Fig.8-2 Calculation Model of Left Bank (Slice Section EL.80m)

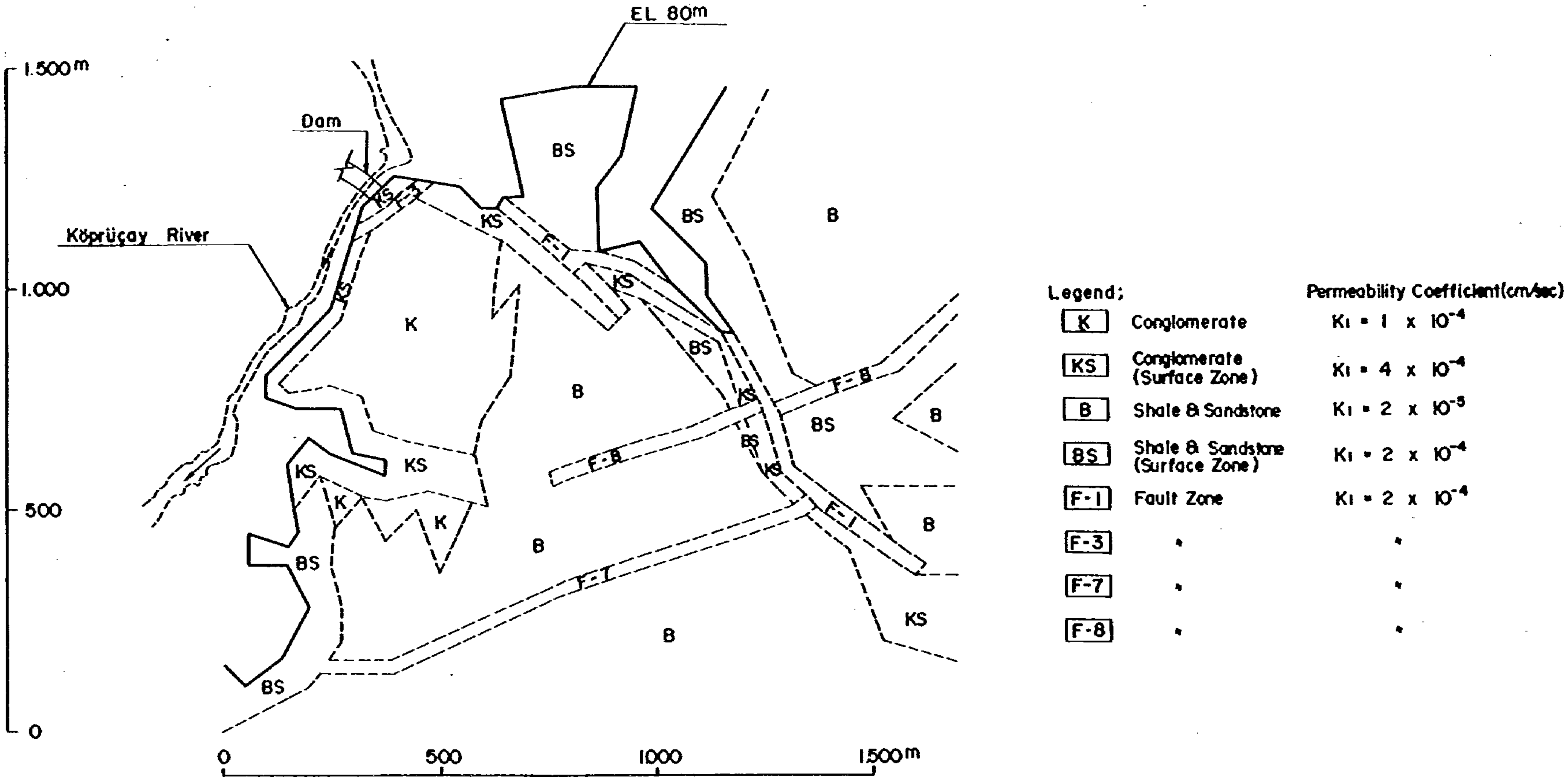


Fig. 8-3 Flow Net Diagram of Right Bank (Slice Section EL.80m)

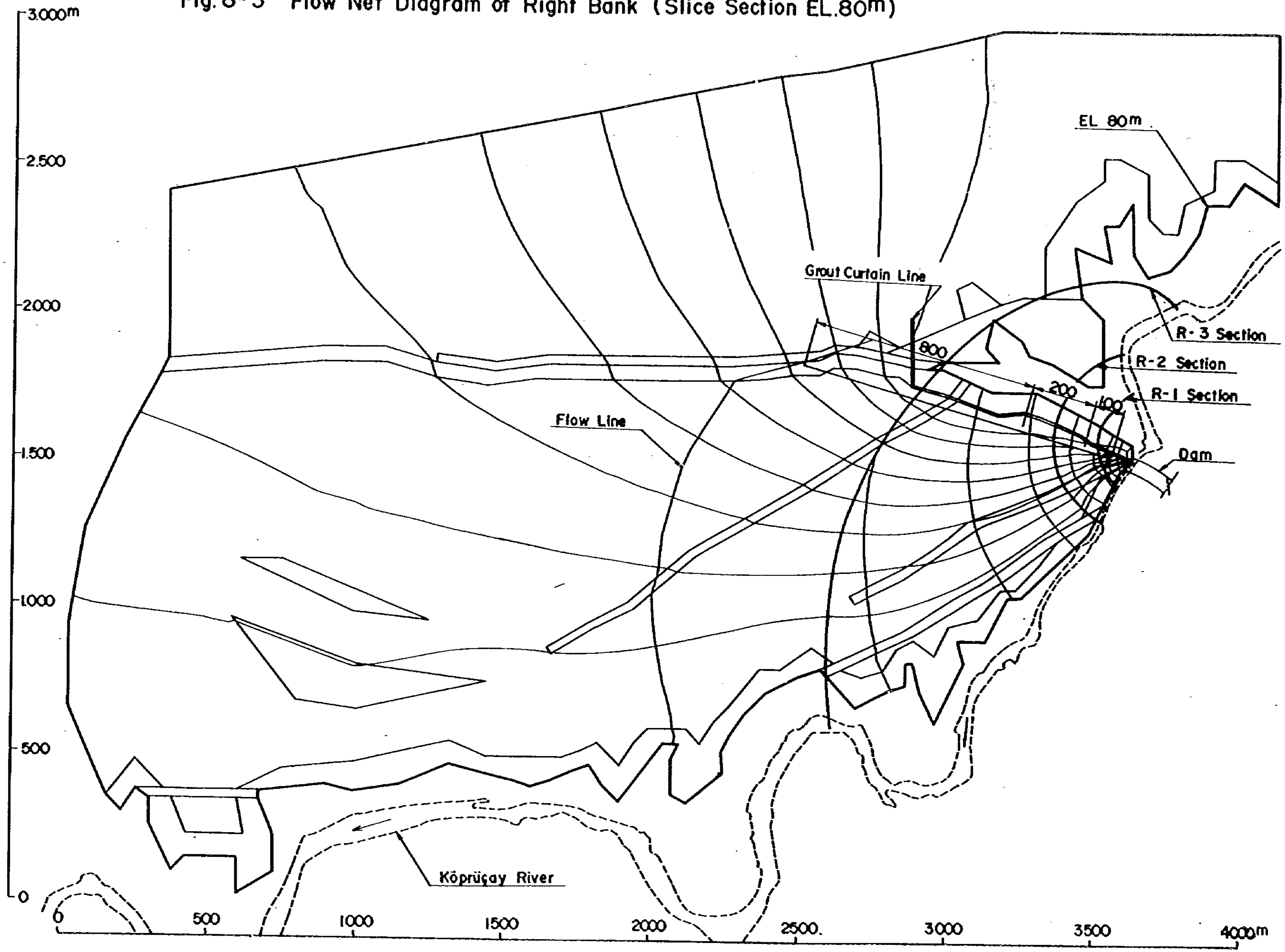


Fig. 8-4 Flow Net Diagram of Left Bank (Slice Section EL.80m)

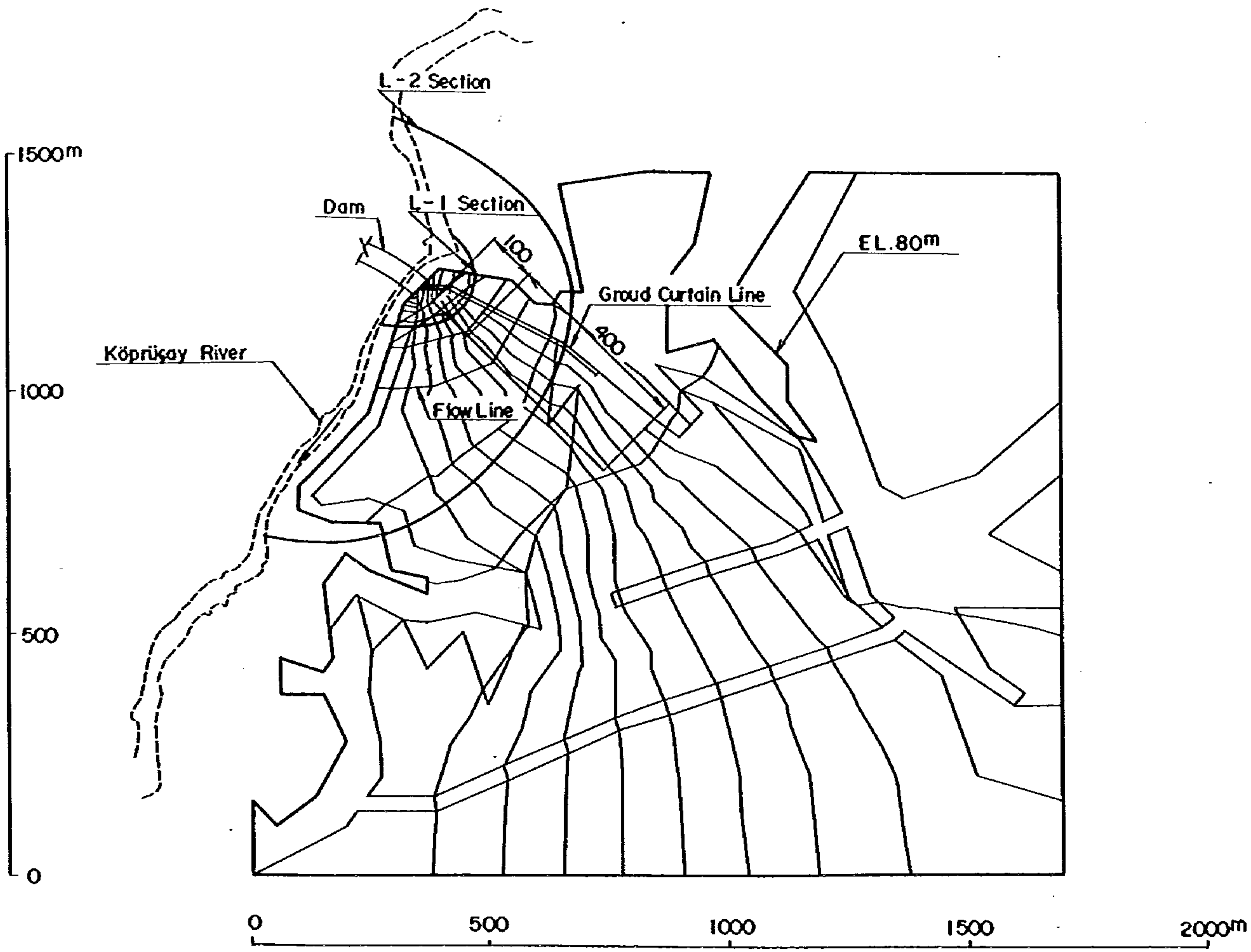
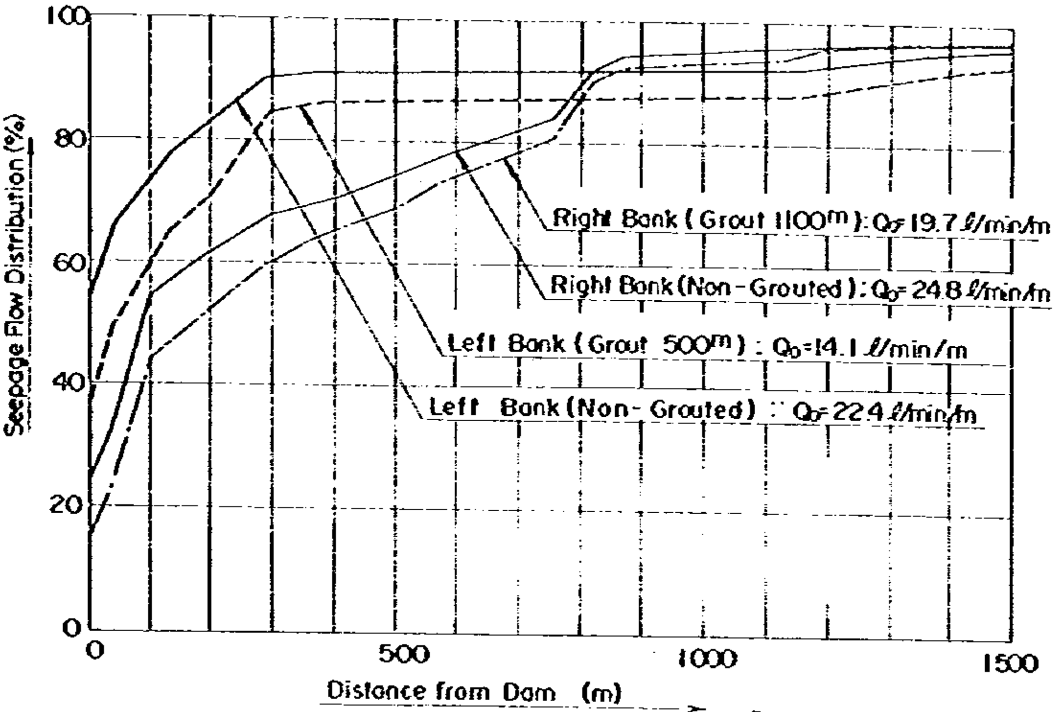


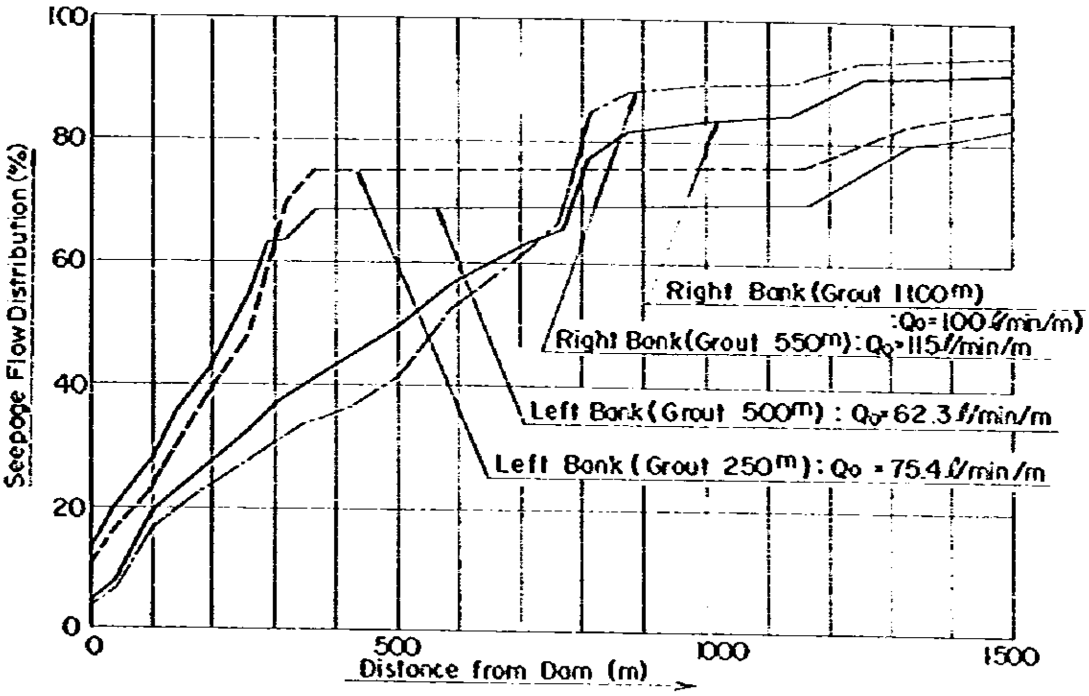
Fig.8-5 Seepage Flow Distribution (Slice Section EL.80m)

(a) Permeability Coefficient K1



Remarks : Distance shows the length along reservoir.

(b) Permeability Coefficient K3



(2) Cross Section Analyses

The cases for which analyses were performed on the cross sections selected in (1), are shown in Table 8-2.

The calculation models for the various cross sections are shown in Figs. 8-6 through 8-11. The results of the calculations are shown in Tables 8-3 through 8-5 and Figs. 8-12 through 8-19. Tables 8-6 and 8-7 indicate total amounts of leakage from the reservoir computed based on the seepage flows shown in Tables 8-3 through 8-5.

Table 8-2 Calculation Cases

Grout & coef. / Section		Non Grouted			Grout (A)			Grout (B)		
		K1	K2	K3	K1	K2	K3	K1	K2	K3
Dam Foundation		D-1-01	D-1-02	-	D-1-A1	-	D-1-A3	D-1-B1	-	D-1-B3
Right Bank	R-1	R-1-01	R-1-02	R-1-03	R-1-A1	R-1-A2	R-1-A3	R-1-B1	R-1-B2	R-1-B3
"	R-2	R-2-01	-	-	-	-	-	-	-	-
"	R-3	R-3-01	-	-	-	-	-	-	-	-
Left Bank	L-1	L-1-01	-	-	L-1-A1	-	-	-	-	-
"	L-2	L-2-01	-	-	-	-	-	-	-	-

(Note) 1. K1 - K3 are permeability coefficients with the values K2 = 5 K1 and K3 = 10 K1 adopted.

2. Grout (A): Case of curtain grouting to EL.-60 m
   Grout (B): Case of curtain grouting to EL.-120 m

Table 8-3 Result of Seepage Flow Analysis
(Case of Permeability Coefficient K_I)

Calculated Section		Distance (m)	Non Grouted		Grout A (EL. - 60 m)		Grout B (EL. - 120 m)	
			Unit Seepage Flow (ℓ/min/m)	Seepage Flow (ℓ/min)	Unit Seepage Flow (ℓ/min/m)	Seepage Flow (ℓ/min)	Unit Seepage Flow (ℓ/min/m)	Seepage Flow (ℓ/min)
Dam Foundation		100	16.80	1,680	8.52	852	8.42	842
Right Bank	R-1	100	7.78	778	7.75	775	7.73	773
	R-2	200	5.54	908	(4.52)	(904)	(4.51)	(902)
	R-3	800	1.65	1,320	(1.64)	(1,312)	(1.64)	(1,312)
	Sub-total	1,100		3,006		(2,991)		(2,987)
Left Bank	L-1	100	8.15	815	8.14	814	(8.10)	(810)
	L-2	400	2.52	1,008	(2.51)	(1,004)	(2.50)	(1,000)
	Sub-total	500		1,823		(1,818)		(1,812)
Grand Total				6,509		(5,661)		(5,639)

Remark; ( ) shows estimated values

Table 8-4 Result of Seepage Flow Analysis
(Case of Permeability Coefficient $5K_1$)

Calculated Section		Distance (m)	Non Grouted		Grout A (EL. - 60 m)		Grout B (EL. - 120 m)	
			Unit Seepage Flow (ℓ/min/m)	Seepage Flow (ℓ/min)	Unit Seepage Flow (ℓ/min/m)	Seepage Flow (ℓ/min)	Unit Seepage Flow (ℓ/min/m)	Seepage Flow (ℓ/min)
Dam Foundation		100	(76.83)	(7,683)	(27.15)	(2,715)	(18.83)	(1,883)
Right Bank	R-1	100	28.28	2,828	23.47	2,347	21.61	2,161
	R-2	200	(16.50)	(3,300)	(13.69)	(2,738)	(12.61)	(2,522)
	R-3	800	(6.00)	(4,800)	(4.98)	(3,984)	(4.58)	(3,664)
	Sub-total	1,100		(10,928)		(9,069)		(8,347)
Left Bank	L-1	100	(29.63)	(2,963)	(24.59)	(2,459)	(22,64)	(2,264)
	L-2	400	(9.16)	(3,664)	(7.60)	(3.040)	(7.00)	(2,800)
	Sub-total	500		(6,627)		(5,499)		(5,064)
Grand Total				(25,238)		(17,283)		(15,294)

Remark; (   ) shows estimated values

Table 8-5 **Result of Seepage Flow Analysis**
(Case of Permeability Coefficient $10K_1$)

Calculated Section		Distance (m)	Non Grouted		Grout A (EL. - 60 m)		Grout B (EL. - 120 m)	
			Unit Seepage Flow (ℓ/min/m)	Seepage Flow (ℓ/min)	Unit Seepage Flow (ℓ/min/m)	Seepage Flow (ℓ/min)	Unit Seepage Flow (ℓ/min/m)	Seepage Flow (ℓ/min)
Dam Foundation		100	(142.09)	(14,209)	46.02	4,602	28.66	2,866
Right Bank	R-1	100	52.30	5,230	39.78	3,978	32.90	3,290
	R-2	200	(30.52)	(6,104)	(23.21)	(4,642)	(19.20)	(3,840)
	R-3	800	(11.09)	(8,872)	(8.44)	(6,752)	(6.98)	(5,584)
	Sub-total	1,100		(20,206)		(15,372)		(12,714)
Left Bank	L-1	100	(54.79)	(5,479)	(41.67)	(4,167)	(34,47)	(3,447)
	L-2	400	(16.94)	(6,776)	(12.88)	(5,152)	(10.66)	(4,264)
	Sub-total	500		(12,255)		(9,139)		(7,711)
Grand Total				(46,670)		(29,293)		(23,291)

Remark; ( ) shows estimated values

Table 8-6 Total Amount of Leakage from the Reservoir

Unit: m^3/min

Permeability Coef.	Non Grouted	Grout (A) (EL. - 60 m)	Grout (B) (EL. - 120 m)
K1	6.51	5.66	5.64
K2	25.24	17.28	15.29
K3	46.67	29.29	23.29

Table 8-7 Amount of Leakage at Dam Foundation

Unit: m^3/min

Permeability Coef.	Non Grouted	Grout (A) (EL. - 60 m)	Grout (B) (EL. - 120 m)
K1	1.68	0.85	0.84
K2	7.68	2.72	1.88
K3	14.21	4.60	2.87

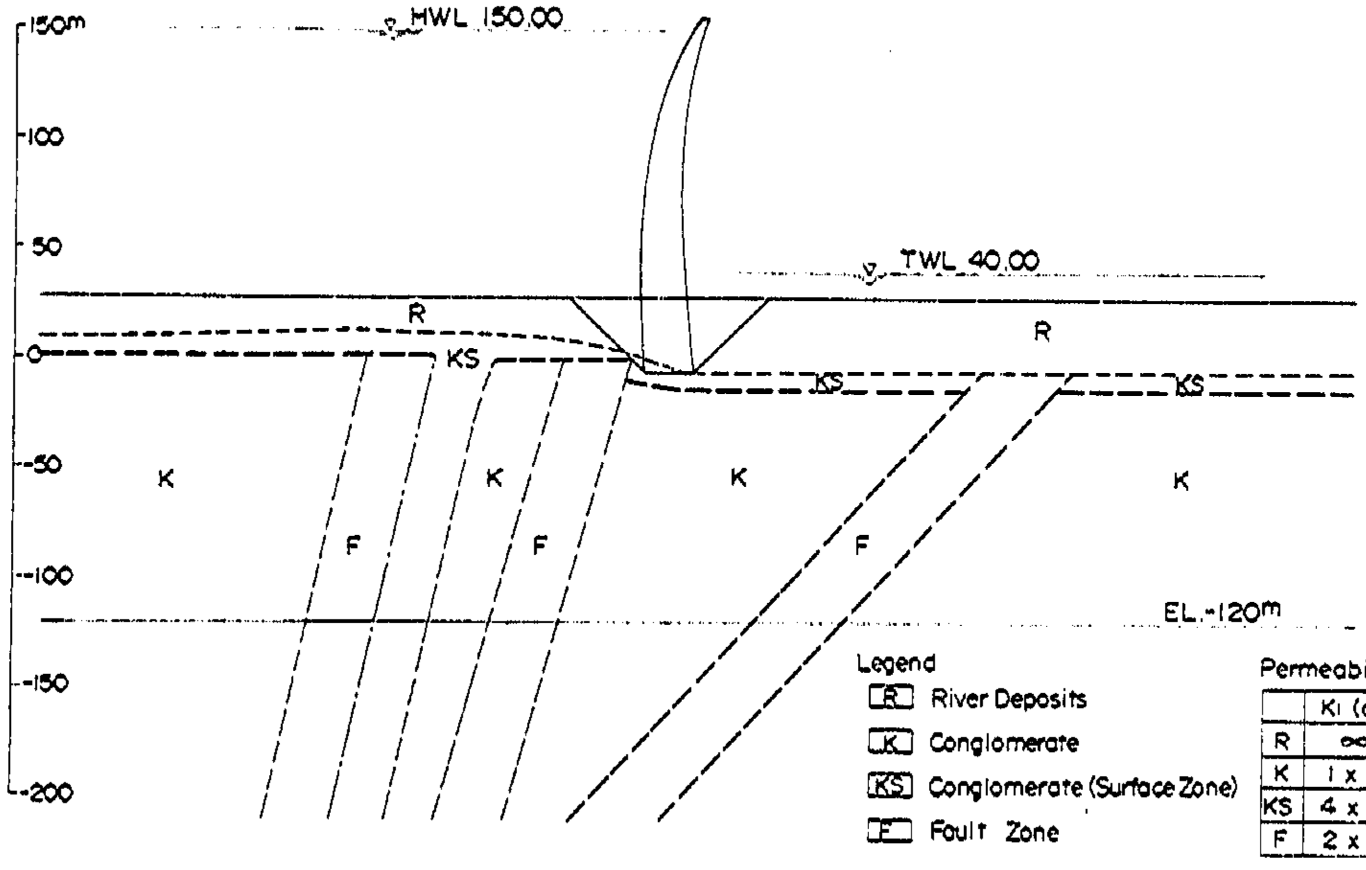
As is clear from Table 8-6, the amounts of leakage for K1 are 6.51 m3/min in case of non grouted and 5.66 m^3/min and 5.64 m^3/min in the case of grouts (A) and (B), and the differences are small. This is because the coefficient of the bedrock (K = 10 Lu, facies of Köprücay Conglomerate) is low and there is practically no difference compared with the coefficient of the grout curtain portion (K = 5 Lu) so that the effect of the grout curtain does not appear prominently. On the other hand, the amount of leakage is 46.67 m^3/min in case of the extended coefficient K3 with non grouted, approximately 7 times the case of K1. As for the cases of K2 and K3, the effects of curtain grouting are prominently seen, and compared with the case of non grouted, the reduction rates in amounts of leakage are 32% (grout EL.-60 m) and 39% (EL. -120 m) for K2, and 37% and 50% for K3.

The relationship between this amount of leakage and the water balance of the reservoir (annual average inflow and storage capacity) expressed graphically is shown in Fig. 8-20, and in case of K3 with non grouted, the amount of leakage will be 0.09% of the annual average inflow, and 0.013% of the total storage capacity. Although there is especially no criteria in the way of an allowable maximum value of quantity of leakage, from the standpoints of safety of the dam and the storage effect of the reservoir, there is some examples of rules shown in Table 8-8, which criteria can be used as a reference.

