPOCOR



Total Number of Personnel : 10,819 (As of Dec. 31, 1987)

Fig. 9.2 ORGANIZATION CHART

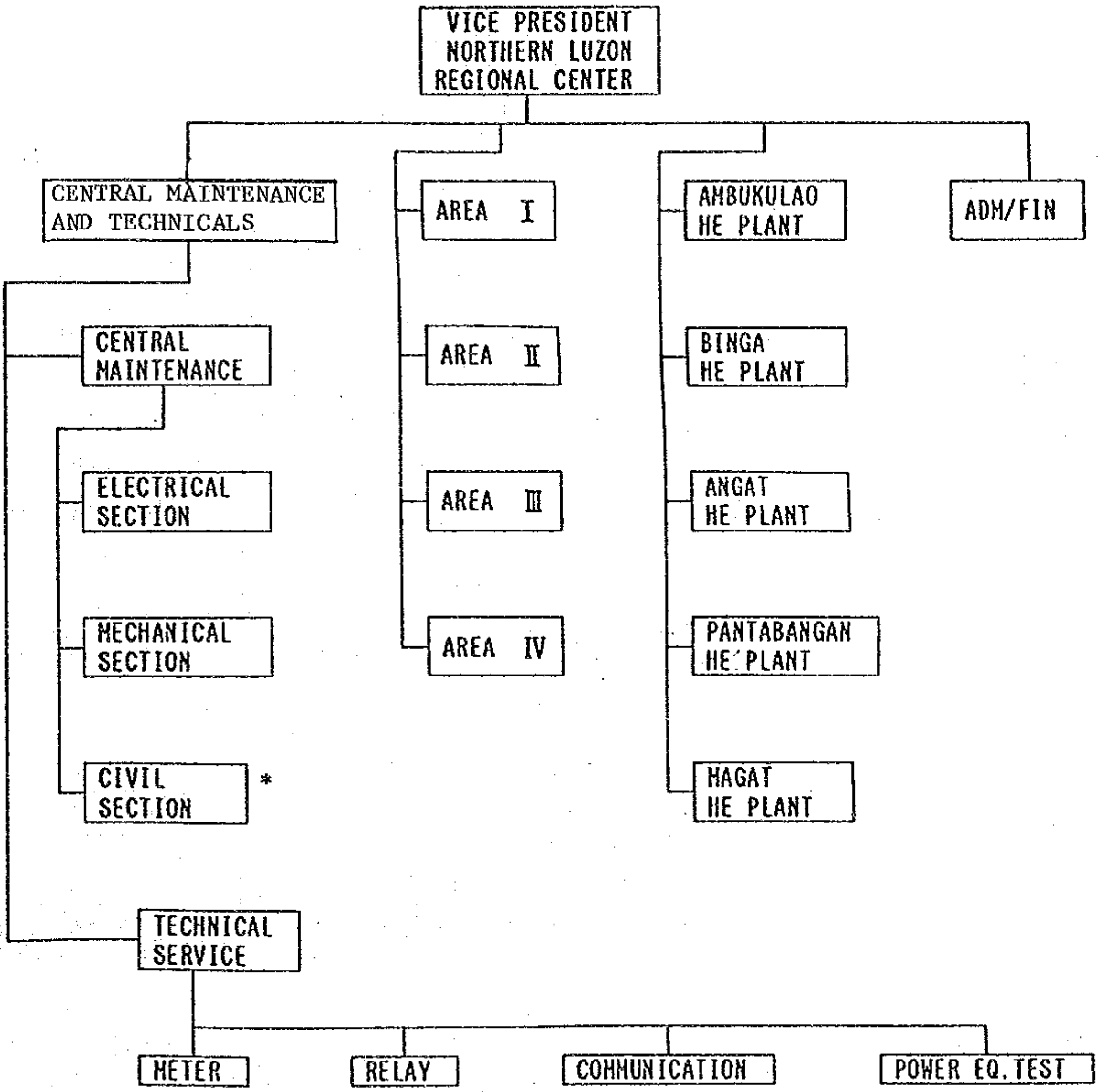
HEAD OFFICE ENGINEERING - MAIN



* A division recommended to be newly established.

Fig.9.3 ORGANIZATION CHART

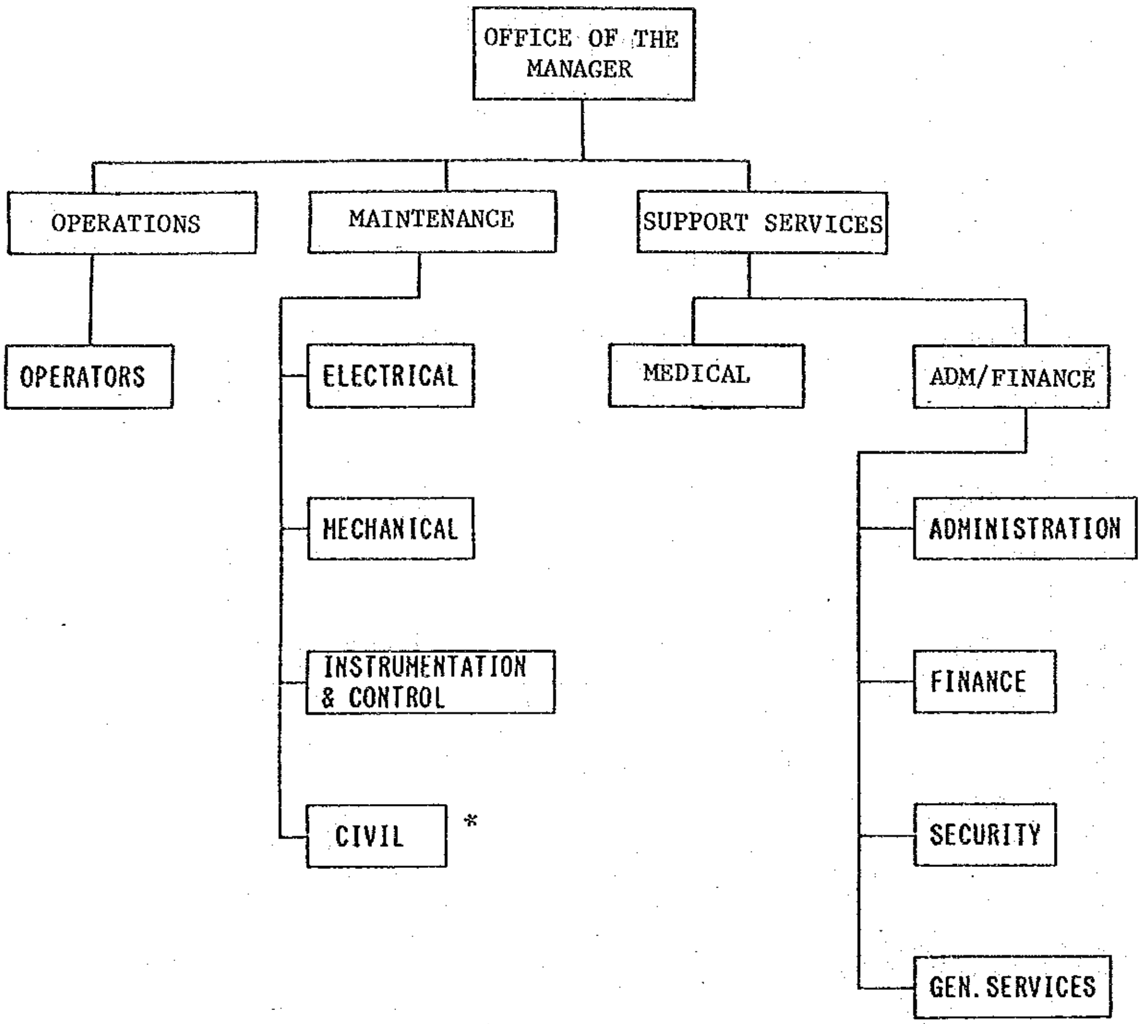
NORTHERN LUZON REGIONAL CENTER



* A section recommended to be newly established.

Fig.9.4 ORGANIZATION CHART

ANGAT HYDROELECTRIC POWER PLANT



* A group recommended to be newly established.

SUPPLEMENT

10. LEAKAGE FROM THE PENSTOCK

SUPPLEMENT

10. Leakage from the Penstock

The investigation of leakage from the penstock and the turbine draft tube was, at the inception of the Study in September 1987, one of the most essential items for the Angat Dam Rehabilitation Project. However, it had to be excluded from the scope of work of the Study, pursuant to the NAPOCOR's written request dated July 15, 1988, Ref. No. 88-HDV-151, because it became impossible for reasons on the Philippine side to dewater the penstock for the internal inspections.

Since the leakage from the penstock poses a very serious problem for the safety of the Angat Power Plant, an attempt was made to look into the problem through analyses to the maximum extent possible of the data on the previous investigations made independently by NAPOCOR.

10.1. Data Available on Leakage from the Penstock

The leakage from the penstock has long been evidenced by the existence of a close relation between the rise of the reservoir water level and the amount of water flowing out of the sinking shaft provided nearby the powerhouse.

It can be imagined that the constant leakage from the penstock would have been keeping the ground water table around the penstock at higher levels. This implies that the penstock is in danger of suffering from buckling due to heavy external water pressures when it is dewatered.

It can also be imagined that the constantly high level of the ground water table in the surrounding mountain masses would have swept fine grain particles away from around the penstock, causing the surrounding mountain masses to be permeable, thereby inducing further leakage from the penstock. Such phenomena, if developing, may ultimately produce a catastrophic effect on the surrounding mountain masses.

If the leakage from the penstock should be attributed to ill defects such as cracks, there is a possibility of further development of crack formation from their tips due to stress concentration of steel materials and their fatigue due to repetition of loadings over a long period of time, thereby causing increase in the leakage rate. There is also a possibility that such crack formations may finally break the steel structure and any fragments may hit and damage the guide vanes or turbines.

Besides, the leakage from the penstock is, by itself, linked with loss of energy output with the resultant loss of revenues from sales. The study should, therefore, be made from such standpoint, too.

The NAPOCOR's intent in their investigation of the leakage problem seems to be primarily to avoid any rupture accident at the time when the penstock is dewatered. To this end, NAPOCOR installed piezometers along the route of the penstock. It is true that the data taken from the piezometric head measurement can be used for investigating the leakage problem, but the result could be insufficient.

It is as a matter of course considered best to conduct visual inspections with the penstock being dewatered, but it was not possible to do so. Hence, it was impossible to work out the solutions to be taken as the repair measure, because it is unknown how serious the status of leakage is (whether there exist any cracks on the welds, or if the leakage should be attributable simply to incomplete fillings in the grouting holes), nor is it known where and to what extent the repair should be made.

Efforts were directed to analyze to the maximum extent possible the data on the piezometric head measurement available from NAPOCOR, but the items to be studied through the analyses had to be limited to the following:

1) Probable locations of the leakage.

2) Estimated amount of leakage.

3) Changes in the amount of leakage with time, if any.

For fear of any rupture accident of the penstock by external water pressure when it is dewatered, NAPOCOR provided a sinking shaft in the vicinity of the bifurcation point, in an attempt to lower the ground water level to release external water pressure on the penstock. The provision of this sinking shaft proved to be very effective in lowering the ground water level.

In addition, five boreholes (shown as DDH-1 through DDH-5 in Fig.10.1) were provided to check the distribution and changes of the ground water level. These boreholds have been used to pump out water to lower the level of the surrounding ground water table for the purpose of protecting the penstock from being ruptured at the time when it is dewatered.

In order to ascertain the effect of such measures on the lowering of the ground water level, nine piezometers were installed in the vicinity of the penstock in July and August 1979. Piezometric heads were measured in September 1979 after completion of the installation work, and in July 1982 with the penstock being dewatered.

The data available for the study of the leakage problem for this Project are only those taken at the measurement in September 1979 and July 1982.

10.2. Piezometric Head Measurement in September 1979

Installation of piezometers commenced on July 23, 1979, beginning with the location shown as PH-2. The number of piezometers installed at PH-2 was three each at the elevation of EL.81.5, EL.70.0 and EL.58.5 m. The work ended on July 24. Subsequently, three piezometers were installed at the location shown as PH-1 at the elevation of EL.80.0, EL.70.0 and EL.57 m, during four days from August 2, 1979. Further, on August 23 and 24, 1979, three piezometers were installed at the location shown as PH-3 at the elevation of EL.108.0, EL.86.5 and EL.67.5 m.

Shown in Fig. 10.1 are the locations of PH-1, PH-2 and PH-3 and boreholes numbered DDH-1 through DDH-5. Shown in Fig. 10.2 is a longitudinal profile of the arrangement of piezometers.

With these newly installed piezometers, piezometric heads at the respective points were measured on September 4, 1979. The result is shown in Table 10.1. The horizontal distance between the piezometers and the boreholes is exhibited in Table 10.2.

10.3. Piezometric Head Measurement in July 1982

The dewatering test of the penstock was conducted from July 4, 1982. The ground water levels measured then at DDH-1 through DDH-5 (each provided with a drainage pump) and the piezometric heads observed at PH-1, PH-2 and PH-3 with a pecific time are tabulated in Table 10.3 and Table 10.4, respectively. Fig.10.3 shows changes in the piezometric head at each point with a lapse of time.

The process of a series of the dewatering test was as follows:

1) The ground water levels were measured at each borehole, numbered DDH-1 thru DDH-5 under the normal plant operation.

2) Each borehole was drained at the rate of 5 to 10 gallons per min., using a small capacity pump.

3) Water was pumped out from the sinking shaft for four hours at the rate of 2,000 gallons per min. The ground water level then lowered to EL.72 m.

4) The intake gate was closed.

5) Under the above condition, the pump for the sinking shaft was put out of operation, but the water level did not recover to the original level.

6) The intake gate was opened and the penstock was filled with water. Then, the water level recovered to the original level within an hour.

   The reservoir water level was at HWL. The sinking shaft is of a size by 2.5 m by 2.5 m, and constructed of concrete lining to a depth of 6-8 m and of no lining to a deeper depth.

10.4. Reservoir Water Level

Piezometric heads in the surrounding mountain masses are subject to the reservoir water level, and particularly so, if water is leaking from the penstock. Table 10.5 shows the reservoir water levels recorded before and after the piezometric measurements in September 1979, and in July 1982 (with the penstock being dewatered).

On September 4, 1979, when the piezometric measurement was done, the reservoir water level stood at a range of EL.190 to 191 m. On July 1, 1982, just before the dewatering test of the penstock was conducted, the reservoir water level stood at EL.178.8 m, but

Fig. 10.1 Route of Power Tunnel
S = 1/500.