Table 8-8 Examples of Permissible Maximum Leakage Volume

Ministry of Agriculture, Forestory and Fisheries	Japan	Within 1% of Reservoir Inflow Within 0.05% of Reservoir capacity
Semator Wash Dam (Earth Dam, H = 29 m)	U.S.A.	240 m^3/min
Feistritz Dam (Rockfill Dam H = 22 m)	Australia	180 m^3/min
Bistrita Dam (Rockfill Dam, H = 130 ft)	Rumania	60 ∿ 120 m^3/min
(Reference) Beşkonak Dam (Concrete Dam, H = 165 m)	Turkey	K3, Non grouted 0.9 % of Reservoir Inflow 0.013 % of Reservoir Capacity 46.67 m^3/min

Fig. 8-6 Calculation Model of Dam Foundation

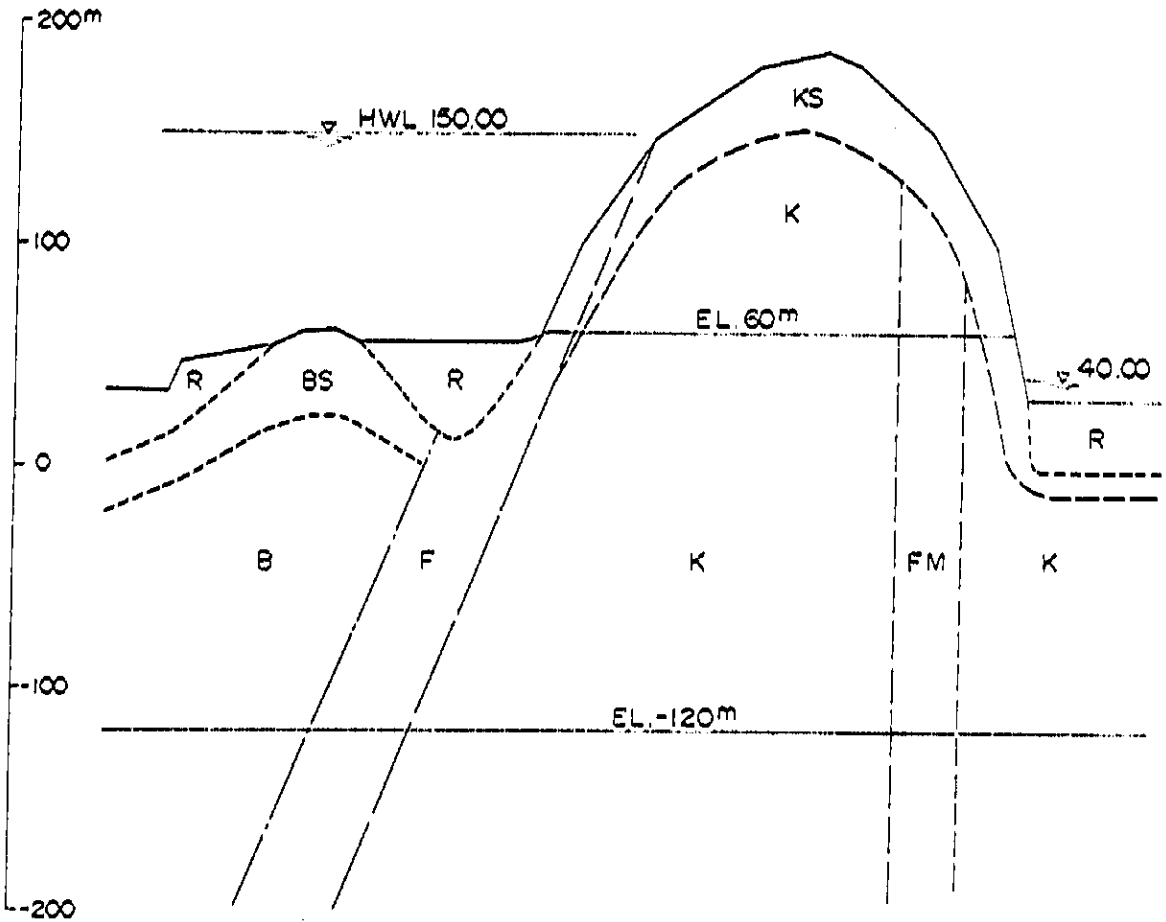


Permeability Coefficient

	K1 (cm/sec)	K2 (cm/sec)	K3 (cm/sec)
R	∞	∞	∞
K	1×10^{-4}	5×10^{-4}	1×10^{-3}
KS	4×10^{-4}	2×10^{-3}	4×10^{-3}
F	2×10^{-4}	1×10^{-3}	2×10^{-3}

Remarks ; K2 & K3 are to be applied to the zone between EL. 60m ~ EL.-120m.

Fig. 8-7 Calculation Model of Right Bank (R-1)



Legend

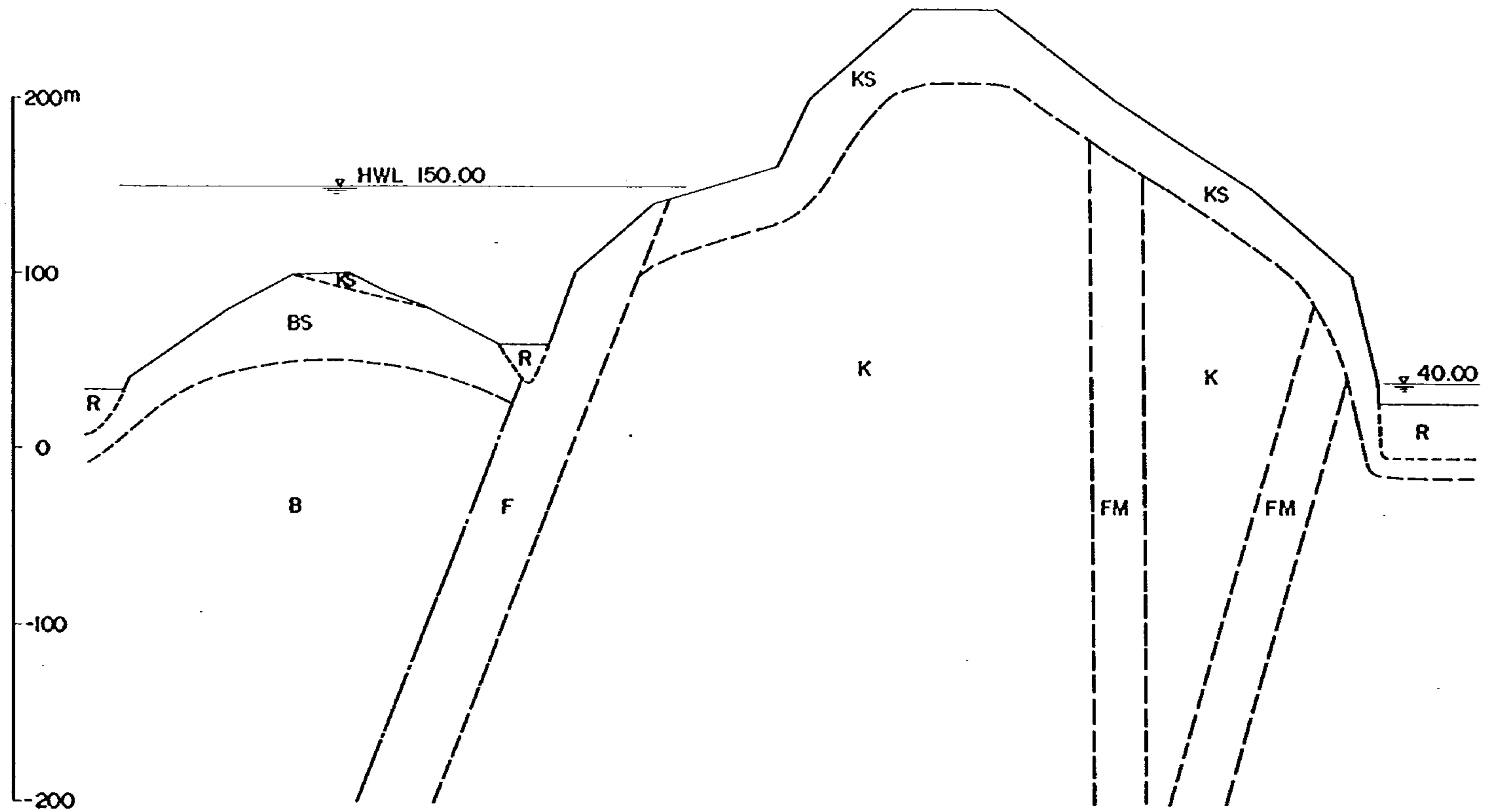
- [R] River Deposits
- [K] Conglomerate
- [KS] Conglomerate (Surface Zone)
- [B] Shale & Sandstone
- [BS] Shale & Sandstone (Surface Zone)
- [F] F-1 Fault Zone
- [FM] Fault Zone

Permeability Coefficient

	K_1 (cm/sec)	K_2 (cm/sec)	K_3 (cm/sec)
R	1×10^{-2}	5×10^{-2}	1×10^{-1}
K	1×10^{-4}	5×10^{-4}	1×10^{-3}
KS	4×10^{-4}	2×10^{-3}	4×10^{-3}
B	2×10^{-5}	1×10^{-4}	2×10^{-4}
BS	2×10^{-4}	1×10^{-3}	2×10^{-3}
F	4×10^{-4}	2×10^{-3}	4×10^{-3}
FM	3×10^{-4}	1.5×10^{-3}	3×10^{-3}

Remarks ; K_2 & K_3 are to be applied to the zone between EL. 60m ~ EL. -120m.

Fig. 8-8 Calculation Model of Right Bank (R-2)



Legend		Permeability Coefficient (cm/sec)
R	River Deposits	$K_1 = 1 \times 10^{-2}$
K	Conglomerate	$K_1 = 1 \times 10^{-4}$
KS	Conglomerate(Surface Zone)	$K_1 = 4 \times 10^{-4}$
B	Shale & Sandstone	$K_1 = 2 \times 10^{-6}$
BS	Shale & Sandstone (Surface Zone)	$K_1 = 2 \times 10^{-4}$
F	F-1 Fault Zone	$K_1 = 4 \times 10^{-4}$
FM	Fault Zone	$K_1 = 3 \times 10^{-4}$

Fig. 8-9 Calculation Model of Right Bank (R-3)

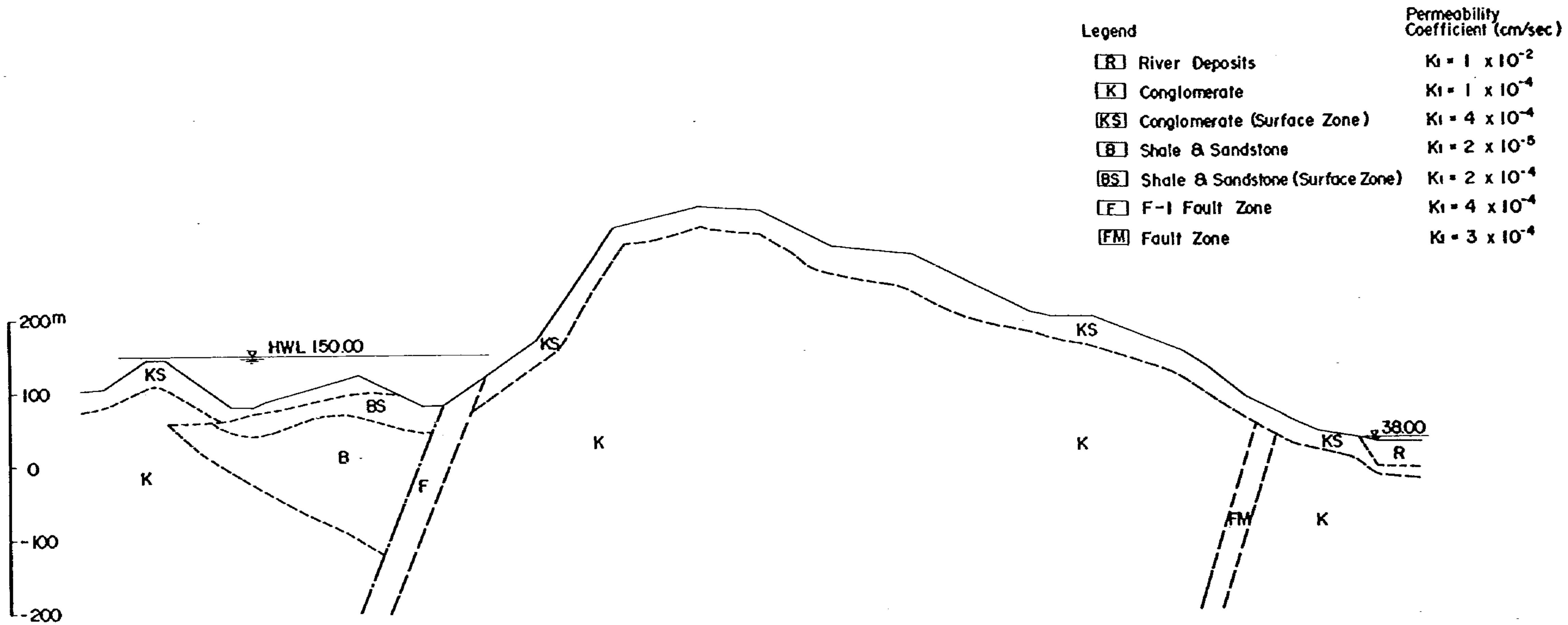
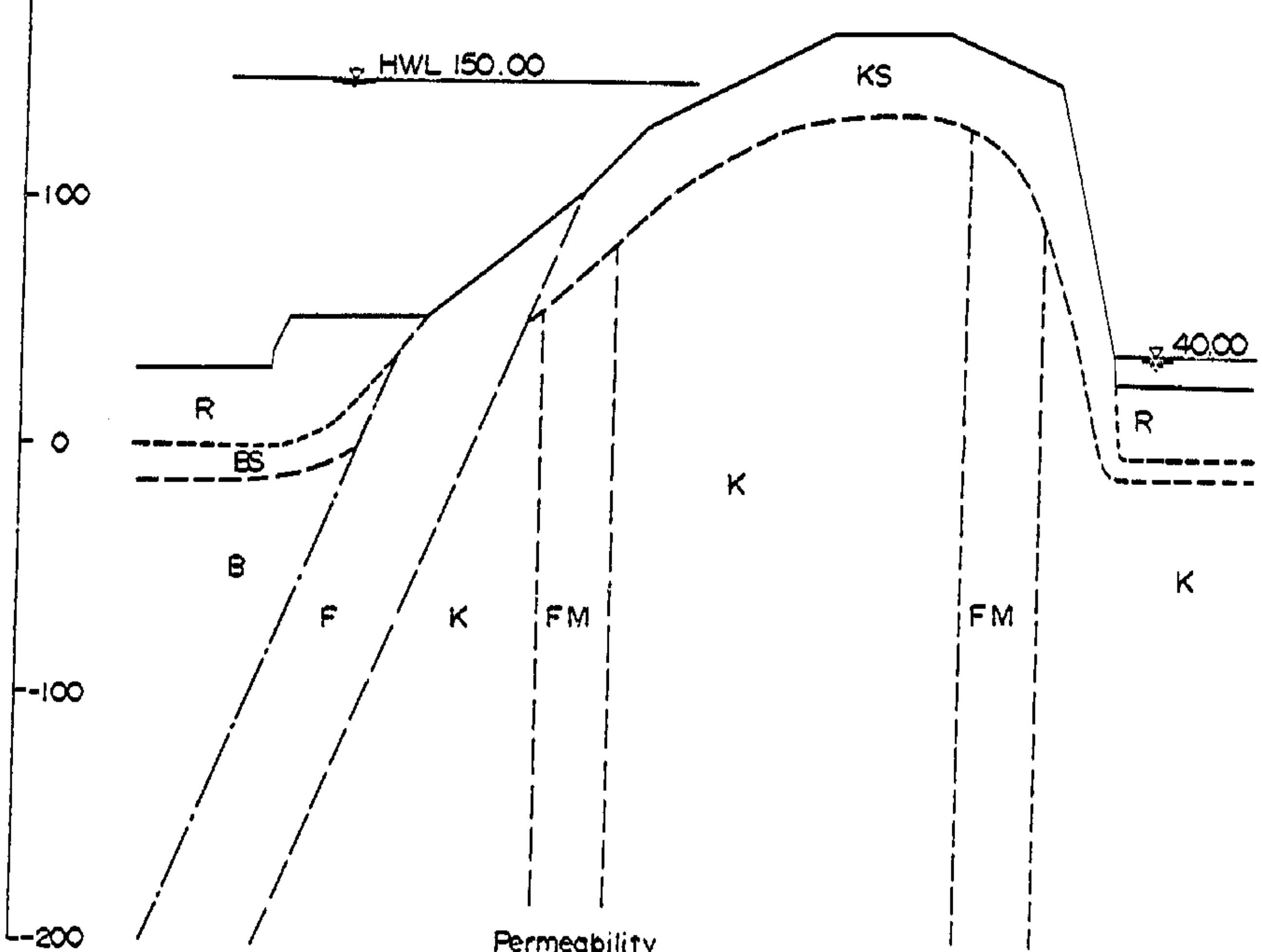


Fig. 8-10 Calculation Model of Left Bank (L-1)



Legend	Permeability Coefficient (cm/sec)	Legend	Permeability Coefficient (cm/sec)
R River Deposits	$K_1 = 1 \times 10^{-2}$	BS Shale & Sandstone (Surface Zone)	$K_1 = 2 \times 10^{-4}$
K Conglomerate	$K_1 = 1 \times 10^{-4}$	F F - I Fault Zone	$K_1 = 4 \times 10^{-4}$
KS Conglomerate (Surface Zone)	$K_1 = 4 \times 10^{-4}$	FM Fault Zone	$K_1 = 3 \times 10^{-4}$
B Shale & Sandstone	$K_1 = 2 \times 10^{-5}$		

Fig. 8-11 Calculation Model of Left Bank (L-2)

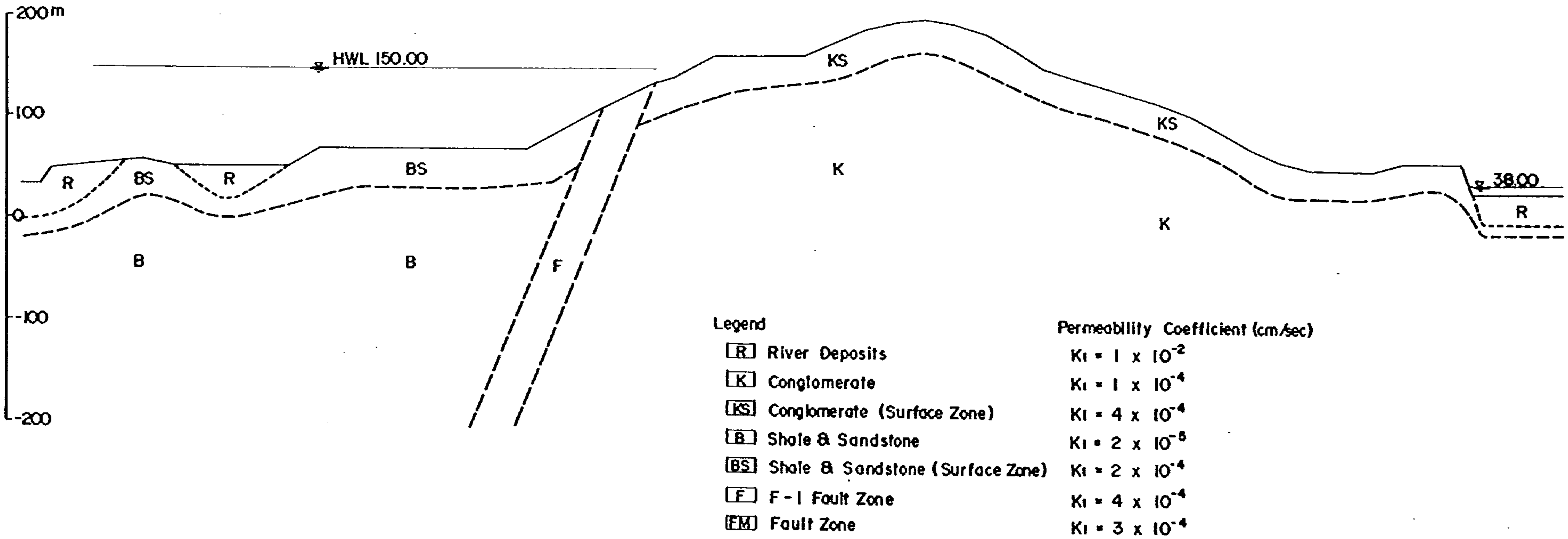


Fig.8-12 Seepage Flow Diagram of Right Bank (R-1-01)

Condition ; Grout None
Permeability Coef. K1

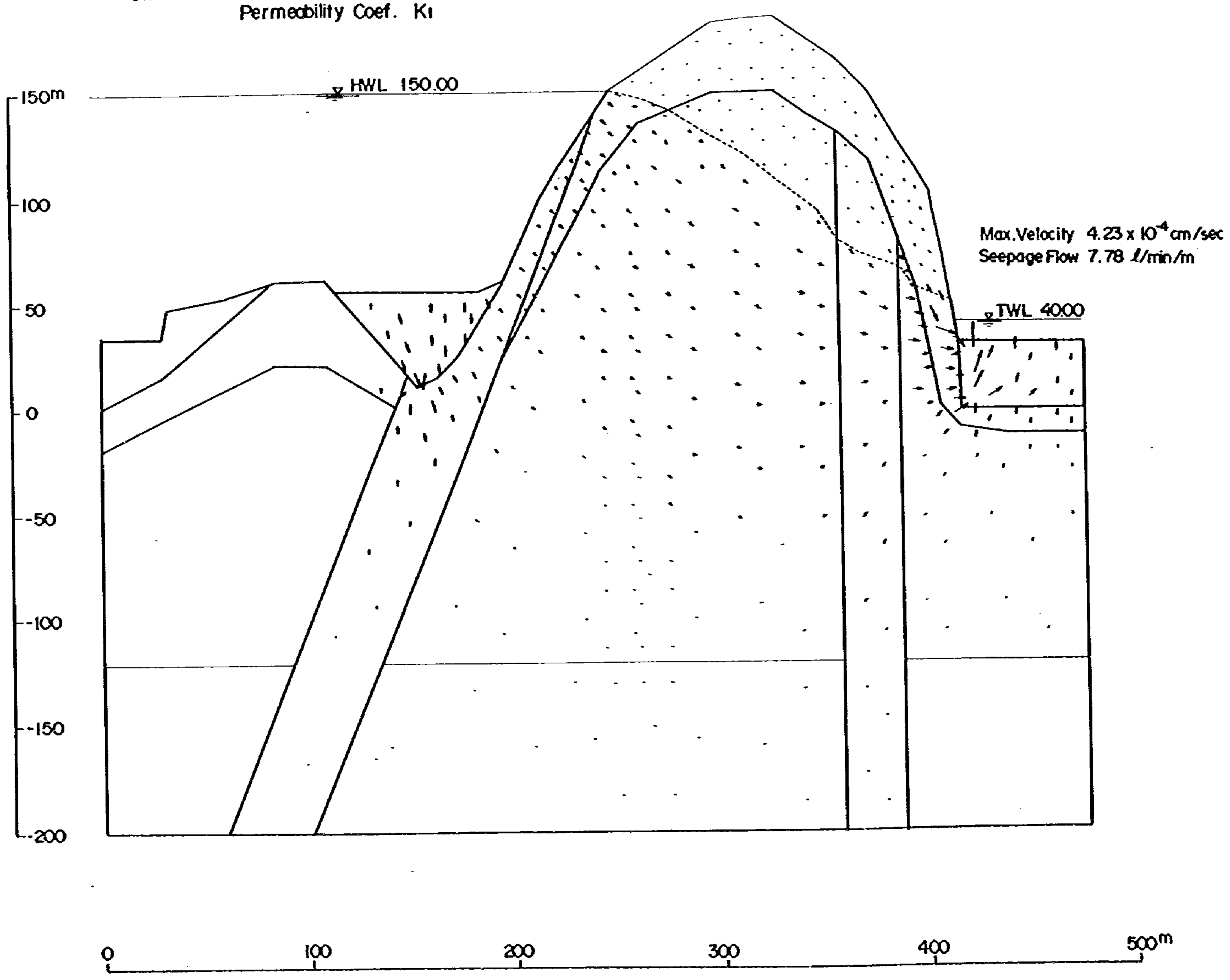


Fig.8-13 Seepage Flow Diagram of Right Bank (R-1-03)

Condition ; Grout None
Permeability Coef. K_3

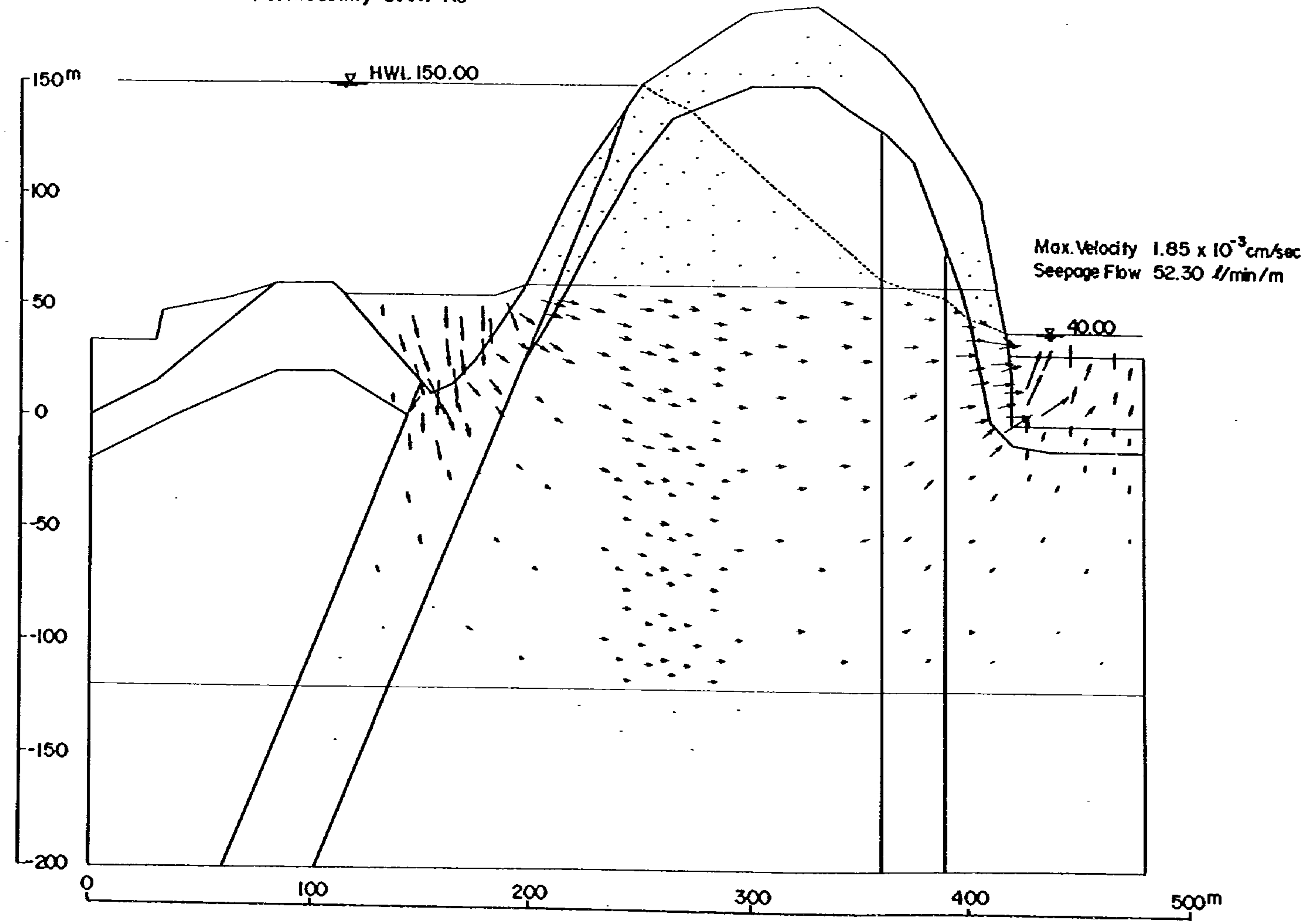


Fig.8-14 Seepage Flow Diagram of Right Bank (R-1-A1)

Condition ; Grout EL.-60m
Permeability Coef. K1

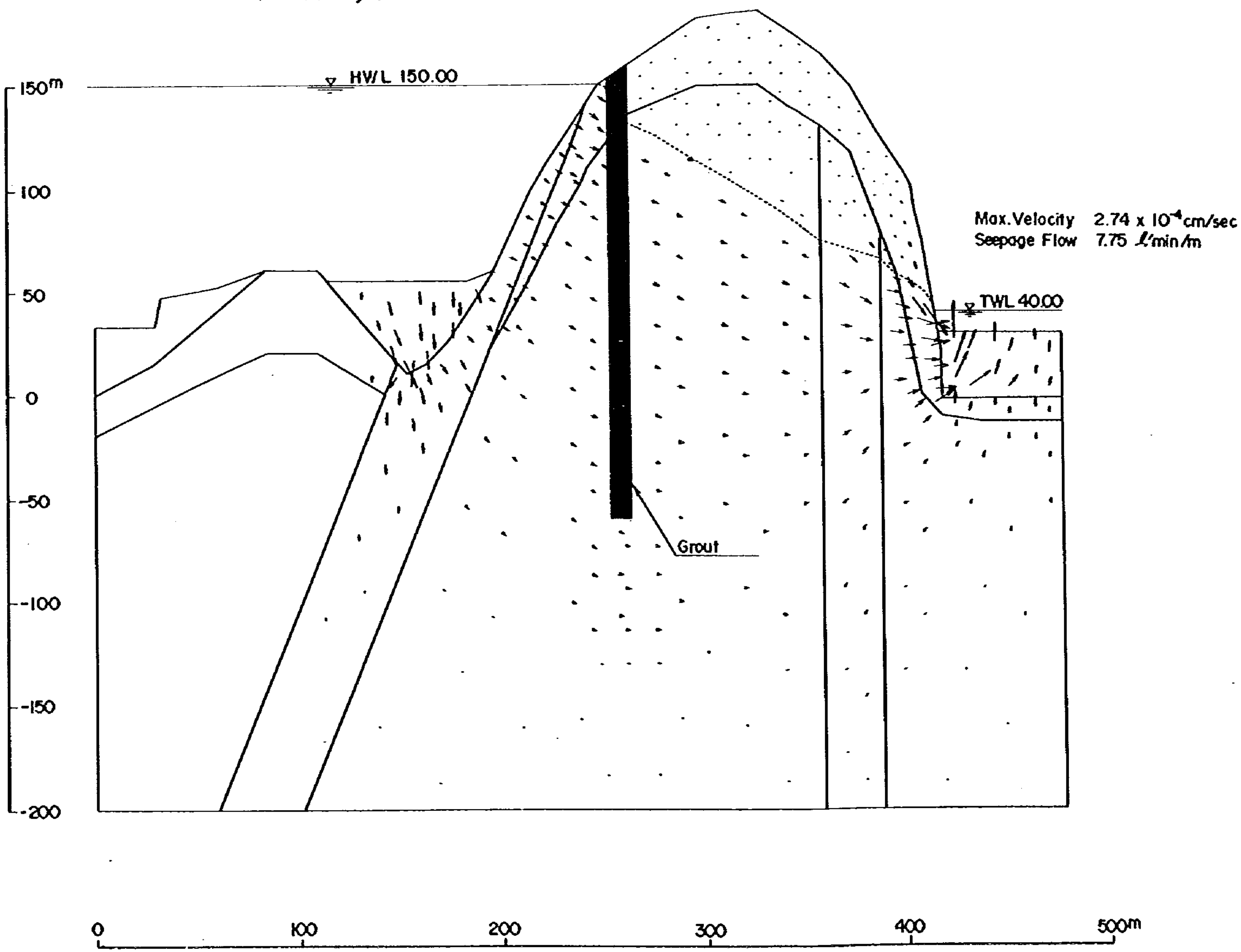


Fig.8-15 Seepage Flow Diagram of Right Bank (R-1-A3)

Condition : Grout EL.-60m
Permeability Coef. K3

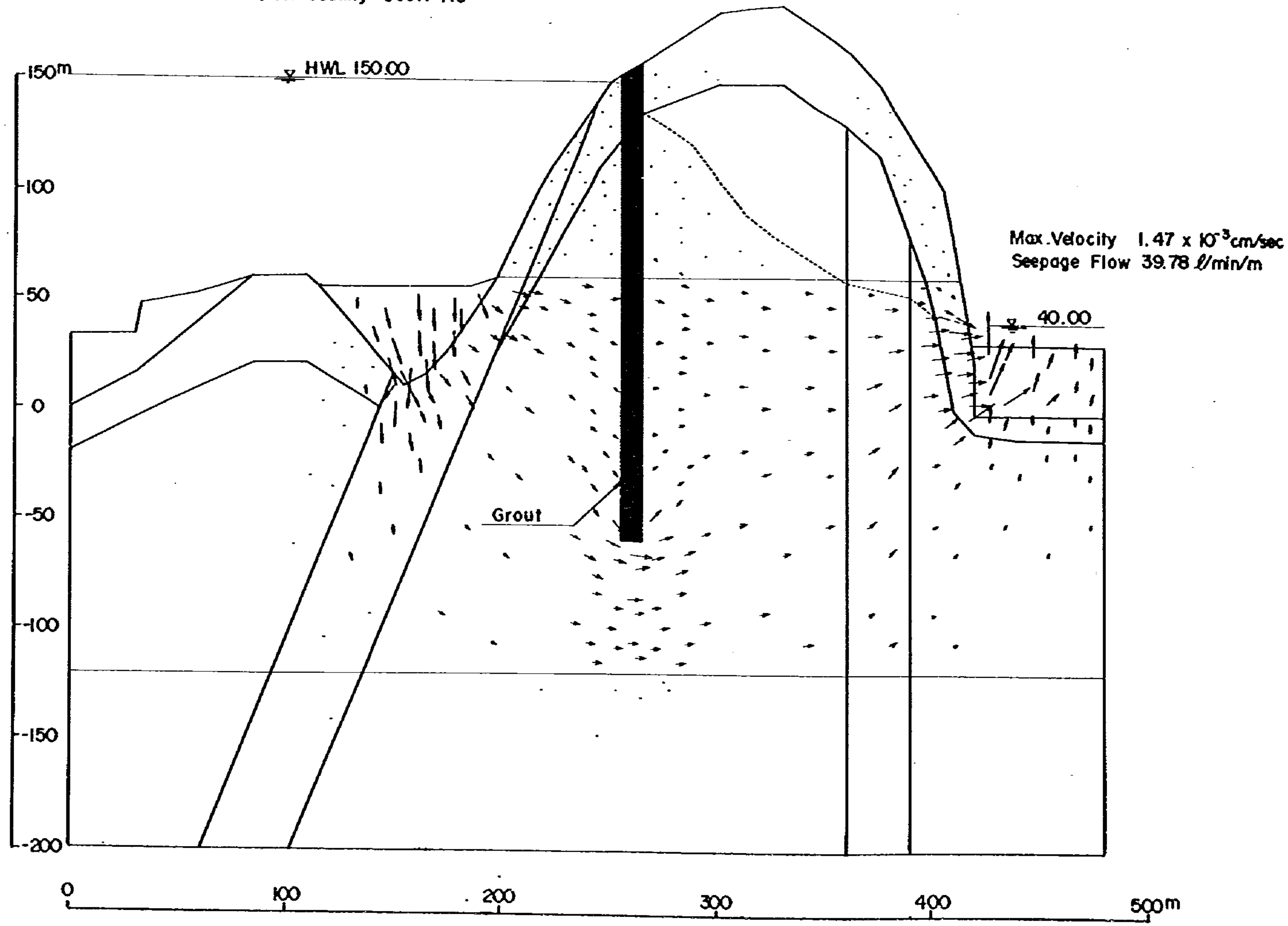


Fig. 8-16 Seepage Flow Diagram of Right Bank (R-1-B1)

Condition ; Grout EL. - 120m
Permeability Coef. K1

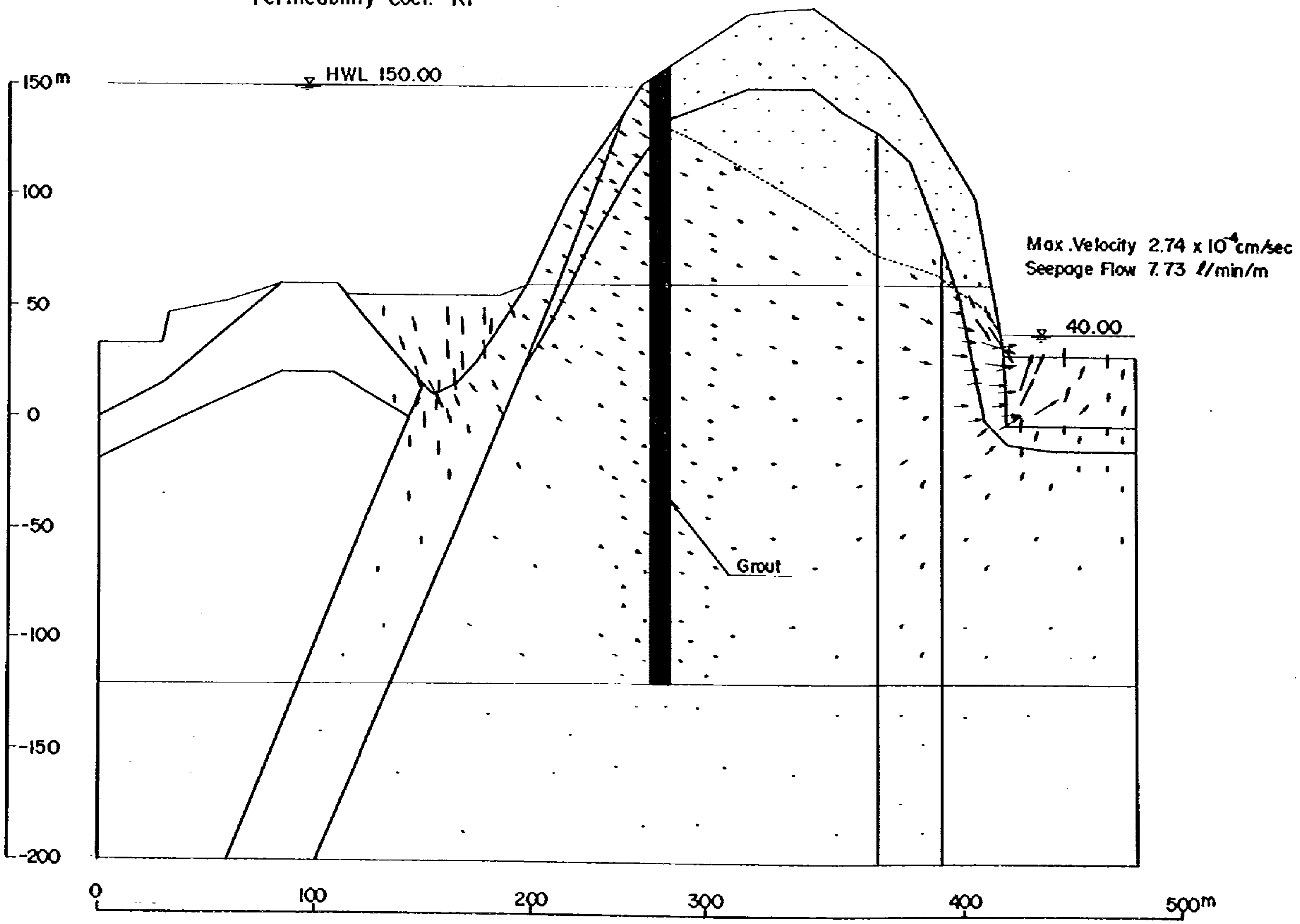


Fig. 8-17 Seepage Flow Diagram of Right Bank (R-1-B3)

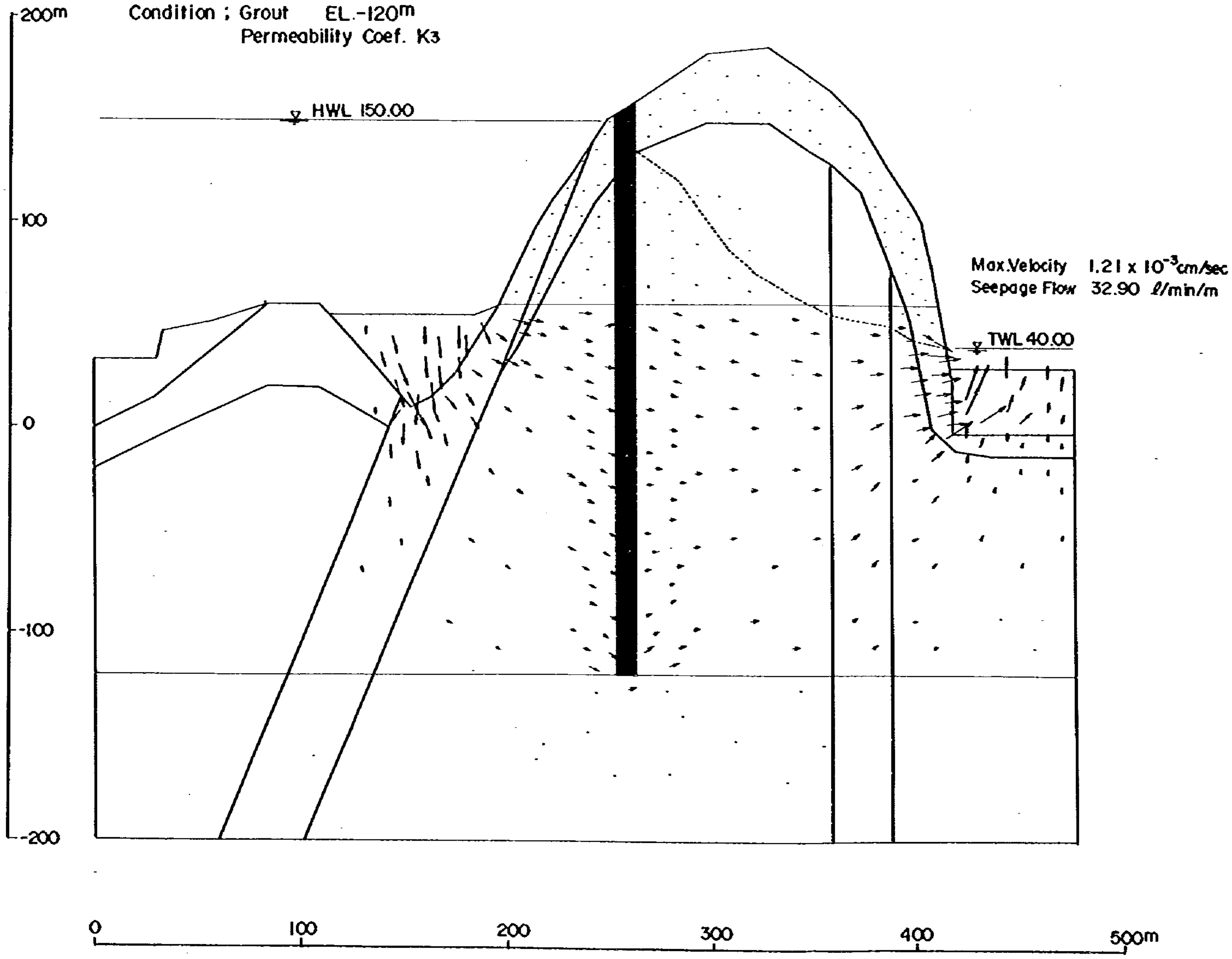


Fig. 8-18 Leakage Per Unit Length

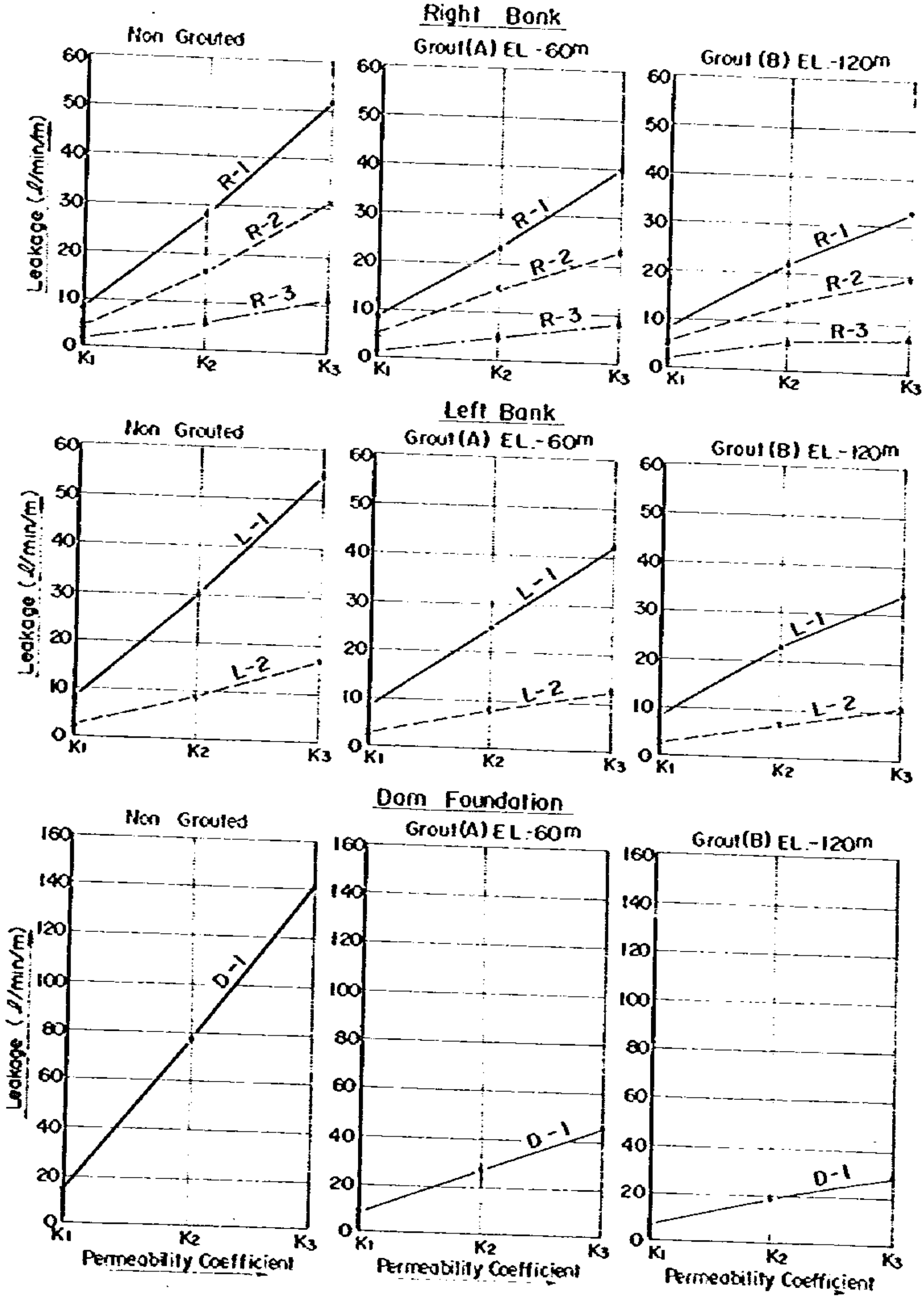


Fig.8-19 Total Amount of Leakage

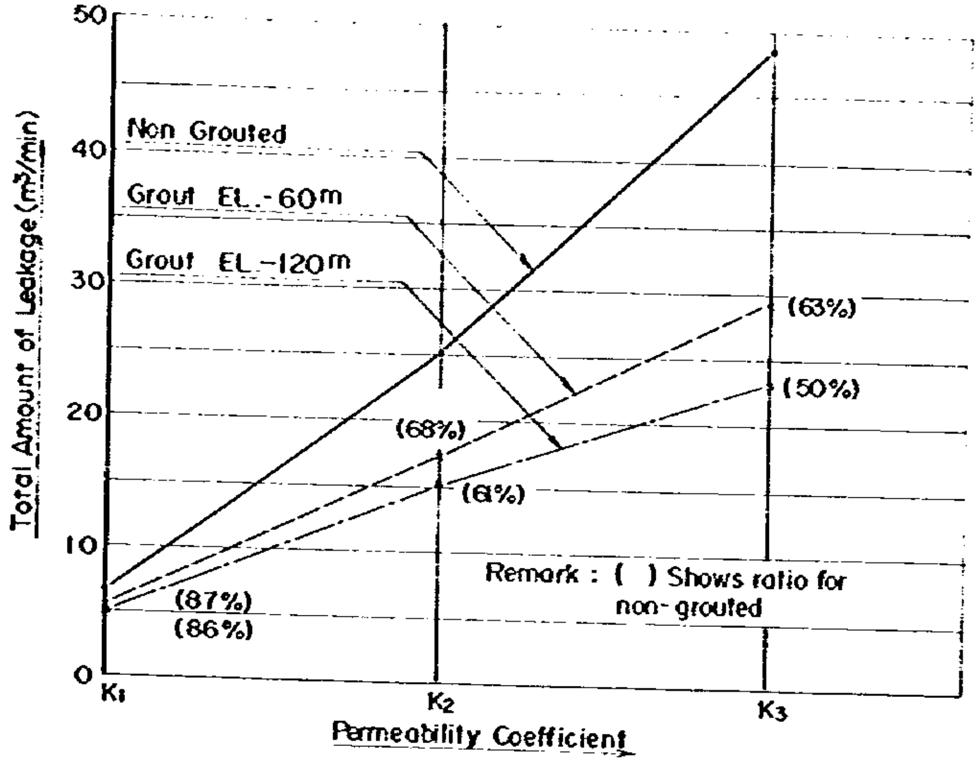
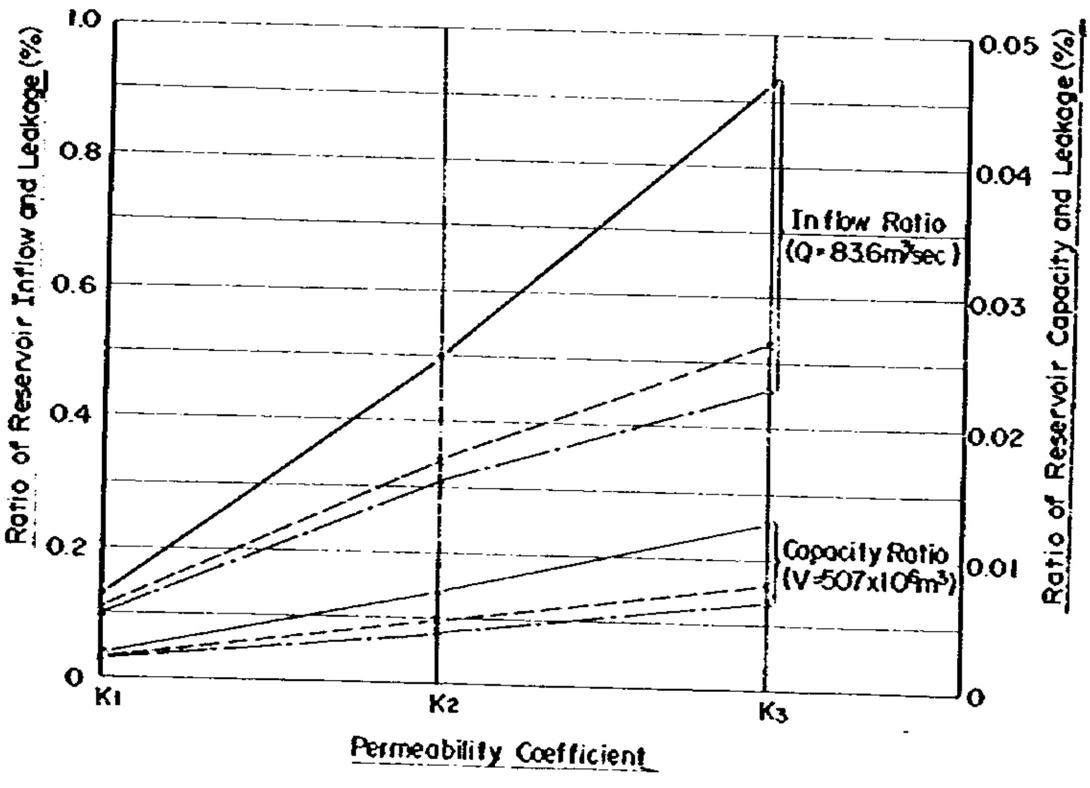


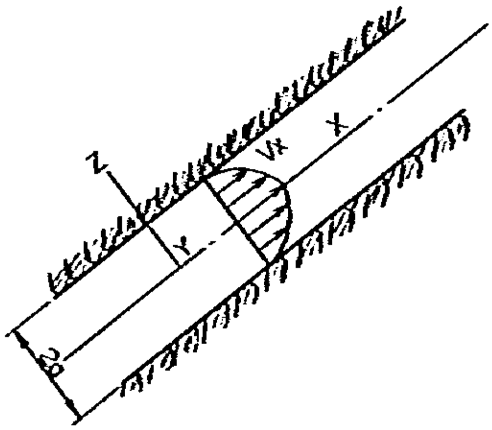
Fig.8-20 Correlation Between Reservoir and Leakage



8.1.5 Seepage Flow Analysis in Case of Existence of Predominant Joint

The seepage flow analysis performed in 8.1.4 considers the bedrock as a homogeneous continuum, and the amount of leakage was computed according to Darcy's law, assuming macroscopically equivalent coefficient of permeability. On the other hand, in case of existence of predominant solution cracks in the bedrock assumed to be a homogeneous continuum, the seepage flow in the bedrock may be considered to pass mainly through the cracks. However, in case cracks is not uniformly distributed and the bedrock cannot be handled as a continuum, it is necessary in seepage flow analysis to consider the joint element as a model of cracks shown in the figure below.

Seepage Flow in the Crack



In considering the seepage flow through cracks in the bedrock, by assuming that

(1) Both sides of the crack are impervious

(2) Soil particles are not contained in the crack, and only seepage water exists

(3) The seepage flow is laminar flow only

(4) The Navier-Stokes equation is applied to seepage flow

the average flow velocities V_{xm}, V_{ym} and V_{zm} in the cracks may be expressed by the following equations:

$$V_{xm} = -\frac{g(2a)^2}{12\gamma} \cdot \frac{\partial H}{\partial x} = -K_j \cdot \frac{\partial H}{\partial x}$$

$$V_{ym} = -\frac{g(2a)^2}{12\gamma} \cdot \frac{\partial H}{\partial y} = -K_j \cdot \frac{\partial H}{\partial y}$$

$$V_{zm} = 0$$

According to the above equations, the hydraulic characteristics of the seepage flow in the bedrock may be said to be a potential flow where permeability coefficient (K_j) is a function of the coefficient of kinematic viscosity γ of water and crack width 2a.

Figs. 7-17 and 7-18 show the results of arrangement of the widths and distribution of solution cracks in the bedrock based on boring logs and test adit data. As is clear from the figure the distribution of cracks in the bedrock is extremely irregular, there are calcite and clay intercalated in the cracks, and it is not an easy matter to select the effective widths of cracks through which seepage flows pass.

According to the core borings,

(1) In the vicinity of the dam site, solution cracks are recognized to occur at the space of 7.8 m on the average.

(2) Average width of solution cracks is 0.9 cm and those cracks, which are more than 2 cm wide, comprise approximately 10% of all cracks.

(3) At the permeability tests in drillholes, those sections where injection pressures didn't reach the required 10 kg/cm^2, are estimated to occure at the space of 25.2 m on the average.

In addition, according to the test adits, average width of solution cracks is 2.3 cm.

In performing seepage flow analysis in case that predominant solution cracks exist in the conglomerate, it is necessary to set the effective width and the space of cracks.

Effective widths of cracks were set to be 2 cm making reference to measured values in drillholes and test adits. While taking consideration of calcite and clay intercalated in the solution cracks of 2 cm wide, effective widths were also set to be 2 mm, which corresponded to 10% of 2 cm.

According to the above items of (1) and (2), solution cracks are estimated to occur at the space of approximately 80 m ($\fallingdotseq$ 7.8 $\div$ 0.1). On the other hand, by the results of permeability tests in drillholes, continuous cracks are estimated to exist at the space of about 25 m. Therefore, solution cracks of approximately 2 cm wide are considered to exist at the space of 25 - 80 m.

In seepage flow calculations, the space of cracks of 2 cm wide were set to be 40 - 60 m in the range of EL. 60 m - EL.-120 m referring to Fig. 7-9.

The calculation cases were shown in Table 8-9. The results of calculations are shown in Fig. 8-21 and Table 8-10.

Table 8-9 Calculation Cases

Section	Effective width of crack	Calculation Cases		
		Non Grouted	Grout (A)	Grout (B)
Right Bank (R - 1)	2 mm	-	R-1-A1-C1	-
	2 cm	R-1-01-C2	R-1-A1-C2	R-1-B1-C2

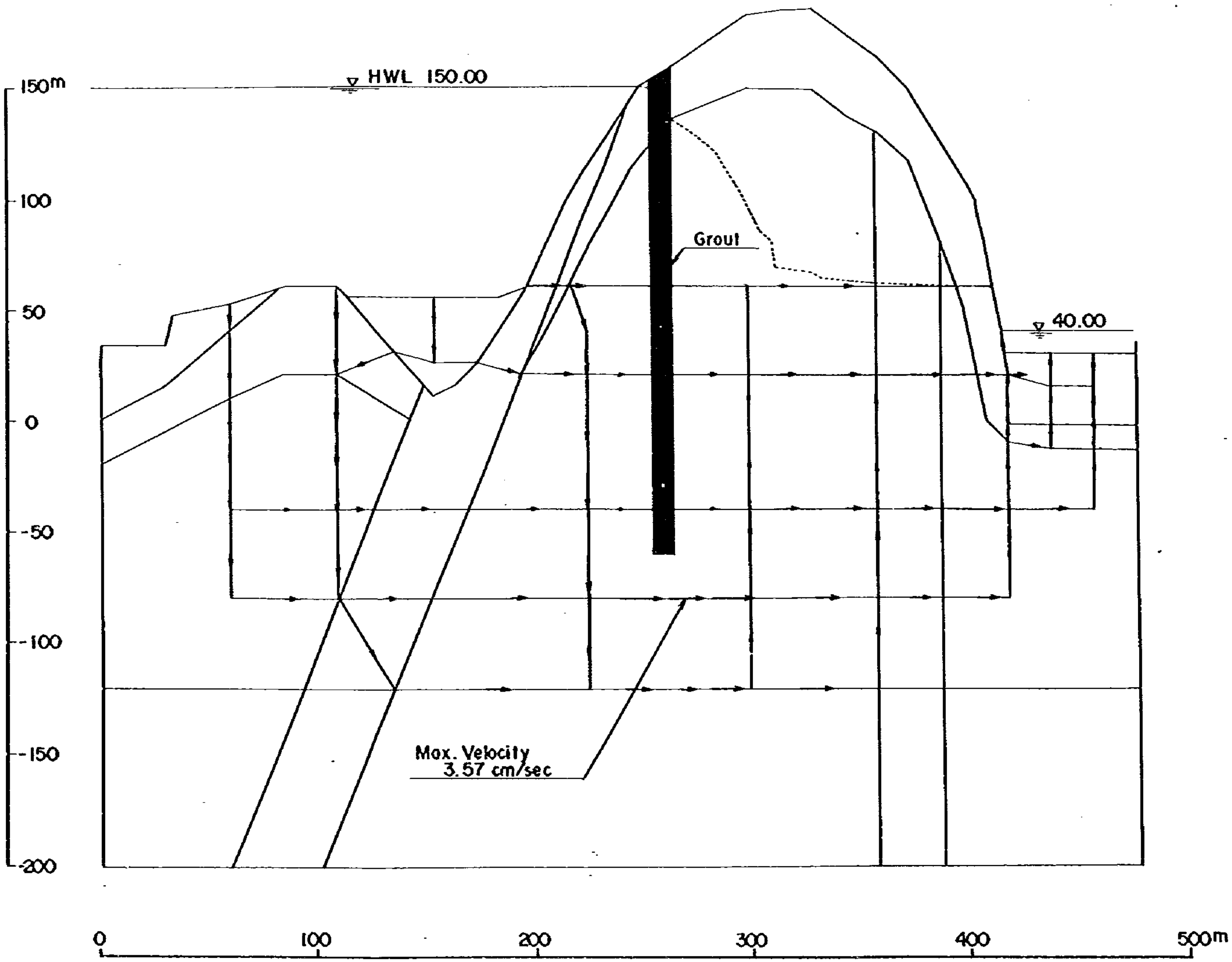
Table 8-10 Result of Seepage Flow Analysis Along Solution Crack (R-1)

Unit: ℓ/min/m

		Non grouted	Grout (A) (EL. - 60 m)	Grout (B) (EL. - 120 m)
Considered Crack	width 2 mm	12.6	7.8	7.7
	width 2 cm	148.0	91.3	40.8
Non Considered Crack (Reference)	Permeability Coef. K1	7.8	7.8	7.7
	" K2	28.3	23.5	21.6
	" K3	52.3	39.8	32.8

Fig. 8-21 Seepage Flow Diagram Along Solution Crack (R-I-AI-C2)

Condition ; Crack W = 2cm, LH = 40m, Lv = 60m
Grout EL. -60m
Permeability Coef. K1



As a result of calculations, compared with the case of no cracks and assuming the bedrock to be a homogeneous continuum, the amounts of leakage in case of crack width of 2 cm with non grouted are 19 times larger (= 148.0/7.8) for K1, 5.2 times for K2, and 2.8 times for K3.