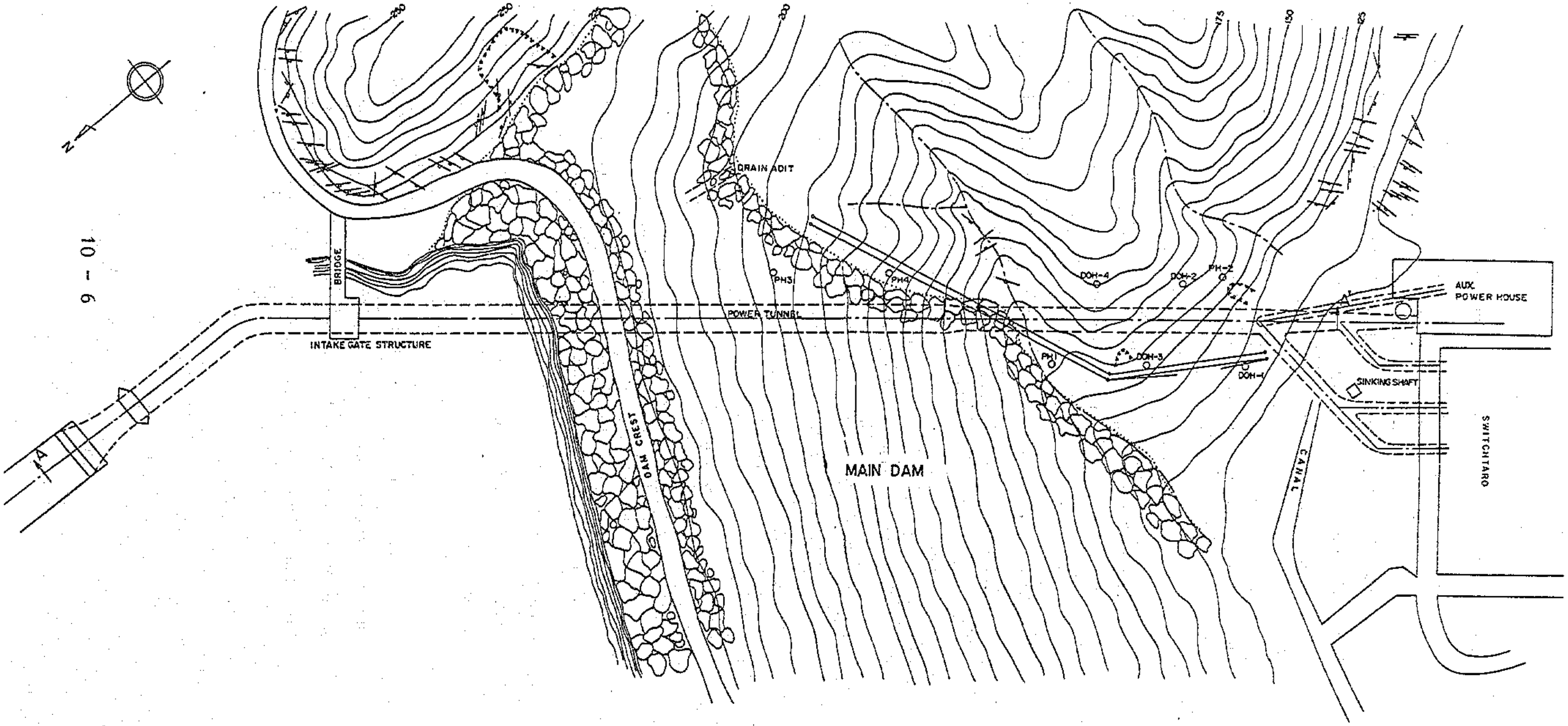


Fig. 10.2 Profile of Power Tunnel

SECTION A-A'
SCALE 1:500

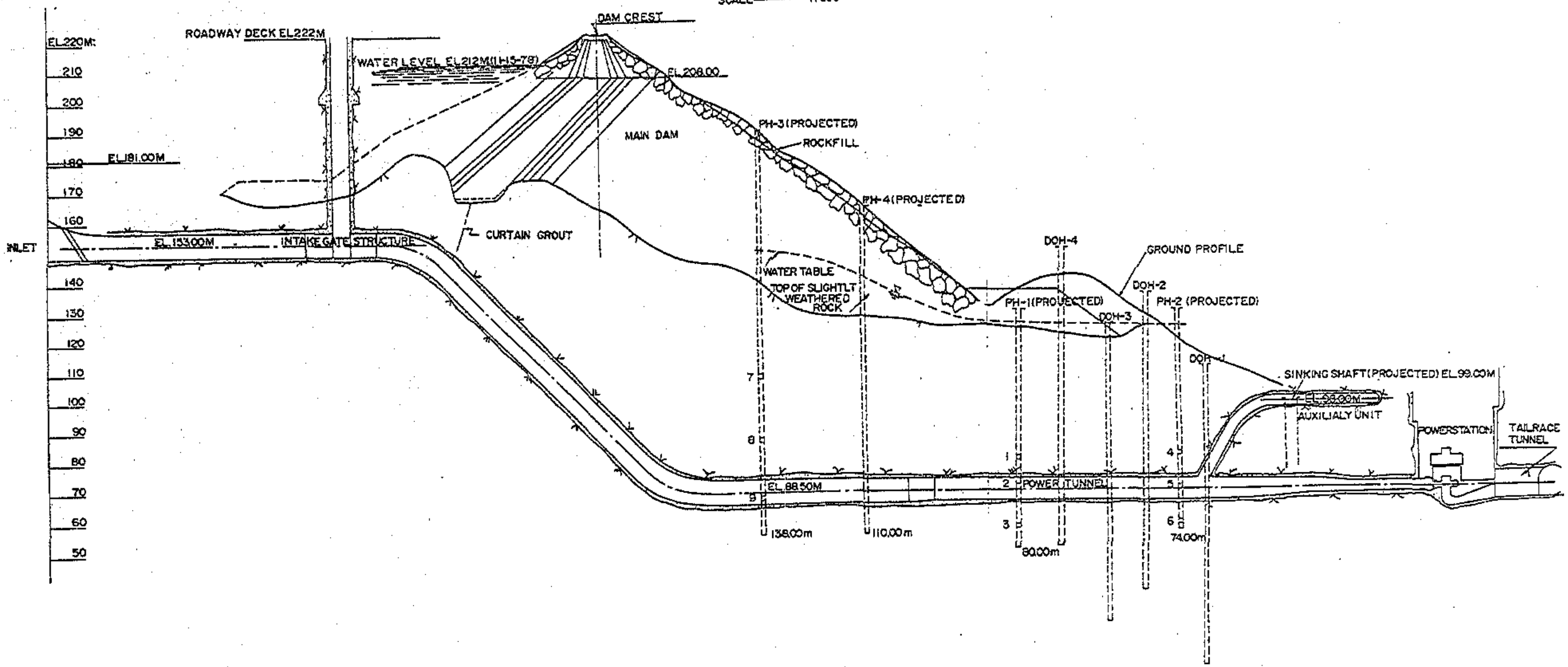


Table 10.1 Piezometric Heads as of Sept., 1979

(Unit : m)

Location & No. of piezometer		Elevation of surface ground	Elevation of installation	Piezometric head	Piezometric water level
	1	131	80	46.2	126.2
PH-1	2	131	70	58.6	128.6
	3	131	57	74.7	131.7
	4	130	81.5	39.8	121.3
PH-2	5	130	70	48.5	118.5
	6	130	58.5	78.6	137.1
	7	189	108.0	41.8	149.8
PH-3	8	189	86.5	54.5	141.0
	9	189	67.5	69.6	137.1

Table 10.2 Distance Between each Installed Piezometer (PH) and each Borehole (DDH)

(Unit : m)

	DDH-1	DDH-2	DDH-3	DDH-4	Sinking Shaft
PH-1	62	48.5	30	28	97
PH-2	28	14	37	41	54
PH-3	152	130	121	102	187

Table 10.3 Water Levels in Boreholes During Penstock Dewatering Test in July 1982

Borehole	EL. of ground surface	W.L. before dewatering	W.L. during dewatering	Drawdown of W.L.	Depth of borehole
DDH-1	110.30	103.60	74.3 - 62.3	29.3 - 41.3	70
DDH-2	116.30	110.30	77.3 - 65.3	33.0 - 45.0	90
DDH-3	113.80	110.80	74.8 - 62.8	36.0 - 48.0	100
DDH-4	127.30	117.30	91.3 - 79.3	25.5 - 38.0	90
DDH-5	110.30	106.60	74.3 - 62.3	32.3 - 44.3	70
Sinking shaft	98.0	98.0	72.0	26	69

Table 10.4 Changes with Time of Piezometric Heads During Penstock Dewatering Test in July 1982

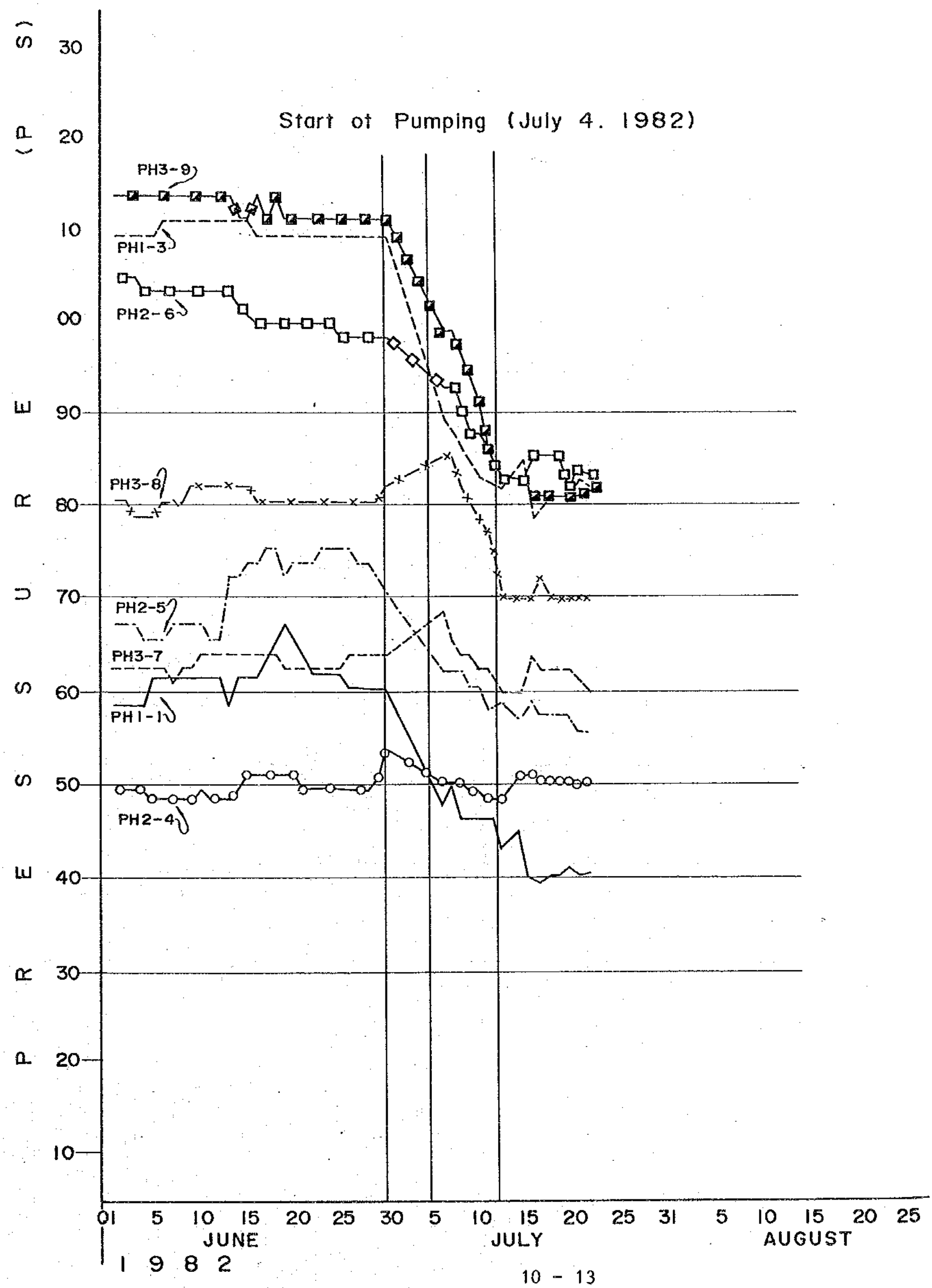
Location & No. of Piezometer		1 June Before Pumping		1 July Start of Pumping		5 July Closing of Intake Gate		12 July Max. W.L. drawdown		Min. Piezometric Head	
		PH	EL	PH	EL	PH	EL	PH	EL	PH	EL
PH-1	1	57.5	137.5	60.0	140	51.0	131.0	43	123	39.5	119.5
	2										
	3	109.0	166	109	166	94.5	151.5	82	139	79	136
	4	49	130.5	53	134.5	51	132.5	48.5	130	48.5	130
PH-2	5	67	137	71	141	64.5	134.5	59.0	129	46.0	116
	6	105	163.5	98	156.5	95	153.5	83	141.5	82	140.5
	7	62	170	64	172	67.5	175.5	60	168	60	168
PH-3	8	81	167.5	82	168.5	85	171.5	70	156.5	70	156.5
	9	114	181.5	111	178.5	102	169.5	83	150.5	79	146.5

Table 10.5 Reservoir Water Levels in Sept. 1979 and in July 1982

(EL m)

Year & date	R.W.L.	Year & date	R.W.L.
1979 Aug. 19	194.57	1982 July 1	178.80
20	194.32	2	178.91
21	193.86	3	179.77
22	193.49	4	180.43
23	193.19	5	180.77
24	192.69	6	181.14
25	192.41	7	181.44
26	192.18	8	181.68
27	191.71	9	181.92
28	191.34	10	182.10
29	190.93	11	182.26
30	190.71	12	182.35
31	190.40	13	182.40
Sept. 1	190.40	14	182.47
2	190.69	15	189.60
3	190.84	16	191.35
4	191.01	17	192.02
5	191.08	18	192.26
6	191.11	19	192.59
7	191.12	20	192.91

Fig. 10.3 Changes in Piezometric Heads with Time



it recovered to approximately EL.193 m at the time of completion of the dewatering test.