Although it cannot be declared at the present that predominant solution cracks are continuous throughout the bedrock in the vicinity of the dam site, it is predicted that leakage of the extent indicated in Table 8-10 would occur at parts.

For the sake of reference, when the ratio between leakage amounts in case of considering cracks of 2 cm widths at this R-1 cross section and in case of assuming a homogeneous continuum, is applied to the entire range studied in 8.1.4, the amounts of leakage from the reservoir will be shown in Table 8-11.

Table 8-11 Total Amount of Leakage along Solution Crack

Unit: m^3/min

Grout Condition	Leakage
Non grouted	123.7
Grout (A) EL. - 60 m	66.2
Grout (B) EL. - 120 m	29.9

8.1.6 Considerations Regarding Seepage Flow Analyses Results

As a result of the seepage flow analysis performed under 8.1.4 and 8.1.5, it was succeeded in numerically grasping the leakage from the reservoir and the leakage prevention effect by the grout curtain.

The Lugeon values of the Köprücay Conglomerate in the vicinity of the dam site obtained from results of pressure tests are high at 10 - 70 Lu in the 20 - 40 m of the surface layer. Deeper than 40 m the average is 5.9 Lu, with 74% of the whole indicating 1 Lu or less and 7% showing 10 Lu or more. While, there were 7.9% of the whole where injection presures at tops of holes were less than 10 kg/cm^2 in permeability tests, and it is thought that karstification has progressed at these sections. The Köprücay Conglomerate at this portion has a complex geological structure, and it is imagined that a considerable amount of cavities such as solution cracks, exist in parts with Lugeon values being higher than the measured average of K1.

In general, it would not be erroneous to assume that seepage flow inside bedrock will conform to Darcy's law in case Lugeon values are low, but when Lugeon values are higher than 20 Lu, the condition of seepage flow may be considered to be close to pipe flow. In addition, observed data of actual leakage greater than calculated values have been reported.

It has been observed that the groundwater level in the vicinity of the dam site is lower than the river water level. This is thought to be due to the Köprücay Conglomerate in this zone having been subjected to karstification so that the watertightness is small. However, the relationship between this state of the groundwater level and the fact that the Lugeon values in permeability tests are low, has not been clarified.

In spite of there being an above problem, according to the geological investigation performed so far, large scale and continuous solution cracks and seepage paths have not been confirmed in the bedrock in the vicinity of the dam site, while measured Lugeon values indicate low values as a whole. Consequently, within the scope that the investigation data provided by DSI are applied, particularly the data of permeability tests and the information about solution cracks, the behaviors of seepage flows and amounts of leakage obtained by this analysis can be used as reference data for the curtain grouting plan. If additional information concerning the bedrock in the vicinity of the dam site can be obtained by more detailed hydrogeological investigations hereafter, it is expected that Lugeon values of even higher accuracies will be attained.

8.2 Curtain Grouting

8.2.1 Proposal for Curtain Grouting

Curtain grouting works were planned to be 200 m of the dam site, 1,200 m of the right bank of the reservoir, and 650 m of the left bank in order to prevent leakage through the bedrock in the vicinity of the dam site.

The directions of faults and solution cracks were northeast to southwest as described in 7.3.4(3). Leakage prevention effect could be secured with a direction orthogonal to the stream line direction of seepage flow obtained as a result of the plane analysis in 8.1.4(1). Therefore, the grout curtain courses were selected to be along the reservoir in a westward direction from the dam site at the right bank and in an eastward direction from the dam site at the left bank.

The greater part of the leakage will occur concentrated in the vicinity of the dam site according to analysis in Figs. 8-7(a) and (b). The amount of leakage will not be increased so much even if the length of the grout curtain were to be halved. However, it was decided that the curtain grouting should be performed in ranges thought to be high-permeability zones. In effect, the grout curtain should cover the faults F-1 to F-3 and their sheared zones at the right bank side, and to the line of partial contact with the impervious Beskonak Formation at the left bank side. With respect to the left bank side, although the boundary between the Beskonak Formation and the subjacent Köprücay Conglomerate is indistinct, since the greater part of the surface portion contacting the reservoir is covered by the Beskonak Formation, it is judged that a natural water barrier is constituted. The grout curtain was planned to contact the Beskonak Formation distributed at the surface.

With regard to depth of grouting, for the dam site vicinity it was taken as EL. -70 m of the zone where solution cracks are prominent, while for the wings at both banks it was made EL. 0 m which reaches below the groundwater level.

According to the above, the total area of the grout curtain will be 380,000 m^2.

Through provision of the above grout curtain, leakage from the reservoir according to numerical analysis would be the following:

(1) In case the bedrock is considered as a homogeneous continuum, if the permeability coefficient is assumed to be approximately 10 times (K3) the average Lugeon value of Köprücay Conglomerate, the total amount of leakage will be approximately 30 m^3/min, and there will be a reduction of approximately 37% from the case of non grouted.

(2) In case it is assumed predominent joints exist in the bedrock and there are cracks of 2 cm wide at intervals of 40 to 60 m, the total amount of leakage will be roughly 70 m^3/min, and the reduction will be approximately 46% from the case of non grouted.

The above matter will thus serve as reference data for the grouting plan.

8.2.2 Considerations Regarding Grout Curtain

Curtain grouting has been planned for a very large area of 380,000 m^2. The main purpose of this grouting is to detect cavities in the bedrock such as solution cracks, and since the non-carstified portion making up the greater part of the Köprüçay Conglomerate indicates low Lugeon values, the actual amount of grout injected is thought to be very small. However, concentrated grouting will be required at the solutions and sheared zones detected as a result of boring.

It can be said similarly regarding Oymapinar dam now under construction on the adjacent Manavgat River, where the amount of grout injected is shown to be very small as a whole.

The range of the grout curtain in the vicinity of the dam site was planned over a horizontal length of approximately 2 km and an area of approximately 380,000 m^2, giving thorough consideration to the geological structure and hydrogeology of the surrounding bedrock, and referring to the results of seepage flow analysis. It is thought some leakage will not be avoidable even when a grout curtain has been provided. The Beskonak dam will be a concrete dam and there will be no problem in particular about the safety of the structure, in addition, it is judged that the function of the reservoir will be amply fulfilled.

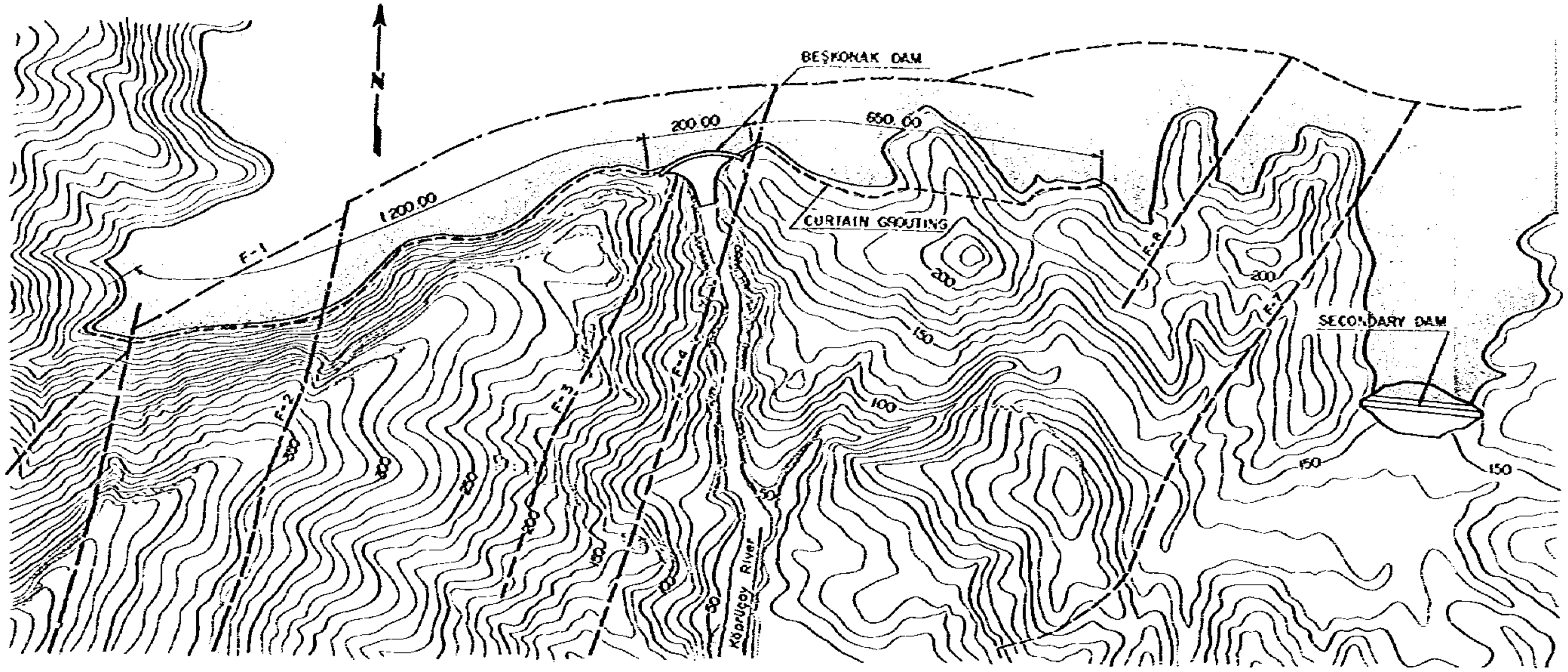
This grouting plan was considered on the basis of the investigation data obtained up to this time. It is necessary for further detailed boring and test adit investigations to be made along the range of this grout curtain course in order to study in succession for confirmation of the range of the grout curtain. Especially, it is necessary for the geological boundary between the Beskonak Formation and the Köprüçay Conglomerate at left bank side to be made distinct.

As reference data, the actual results of the curtain grout for the dam and reservoir at limestone and karst zone are shown in Table 8-12.

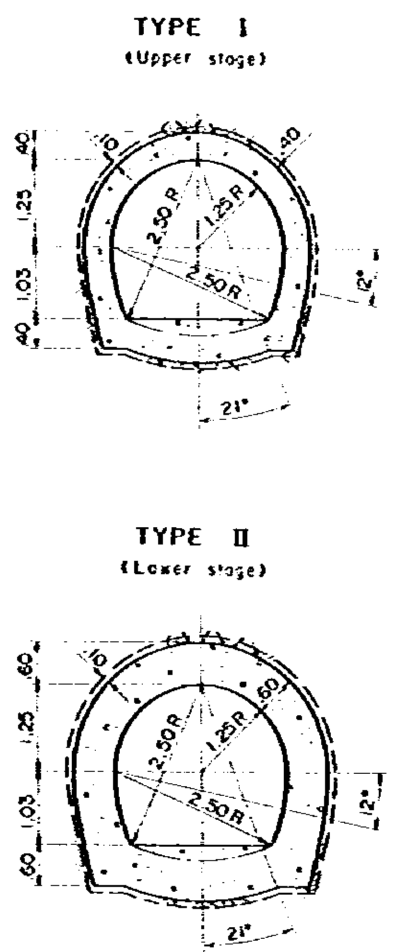
Table 8-12 Actual Result of Grout Curtain at Limestone and Karst Zone

Name of Dam	Nationality	Dam Style	Dam Height (m)	Grouting Area (m^2)	Total Grout Length (m)
Grands Rapides	Canada	Gravity, Rockfill	28	500,000	-
La Amistad	Mexico	Gravity, Rockfill	85	305,000	151,000
Peruca	Yugoslavia	Rockfill	60	240,000	169,600
Keban	Turkey	Gravity, Rockfill	207	200,000	320,000
Srinagarind	Thailand	Rockfill	140	76,800	132,000
Beşkonak (Planning)	Turkey	Archgravity	165	380,000	290,000

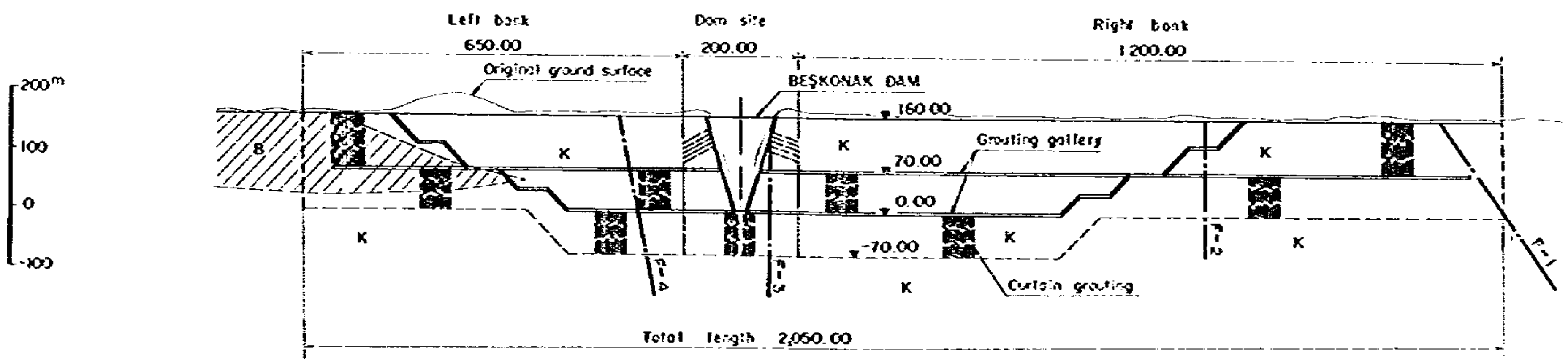
PLAN



SECTION OF GROUTING GALLERY



PROFILE



0 — 5m (Section of grouting gallery)

0 — 500m (Plan, Profile)

Legend

K: Conglomerate

B: Beşkonak formation

F: Fault zone

BEŞKONAK PROJECT	
BEŞKONAK RESERVOIR GROUT CURTAIN COARSES GENERAL	
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CHAPTER 9

DEVELOPMENT PLAN

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CHAPTER 9 DEVELOPMENT PLAN

9.1 Fundamental Matters

A survey and study on the Beskonak project at the reconnaissance level had been made by DSI since 1965, and in 1975 it was incorporated in the long-range development program with a scale of around 100 MW. The original plan formulated by DSI consisted of a concrete arch dam and a power station of 86 MW at the Beskonak site and a power station of 16 MW downstream.

As a result of the latest survey and study, the following has come to light:

(1) In southern part of Turkey (especially in Antalya region), there have been few power facilities to supply power energy during peak time.

(2) The discharge of the Köprücay River shows a substantial seasonal variation between the rainy and dry seasons, and therefore, it will be effective to carry out annual regulation.

Taking into consideration the above matter, it was decided to study the optimum development scale as a reservoir-type peaking power project.

9.1.1 Development System

The Köprücay River, which is of medium scale in Turkey, is an undeveloped one so far as hydroelectric power generation is concerned, but has been developed for irrigation purposes. Köprücay diversion dam (an intake dam for irrigation) was constructed by DSI in 1966 approximately 18 km downstream of the Beskonak dam site. Fig. 9-1 shows the existing and projected irrigation areas and the intake quantities on both banks of the Köprücay River. In the downstream area of the dam site, a total of 31.19 m^3/sec

(25.83 m^3/sec existing and 5.36 m^3/sec planned), is to be drawn from the Köprücay River during the irrigation period (June - September). Accordingly, in formulation of the Project, it is necessary to consider securing of irrigation water for the downstream area. The following three cases are considered in order to secure the downstream water utilization.

(1) Provide a small-scale turbine & generator for irrigation at Beskonak power station.

(2) Provide a regulating pond (Kisik dam) between Beskonak power station and Köprücay diversion dam.

(3) Utilize the diversion dam as a regulating dam after rebuilding.

The existing diversion dam doesn't have any regulating capacity. In order to utilize it as a regulating dam it is recessary to install gates at its crest. Dam body, piers and connecting bridge should be rebuilt to install the gates. At this study there are no detail drawings of the diversion dam and its appurtenant structures, in addition, the dam foundation is unknown. Therefore the study of case (3) was not performed. It would be favorable to perform its study at final design.

Studies were made of the two cases of (1) and (2). As will be described in 9.4, the two cases are approximately the same in the aspect of economics. However, the construction of Kisik dam in case (2) would inundate agricultural land corresponding to approximately 70% of the Bucak-Akbas-Karatas irrigation project presently under development by DSI. A part of irrigation works now under construction would also be affected. Consequently, it is feared that there will be various difficulties with regard to land acquisition within this irrigation project area.

For the above reasons, it was decided that the Project was to be made of the various pertinent studies on only case (1) of providing a small unit without a regulating pond.

9.1.2 Economic Evaluation

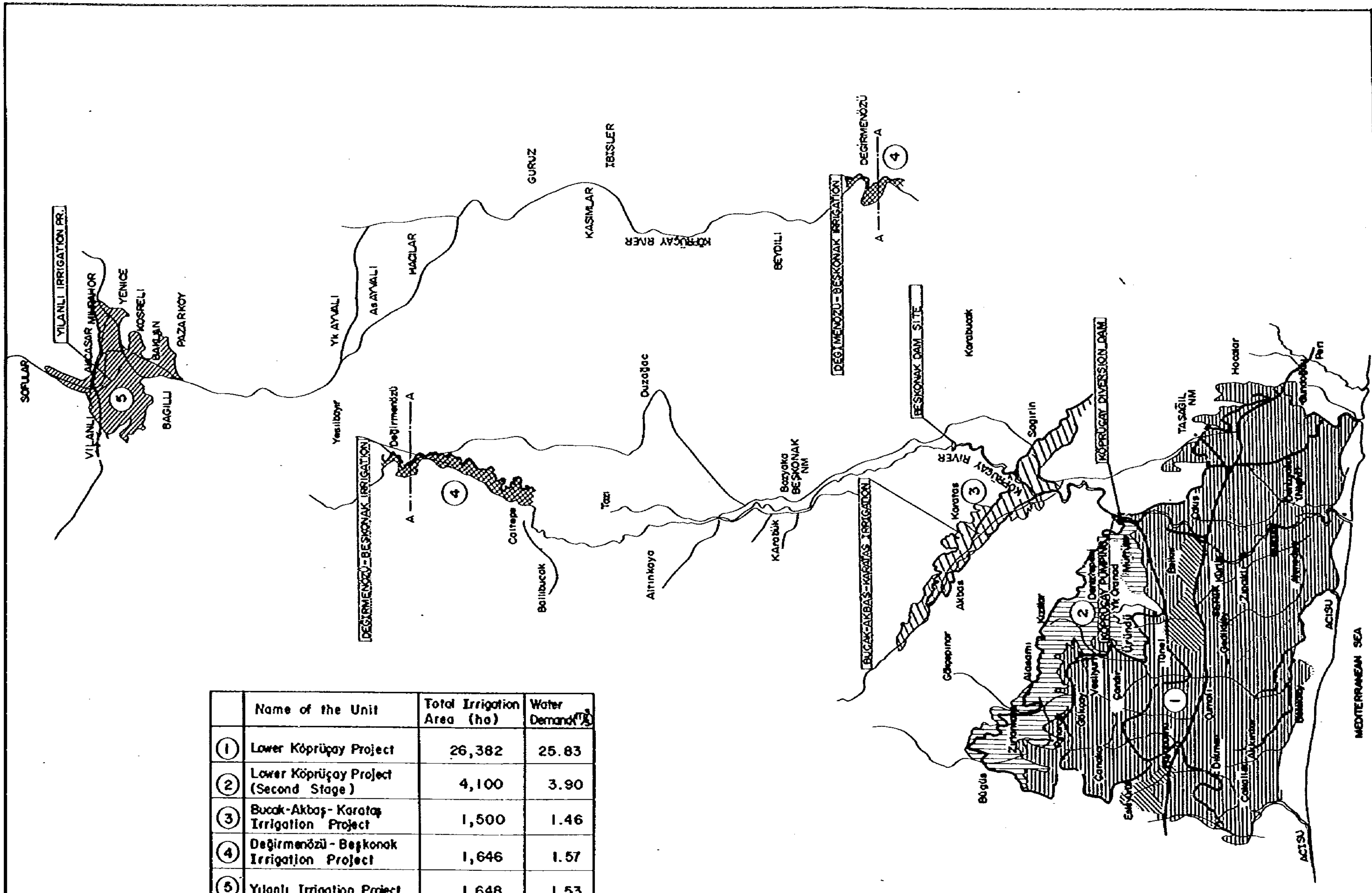
In order to select the optimum development scale, an economic evaluation was made based on construction costs as of March 1982 without considering cost escalation in future.

The annual cost (C) and benefit (B) equalized for the service life were obtained for this purpose and the economic effects was studied by net present value (B - C) and benefit-cost ratio (B/C). The costs of a hydroelectric power station are composed of interest, depreciation, repair costs, and operation and maintenance costs. The annual cost is determined as the product of multiplying equalized annual cost factor by total construction cost.

The benefit is considered to be the annual cost required of an alternative power station having an equal power generating capability to the Beskonak project. The alternative power source for this project was taken to be the oil-fired thermal power plant indicated in Table 13-4.

The kW and kWh prices obtained based on the above are as follows:

kW price :	15,509 TL/kW
kWh price:	7.58 TL/kWh



	Name of the Unit	Total Irrigation Area (ha)	Water Demand (m^3/s)
(1)	Lower Köprüçay Project	26,382	25.83
(2)	Lower Köprüçay Project (Second Stage)	4,100	3.90
(3)	Bucak-Akbaş-Karataş Irrigation Project	1,500	1.46
(4)	Değirmenözü - Beşkonak Irrigation Project	1,646	1.57
(5)	Yılanlı Irrigation Project	1,648	1.53

BEŞKONAK PROJECT

Köprüçay Irrigation Areas

Fig. 9-1 | Nov. 1983

9.2 Reservoir Operation Plan

9.2.1 Reservoir Operation Plan

The operation rules for the reservoir are to be prepared considering the following:

(1) In order to make the dependable discharge in the dry season as large as possible, operation is to be carried out with the runoff during the rainy season stored and regulated for supply in the dry season.

(2) Operation is to be carried out to secure output as stable as possible over a long period, and further, to increase energy production.

(3) Operation is to be carried out to make waste overflow from the reservoir as less as possible.

(4) The irrigation water for the downstream area is to be secured from June to September through the operation of the reservoir and power station.

9.2.2 Energy Calculation

Calculation of electric energy production at the power station is to be performed according to the conditions below and the procedure shown in Fig. 9-2.

(1) The monthly average runoffs in Table 6-1 are used for runoff data, and upon consideration of evaporation from the reservoir surface, calculations are made for the 40 years from October 1940 to September 1980.

(2) Giving consideration to variation in efficiencies of turbines and generators according to water level of the reservoir, the available discharge is held down in step

with the maximum output when the water level is higher than the normal intake level.

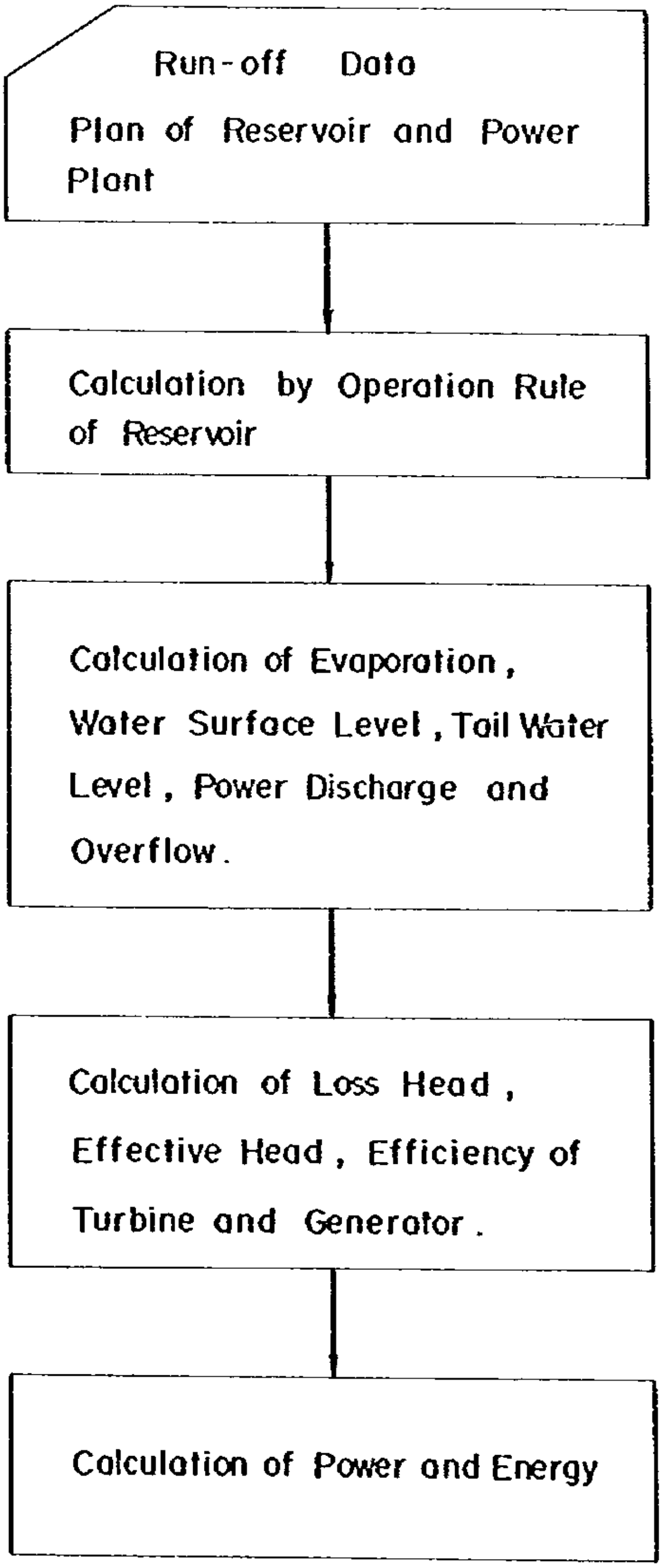
(3) In order to secure the irrigation water (June - Sept.), 30 m^3/sec of power discharge of a small unit will be made 24 hours a day even on a day when downstream water utilization can be adequately satisfied with the residual runoff.

(4) With the discharge mass curve at the dam site as the basis, dependable discharges are respectively determined for the irrigation season (Jun. - Sep.) and non-irrigation season (Oct. - May) in order that a stable output can be secured throughout the year.

(5) The probable annual energy production is to be the average annual energy production for the 40-year period.

(6) The dependable capacity is to be the 40-year average of monthly minimum 5-day averages at the required peak duration (6-hour).

Fig. 9-2 Procedure of Calculation of Power and Energy



9.3 Study on Development Scale

9.3.1 Study on Reservoir Scale

Fig. 9-3 shows the mass curve prepared based on the 40-year monthly average runoffs of the reservoir. River flow is divided into rainy season (December - May) and dry season (June - November). The average annual total inflow for the 40 years is 2,635 x 10^6 m^3, the inflow in the rainy season (December - May) is 1,934 x 10^6 m^3, and that in the dry season (June - November) is 701 x 10^6 m^3. The inflow in the rainy season is approximately 2.8 times that of the dry season. The driest year during the 40 years was 1950 with inflow of 1,539 x 10^6 m^3, while the wettest year was 1953 with 4,387 x 10^6 m^3.

In formulation of the Project, it is necessary for thorough consideration to be given to effectively utilizing the abundant water resources of the Köprücay River by storing and regulating the inflow which varies by season and by year. The storage capacity of the reservoir (see Fig. 9-4) is not of a size to regulate the inflow over the years, but of a size that the seasonal variations of each year can be annually regulated. It is necessary to aim for stabilization of generating output over a long term by both efficiently operating the reservoir and effectively regulating the inflow in the rainy season for supply during the dry season.

The high water level and the effective capacity of the reservoir must be selected so as to be most advantageous from the standpoint of economics. With respect to the high water level of the reservoir, the upper limit is to be EL. 155 m taking into account the topographical conditions of the dam site. Further, considering sedimentation volume, effective storage capacity, etc., comparison studies were made of the three cases of EL. 155 m, 150 m and 145 m.

On the other hand, from the result of sedimentation calculations described in 6.7, the lower limit of the low water level of the reservoir was set to be EL. 115 m. Concerning the effective capacity of the reservoir, comparison studies were made of the cases below for the various high water levels in the range of 160 x 10^6 m^3 to 390 x 10^6 m^3.

High Water Level (m)	Effective Storage Capacity (10^6 m^3)			
155	390	330	275	230
150	305	260	220	180
145	230	195	160	

The followings are various conditions considered in the comparison studies of high water level and effective storage capacity:

(1) That the project site is relatively close to the demand area, and that the daily-load curve would gradually peak in the future were taken into consideration and it was judged desirable for the power station to bear a peak duration of approximately 6 hours.

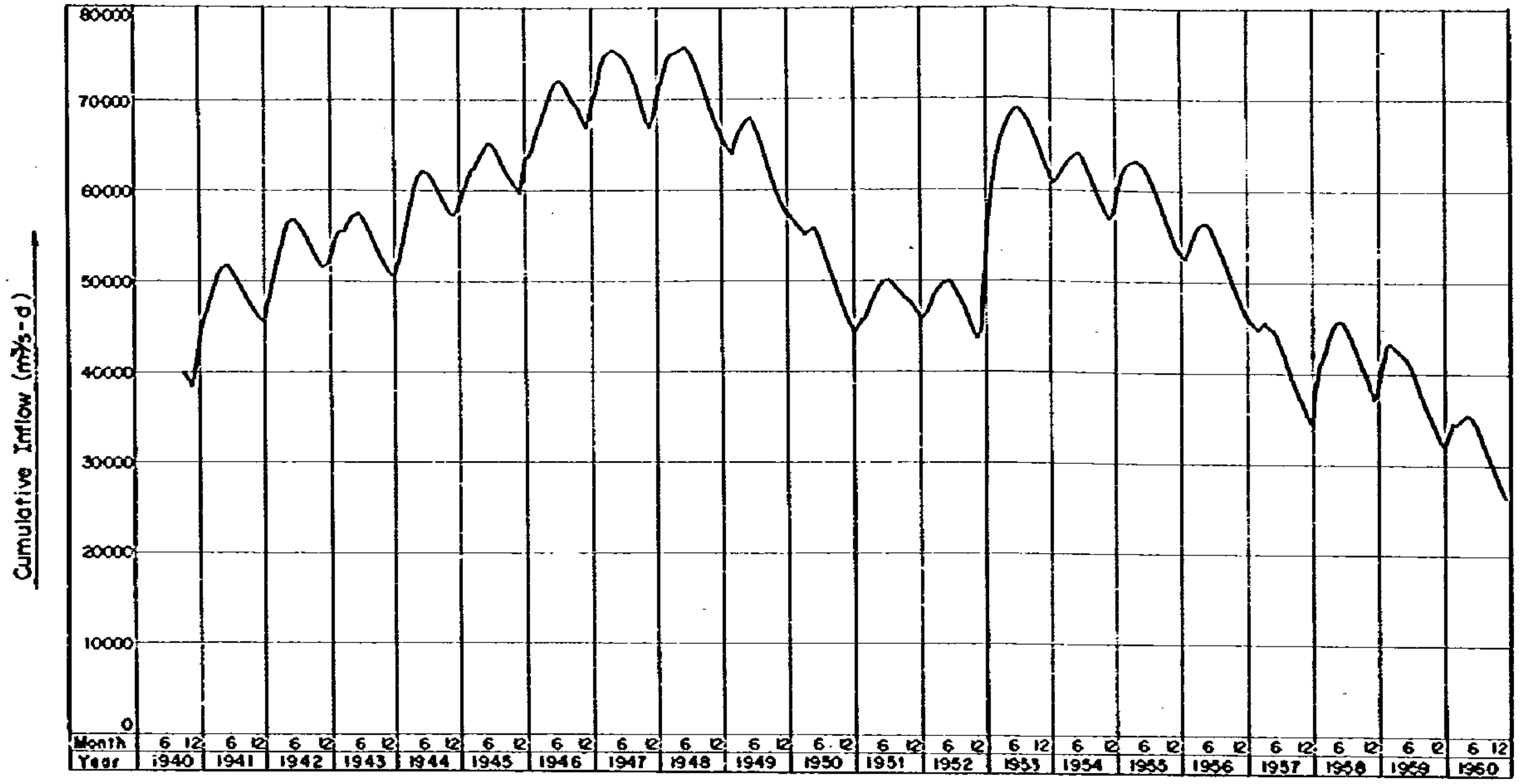
(2) The maximum available discharge and installed capacity of the power station were selected to match the peak duration time (6 hours) for the dependable discharge.

(3) The benefit was computed based on the annual costs per kW and kWh of the alternative thermal power station described in 13.2.

(4) The cost was computed multiplying construction cost by the equalized annual cost factor.

Table 9-1 and Fig. 9-5 indicate the results of comparison studies of high water levels and effective storage capacities of the reservoir. As a result of comparison studies of the various cases, it was found that the case of high water level of EL. 155 m and effective storage capacity of 275×10^6 m^3 gave the highest B/C and (B - C), for the greatest economic effect. Consequently, it was decided that the reservoir should have a high water level of EL. 155 m, available drawdown of 20.5 m, and effective storage capacity of 275×10^6 m^3.

Fig. 9-3 Mass Curve at Beşkonak Dam Site



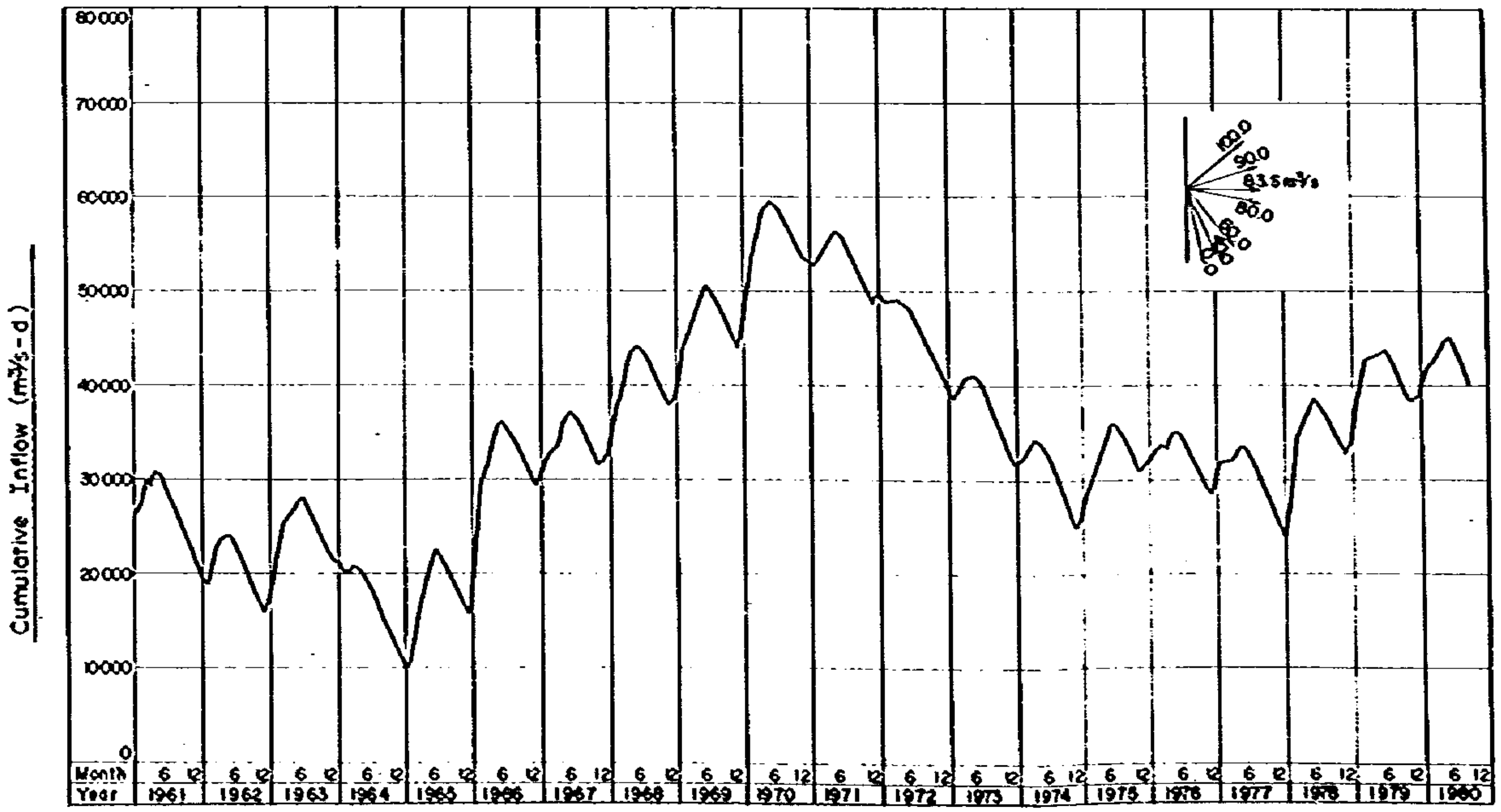
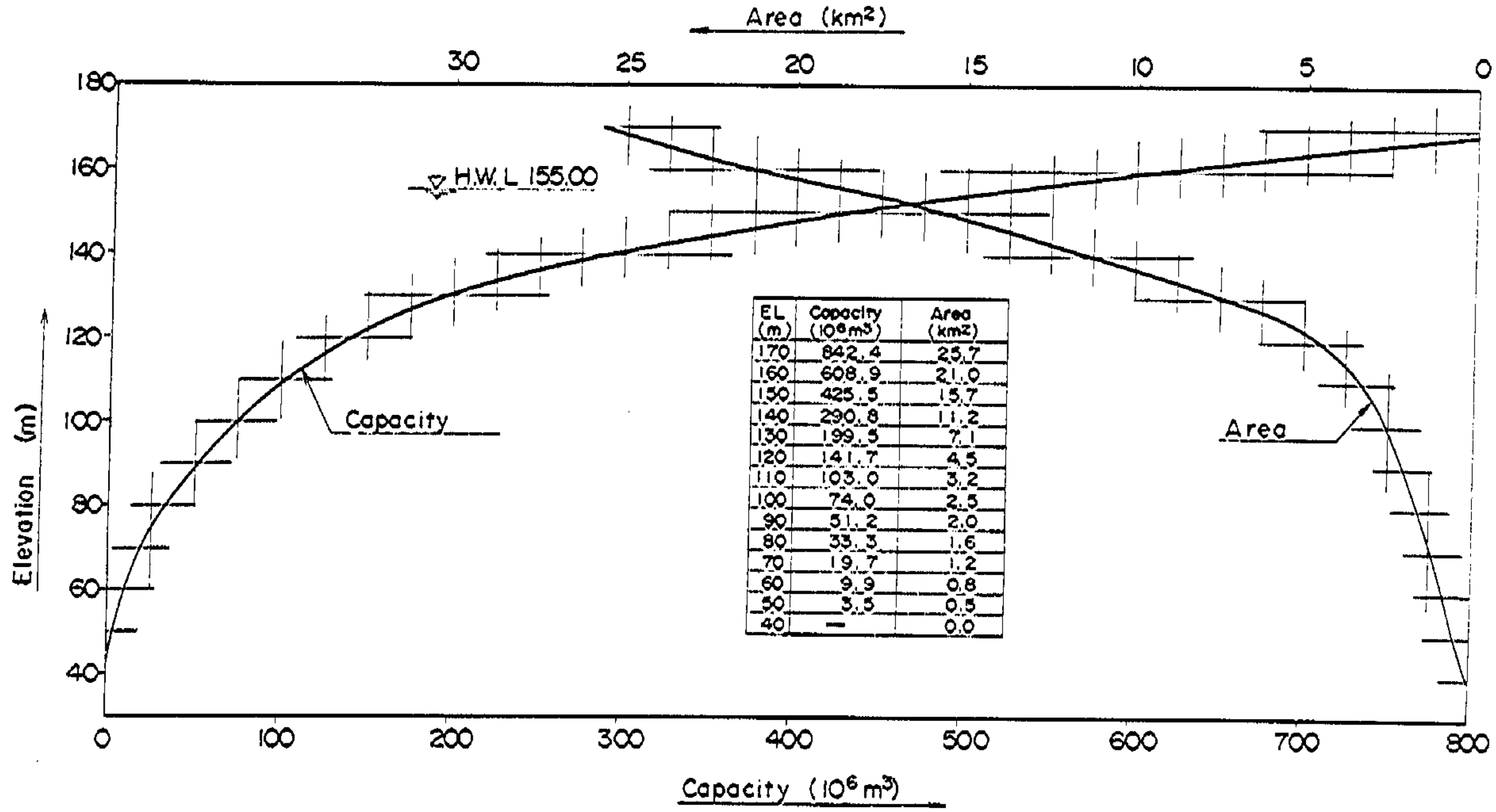


Fig. 9-4 Beşkonak Reservoir Capacity and Area Curve

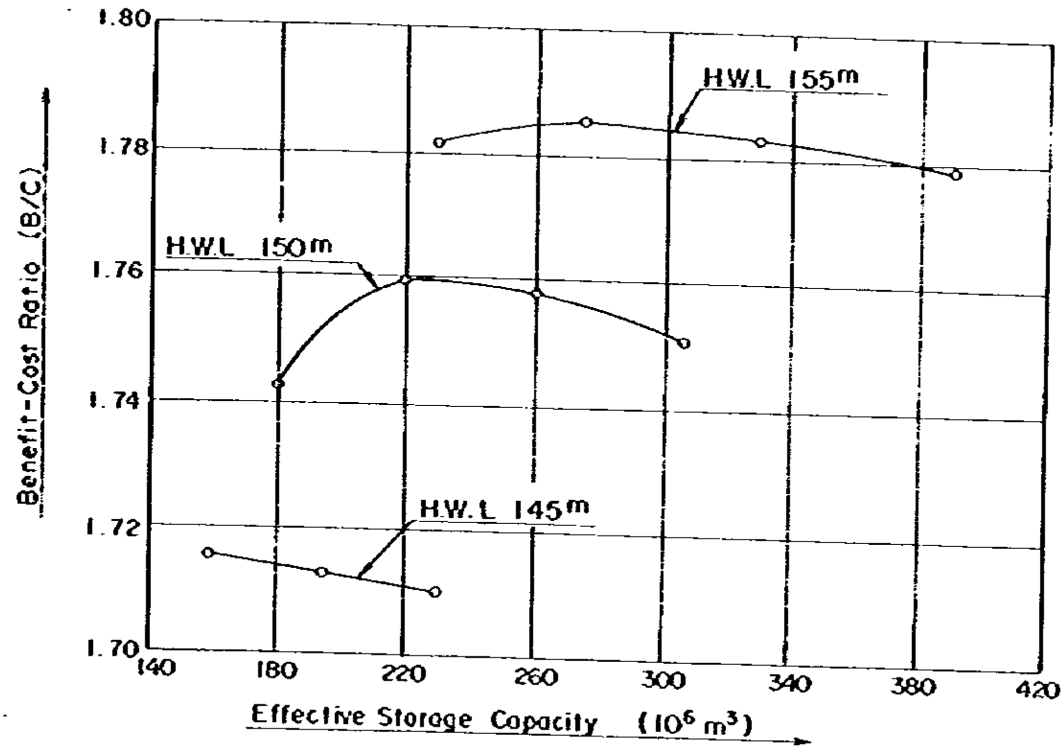


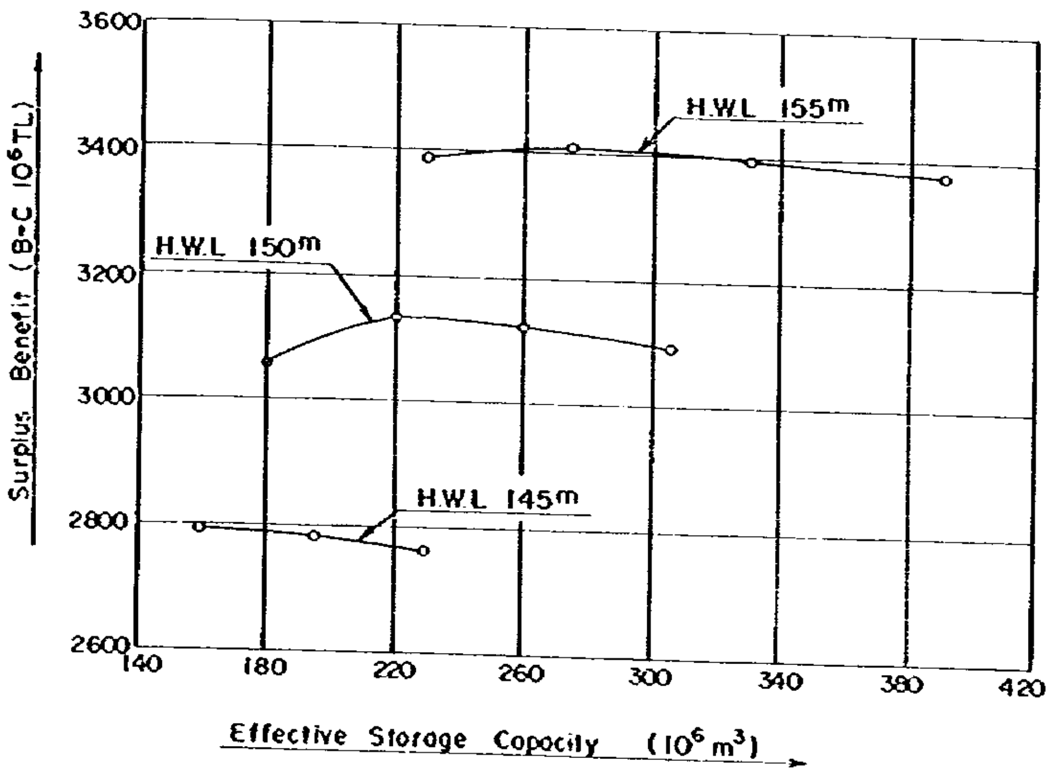
EL (m)	Capacity ($10^6 m^3$)	Area (km^2)
170	842.4	25.7
160	608.9	21.0
150	425.5	15.7
140	290.8	11.2
130	199.5	7.1
120	141.7	4.5
110	103.0	3.2
100	74.0	2.5
90	51.2	2.0
80	33.3	1.6
70	19.7	1.2
60	9.9	0.8
50	3.5	0.5
40	—	0.0

Table 9-1 Study on Optimum High Water Level and Effective Storage Capacity of Reservoir

Item	Unit	A-1	A-2	A-3	A-4	B-1	B-2	B-3	B-4	C-1	C-2	C-3
High Water Level	m	155	155	155	155	150	150	150	150	145	145	145
Low Water Level	m	115	127	134.5	139	115	125	131	136	115	122.5	128.5
Gross Storage Capacity	10^6m^3	507	507	507	507	425	425	425	425	349	349	349
Effective Storage Capacity	10^6m^3	390	330	275	230	305	260	220	180	230	195	160
Available Drawdown	m	40	28	20.5	16	35	25	19	14	30	22.5	16.5
Maximum Power Discharge	m^3/sec	226	218	214	210	218	202	199	195	191	187	183
Effective Head	m	99	103	105	107	96	99	101	103	92	94.5	96.5
Installed Capacity	MW	200	200	200	200	180	180	180	180	160	160	160
Dependable Capacity	MW	169.7	167.5	165.4	163.3	150.9	149.7	148.4	142.6	132.7	131.8	130.4
Annual Energy Production	10^6kWh	671.0	677.7	683.7	685.9	643.3	649.2	652.9	655.4	608.3	611.9	616.3
Annual Benefit												
kWh Benefit	10^6TL	5,086	5,137	5,183	5,199	4,876	4,921	4,949	4,968	4,611	4,638	4,672
kW Benefit	10^6TL	2,632	2,598	2,565	2,533	2,340	2,322	2,302	2,212	2,058	2,044	2,022
Total (B)	10^6TL	7,718	7,735	7,748	7,732	7,216	7,243	7,251	7,180	6,669	6,682	6,694
Construction Cost	10^6TL	37,727	37,727	37,727	37,727	35,825	35,825	35,825	35,825	33,921	33,921	33,921
Annual Cost (C)	10^6TL	4,339	4,339	4,339	4,339	4,120	4,120	4,120	4,120	3,901	3,901	3,901
Net Present Value (B - C)	10^6TL	3,379	3,396	3,409	3,393	3,096	3,123	3,131	3,060	2,768	2,781	2,793
Benefit - Cost Ratio (B/C)	-	1.779	1.783	1.786	1.782	1.751	1.758	1.760	1.743	1.710	1.713	1.716

Fig.9-5 Study on Optimum High Water Level and Effective Storage Capacity of Reservoir





9.3.2 Study on Installed Capacity

The maximum available discharge and installed capacity of the power station must be of a scale matching the peak duration required from the aspect of demand and supply balance, and further, be the most economical.

In general, if installed capacity is made larger, the output would become latent and the economics poor, while in contrast, when installed capacity is made smaller, the dependable peak capacity would be held down by the critical maximum output, the peak duration will become long, and the economics also poor.

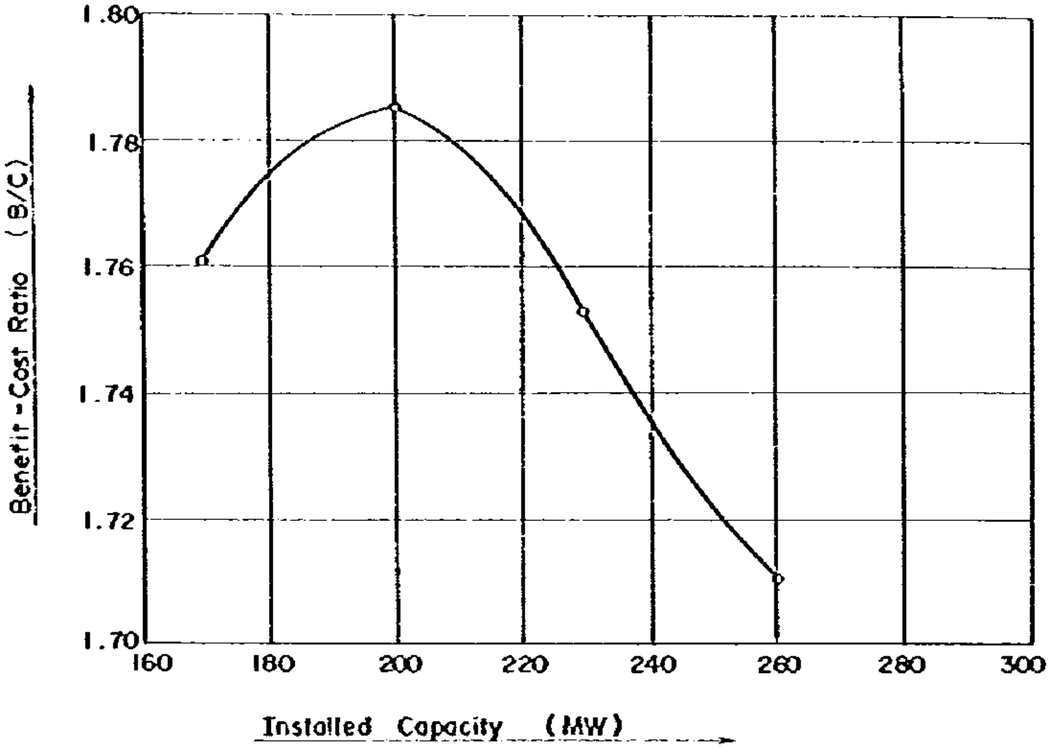
Comparison studies were made of the four cases of installed capacity of 170 MW, 200 MW, 230 MW and 260 MW with the purpose of selecting the optimum development scale for the Project. The results of the studies are indicated in Table 9-2 and Fig. 9-6.

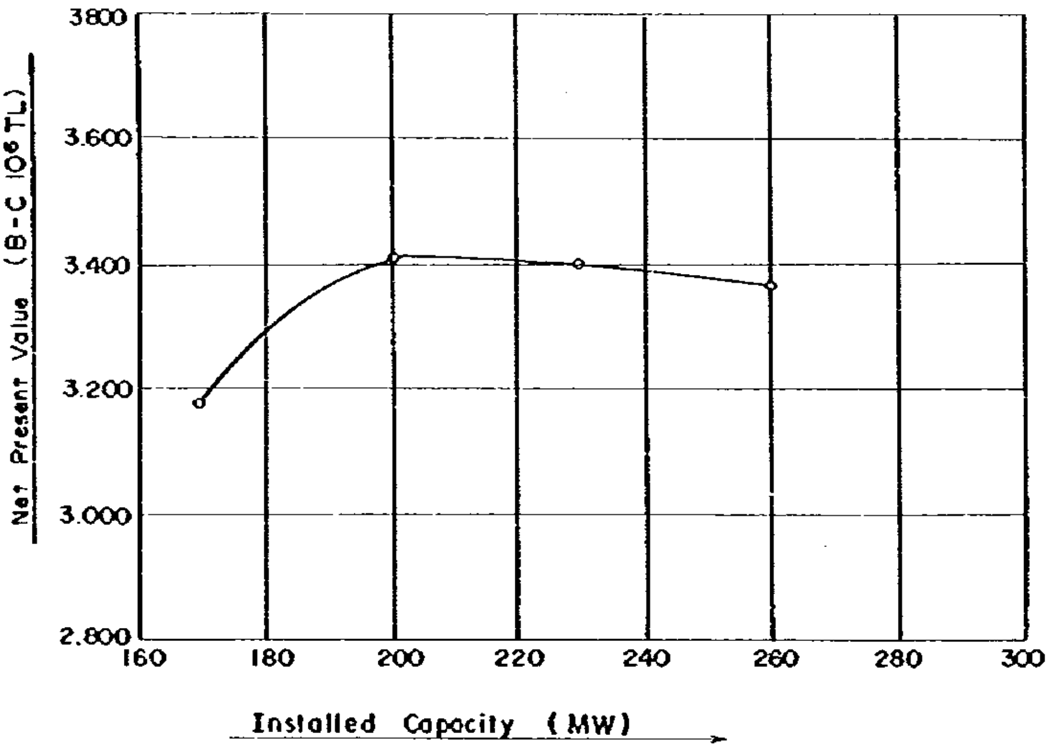
As a result of comparison studies, an installed capacity of 200 MW was selected as the optimum development scale for the Project.

Table 9-2 Study on Optimum Installed Capacity

Item	Unit	A	B	C	D
High Water Level	m	155.0	155.0	155.0	155.0
Effective Storage Capacity	$10^6 m^3$	275	275	275	275
Maximum Power Discharge	m^3/sec	182	214	246	279
Installed Capacity	MW	170	200	230	260
Dependable Capacity	MW	143.5	165.4	175.6	187.5
Annual Energy Production	10^6kWh	676.6	683.7	684.8	685.4
Annual Benefit					
kWh Benefit	10^6TL	5,129	5,182	5,191	5,195
kW Benefit	10^6TL	2,226	2,565	2,723	2,908
Total (B)	10^6TL	7,355	7,747	7,914	8,103
Cost					
Construction Cost	10^6TL	36,309	37,727	39,250	41,186
Annual Cost (C)	10^6TL	4,176	4,339	4,514	4,736
Net Present Value (B - C)	10^6TL	3,179	3,408	3,400	3,367
Benefit - Cost Ratio (B/C)	-	1.761	1.785	1.753	1.711

Fig. 9-6 Study on Optimum Installed Capacity





9.3.3 Study on Number of Units

In setting up operation of the power station, power discharge of 30 m^3/sec must be considered for 24 hours a day during the irrigation period (June - September) in order to secure irrigation water for the downstream area. Turbine operation must be performed with the small amount of 30 m^3/sec for a long period of 4 months. It will be necessary for the turbine and generator to be of small scale from the electro-mechanical aspects.

A comparison study was made on the two cases below, fixing the installed capacity at 200 MW, with regard to the number of units including turbines and generators of small scale.

Case	Installed Capacity (MW)	Units
A	200	155 MW 1 unit, 45 MW 1 unit
B	200	85 MW 2 units, 30 MW 1 unit

From the aspect of operation and maintenance in the future, Case A (155 MW, 45 MW) is inferior to Case B because of its different size unit. While investment cost of Case A is less than Case B especially in electro-mechanical equipment and civil works of power structures.

The results of the study are indicated in Table 9-3. As a result of the comparison study of the two cases, Case A was found to be more economical, and it was decided that the power station should be provided with a total of 200 MW of turbine-generators, consisting of 1 unit of 155 MW and 1 unit of 45 MW.

Table 9-3 Study on Number of Units of Beskonak P.S.

Item	Unit	Case A	Case B
Installed Capacity	MW	200	200
Maximum Power Discharge	m^3/sec	217	214
Number of Units	-	155 MW @1 unit 45 MW @1 unit	85 MW @2 units 30 MW @1 unit
Annual Energy Production	10^6kWh	659.9	683.7
Dependable Capacity	MW	161.8	165.4
Annual Benefit			
kWh Benefit	10^6TL	5,002	5,182
kW Benefit	10^6TL	2,509	2,565
Total (B)	10^6TL	7,511	7,747
Construction Cost	10^6TL	35,478	37,727
Annual Cost (C)	10^6TL	4,080	4,339
Net Present Value	10^6TL	3,431	3,408
Benefit - Cost Ratio (B/C)	-	1.841	1.785

9.4 Study on Regulating Pondage

As described in 9.2 and 9.3, the Project is to consist of high water level of EL. 155 m, effective storage capacity of 275 x 10^6 m^3, maximum available discharge of 217 m^3/sec and installed capacity of 200 MW (155 MW, 45 MW). Requirements of downstream water utilization during the irrigation season is to be met through operation of unit of 45 MW.