In discussing the leakage from the penstock, attention should be directed to the piezometric head just before the dewatering test, i.e. under the condition that the water pressure still remains in the penstock, rather than to that at the time of the dewatering test. From this point of view, the leakage in 1982 must be less in the amount than that in 1979, because the reservoir water level was lower by about 11 m, assuming that there would have been no changes in the condition of the penstock between September 1979 and July 1982.

10.5. Relation Between Piezometric Heads in September 1979 and July 1982

Table 10.6 shows values of the piezometric head measured at each point in September 1979 and July 1982.

Table 10.6 Piezometric Head Measurement

Location	Ground Surface (EL)	Piezometer No. *	Piezometric Head (EL)		Difference of Water Level
			July, 1982	Sept.,1979	
PH-1	131	1	137.5 m	126.2 m	11.3 m
		2	-	128.6	-
		3	166.0	131.7	34.3
PH-2	130	4	130.5	121.3	9.2
		5	137.0	118.5	18.5
		6	163.5	137.1	26.4
PH-3	189	7	170.0	149.8	20.2
		8	167.5	141.0	26.5
		9	181.5	137.1	44.4

* Piezometer installation elevations are shown in Table 10.1.

The comparison of the piezometric heads in Table 10.6 above reveals the following:

1) As of September 1979, any value of the piezometric head is found to be a little higher or lower than the ground surface elevation, but as of July 1982, all values except those at PH-3 are above the ground surface elevation.

2) All values of the piezometric head at PH-1, PH-2 and PH-3 in July 1982 are found to be higher than those in September 1979.

3) As of September 1979, the piezometrers at PH-1 and PH-2 tend to show relatively higher values with distance closer to the penstock, but those at PH-3 seem to have nothing to do with the distance to the penstock. As of July 1982, the piezometers installed closer to the penstock show higher values than those distant from the penstock regardless of the measuring points.

   It appears from the above that there might have been some change in the amount of leakage from the penstock between two periods, September 1979 and July 1982, in that the 1982 measurement of piezometric heads is higher than the 1979 measurement regardless of measuring points, despite the fact that the reservoir water level was lower in July 1982 than in September 1979 (EL.178.8 vs. EL.193 m).

10.6. Changes in Piezometric Heads during Pumping Operation in July 1982

The change in piezometric heads measured at each point during the period of pumping operation at the boreholes and the sinking shaft for the penstock dewatering test is as shown in Table 10.4.

Table 10.7 shows the change in piezometric heads caused by pumping operation, i.e. the relation between the piezometric head just before pumping operation and the minimum head measured during pumping operation.

Table 10.7 Change in Piezometric Heads During Dewatering Test

(Unit: m)

Location		1 July Head just before pumping	12 July Min. head during pumping	Difference of head
PH-1	1	140.0	123.0	17.0
	2	-	-	-
	3	166.0	139.0	27.0
PH-2	4	134.5	130.0	4.5
	5	141.0	129.0	12.0
	6	156.5	141.5	15.0
PH-3	7	172.0	168.0	4.0
	8	168.5	156.5	12.0
	9	178.5	150.5	28.0

The biggest drop of piezometric heads was observed at the piezometers installed at PH-1-3 and PH-3-9, the lowest point of each location at PH-1 and PH-3. The next biggest drop was observed at PH-1-1, PH-2-5 and PH-3-8, the highest point of PH-1, the middle point of PH-2 and PH-3, respectively. A few drop was observed at PH-2-4 and PH-3-7, the highest point of PH-2 and PH-3.

The difference of piezometric heads observed on July 12, 1982 at the highest and lowest points of each location PH-1, PH-2 and PH-3 is compared with that observed in June, 1982 just before pumping operation, as shown in Table 10.8 below:

Table 10.8. Difference of Piezometric Heads between the Highest and Lowest Points of Each Location

Location	June 1982 (before pumping oper.)	July 12, 1982 (after pumping oper.)
PH-1 (3-1)	28.5 m	16.0 m
PH-2 (6-4)	33.0	11.5
PH-3 (9-7)	11.5	-17.5

As is obvious from this table, the difference of heads among all locations is found relatively small on July 12, when the penstock was dewatered. This means that the hydraulic gradient from the bottom to upward becomes gentler, or the movement of leaked water, particularly from the bottom to upward, becomes slow. Furthermore, a conspicuous drop of piezometric heads observed at the piezometers installed at the lowest point during the dewatering test suggests that the amount of leakage from the penstock would be significant when the penstock is full of water.

10.7. Considerations on the Results of Piezometric Head Measurements in 1979 and 1982

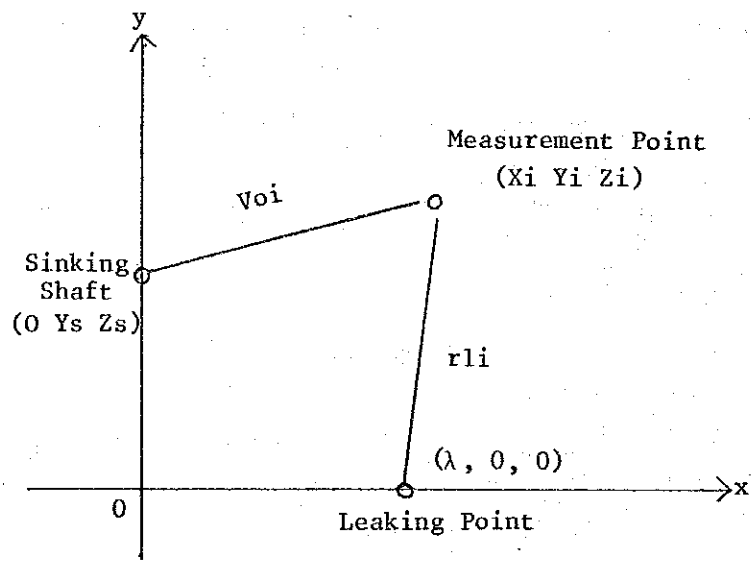
As mentioned in the foregoing subsection, the results of piezometric head measurements effected in 1979 and 1982 differ in context. The piezometric head measurements in 1982 were found higher than those in 1979, regardless of measuring points, despite the fact that the reservoir water level was lower in 1982 than in 1979. This suggests that there might have occurred changes in the amount of leakage from the penstock during a period from 1979 to 1982.

It is extremely difficult to make a quantitative analysis of the change in the amount of leakage from the penstock on the basis of these measurement data only. However, an attempt was made to estimate it using a theoretically simplified method as follows.

10.7.1. Simplified Theory to Assume Locations of Leakage

Fig. 10.4 shows a three-dimensional, rectangular coordinate system, comprised of "X" axis (horizontal) showing the penstock route, "Y" axis (vertical) showing a line passing through the sinking shaft, and "Z" axis in an upward direction. A leaking point should be on the "X" axis as it is very probable that the penstock is leaking. In the Figure, measurement points can be expressed as "Xi", "Yi" and "Zi", with i representing the number of piezometers, i.e., 1 through 9.

Fig. 10.4



Assuming that the penstock would be leaking at one point, and when indicating it as (λ, 0, 0), in which λ is unknown figure and must be determined by calculation, then the location of the sinking shaft, leakage point and measurement points in the above coordinate system are expressed as below:

- Sinking shaft : (0, Ys, Zs)
- Leakage point : (λ, 0, 0)
- Measurement point: (Xi, Yi, Zi)

Leakage is assumed to start from the point and get into surrounding mountain masses in a spherical form. Applying the Darcy's equation with a permeability coefficient in the surrounding mountain masses designated as "k", the following formulae can be established:

$$V = -k\frac{\partial h}{\partial r}\ , \quad q = 4\pi r^2 \cdot V \ \ldots\ldots\ldots\ldots\ (1\text{-}1)$$

Here, it is assumed that the water pressure would be constant ("ho") under the condition of $r \leqq r_{10}$, and leakage would start propagating at $r = r_{10}$. Where, r_{10} is an assumed leakage radius and unknown figure.