Meanwhile, another study was also made of a scheme in which a regulating pond (Kisik dam) and power station would be provided between Beskonak power station and Köprücay diversion dam. In this case, it would be possible for Beskonak power station to be operated only at peak time throughout the year, and it would be unnecessary to provide a small-scale unit, and the installed capacity would be 200 MW with the 2 units of 100 MW.

Regarding the downstream Kisik project, it will be discussed in detail in Chapter 15. Table 9-4 indicates the results of a comparison study of the following different development systems.

Case A: Beskonak power station (155 MW + 45 MW = 200 MW)
Case B: Beskonak power station (100 MW @2 = 200 MW)
Kisik power station (8 MW @2 = 16 MW)

Case A is a scheme for independent development without providing a regulating pond, the downstream irrigation water being secured with the power discharge of Beskonak power station. On the other hand, Case B is a scheme for Beskonak and Kisik projects to be developed simultaneously. The peak power discharge of Beskonak power station is to be stored and regulated at Kisik Dam for uniform discharge to the downstream Köprücay diversion dam.

Case A, because of 24-hour power generation discharge during the irrigation season, will have annual energy production of 659.9 x 10^6 kWh, which is the smaller than that of Case B. Case B is a scheme for developing the Köprücay River in the most ideal manner, with the annual energy production (including Kisik power station) being 139 x 10^6 kWh larger than Case A, and B/C will be 1.789 and (B - C) 4,039 x 10^6 TL.

Case A does not involve any problem in particular regarding land acquisition and other aspects. However Case B, by construction of Kisik dam, will result in inundation of flat cultivated land upstream of the dam to affect the Bucak-Akbas-Karatas irrigation project of DSI.

As a result of various studies regarding the above two cases, Case A was selected as the optimum proposal for the Project. The reasons for the selection are given below.

(1) Case A has slightly less annual energy production and less effective output compared with Case B, but is not accompanied by any problem in particular. On the other hand, Case B, as a result of construction of Kisik dam, would conflict with the Bucak-Akbas-Karatas irrigation project which is already partially under construction, and approximately 70% of the agricultural land in the irrigation project would be inundated. Consequently, there is concern that various difficulties will be encountered in land acquisition.

(2) If, after Beskonak project of Case A has been developed independently, the above various problems concerning the Kisik project are resolved to make construction of Kisik dam and power station possible, it will be possible to use Beskonak power station (155 MW, 45 MW) exclusively for peak power generation by changing the operation rule. In such case the annual energy production of Beskonak power station is estimated to be 43.1 x 10^6 kWh larger, and effective output also about 29.5 MW larger. The economic effect of the Kisik project will be B/C of 1.638 and (B - C) of 654 x 10^6 TL as of March 1982 when the above incremental benefit of the upstream Beskonak project is considered.

Table 9-4 Study on Optimum Development System

Item	Unit	Case A	Case B		
Name of Power Station	-	Beskonak	Beskonak	Kisik	Total
Installed Capacity	MW	200	200	16	216
Number of Units	-	155 MW@1 unit 45 MW@1 unit	100 MW @2 units	8 MW @2 units	-
Annual Energy Production	10^6kWh	659.9	703.0	95.9	798.9
Dependable Capacity	MW	161.8	191.3	10.8	202.1
Annual Benefit					
kWh Benefit	10^6TL	5,002	5,329	727	6,056
kW Benefit	10^6TL	2,509	2,967	167	3,134
Total (B)	10^6TL	7,511	8,296	894	9,190
Construction Cost	10^6TL	35,478	35,876	8,910	44,786
Annual Cost (C)	10^6TL	4,080	-	-	5,151
Net Present Value (B - C)	10^6TL	3,431	-	-	4,039
Benefit - Cost Ratio (B/C)	-	1.841	-	-	1.784

As a result of the studies of 9.3 and 9.4, it was decided that the Beskonak Hydroelectric Power Development Project should consist of Beskonak reservoir with high water level of EL. 155 m, effective storage capacity of 275 x 10^6 m^3, and available drawdown of 20.5 m, and of Beskonak power station with maximum available discharge of 217 m^3/sec and installed capacity of 200 MW (155 MW, 45 MW).

Table 9-5 indicates the inflow, the available discharge, evaporation, and overflow when operating Beskonak reservoir for the 40-year period of 1941 to 1980. The storage quantities, supply quantities and reservoir water levels for the individual months are shown in Fig. 9-7, and the monthly energy productions of Beskonak power station in Table 9-6 and Fig. 9-8.

Table 9-5 Summary Operation Study of Beşkonak Reservoir

Year	Inflow ($10^6 m^3$)	Evaporation ($10^6 m^3$)	Outflow for Energy ($10^6 m^3$)	Outflow from Spillway ($10^6 m^3$)
1941	3,277.9	20.0	3,195.7	62.1
1942	3,077.9	19.8	3,058.1	0.0
1943	2,670.2	20.1	2,650.2	0.0
1944	3,138.7	19.7	3,112.9	6.1
1945	2,861.9	20.0	2,841.9	0.0
1946	3,324.0	20.6	3,254.2	0.0
1947	2,697.3	20.0	2,636.4	0.0
1948	2,779.2	19.9	2,759.2	0.0
1949	1,935.0	18.9	1,858.0	0.0
1950	1,539.3	18.7	1,551.3	0.0
1951	2,536.8	20.0	2,365.3	0.0
1952	2,361.5	20.7	2,459.9	0.0
1953	4,386.9	20.2	3,578.1	788.6
1954	2,081.9	19.2	2,062.8	0.0
1955	2,438.3	19.6	2,418.7	0.0
1956	2,111.0	19.3	2,091.7	0.0
1957	1,574.3	18.6	1,584.6	0.0
1958	2,850.7	19.5	2,632.2	170.0
1959	2,180.1	19.3	2,121.9	67.5
1960	2,022.1	19.3	1,974.2	0.0
1961	2,145.5	18.7	2,161.3	0.0
1962	2,167.0	19.1	2,113.4	0.0
1963	3,129.8	19.8	3,109.9	0.0
1964	1,734.5	19.3	1,715.2	0.0
1965	3,048.8	19.5	2,976.0	53.3
1966	3,839.7	19.9	3,330.2	439.7
1967	2,700.0	19.7	2,680.2	0.0
1968	3,228.1	20.0	3,179.1	28.5
1969	3,250.9	19.9	3,201.3	30.1
1970	3,371.6	19.8	3,278.1	73.5
1971	2,279.1	19.5	2,259.6	0.0
1972	1,957.2	19.6	1,937.6	0.0
1973	1,954.1	19.1	1,934.9	0.0
1974	1,964.4	19.2	1,945.3	0.0
1975	3,021.3	19.7	3,001.5	0.0
1976	2,485.8	20.1	2,465.7	0.0
1977	2,439.6	20.0	2,419.6	0.0
1978	3,185.5	19.5	2,951.6	215.4
1979	3,082.2	20.1	2,914.7	147.4
1980	2,682.1	20.0	2,662.1	0.0
Average	2,634.8	19.7	2,561.8	53.3

Fig. 9-7 Reservoir Operation (1)

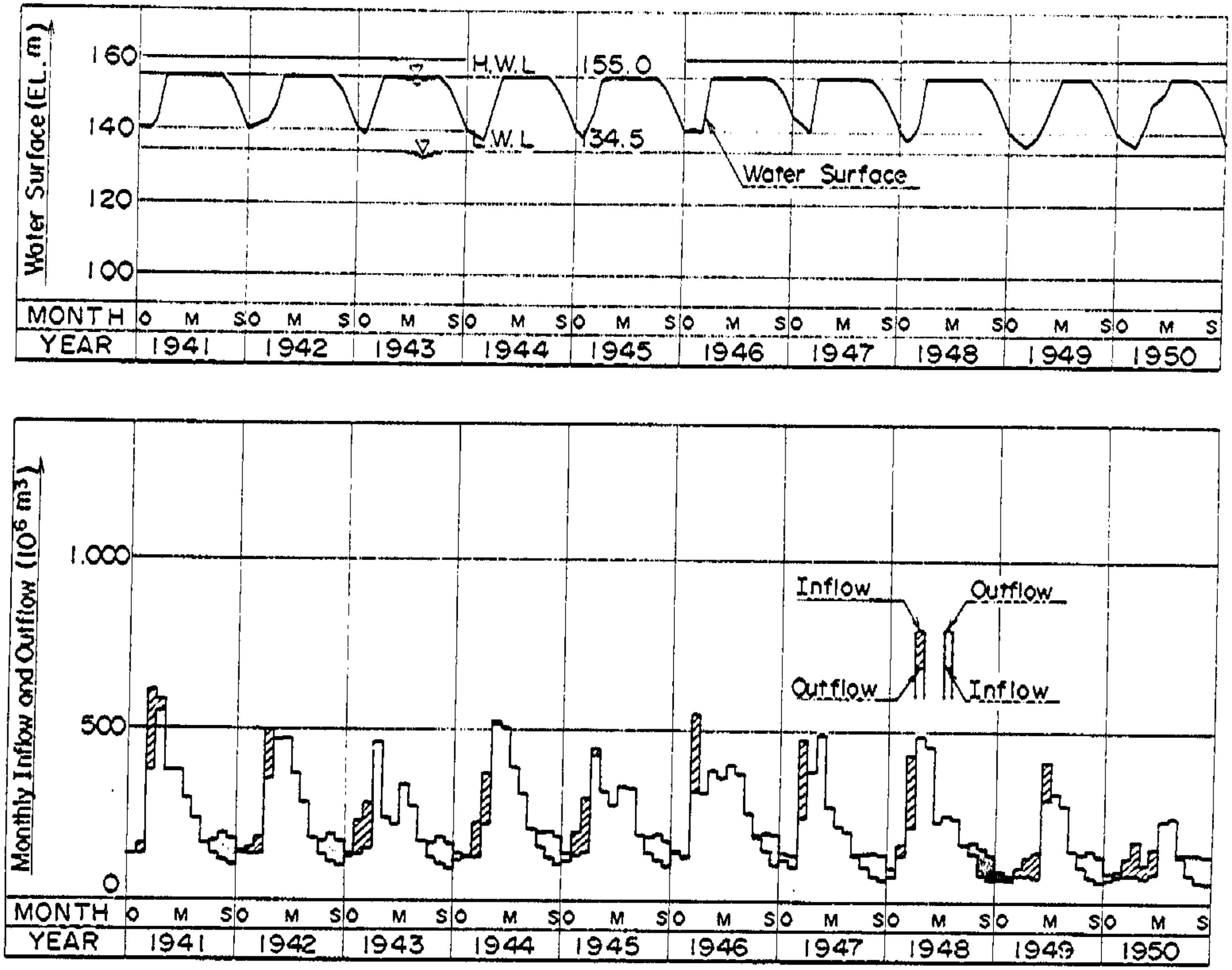


Fig. 9-7 Reservoir Operation (2)

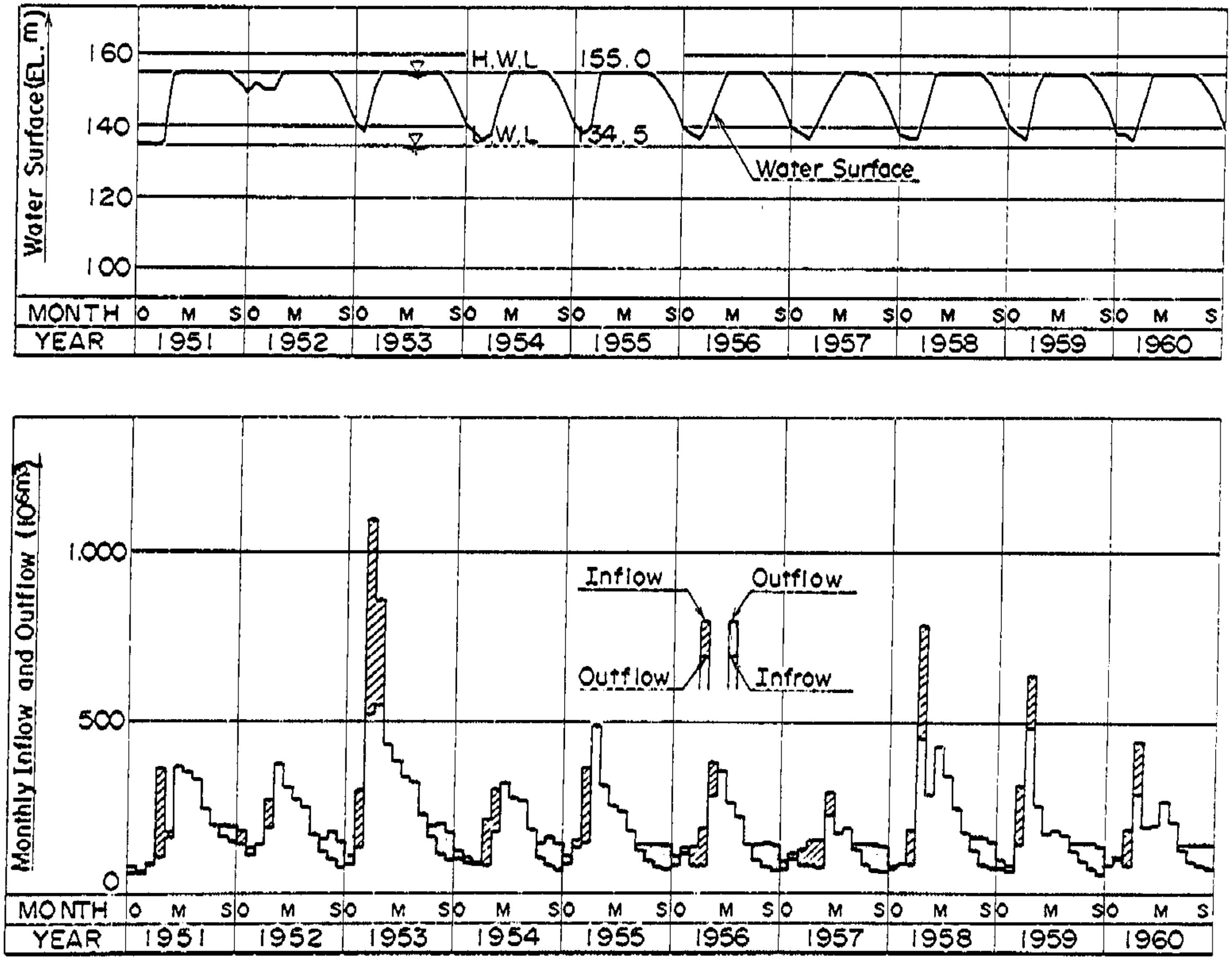


Fig. 9-7 Reservoir Operation (3)

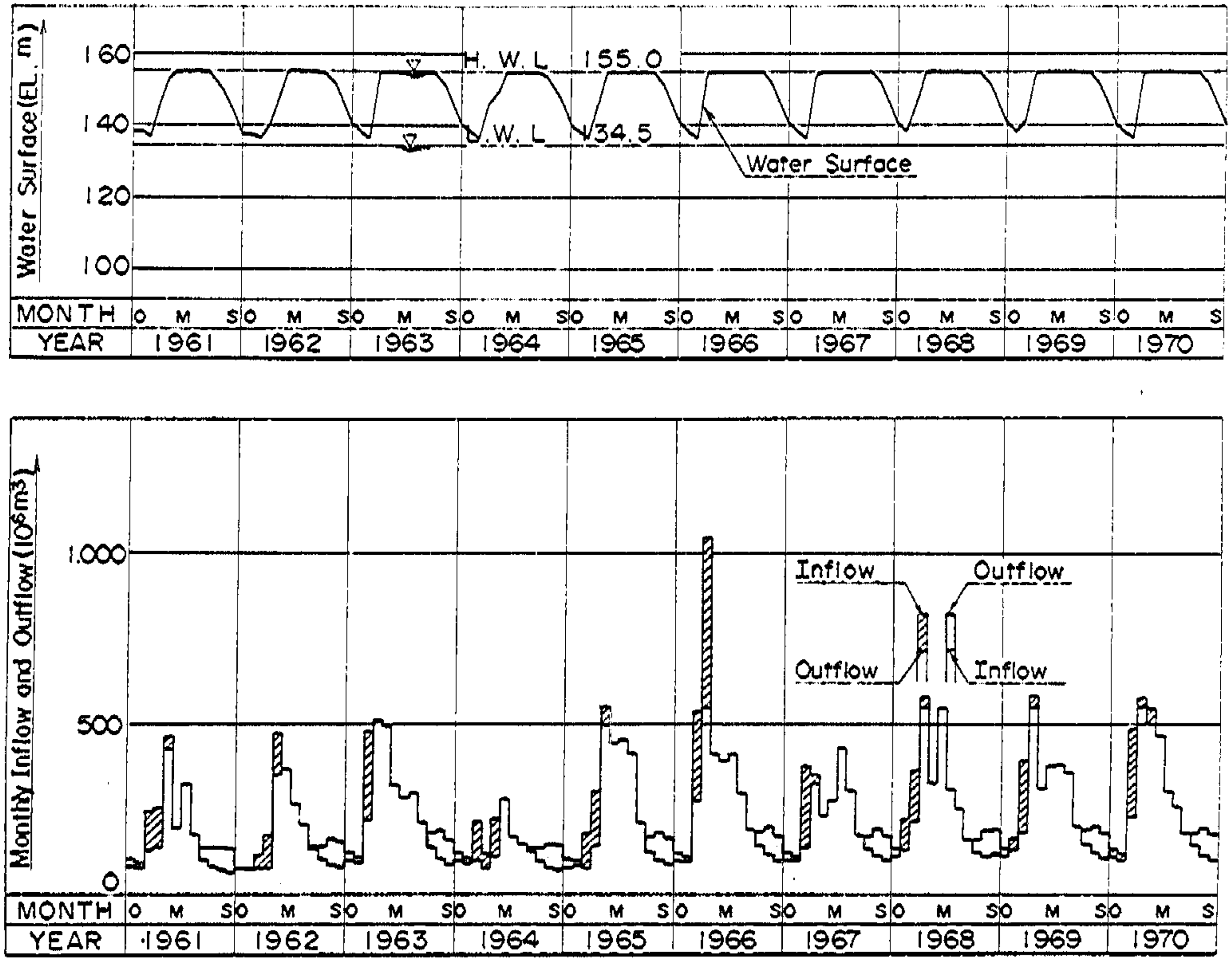


Fig. 9-7 Reservoir Operation (4)

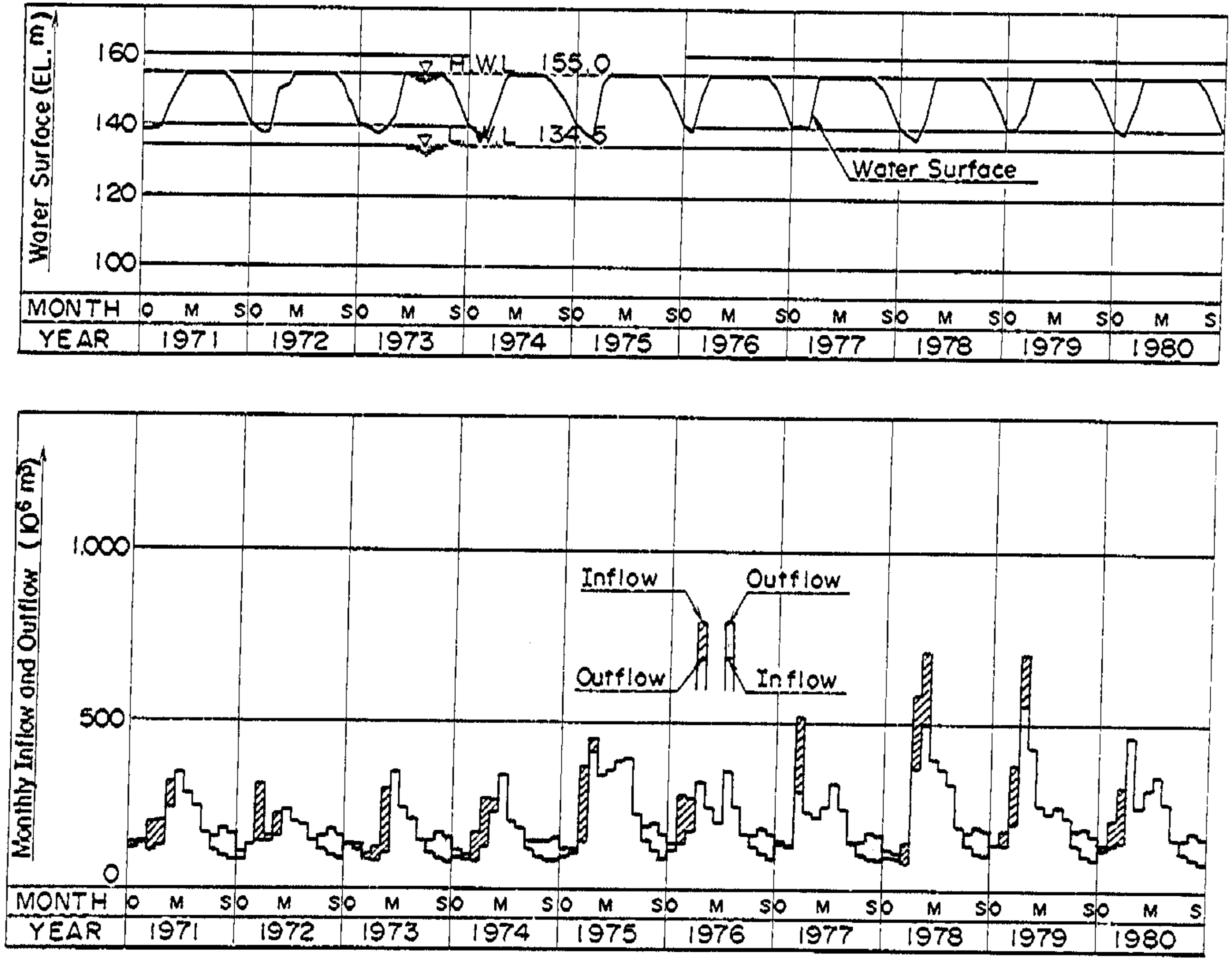
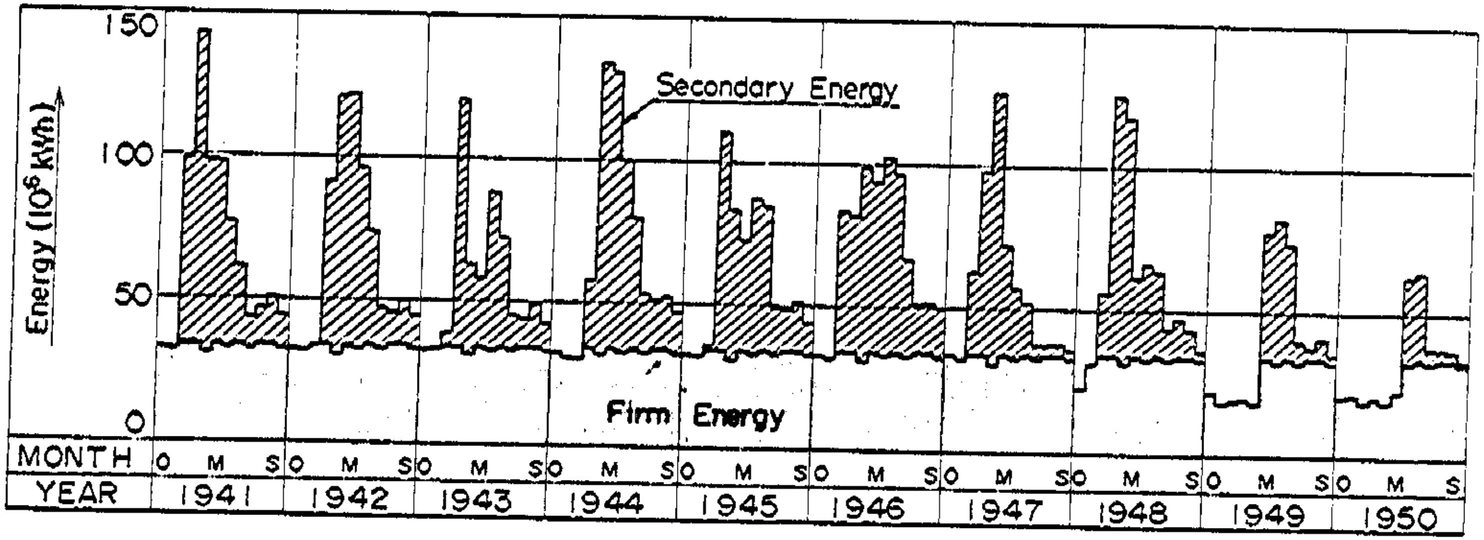


Table 9-6 Energy Production of Beşkonak P.S.