When placing $h = ho$ at $r \doteq r_{10}$ and $h = h\infty$ at $r \to \infty$ in the above formulae, an augmentation of piezometric head based on a water level $h\infty$ at $r \to \infty$, at an arbitrary point r_{1i} which represents a distance to an arbitrary measurement point of the piezometric head from the center of a leakage point, is expressed as below.

$$\Delta hi = (ho - h\infty)\frac{r_{10}}{r_{1i}} \ \ldots\ldots\ldots\ldots\ldots\ (1\text{-}2)$$

This value represents the augmentation due solely to leakage from the penstock. The actual piezometric head, however, has to include an effect of the sinking shaft. Hence, a water level at an arbitrary point of r_{1i} should be expressed as below.

$$hi = h\infty + (ho - h\infty)\frac{r_{10}}{r_{1i}} + (hs - h\infty)\frac{r_{oo}}{r_{oi}} \ \ldots\ldots\ (1\text{-}3)$$

Where,

- r_{oo} : a radius of the sinking shaft
- r_{oi} : a distance to an arbitrary measurement point from the center of the sinking shaft
- hi : a groundwater level at an arbitrary point based on EL.68.5 m (at the center line of the penstock)
- $h\infty$: a groundwater level at an infinite point based on the same EL
- hs : a water level in the sinking shaft based on the same EL

When placing $\frac{r10}{r1i} = x_i$, then,

$$hi = ho[xi + \frac{hs}{ho} \cdot \frac{roo}{roi}] - h\infty[xi + \frac{roo}{roi} - 1]$$

Hence,

$$xi = \frac{\frac{hi}{ho} + \frac{h\infty}{ho}(\frac{roo}{roi} - 1) - \frac{hs}{ho} \cdot \frac{roo}{roi}}{1 - \frac{h\infty}{ho}} \quad \ldots\ldots \quad (1\text{-}4)$$

Further, when placing $\frac{roo}{roi} - 1 = P_1$,

and $\frac{hs}{ho} \cdot \frac{roo}{roi} = P_2$, then,

$$x_i = \frac{\frac{hi}{ho} + \frac{h\infty}{ho} \cdot P_1 - P_2}{1 - \frac{h\infty}{ho}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1\text{-}5)$$

A value of "i" means an arbitary number and can be selective from 1 to 9. Any one of those numbers selected is designated as "m" to serve as the base value in the following calculations.

When, placing $xm = \frac{r10}{r1m}$, then,

$$x_i = x_m(\frac{r1m}{r1i})$$

On the basis of geometrical condition,

$$r_{1m}^{2} = (Xm - \lambda)^2 + Ym^2 + Zm^2 = \lambda^2 - 2\lambda Xm + M^2$$

$$r_{1i}^{2} = (Xi - \lambda)^2 + Yi^2 + Zi^2 = \lambda^2 - 2\lambda xi + I^2$$

Where, $M^2 = Xm^2 + Ym^2 + Zm^2$

$I^2 = Xi^2 + Yi^2 + Zi^2$

Hence,

$$\left(\frac{xi}{xm}\right)^2 = \frac{\lambda^2 - 2\lambda Xm + M^2}{\lambda^2 - 2\lambda xi + I^2} \quad \ldots\ldots\ldots\ldots\ldots (1\text{-}6)$$

Further, when placing $\left(\frac{xi}{xm}\right)^2 = Si$, then, an assumed leakage point (λ) can be obtained by solving the above quadratic equation as follows:

$$\lambda = \frac{(Xm - SiXi) \pm \sqrt{(Xm - SiXi)^2 - (1-Si)(M^2 - SiI^2)}}{(1 - Si)} \quad \ldots (1\text{-}7)$$

Fig. 10.4 refers to the location of λ.

10.7.2. Amount of Leakage at the Assumed Leakage Point

Equation of continuity of water flow and the Darcy's equation of motion are as given below:

$$q = 4\pi r^2 \cdot v, \quad v = -k\frac{\partial h}{\partial r}$$

Hence, when placing $h = ho$ at $r = r_{10}$ and $h = h\infty$ at $r \to \infty$ as the boundary condition, then,

$$q = 4\pi k(ho - h\infty)r_{10} \quad \ldots\ldots\ldots\ldots\ldots (2\text{-}1)$$

Where,

k : a coefficient of permeability in mountain masses

When placing the growth of the leakage rate observed in 1982 over that in 1979 as "μ", and assuming no change in the coefficient of permeability of the surrounding mountain masses, then,

$$\mu = \frac{q_{(82)}}{q_{(79)}} = \frac{[ho_{(82)} - h\infty_{(82)}] \times m_{(82)} \cdot rlm}{[ho_{(79)} - h\infty_{(79)}] \times m_{(79)} \cdot rlm} \quad \cdots\cdots\cdots \quad (2\text{-}2)$$

Where,

rlm : a distance from the leakage point to the measurement point "m"

If a leakage point is assumed to have remained unchanged between 1979 and 1982, rlm should remain unchanged.

Therefore, μ can be expressed as:

$$\mu = \frac{[(ho - h\infty)x_m]_{82}}{[(ho - h\infty)x_m]_{79}} \quad \cdots\cdots\cdots\cdots \quad (2\text{-}3)$$

10.7.3. Results of Calculation by Simplified Method

1) Calculation of x_i

The value x_i is expressed as Eq.(1-4), in which ho is an internal pressure of the penstock, hi is a piezometric head measured at each point, roo is a radius of the sphere with a diameter equal to a half of the depth of the sinking shaft, and roi is a distance from the center of the sinking shaft to each piezometer installed, all of whom are known values.

When applying these known values to Eq.(1-4) with a head at an infinite point ($h\infty$) assumed arbitrarily, the values x_i in 1979 and 1982 can be obtained. Table 10.9 shows those values thus obtained. Shown in Table 10.10 are data necessary for the calculation of values, Xi, Yi, Zi, I^2, roi, P_1 and P_2.

2) Estimation of Leakage Point

An assumed leakage point (λ) can be obtained using Eq. (1-7). Values "Si" necessary for the calculation are shown in Table 10.11. These values are obtained by squaring the ratio of a distance between the measurement point selected as the base point "m" and the leakage point, to that between an arbitrary measurement point "i" and the leakage point. In the Table, figures in the columns "x_m=1", "x_m=4" and "x_m=7" are the values "Si", or $(\frac{Xi}{xm})^2$ in case the measurement points 1, 4 and 7 are selected as the base point, respectively. The values "xm" means "xi" at each base point.

Values "λ" calculated using the above "Si" values are exhibited in Tables 10.12.1 and 10.12.2. Figures in the columns "xm=1", "xm=4" and "xm=7" are values "λ" in case the measurement points 1,4 and 7 are selected as the base point, respectively. Water levels at infinite point "$h\infty$" are assumed for three different cases of -30, 0 and +30. The formula regarding "λ" is expressed by the quadratic equation and entails two solutions, but either of them is meaningless. Table 10.12.1 shows the result of the calculation based on the measurement data taken in September 1979, which can be summarized as given in Table 10.13.1.

Table 10.9 <u>Values of Xi</u>

	1979		1982		Xi 1979			Xi 1982		
	ho	hi	ho	hi	$h_{\infty i}$-30	0	+30	$h_{\infty i}$-30	0	+30
1	121.5 m	57.7 m	111.5 m	69.0 m	0.5882	0.5737	0.5496	0.7096	0.7265	0.7557
2	"	60.1	"	-	0.6041	0.5945	0.5787			
3	"	63.2	"	97.5	0.6245	0.6189	0.6097	0.9110	0.9821	1.1054
4	"	52.8	"	62.0	0.5623	0.6022	0.6682	0.6671	0.7387	0.8630
5	"	50.0	"	68.5	0.5446	0.5870	0.6571	0.7138	0.8055	0.9648
6	"	68.6	"	95.0	0.6669	0.7350	0.8478	0.9006	1.0377	1.2758
7	"	81.3	"	101.5	0.7393	0.6144	0.6852	0.9344	0.9646	1.0171
8	"	72.5	"	99.0	0.6814	0.6476	0.5918	0.9168	0.9434	0.9895
9	"	68.6	"	113.0	0.6557	0.6158	0.5499	1.0158	1.0693	1.1621
Sinking shaft	"	-34.0	"	-34.0						

Table 10.10 List of Relevant Principal Values

		Xi	Yi	Zi	Piezometric head, 1979	Piezometric head, 1982	I^2	roi	P1	1979 P2	1982 P2
PH-1	1	96	13	11.5	126.2	137.5	9517.25	97.74	-0.647	-0.0988	-0.1076
	2	96	13	1.5	128.6	-	9387.25	96.55	-0.643	-0.0999	-0.1089
	3	96	13	-11.5	131.7	166.0	9517.25	97.74	-0.647	-0.0988	-0.1076
PH-2	4	42	-14	13	121.3	130.5	2129.00	57.56	-0.401	-0.1676	-0.1827
	5	42	-14	1.5	118.5	137	1962.25	55.06	-0.373	-0.1755	-0.1912
	6	42	-14	-10.0	137.1	163.5	2060.00	56.69	-0.391	-0.1704	-0.1857
PH-3	7	185	-14	39.5	149.8	170	35981.25	193.47	-0.822	-0.0498	-0.0543
	8	185	-14	18	141.0	167.5	34745.00	189.68	-0.818	-0.0509	-0.0555
	9	185	-14	-1.0	137.1	181.5	34422.00	188.32	-0.817	-0.0512	-0.0558
Sinking shaft		0	21	-5.0	34.5 *	34.5					