Unit: GWh

Year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Total
1941	32.7	32.2	93.7	142.7	97.7	97.7	76.6	61.3	43.3	45.4	50.9	44.6	825.0
1942	33.0	32.4	33.6	91.0	120.7	121.2	95.5	73.5	45.8	45.6	50.4	44.1	787.7
1943	32.6	33.3	38.8	119.8	62.6	57.6	87.7	71.5	45.0	44.1	49.0	42.2	684.1
1944	32.5	30.8	30.3	57.6	133.5	130.6	59.7	79.6	53.1	51.3	52.3	47.0	798.2
1945	32.4	32.3	35.1	110.2	82.8	72.3	86.7	84.6	49.3	45.6	51.6	43.9	730.8
1946	33.0	31.8	83.1	81.5	98.7	93.3	102.2	95.1	66.4	51.1	51.6	50.4	839.2
1947	34.6	32.3	62.7	97.6	125.2	72.0	55.8	52.2	37.8	37.7	38.1	35.3	692.2
1948	23.3	31.5	56.0	124.5	117.0	61.2	66.0	63.6	43.8	47.3	43.1	35.6	712.8
1949	22.2	18.4	19.1	19.7	18.7	77.6	82.2	73.7	39.9	38.2	41.1	34.9	485.7
1950	21.0	21.8	19.3	20.5	19.2	22.9	62.7	65.5	37.8	37.7	37.4	34.1	398.9
1951	18.9	14.5	18.6	26.8	42.4	95.4	90.5	85.5	62.3	51.0	51.3	50.4	607.6
1952	36.3	35.1	35.4	48.2	97.3	79.3	70.3	64.4	44.6	42.6	47.2	37.8	639.5
1953	25.5	33.8	135.5	142.7	112.3	99.4	86.6	83.4	58.7	51.3	52.3	43.7	925.2
1954	29.3	24.5	19.0	20.1	45.5	83.0	71.4	69.2	48.5	38.2	43.0	36.2	528.8
1955	26.2	31.2	35.7	126.8	81.5	65.6	61.4	46.7	37.3	37.7	37.6	35.5	624.1
1956	25.4	30.4	19.4	20.6	73.7	92.1	68.2	57.3	37.7	37.7	39.2	35.9	537.6
1957	22.3	26.6	19.3	20.3	19.1	59.1	44.1	45.5	37.3	37.7	37.3	34.1	405.8
1958	19.0	19.8	19.8	117.8	73.4	110.9	87.8	63.4	45.5	43.3	42.7	37.2	680.6
1959	23.1	18.7	34.5	124.7	65.8	43.9	47.1	42.5	37.3	37.7	37.4	34.2	547.0
1960	19.2	25.4	19.8	75.1	49.4	50.5	68.6	52.4	37.3	37.7	37.6	36.1	509.0
1961	25.5	22.1	32.2	35.3	111.4	51.1	84.6	45.6	37.3	37.7	37.0	33.8	554.7
1962	19.0	19.0	19.3	20.4	91.1	95.1	68.5	53.5	37.8	37.7	44.0	39.5	544.8
1963	30.2	25.8	56.4	132.7	128.2	83.2	73.7	77.6	54.0	48.5	49.4	41.4	801.1
1964	29.9	25.5	25.1	20.9	39.4	73.3	44.6	38.3	37.3	37.7	39.6	37.6	449.2
1965	25.4	23.8	19.8	37.4	123.9	115.8	118.1	107.3	54.3	45.1	48.8	42.2	761.8
1966	29.7	26.6	71.5	142.7	106.5	102.1	107.2	76.9	49.3	43.9	52.3	44.1	852.8
1967	29.9	27.2	34.9	85.7	60.9	71.7	111.8	73.0	45.3	45.5	51.0	44.0	669.1
1968	32.4	32.6	55.8	142.7	85.7	141.7	80.0	65.6	42.5	43.4	49.7	48.3	820.5
1969	32.4	31.5	47.0	142.7	80.8	93.5	93.7	92.8	51.5	49.9	52.3	45.6	823.7
1970	31.7	27.3	58.1	142.7	128.9	121.1	78.1	66.3	45.8	47.4	51.1	44.4	843.9
1971	32.5	31.2	27.4	32.6	62.1	87.9	72.9	62.7	42.9	40.4	45.8	40.4	579.8
1972	26.1	30.7	34.9	36.3	40.5	60.5	51.3	49.1	37.7	41.8	47.1	40.8	495.9
1973	32.7	31.1	20.5	19.8	27.6	89.6	62.0	53.2	37.8	37.7	44.5	38.6	495.0
1974	27.8	23.7	19.7	32.8	59.5	87.0	51.9	45.7	37.3	37.7	37.6	38.8	500.4
1975	28.7	28.2	34.9	155.9	86.7	91.0	95.3	93.7	57.0	48.8	51.2	41.3	771.6
1976	32.4	33.7	44.5	82.7	62.7	51.7	90.7	63.7	42.5	43.2	48.3	41.1	637.1
1977	33.0	31.9	75.5	59.8	55.3	63.7	81.5	61.8	37.8	37.7	44.8	41.3	624.0
1978	28.9	25.7	19.4	93.9	128.9	100.4	93.2	81.3	48.9	45.5	49.3	44.5	759.8
1979	32.9	32.8	51.5	142.7	109.8	65.1	59.3	64.6	55.4	47.9	48.8	41.7	752.5
1980	32.5	32.6	37.8	118.4	64.6	78.2	68.1	67.4	40.7	40.3	45.4	40.1	632.4
Ave.	28.4	28.0	40.6	80.4	79.7	82.8	78.2	67.2	45.1	43.2	45.8	40.5	659.9

Fig. 9-8 Monthly Energy Production of Beşkonak P.S (1)



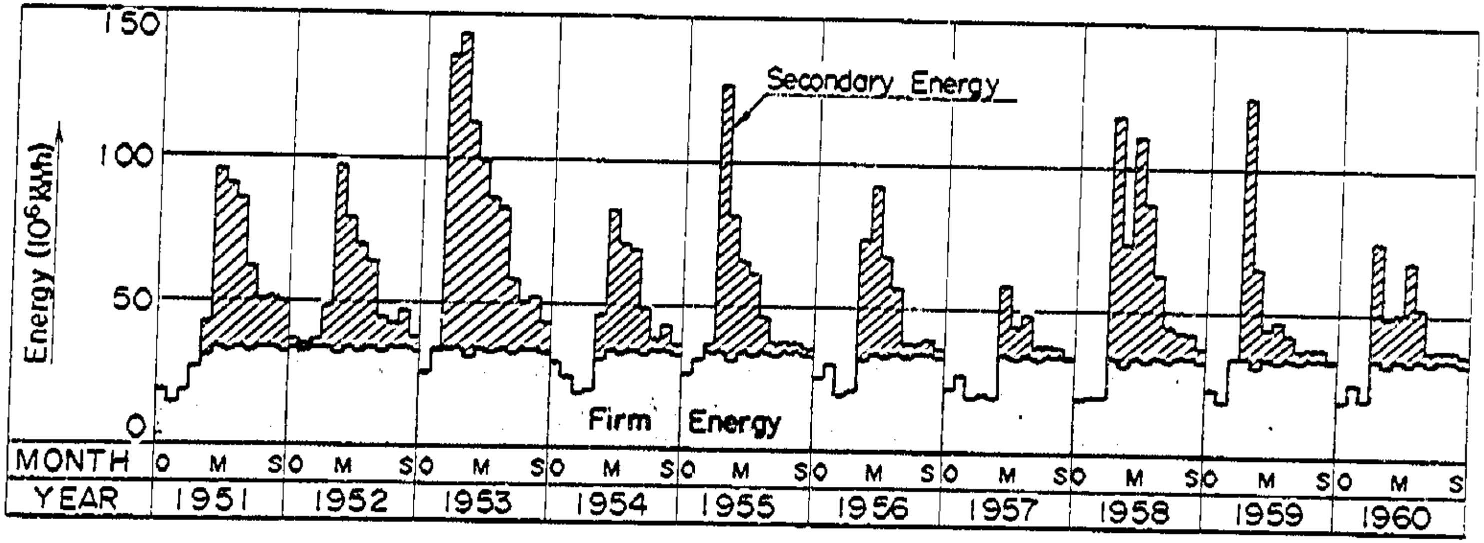
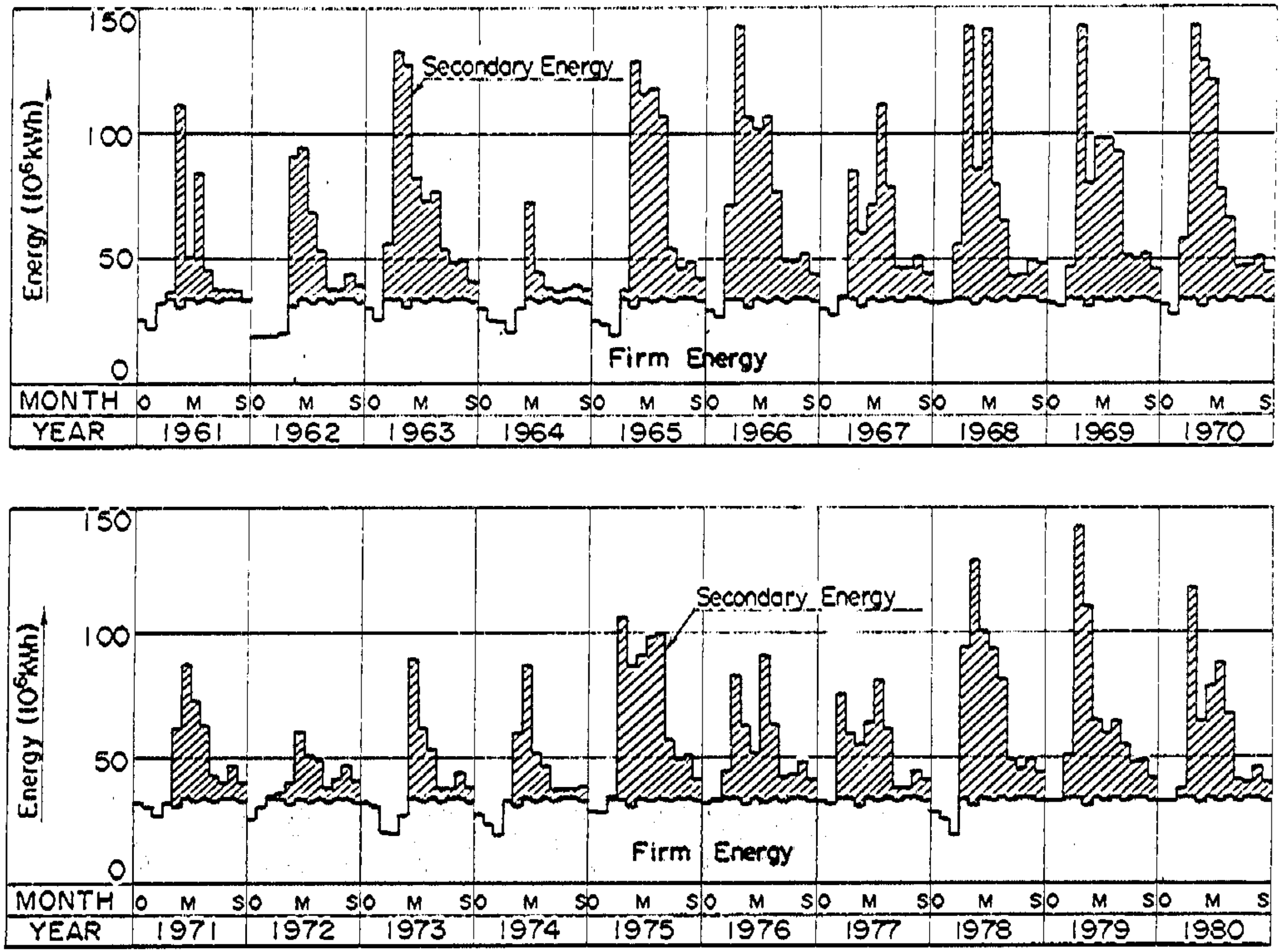


Fig. 9-8 Monthly Energy Production of Beşkonak P.S (2)



CHAPTER 10

TRANSMISSION LINE PLAN AND SYSTEM ANALYSIS

CHAPTER 10 TRANSMISSION LINE PLAN AND SYSTEM ANALYSIS

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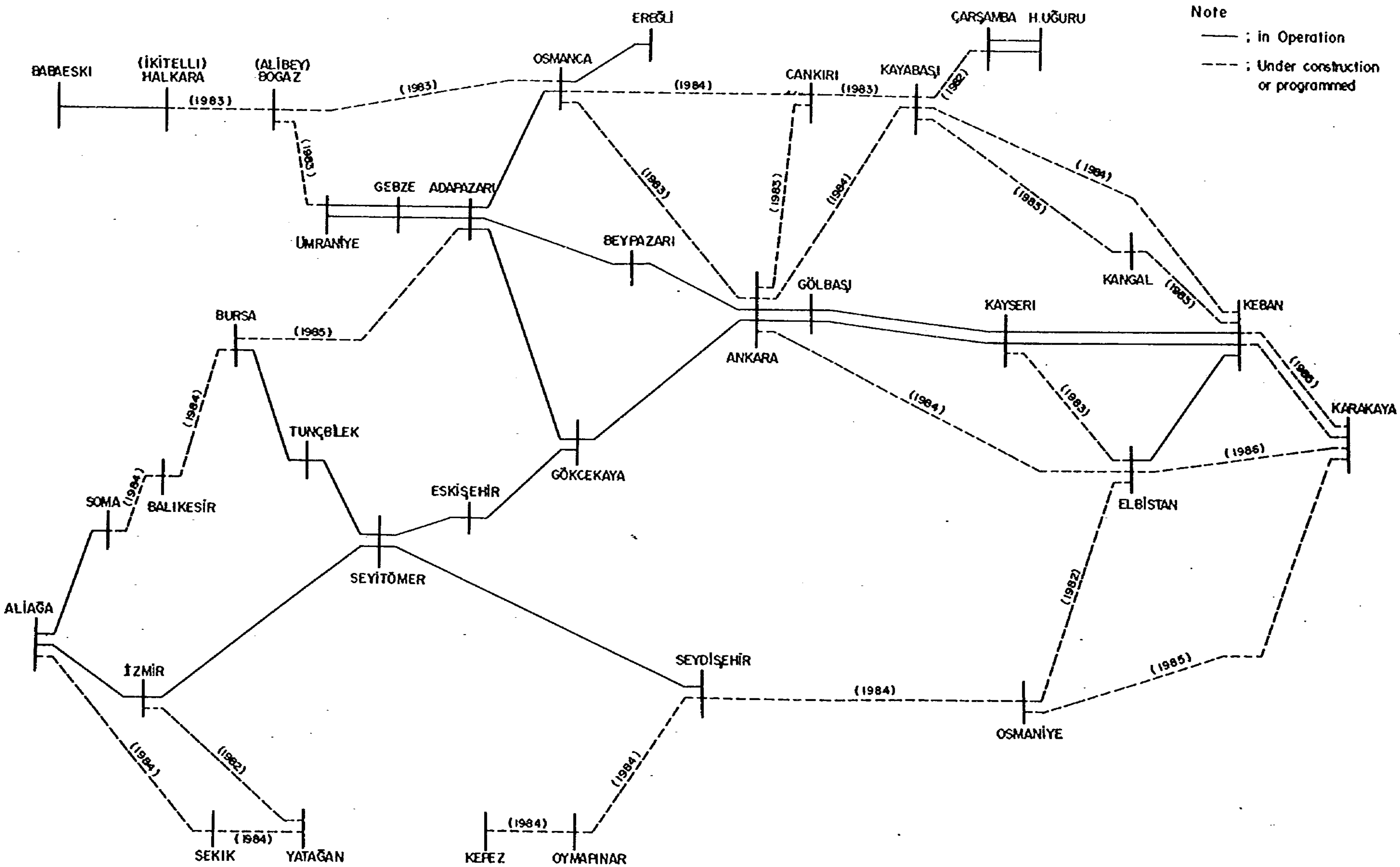
CHAPTER 10 TRANSMISSION LINE PLAN AND SYSTEM ANALYSIS

10.1 Outline of Power System

The major transmission lines of the electric power system of Turkey are composed, in order from high voltage, of 380 kV and 154 kV. Whereas power consumption areas are situated in the western part of Turkey centered at Istanbul and Izmir, the principal power generation areas are concentrated in the eastern Turkey more than 1,000 km distant from these load centers. Because of this, the power flows of the 380 kV transmission lines connecting the two areas are toward the west in all sections, and series capacitors are employed in some of the sections to maintain the system stability. The 380 kV power transmission system is shown in Fig. 10-1.

Fig. 10-1 380kV Transmission System in 1982



10.2 Power Transmission Scheme for Beskonak Project

It is forecast that the power demand in Antalya region will be approximately 500 MW around 1993 when the Beskonak project is scheduled to be completed. Consequently, the power transmission scheme was set up assuming that all of the electric power of the Project would be consumed in Antalya region.

The following items were studied in formulating the power transmission pattern:

- Receiving substation
- Transmitting voltage
- Conductor size
- Number of circuits
- Transmission line length

10.2.1 Receiving Substation and Lead-in Point

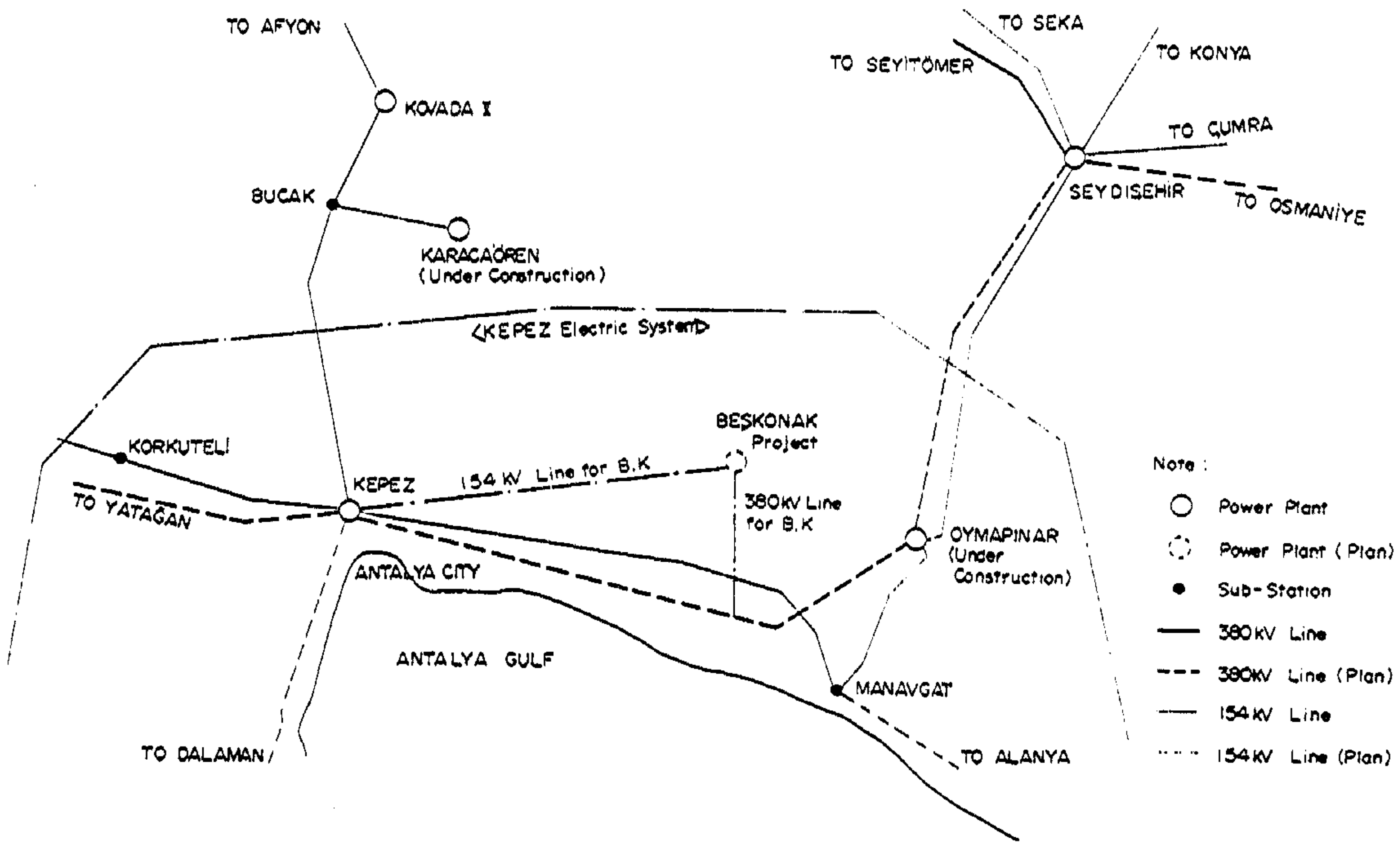
Kepez substation located in Antalya city was taken as the receiving substation. The following two places were selected as the lead-in points for transmission lines.

(1) Kepez substation in Antalya city

(2) Projected 380 kV transmission line passing vicinity of the project site

Besides the above, there could be a scheme to connect to an existing 154 kV transmission line, but power-carrying thermal capacity of that transmission line would be insufficient for transmitting the power of the Project, and it was therefore taken out of consideration. A diagram of the Kepez power system and the route of the Beskonak transmission line are given in Fig. 10-2.

Fig. IC-2 Transmission System of Antalya Region (in 1982)



10.2.2 Selection of Voltage and Conductor Size

The conductor sizes presently adopted by TEK for the different classes of voltage are

154 kV: 477 MCM and 795 MCM

380 kV: 954 MCM x 2B and 954 MCM x 3B

and the appropriate sizes were selected from the above.

10.2.3 Number of Circuits

The number of circuits for the transmission line was studied respectively for one- and two-circuit.

10.2.4 Length of Transmission Line

The project site - Kepez substation : 65 km

The project site - projected 380 kV transmission line : 25 km

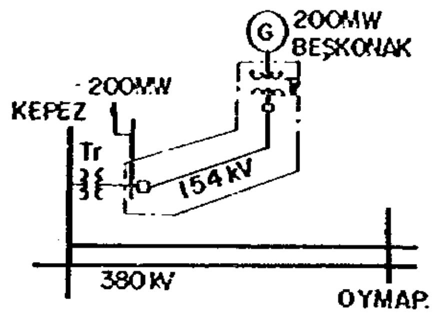
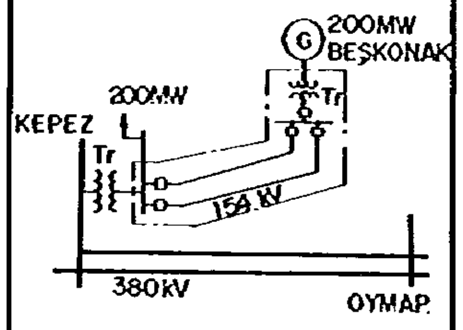
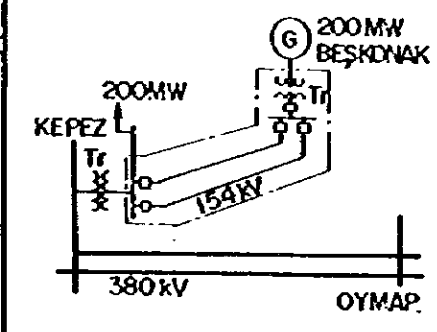
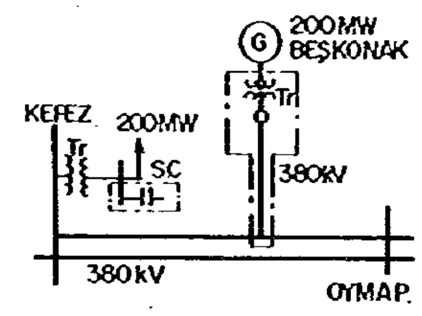
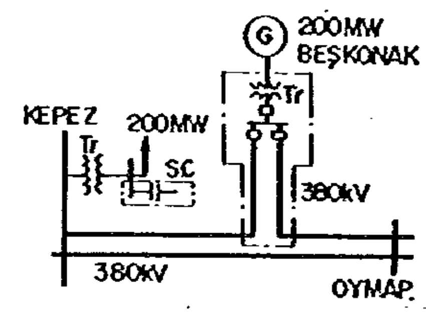
10.2.5 Transmission Pattern

The foregoing study items were rearranged and the five patterns below were made up.

Transmission Pattern	Voltage (kV)	Conductor	No. of Circuits	Length (km)	Lead-in Point
A	154	795 MCM	1	65	Kepez
B	154	477 MCM	2	65	Kepez
C	154	795 MCM	2	65	Kepez
D	380	954 MCM x 2B	1	25	380 kV line
E	380	954 MCM x 2B	2	25	380 kV line

The networks of the five patterns are shown in Table 10-1.

Table 10-1 . Economic Comparison of Transmission for Beskonak

Transmission Pattern	(A) 154 kV Line (1-circuit)	(B) 154 kV Line (2-circuit) 477 MCM x 1 B	(C) 154 kV Line (2-circuit) 795 MCM x 1 B	(D) 380 kV Line (1-circuit)	(E) 380 kV Line (2-circuit)
Power System Diagram Items Unit of cost: x 10^6TL					
Transmission Line					
Voltage	154 kV	154 kV	154 kV	380 kV	380 kV
Length to be Constructed	65 km	65 km	65 km	25 km	25 km
Number of Circuits	1 Circuit	2 Circuits	2 Circuits	1 Circuit	2 Circuits
Size of Conductor	ACSR 795 MCM	ACSR 477 MCM	ACSR 795 MCM	ACSR 954 MCM x 2 B	ACSR 954 MCM x 2 B
Construction Cost	344.5	416.0	546.0	290.0	580.0
Annual Cost	33.2	40.1	52.7	38.0	56.0
Station Equipments					
Step-up Trans. and Feeders	14.4/154 kV, 225 MVA	14.4/150 kV, 225 MVA, 1 Feeder	14.4/154 kV, 225 MVA, 1 Feeder	14.4/380 kV, 225 MVA	14.4/380 kV, 225 MVA, 1 Feeder
154 kV Outline Feeder	2 Feeders	4 Feeders	4 Feeders	—	—
380 kV Outline Feeder	—	—	—	1 Feeder	2 Feeders
Static Condenser	—	—	—	22 MVA	35 MVA
Construction Cost	405.5	459.7	459.7	482.8	601.9
Annual Cost	42.4	48.1	48.1	50.5	63.0
Transmission Losses					
Peek Power Loss	(8.8 MW) 136.5	(7.32 MW) 113.5	(4.4 MW) 68.2	(3.38 MW) 52.5	(6.18 MW) 95.9
Annual Energy Loss	(16,728 x 10^6 kwh) 134.7	(13,915 x 10^6 kwh) 112.0	(8,364 x 10^6 kwh) 67.3	(6,433 x 10^6 kwh) 51.8	(11,753 x 10^6 kwh) 94.6
Annual Cost	271.2	225.5	135.5	104.3	190.5
Total Construction Cost	750.0	875.7	1,005.7	772.8	1,181.9
Total Annual Cost	346.8	313.7	236.3	182.8	309.5

Note 1: [- - -]; Scope of construction cost for economic comparison

2: Annual factor
(a) 0.0965 for transmission line
(b) 0.1046 for station equipment

3: Cost for power loss and energy loss
(a) 15.5 x 10^3 TL/KW/year
(b) 8.05 TL/kwh

10.3 System Analysis

System analysis were performed on the two cases below.

- Calculations for selecting optimum power transmission method for Beskonak project

- Calculations of entire 380 kV system after start-up of Beskonak project

10.3.1 Conditions for System Calculations

The following conditions were set for carrying out various system calculations.

(1) Power Flow and Voltage Calculations

System voltage	:	95 - 110%
Generator operating voltage	:	100 ± 5%
Transformer tap ratio	:	1.00 - 1.05
Load power factor	:	0.95

(2) Stability Calculations

For stability calculations, the faulting conditions of a transmission line should be set in accordance with the reliability standards of the system, but here, a 3-phase ground fault (3∅G) was applied.

The transmission voltage classes and their fault sequences are as follows:

(a) 154 kV transmissionline; fault is to be cleared in 0.12 sec. after occurrence of 3∅G

(b) 380 kV transmission line; fault is to be cleared in 0.10 sec. after occurrence of 3∅G

(a) 154 kV Transmission Line

(Fault) (Clear)

C.B.on 3ϕG C.B.off

0 0.12 sec.

——>Time

(b) 380 kV Transmission Line

(Fault) (Clear)

C.B.on 3ϕG C.B.off

0 0.10 sec.

——>Time

(3) Short-circuit Current Calculation

Generator reactance	:	Transient reactance (Xd')
Transformer tap ratio	:	1.00
Operating generators	:	All generators (22,780 MVA) in Table 10-2 are paralleled in the system

(4) Demand and Supply, and Power Flow Patterns of Trunk Systems

The conditions for demand and supply were set as indicated below based on the power development program and load sorecast up to around 1993 when the Beskonak power station is scheduled to be started.

Power source output	:	20,378 MW (rating)
Demand	:	16,500 MW

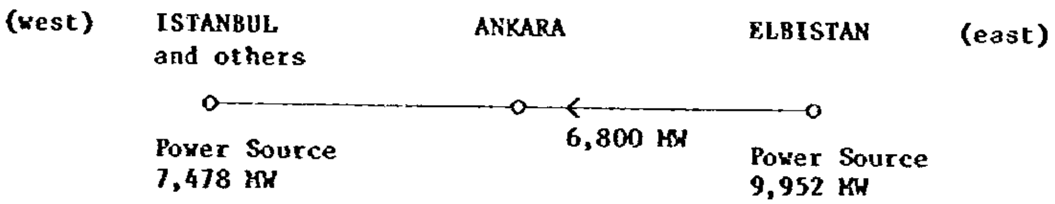
The outputs of power stations are given in Table 10-2 and the demands of substations in Table 10-3.

For power flow conditions, two patterns were set up in accordance with the power flows from the power source area in the eastern part of Turkey toward the Ankara area.

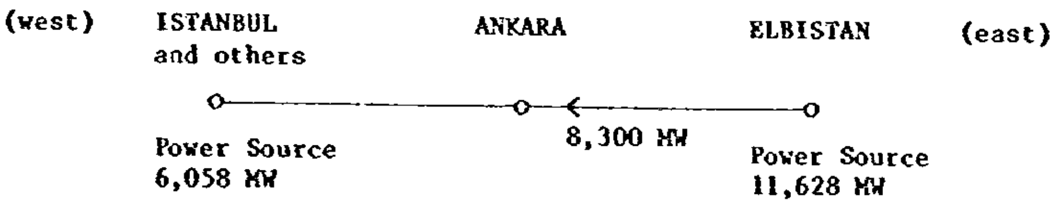
These two patterns were set up in order to obtain a yardstic on the extent to which the electric power of eastern Tuekey can be transmitted through the transmission line facility according to the expansion plan for the 380 kV system. For the power flow patterns, the ratios of power source outputs of the west and east with ankara at the middle were varied and the power flows from the east to the ankara area were made the two kinds below. Pattern-1 is of small power flow to the west while Pattern-2 is of large power flow.

	Power Flow Pattern-1	Power Flow Pattern-2
Power source output (west of Ankara)	7,478 MW	6,058 MW
Power source output (east of Ankara)	9,952	11,728 MW
(Total)	(17,430)	(17,686)
Power flow toward Ankara	6,800	8,300

(a) Power Flow Pattern-1



(b) Power Flow Pattern-2



(5) Network

The network was based on the 380 kV transmission lines for around 1993 planned by TEK. The network of the 380 kV system and its details are shown in Fig. 10-3.