* Sinking shaft

W.L. 98.00 - shaft center EL 63.5 roo = 1/2(EL 98 - EL 29) = 34.5

Sinking shaft bottom EL 27.00 Zi = 63.5 - 68.5 = -5.0 m

Overflow level EL 98.00

Penstock center line EL = 68.5

Table 10.11.1 Values of $Si(x_i/x_m)^2$ in 1979

Si / PH	$h_\infty = -30$			$h_\infty = 0$			$h_\infty = +30$		
	$x_m = 1^*$	$x_m = 4^*$	$x_m = 7^*$	$x_m = 1$	$x_m = 4$	$x_m = 7$	$x_m = 1$	$x_m = 4$	$x_m = 7$
1	1	1.0942	0.6330	1	0.9076	0.8719	1	0.6765	0.6434
2	1.0548	1.1542	0.6677	1.0738	0.9746	0.9363	1.1087	0.7501	0.7133
3	1.1272	1.2335	0.7135	1.1638	1.0562	1.0147	1.2295	0.8326	0.7918
4	0.9139	1	0.5785	1.1018	1	0.9607	1.4782	1	0.9510
5	0.8572	0.9380	0.5426	1.0469	0.9502	0.9128	1.4295	0.9671	0.9197
6	1.2855	1.4066	0.8137	1.6414	1.4897	1.4311	2.3795	1.6098	1.5309
7	1.5798	1.7286	1	1.1469	1.0409	1	1.5543	1.0515	1
8	1.3420	1.4685	0.8495	1.2742	1.1565	1.1110	1.1595	0.7844	0.7460
9	1.2427	1.3598	0.7866	1.1522	1.0457	1.0046	1.0011	0.6773	0.6441

* $x_m = 1$, $x_m = 4$, and $x_m = 7$ mean that a value of Si is calculated with the measurement points 1, 4 and 7 set as a base point, respectively.

Table 10.11.2 Values of Si $(x_i/x_m)^2$ in 1982

Si / PH	$h_\infty = -30$			$h_\infty = 0$			$h_\infty = +30$		
	$x_m = 1$	$x_m = 4$	$x_m = 7$	$x_m = 1$	$x_m = 4$	$x_m = 7$	$x_m = 1$	$x_m = 4$	$x_m = 7$
1	1	1.1315	0.5767	1	0.9672	0.5673	1	0.7668	0.5520
2	-	-	=	-	-	-	-	-	-
3	1.6482	1.8649	0.9505	1.8274	1.7676	1.0366	2.1396	1.6407	1.1812
4	0.8838	1	0.5097	1.0339	1	0.5865	1.3041	1	0.7199
5	1.0119	1.1449	0.5836	1.2293	1.1890	0.6973	1.6300	1.2498	0.8998
6	1.6108	1.8226	0.9290	2.0402	1.9734	1.1573	2.8501	2.1855	1.5734
7	1.7339	1.9619	1	1.7629	1.7051	1	1.8115	1.3890	1
8	1.6693	1.8887	0.9627	1.6862	1.6310	0.9565	1.7145	1.3146	0.9465
9	2.0492	2.3186	1.1818	2.1663	2.0954	1.2289	2.3648	1.8133	1.3054

Table 10.12.1 Values of λ in 1979.

PH \ λ	$h_\infty = -30$			$h_\infty = 0$			$h_\infty = +30$		
	$x_m = 1$	$x_m = 4$	$x_m = 7$	$x_m = 1$	$x_m = 4$	$x_m = 7$	$x_m = 1$	$x_m = 4$	$x_m = 7$
1	-	69.29 1,269.20	156.95 520.06	-	67.45 -1,044.36	1,430.47 151.07	-	64.54 - 206.36	534.51 156.65
2	49.09 142.91	68.53 823.86	156.57 571.08	56.12 135.88	66.98 -4,126.94	2,835.82 150.23	50.78 141.22	64.52 - 304.69	657.50 155.35
3	-	70.47 584.06	154.71 658.59	-	68.95 2,044.76	-12,065.27 148.42	-	66.60 - 519.76	894.15 152.80
4	69.29 1,269.1	-	130.43 632.10	-1,044.36 67.45	-	7,242.12 119.23	-206.39 64.54	-	5,801.28 119.45
5	71.36 768.95	-	132.31 576.97	-2,288.31 69.54	-	3,242.91 120.91	-234.19 66.73	-	3,524.90 120.74
6	65.80 -247.42	-	123.08 1,496.08	-148.71 64.33	-	-690.41 110.99	-55.06 60.77	-	-564.29 109.58
7	156.96 520.05	130.42 632.11	-	151.07 1,430.64	119.23 7,243.43	-	156.65 534.48	119.45 5,803.94	-
8	145.70 744.77	121.50 858.96	-	145.00 874.17	116.87 2,080.61	81.96 288.04	143.72 1,342.27	109.33 -1,065.86	-
9	142.63 960.79	118.69 1,046.2	-	141.69 1,397.83	113.76 6,514.45	-397.04 767.04	139.93 162,048.25	105.57 - 621.84	-

Table 10.12.2 Values of λ in 1982

λ / PH	$h_\infty = -30$			$h_\infty = 0$			$h_\infty = +30$		
	$x_m = 1$	$x_m = 4$	$x_m = 7$	$x_m = 1$	$x_m = 4$	$x_m = 7$	$x_m = 1$	$x_m = 4$	$x_m = 7$
1	-	69.62 943.67	453.75 158.76	-	68.08 -3,168.76	444.29 159.08	-	65.79 -336.91	429.70 159.62
2	=	-	-	-	-	-	-	-	-
3	-	74.52 242.35	3,638.40 149.56	-	73.99 258.71	-4,819.44 148.06	-	73.26 287.30	-936.18 145.84
4	943.81 69.62	-	534.04 133.27	-3,169.92 68.08	-	645.54 130.12	-336.93 65.79	-	979.50 125.56
5	-9,061.47 69.85	11.14 72.86	640.15 130.69	-455.09 68.09	15.85 68.15	902.07 126.76	-152.99 65.56	20.34 63.66	2,817.08 121.22
6	-157.33 64.51	-	3,991.97 120.20	-82.07 62.25	-	-1,849.66 115.47	-33.39 59.02	-	-523.79 109.01
7	158.76 453.78	133.27 534.06	-	159.08 444.24	130.12 645.50	-	159.62 429.73	125.56 979.66	-
8	148.65 487.30	126.35 565.47	-	148.79 480.61	123.53 699.22	-	149.01 470.11	119.35 1,159.74	-
9	148.80 390.85	128.53 458.37	93.46 276.54	149.47 373.14	126.69 504.40	103.67 266.33	150.53 349.89	124.04 597.61	114.94 255.06

Table 10.13.1. Summarized Values of λ in 1979

(Unit: m)

Point to be selected as "x_m"	h∞ = -30	h∞ = 0	h∞ = +30
Point 1	50 - 70 140 - 160	70 120 - 130	50 - 60 140 - 150
Point 4	60 - 70 140 - 150	70 120	65 100 - 120
Point 7	120 - 160	100 - 150	100 - 150

Table 10.12.2 shows the result of the calculation based on the measurement data taken in July 1982, which can be summarized as below:

Table 10.13.2. Summarized Values of λ in 1982

(Unint: m)

Point to be selected as "x_m"	h∞ = -30	h∞ = 0	h∞ = +30
Point 1	65 - 70 150	60 - 70 150 - 160	60 - 65 150 - 160
Point 4	70 - 72 130	68 - 75 120 - 130	63 - 70 120 - 125
Point 7	100 - 160	100 - 160	110 - 160

The comparison between the results of calculations based on the measurement data in 1979 and those in 1982, reveals, with regard to the leakage point, the following:

a) Regardless of the values "h∞", there is no substantial difference in the values "λ", or the location of an assumed leakage point.

b) When the points 1 and 4 are selected as the base point, an assumed leakage point can be sought at two different places 50-70 m and 100-160 m distant therefrom, but when the point 7 is selected as the base point, it can be sought at a place 100 - 160 m distant therefrom.

c) There is practically no change in the assumed leakage point between data in 1979 and 1982. This implies that no new, additional leakage would have taken place during the period from 1979 to 1982.

d) The distances from the center and bottom of the sinking shaft to the bifurcation point of the penstock are about 40 m and 60 m respectively. And, the distance to the connection of the penstock with the concrete structure is approximately 125 m.

Therefore, supposing that the penstock leaks at two points, it can be assumed that they would be at the neighborhood of the bifurcation point and the connection with the concrete structure. This, however, must be evidenced by visual inspections.

3) Change in Leakage Rate from 1979 to 1982

As shown in the preceding subsection, the increase in leakage rate from 1979 to 1982 is given by Eq.(2-3). The result of the calculation is tabulated as below:

Table 10.14. Increase in Leakage Rate from 1979 to 1982 (Q82/Q79)

No. of Piezometer	h_∞ = -30	h_∞ = 0	h_∞ = +30
1	1.127	1.163	1.225
2	-	-	-
3	1.362	1.457	1.615
4	1.108	1.126	1.151
5	1.224	1.260	1.308
6	1.261	1.296	1.341
7	1.180	1.441	1.323
8	1.257	1.337	1.490
9	1.447	1.594	1.883

The result in this Table reveals the following:

a) The growth varies to some extent depending on assumed water levels at an infinite point.

   It becomes higher in proportion to the rise in the assumed water level at an infinite point.

b) The leakage rate in 1982 is greater than that in 1979 irrespective of measuring points of the piezometric head. It increases by 10-40% in case of the lower ground water level at an infinite point, and by 20-80% in case of the higher ground water level.

10.8. Estimation of Leakage Rate

In the preceding subsection, a study was conducted, using the simplified theory, to detect an assumed leakage point of the penstock and to check changes in the leakage rate during a period from 1979 to 1982. However, a quantitative appraisal on the amount of leakage was not possible in the study. As discussed in 10.7.2, the amount of leakage is expressed as:

$$q = 4\pi k(ho - h\infty)r_{10} \quad \text{(Eq.2-1)}$$

In order to make a quantitative appraisal using this equation, it is necessary to obtain the coefficient of permeability of the surrounding mountain masses. A method to obtain the coefficient of permeability is discussed in the following subsection.

10.8.1. Simplified Theory for Estimating Coefficient of Permeability

The amount of leakage from the penstock is affected by the coefficient of permeability of the surrounding mountain masses as well as water pressures at the leakage point. At the present stage, it is extremely difficult to estimate the coefficient due to lack of relevant data. Only available are the data on the pumping test at the sinking shaft performed in July, 1982. Fig. 10.3 and Table 10.3 show changes with time of the piezometric heads during the pumping test in July, 1982.

An attempt was made to estimate the coefficient of permeability of the surrounding mountain masses, using the measured data through a simplified method under the following condition:

- Sinking shaft:
  Converted radius: 1.41 m (cross section of 6.25 m^2)
  Elevation of the shaft invert: EL.27.0 m
  Below the shaft invert are assumed impermeable layers.

- Water level in the sinking shaft:
  EL.98.0 m at t=0, and EL.72.0 m at t=4 hrs, during which the water level would decrease gradually.

- The rate of pumping water from the sinking shaft would be constant at all times.

10.8.2. Ground Water Levels at Each Piezometer Measurement Point

Table 10.4 shows the estimated ground water levels at each piezometer measurement point before, during and after the penstock dewatering test. Shown below is the reproduction from the Table on July 1 (t=0) and July 12 (t=11 days).

Table 10.15 Ground Water Levels Before and After Pumping Test at the Sinking Shaft

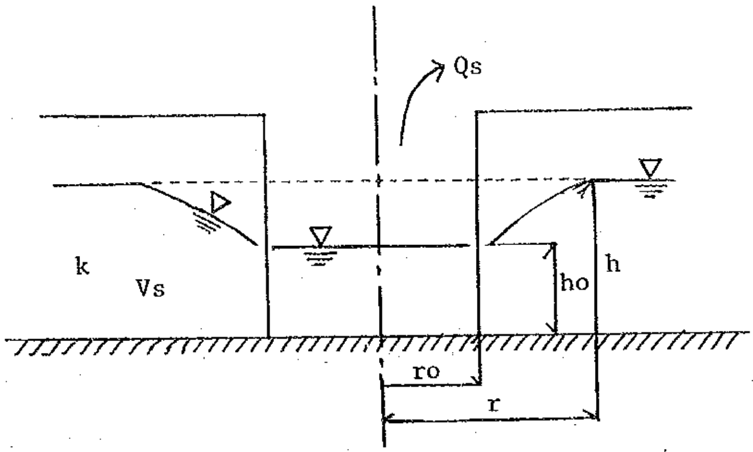
(Unit: m)

Measurement Point		Distance from Sinking Shaft	Water Level before Pumping	Water Level 11 Days after Pumping	Balance
PH-1	1	97	140	123	17
	2		-	-	-
	3		166	139	27
PH-2	4	54	134.5	130	4.5
	5		141	129	12.0
	6		156.5	141.5	15.0
PH-3	7	187	172	168	4.0
	8		168.5	156.5	12.0
	9		178.5	150	28.0

10.8.3 Deployment of Theory

The ground water table in the surrounding mountain masses is assumed to have a free surface developing semi-infinitely as shown in Fig. 10.5 below.

Fig. 10.5 Ground Water Table in the Mountain Masses Surrounding the Sinking Shaft



It is also assumed that the ground water would flock to the sinking shaft in the steady state at t=0.

The upper end of the sinking shaft is of a structure to allow overflow, and the maximum water level depends on the elevation of the overflow crest.

1) Solution of Steady Flow:

Definitions of the symbols used in the equations for the solution are as follows:

- Coefficient of permeability of the surrounding mountain masses : k
- Overflow discharge from the sinking shaft : Qs
- Water level in the sinking shaft : h_o
- Water flow velocity in surrounding mountain masses (steady flow) : Vs
- Head at an arbitrary point : h
- Distance from the center of the sinking shaft to an arbitrary point : r
- Radius of the sinking shaft : ro

$$Qs = -2\pi rhVs$$

$$Vs = -k \frac{dh}{dr}$$

Hence,

$$q = 2\pi rh\frac{dh}{dr}$$

When solving the above equation with h=ho at r=ro, then,

$$h^2 - ho^2 = \frac{Qs}{\pi k} \log \frac{r}{ro} \quad \text{............. (3-1)}$$

2) Solution of Unsteady Flow:

The dewatering test of the sinking shaft performed in 1982 may be regarded as the pumping test of a well. To simplify the process of calculations, the following assumptions were adopted in obtaining an unsteady flow at the time of pumping water from the well.

a) The surrounding mountain masses would continue infinitely and have a uniform coefficient of permeability.

b) The ground water table in the mountain masses would have a a constant level of "H" at r=R.

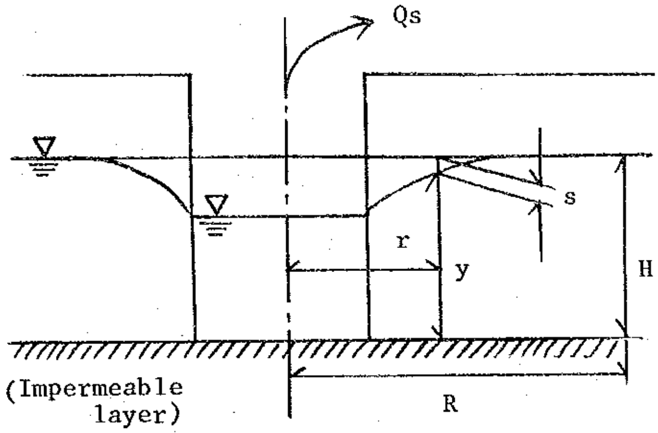
c) Water would flow out of the well only.

d) No leakage would take place from the penstock at the pumping test since the intake gates have been closed.

e) Mountain masses below EL.27 m, the shaft invert elevation, would be comprised of impermeable layers.

f) A drop (S) of the water level at an arbitrary point would be smaller than the water level "H" at r=R, as shown below.

Fig.10.6



g) A ground water table used for the calculation is as given in Fig.10.6 above.

The equations of motion and continuity for the ground water flow at an arbitrary point r are expressed as:

$$Qr = 2\pi rykv = 2\pi ryk\frac{\partial y}{\partial r} \quad \cdots\cdots\cdots\cdots \quad (4\text{-}1)$$

$$\frac{\partial Qr}{\partial r} \cdot dr = 2\pi rdr \cdot \frac{\partial y}{\partial t} \cdot \mu \quad \cdots\cdots\cdots \quad (4\text{-}2)$$

Where,

y : Ground water level at an arbitrary point (at a distance "r" from the center line of the well).

k : Coefficient of permeability in the surrounding mountain masses.

ro : Radius of the well

μ : Porosity of the surrounding mountain masses

When eliminating "Q" from the above equations, then,

$$\frac{\partial^2 y^2}{\partial r2} + \frac{1}{r} \cdot \frac{\partial y^2}{\partial r} = \frac{\mu}{ky} \cdot \frac{\partial y^2}{\partial t} \text{ .... } (4\text{-}3)$$

When converting "y" to "S" by applying "$y=H-S = H(1-\frac{S}{H})$" with $\frac{S}{H}$ <<1, the equation (4-3) can be expressed as:

$$\frac{\partial^2 S}{\partial r2} + \frac{1}{r} \cdot \frac{\partial S}{\partial r} = \frac{\mu}{kH} \cdot \frac{\partial S}{\partial t} \text{ .... } (4\text{-}4)$$

When pumping water from the sinking shaft at a uniform rate of (Q), and if it reaches the steady state at $t=t_s$, then, a solution for steady flow (S1) in an ultimate time can be expressed as below:

Since $H^2-y^2 \doteqdot 2HS1 = \frac{Q}{\pi k} \cdot \ln\frac{R}{r}$,

$$S1 = \frac{Q}{2\pi kH} \ln \frac{R}{r} \text{ .... } (4\text{-}5)$$

Strictly speaking, the equation (4-4) should be solved under the conditions prior to pumping water from the sinking shaft, i.e. just before the water level in the shaft begins lowering. However, for the sake of simplification, it was assumed that S=0 at t=0.

If a solution of the equation (4-4) can be derived by adding the solution of unsteady flow to that of steady flow, and if the following equation can be established,

S = S1 + S2 (solution of unsteady flow)

Then,

$$S = S1 + S2 = 0, \text{ when } t=0$$

and,

$$S = \frac{Q}{2\pi kH} \ln \frac{R}{r}, \text{ when } t=ts$$

When r=R, then S=0 irrespective of time.

Hence, with regard to the solution of unsteady flow (S2), the following are boundary conditions for the equation (4-4):

$$S2 = -\frac{Q}{2\pi kH} \ln \frac{R}{r}, \text{ when } t=0$$

$$S2 = 0, \text{ when } t=t_s$$

$$S2 = 0, \text{ when } r=R$$

With "S" in the equation (4-4) placed as S=f(r)T(t), where f(r) is a function relating only to "f" and, T(t) a function relating only to "t", segregation of variables can be made as below:

$$\frac{1}{f}\left(\frac{d^2f}{dr2} + \frac{1}{r} \cdot \frac{df}{dr}\right) = \frac{\mu}{kH} \cdot \frac{1}{T} \cdot \frac{dT}{dt} \quad \ldots\ldots\ldots \quad (4\text{-}6)$$

When placing an arbitrary constant as $-(\frac{\alpha}{R})^2$, then

$$\frac{1}{T} \cdot \frac{dT}{dt} = -\frac{H}{\mu}\left(\frac{\alpha}{R}\right)^2 k$$

Further, when placing $\tau = \frac{kHt}{\mu R^2}$, then,

$$T = e^{-\frac{\alpha^2 \cdot kHt}{\mu R^2}} = e^{-\alpha^2 \tau} \quad \ldots\ldots\ldots\ldots \quad (4\text{-}7)$$

In the meantime, an equation satisfying f(r) can be expressed as below, when placing $\rho = \frac{\alpha r}{K}$:

$$\frac{d^2 f}{d\rho^2} + \frac{1}{\rho} \cdot \frac{df}{d\rho} + f = 0 \quad \ldots\ldots\ldots\ldots \quad (4\text{-}8)$$

Since this equation is of Bessel function of 0 order, a final solution of S_2 can be obtained as follows:

$$S_2 = \sum^{\alpha} A \cdot Jo(\rho) \cdot e^{-\alpha^2 \tau} \quad \ldots\ldots\ldots\ldots \quad (4\text{-}9)$$

As $S_2 = 0$, when r=R, therefore,

$$0 = \sum^{\alpha} A \cdot Jo(\alpha) \cdot e^{-\alpha^2 \tau}$$

Hence, to make the value "A" anything but 0, it is necessary to choose as "α" such values that enable Jo(α)=0. Such "a" values are innumerable. They can be tentatively determined in the order of small numbers starting from 1 as shown below:

n	1	2	3	4	5	....
αn	2.4048	5.5201	8.6537	11.795	14.9309	

Now that, $S_2 = -\frac{Q}{2\pi kH} \ln \frac{r}{R}$ at t=0, and if this is replaced with $-\psi(r)$, then, $-\psi(r)$ can be expressed as $\Sigma AJo(\rho)$.

Therefore,

$$A = \frac{2}{J_1(\alpha)^2} \int_0^1 \psi(r) Jo(\alpha \frac{r}{R}) \cdot \frac{r}{R} d(\frac{r}{R}) = -\frac{Q}{\pi k H \alpha^2 J_1(\alpha)^2}$$

Consequently,

$$S2 = -\frac{Q}{\pi kH} \cdot \frac{\alpha}{\Sigma} \cdot \frac{Jo(\alpha\frac{r}{R})}{\alpha^2 J_1(\alpha)^2} \cdot e^{-\alpha^2\tau} \quad \text{.........} \quad (4\text{-}10)$$

From the above, a drop of the water level "S" can be expressed as:

$$S = S_1 + S_2 = \frac{Q}{2\pi kH}\left[\ln\frac{R}{r} - 2\sum^{\alpha} \frac{Jo(\alpha\frac{r}{R})}{\alpha^2 J_1(\alpha)^2} \cdot e^{-\alpha^2\tau}\right]$$

The above equation is further developed, when placing

$\frac{r^2}{R^2 \cdot \tau} = \varepsilon$, as follows:

$$S\frac{kH}{Q} = \frac{1}{4\pi}\left[\ln\frac{1.5^2}{\varepsilon} + \frac{\varepsilon}{4.1!} - \frac{\varepsilon^2}{4^2 \cdot 2 \cdot 2!} + \frac{\varepsilon^4}{4^2 \cdot 3.3!} \cdots\cdots\right]$$

When the value ε is small, the above can be represented as:

$$S\frac{kH}{Q} = \frac{1}{4\pi}\ln\frac{1.5^2}{\varepsilon} = \frac{1}{2\pi}\ln\left(\frac{1.5\sqrt{\frac{Hkt}{\mu}}}{r}\right) \quad \text{........} \quad (4\text{-}11)$$

10.8.4. <u>Result of Calculations</u>

As given in the equation (4-11), a drop of the water level "S" in an arbitrary time at an arbitrary point can be expressed as below:

$$S = \frac{Q}{2\pi kH}\ln\left(\frac{1.5\sqrt{\frac{Hkt}{\mu}}}{r}\right) \quad \text{..............} \quad (4\text{-}12)$$

As the groundwater level at t=0 is EL.98 m and the shaft invert elevation is at EL.27.0 m, "ho" should be 71 m, and the water depth (h) based on EL.27 m at t=0 would be as exhibited in Table 10.16 below.

Table 10.16 Water Depth Based on EL.27 m

No. of Piezometer		1 July	12 July
PH-1	1	113 m	96 m
	2	-	-
	3	139	112
PH-2	4	107.5	103
	5	114	102
	6	129.5	114.5
PH-3	7	145	141.0
	8	141.5	129.5
	9	151.5	123.0

Water depths at PH-1, PH-2 and PH-3, taking an average of those at points 1 and 3; 4, 5 and 6; and 7, 8 and 9, respectively, are as follows:

	1 July	12 July
PH-1	126 m	104 m
PH-2	117.0	106.5
PH-3	146.0	131.2

Given that Yo is 1.41 in the equation of $\frac{Qs}{k} = \frac{\pi(h^2-h^2o)}{\log\frac{r}{ro}}$,

then the value Qs/k on July 1, 1982 can be obtained as below:

Table 10.17 Value Qs/k on July 1, 1982

(m^2)

Back calculation from PH-1 data	6,912
- do - from PH-2 data	6,262
- do - from PH-3 data	9,587
Average	7,587

The drop of water level at each point shown in Table 10.16 took place throughout a period from July 1 to July 12, 1982, but the water level in the sinking shaft lowered from EL.98 m to EL.72 m in four hours. The drop of water level at each point after 4 hours estimated from the total drop which took place for 11 days (1 July to 12 July) are as follows:

	Total Drop	Drop after four hours
PH-1	22 m	0.33 m
PH-2	23 m	0.35 m
PH-3	14.7 m	0.22 m

The value "H" at an infinite point is unknown. If it is assumed to be same as the elevation of the overflowing crest of the sinking shaft, Q is expressed as below:

$$Q = Qs + \frac{6.25 \cdot (98 - 72)}{t}$$

Where, H = 98-27 = 71 m, t=4 hrs. = 14,400 sec.

Now that $\frac{Qs}{k}$ = 7,587, Q can be further developed as below:

$$Q = 7{,}587\ k + 0.011$$

The value "k" can be obtained by back calculation using the equation (4-12), assuming that the porosity of the surrounding mountain masses would be 2%.

The result of the back calculation indicates the value "k" for each PH-1, PH-2 and PH-3 to be in the range of 9×10^{-5} to 3×10^{-4} m/sec.

In addition, the value "Qs" obtained from the relation of $\frac{Qs}{k}$ = 7,587, is found to be in the range of 0.7 m^3/sec. to 2.3 m^3/sec. This can be considered as the rate of leakage from the penstock in a steady state.

APPENDIX

LIST OF TABLES AND FIGURES

Table A2.1 Annual Rainfall Data at the Norzagaray Gauging Station

1	1925	3641.90
2	1926	3743.80
3	1927	4281.89
4	1928	2901.00
5	1929	4092.10
6	1930	4533.89
7	1931	3529.90
8	1932	3526.40
9	1933	3072.20
10	1934	4391.70
11	1936	2585.90
12	1937	4280.30
13	1938	2757.70
14	1939	3324.90
15	1948	3532.60
16	1949	2000.00
17	1950	3304.90
18	1951	2707.30
19	1952	3019.60
20	1953	3475.20
21	1954	2168.70
22	1955	2297.30
23	1956	2948.20
24	1957	3526.20
25	1958	2840.60
26	1959	2855.80
27	1960	3781.20
28	1961	3425.70
29	1962	3564.79
30	1963	2392.87
31	1964	2971.96
32	1965	2624.80
33	1967	3212.00
34	1969	2780.90
35	1970	3017.00
36	1971	3338.90
37	1972	4918.39
38	1974	4370.00
39	1975	2565.40
40	1976	3859.02
41	1977	1425.70
42	1978	3001.80
43	1981	2894.00
44	1982	744.30
45	1985	3234.00
46	1986	3433.40

SGM= 0.6957