Table 10-2 Ratings of Generators and Transformers used for Calculation

Sym.of Gen.	Power Stations	Output in power flow study (MW)	Rating of Generators		Rating of Transformers	
			Output (MW)	Capacity (MVA)	Capacity (MVA)	Imp.Voltage(%) 100 MVA base
	(Pattern-1)					
A	BABAESKI	540	600	670	670	2.39
B	SOMA	620	660	740	740	2.16
C	ALIAGA	240	260	290	290	5.52
D	SEKIK	540	570	640	640	2.50
E	YATAGAN	590	630	700	700	2.29
F	SEYITÖMER	570	600	670	670	2.39
G	GÖKCEKAYA	278	278	310	310	5.16
H	BAYPAZARI	570	600	670	670	2.39
I	OYMAPINAR	540	540	600	600	2.67
J	AKKUYU	1,420	1,600	1,800	1,800	0.89
K	KAYRAKTEPE	420	420	470	470	3.40
L	BURSA	380	400	450	450	3.55
M	BESKONAK	200	200	230	230	5.22
N	AMBARLI	570	600	670	670	2.38
O	BOYABAT	500	510	570	570	2.81
P	ALTINKAYA	500	700	780	780	2.05
Q	H.UGURLU	400	500	580	580	2.76
R	KANGAL	280	320	360	360	4.44

Sym.of Gen.	Power Stations	Output in power flow study (MW)	Rating of Generators		Rating of Transformers	
			Output (MW)	Capacity (MVA)	Capacity (MVA)	Imp.Voltage(%) 100 MVA base
S	KEBAN	1,400	1,530	1,700	1,700	0.94
T	KARAKAYA	1,400	1,800	2,000	2,000	0.80
U	ELBISTAN	4,402	5,560	6,200	6,200	0.26
V	ATATÜRK	300	300	340	340	4.71
W	ILISU	1,000	1,200	1,340	1,340	1.19
	TOTAL	17,660	20,378	22,780	22,780	-
	(Pattern 2)... Difference of "Pattern-2" from "Pattern-1"					
J	AKKUYU	570	1,600	1,800	1,800	0.89
N	AMBARLI	0	-	-	-	-
O	BOYABAT	510	510	570	570	2.81
P	ALTINKAYA	700	500	700	780	2.05
Q	H.UGURLU	500	500	580	580	2.76
S	KEBAN	1,530	1,530	1,700	1,700	0.94
T	KARAKAYA	1,800	1,800	2,000	2,000	0.80
U	ELBISTAN	4,738	5,560	6,200	6,200	0.26
V	ATATÜRK	600	600	670	670	2.38
W	ILISU	1,200	1,200	1,340	1,340	1.19

Table 10-3 Demands of Sub-stations Used for Calculation

(in 1993)

Sub-stations	Demands (MW)	Sub-stations	Demands (MW)
BABAESKI	558	ALIAGA	1,709
HALKARA	808	IZMIR	1,264
OSMANCA	259	YATAGAN	324
EREGLI	170	KEPEZ	500
G.O.P.	939	OYMAPINAR	284
ÜMRANIYE	1,109	SEYDISEHIR	461
GEBZE	1,068	CARSAMBA	315
ADAPAZARI	1,060	KEBAN	110
ANKARA	874	FERROKROM	182
GÖLBASI	1,109	KAYSERI	394
BURSA	268	ELBISTAN	213
SEYITÖMAR	292	OSMANIYE	710
ESKISEHIR	153	KARAKAYA	630
SOMA	737	Total	16,500

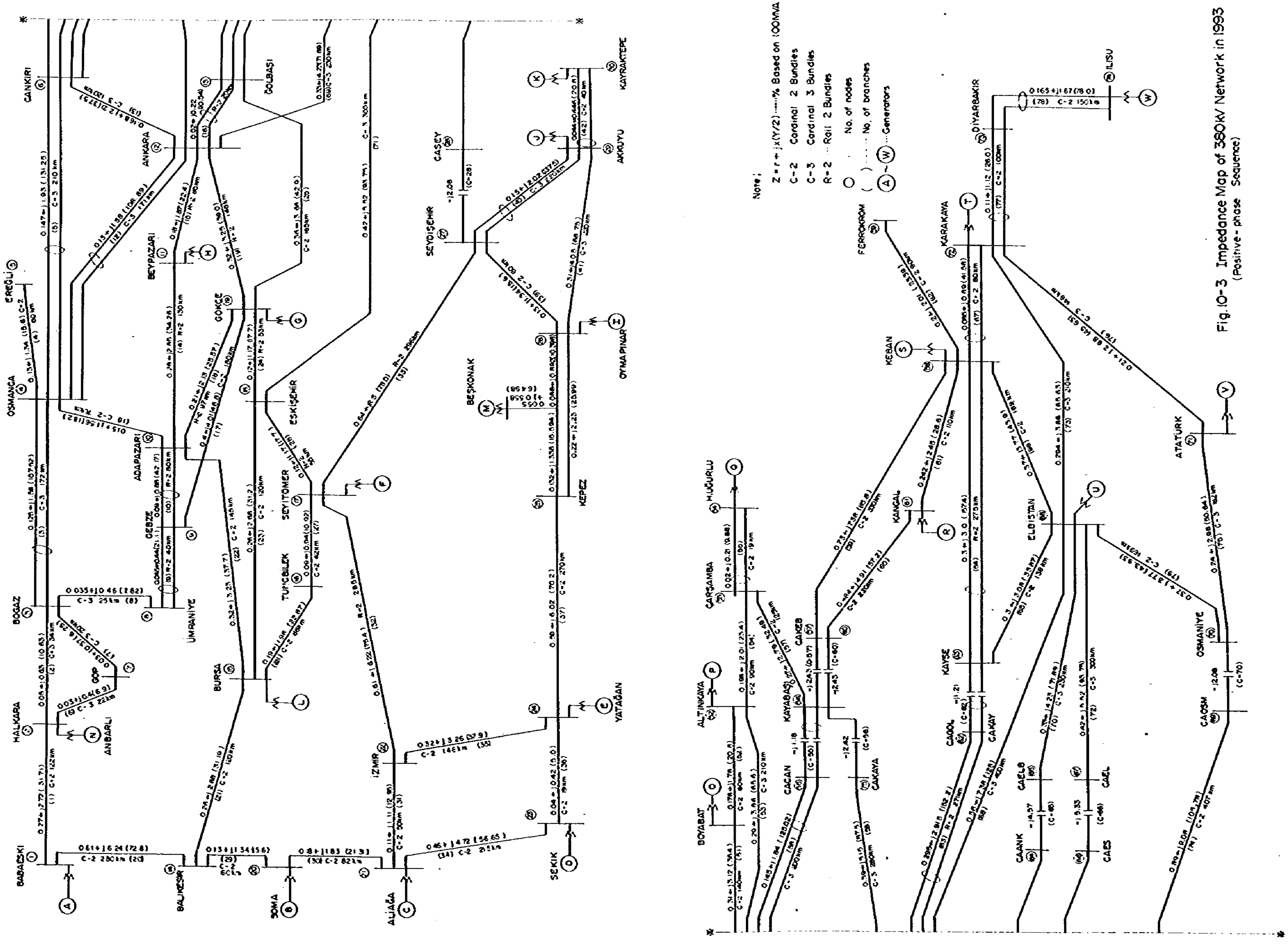


Fig.10-3 Impedance Map of 380kV Network in 1993
(Positive-phase Sequence)

10.3.2 Transmission Scheme of Beskonak Project

System calculations were performed on the various transmission patterns given in 10.2.5 in order to select the optimum transmission method for the Project. The power flow pattern of the 380 kV system to be the condition for calculations was Pattern-1 described in 10.3.1. The reasons for this are that there are small effects by difference in power flow patterns on the 380 kV system in the vicinity of Antalya, and that Pattern-1 is considered to be similar to the actual system operation.

(1) Power Flow and Voltage Calculations

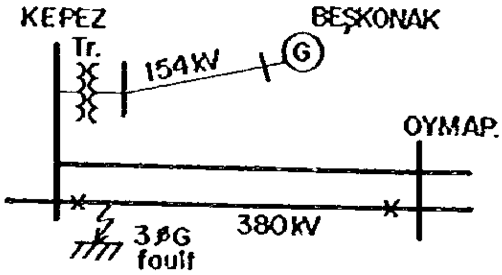
The results of power flow and voltage calculations for the individual transmission patterns are shown in Fig. 10-4. In 380 kV transmission method (Pattern-D and -E), since power would be supplied through the transformers (380/154 kV) of Kepez substation, the voltage of the 154 kV bus of the substation will be lowered to 93-94% due to reactive power consumption by the transformers. In order to compensate for this voltage drop to the same voltage level as the case of 154 kV transmission method (Pattern-A to -C), static condensers of 25 to 35 MVA will be necessary.

In the aspect of voltage maintenance at Kepez substation, 154 kV transmission will be advantageous compared with 380 kV transmission. In the aspect of transmission loss, 380 kV transmission is advantageous as losses are smaller compared with 154 kV transmission.

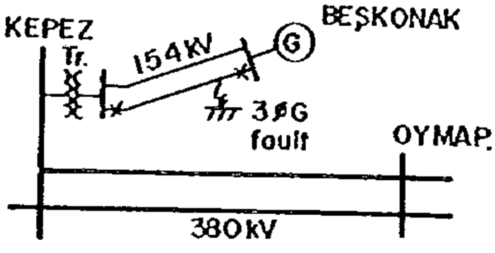
(2) Stability Calculations

The points of 3-phase ground faults applied in calculations of transient stabilities of the each transmission pattern were set in accordance with the number of circuits as indicated in the diagrams below.

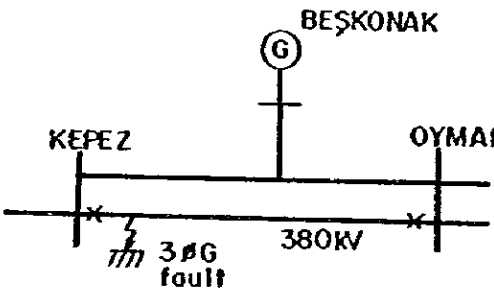
(a) Pattern - A



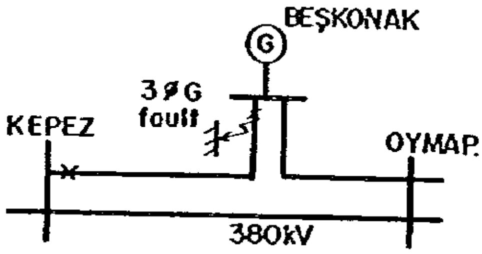
(b) Pattern-B,C



(c) Pattern - D



(d) Pattern-E

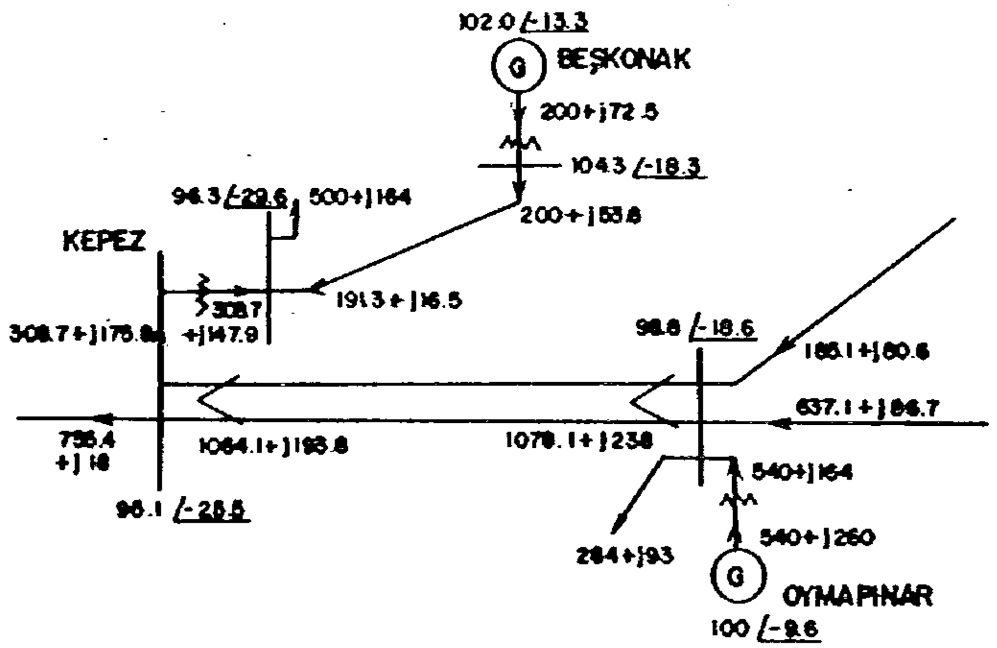


The swing curves according to transient stability calculations are shown by the pattern in Figs. 10-5 to 10-8, while the voltage and power output perturbations of the Beskonak generators are shown in Figs. 10-9 to 10-11 (Patterns-B to -E). Among these transmission patterns the swing of the Beskonak generators is greatest in case of Pattern-C (Figs. 10-6 and 10-9), followed by Pattern-E (Figs. 10-8 and 10-11). These swings are of small amplitudes and since they are staying in synchronism in the each transmission pattern, there is no problem from the standpoint of stability.

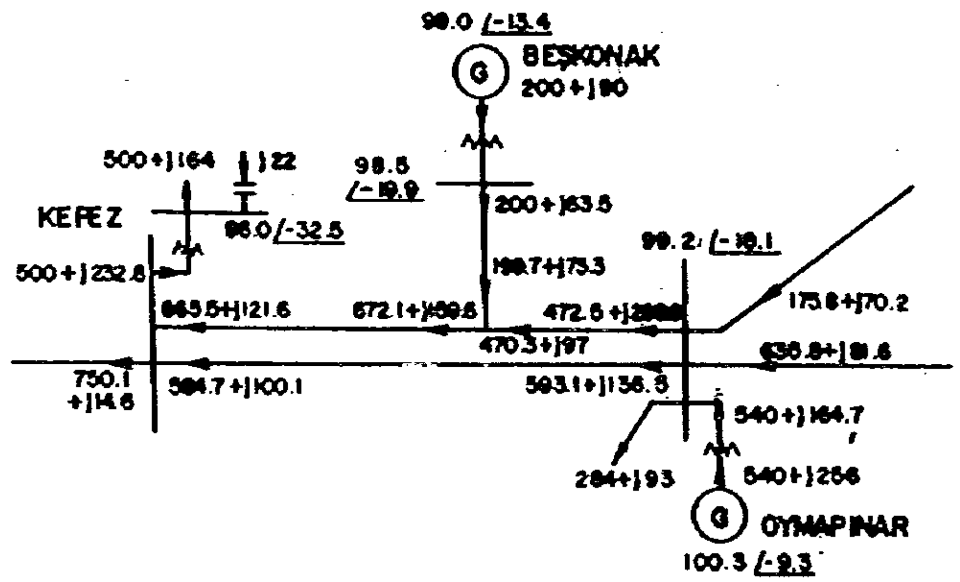
Fig. 10-4 Transmission Patterns of Beşkonak Power Station and their Power Flow Diagrams

Note ;
P+jQ ; Active Power (MW) and Reactive Power (MVar)
V /θ ; Bus Voltage (%) and Leading Angle (deg)
Base Genrator is ELBISTAN.

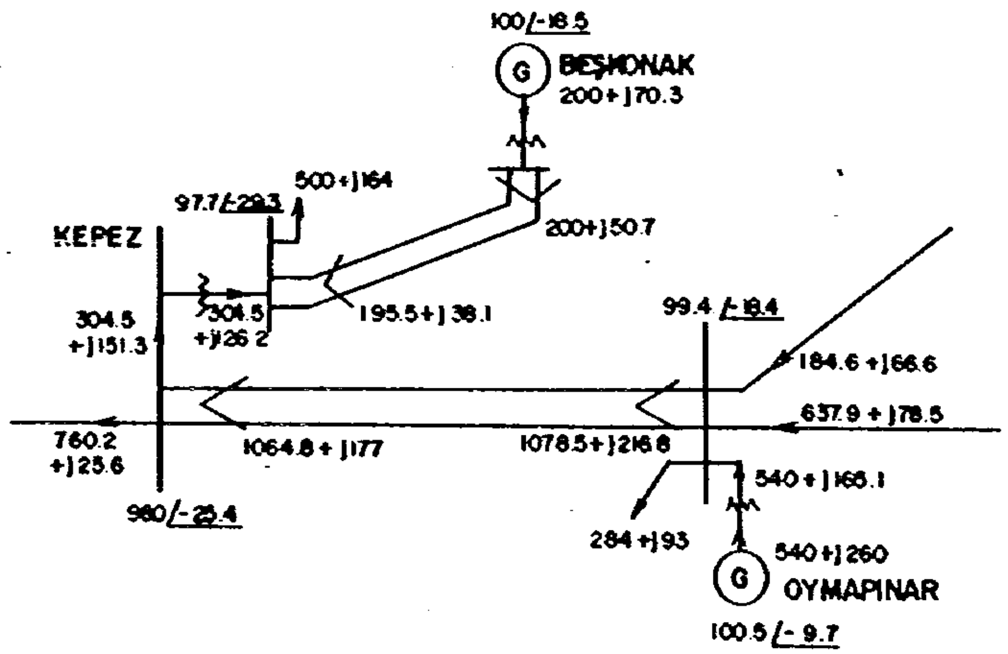
Pattern-A ; 154kV Transm. Line (1-Circuit) 795MCM x 1B



Pattern-D ; 380kV Transm. Line (1-Circuit) 954 MCM x 2B



Pattern - B ; 154kV Transm. Line (2-Circuits) 477MCM x 1B
Pattern - C ; 154kV Transm. Line (2-Circuits) 795MCM x 1B



Pattern-E ; 380kV Transm. Line (2-Circuits) 954MCM x 2B

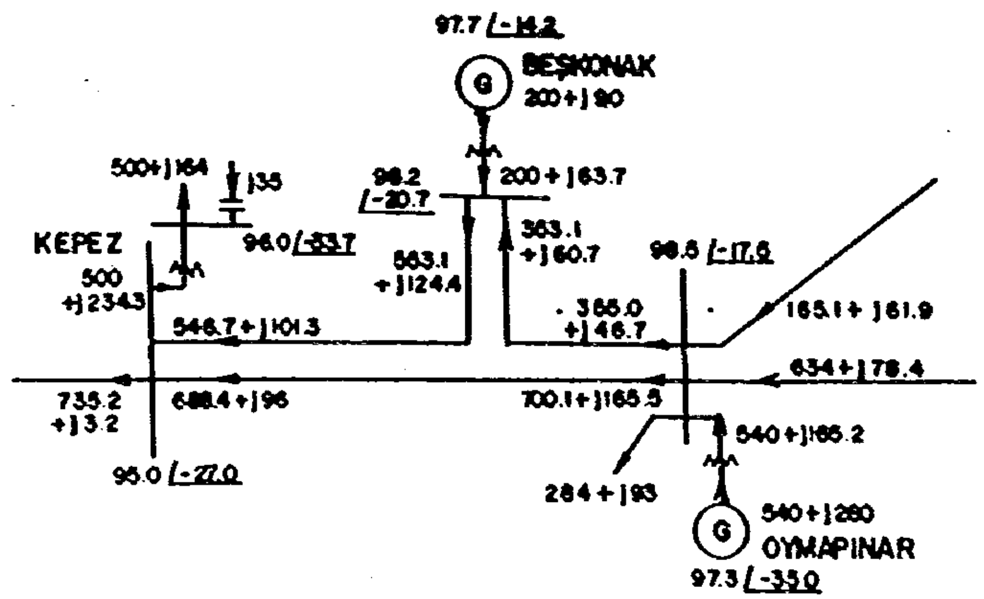
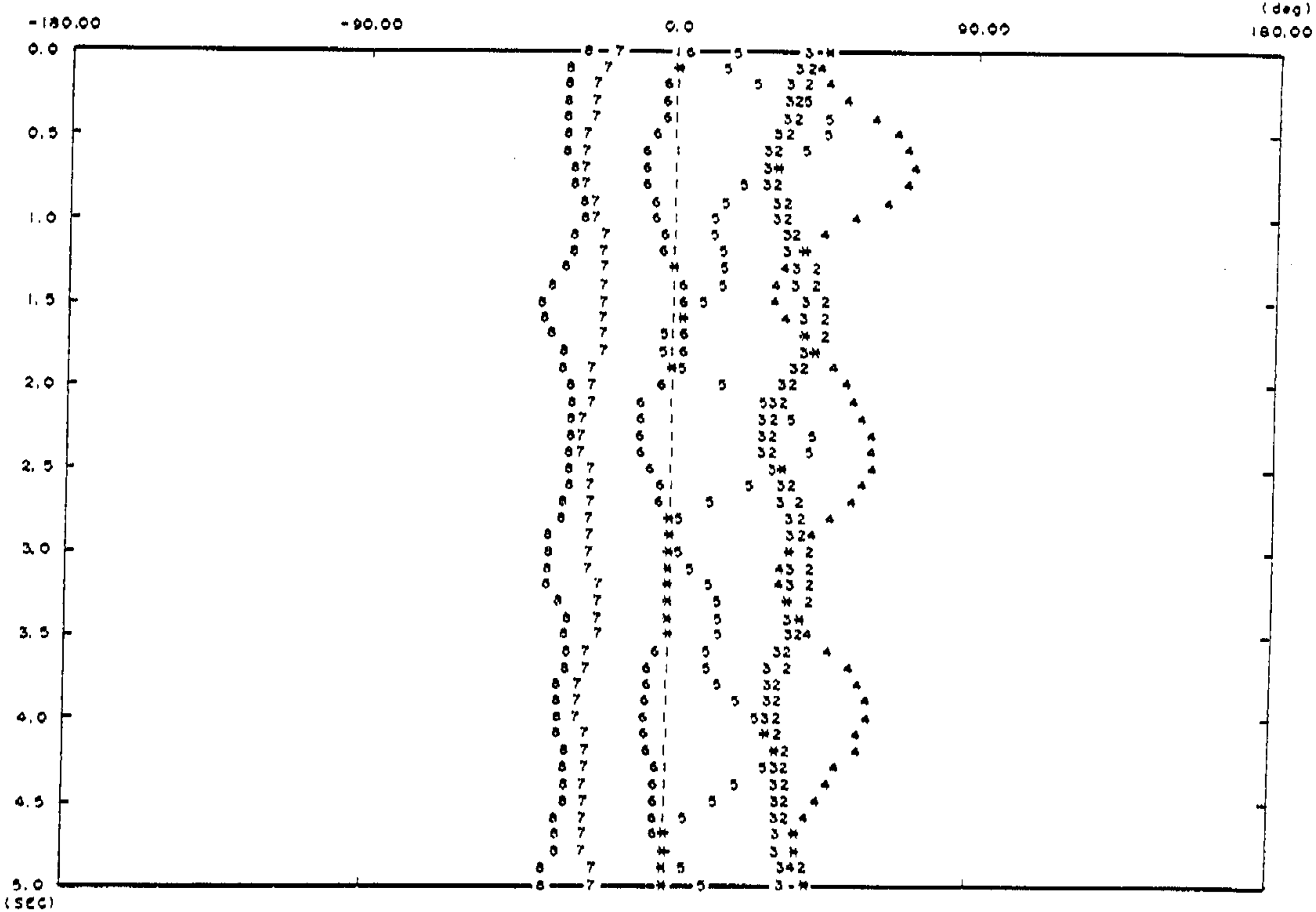


Fig.10-5 Transient-stability Swing Curve following 3øG-fault (Pattern-A)

BASE GENERATOR=F SEYİTÖMR



SYMBOL	GND	SYMBOL	GND	SYMBOL	GND
1=F	SEYİTÖMR	2=W	ILISU	3=U	ELBİSTAN
4=J	AKKUYU	5=M	BEŞKONAK	6=O	H. UĞURLU
7=A	BABAESKI	8=C	ALİAĞA		

Fig. 10-6 Transient-stability Swing Curve following 3ØG-fault (Pattern-B,C)

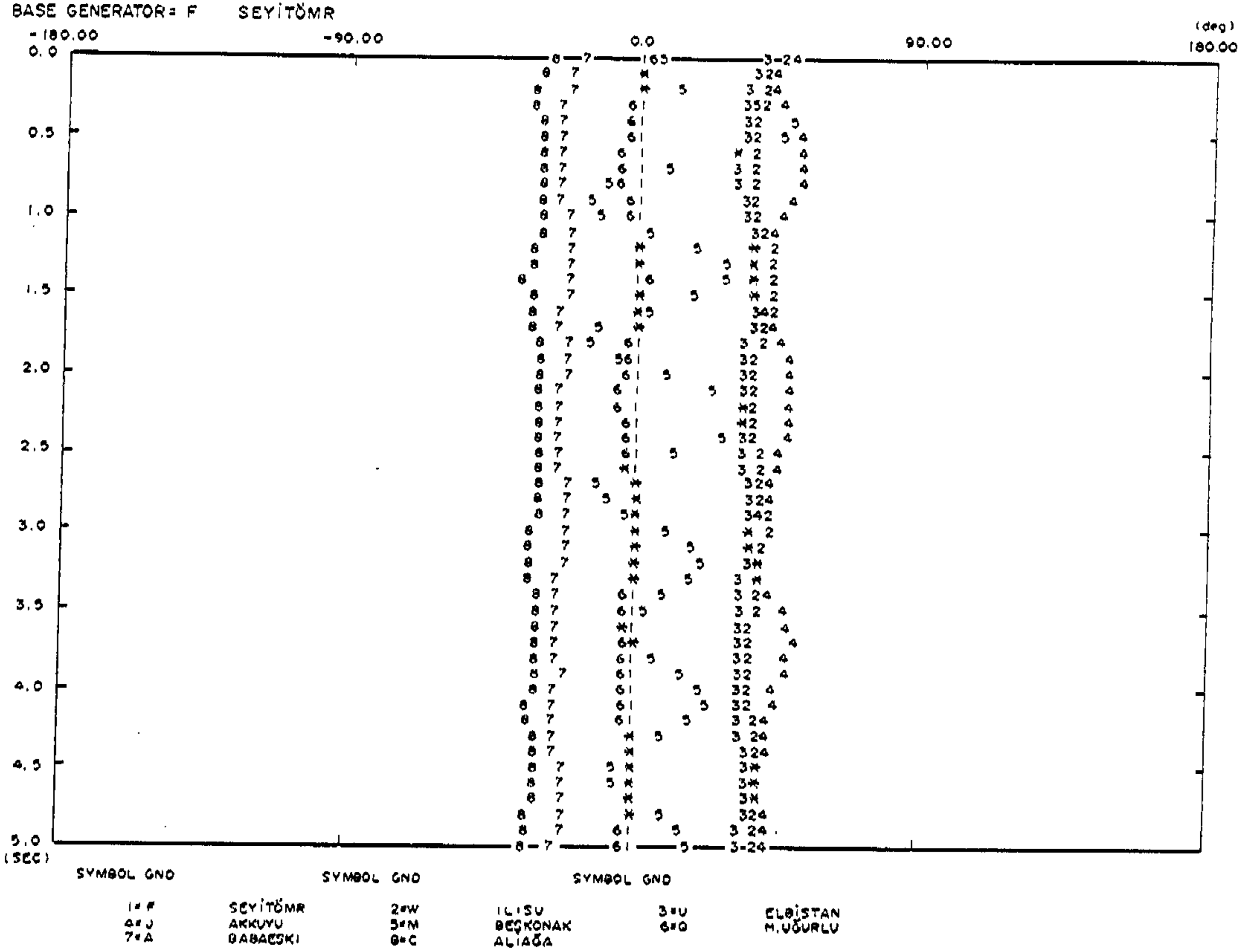


Fig. 10-7 Transient-stability Swing Curve following 3ØG-fault (Pattern-D)

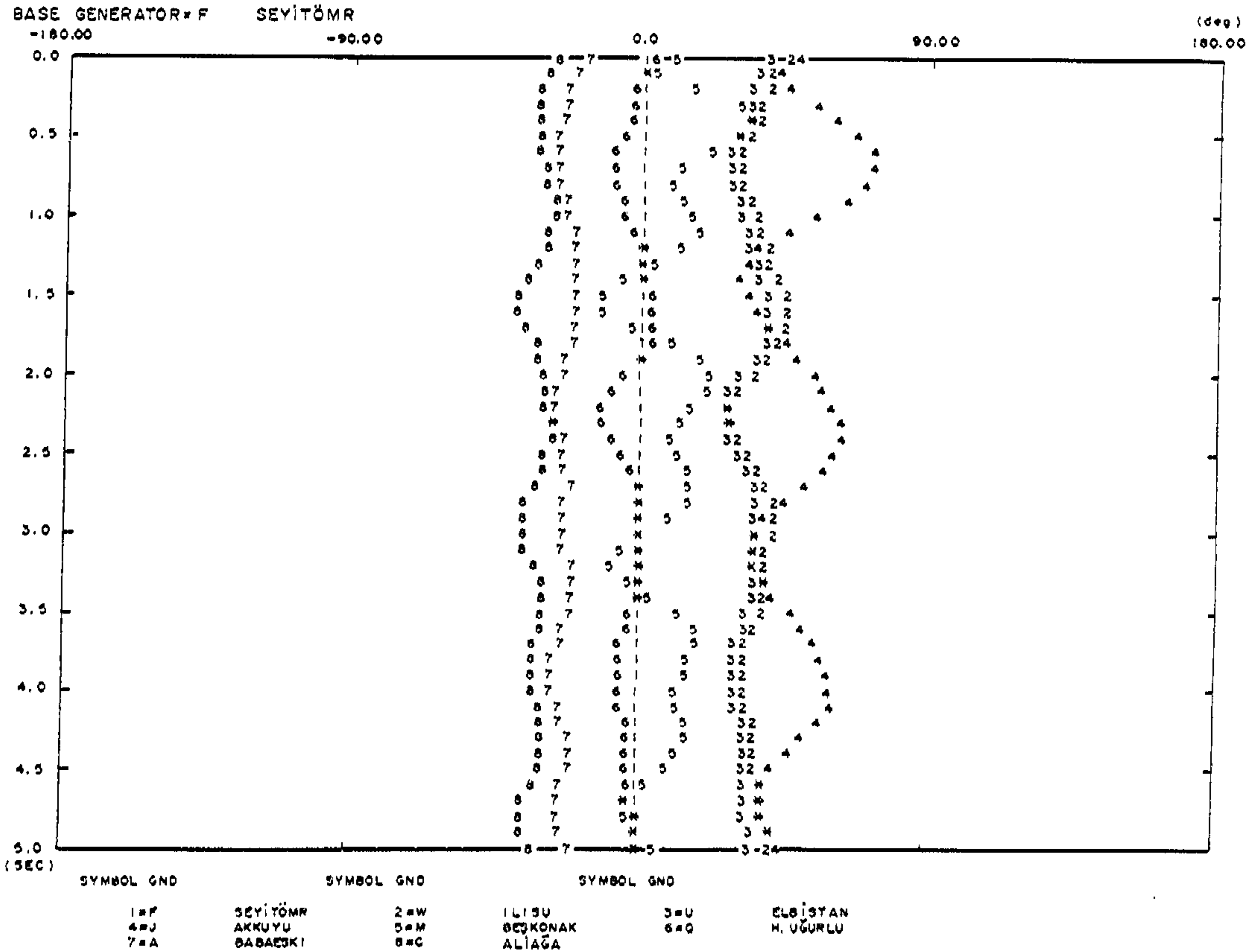
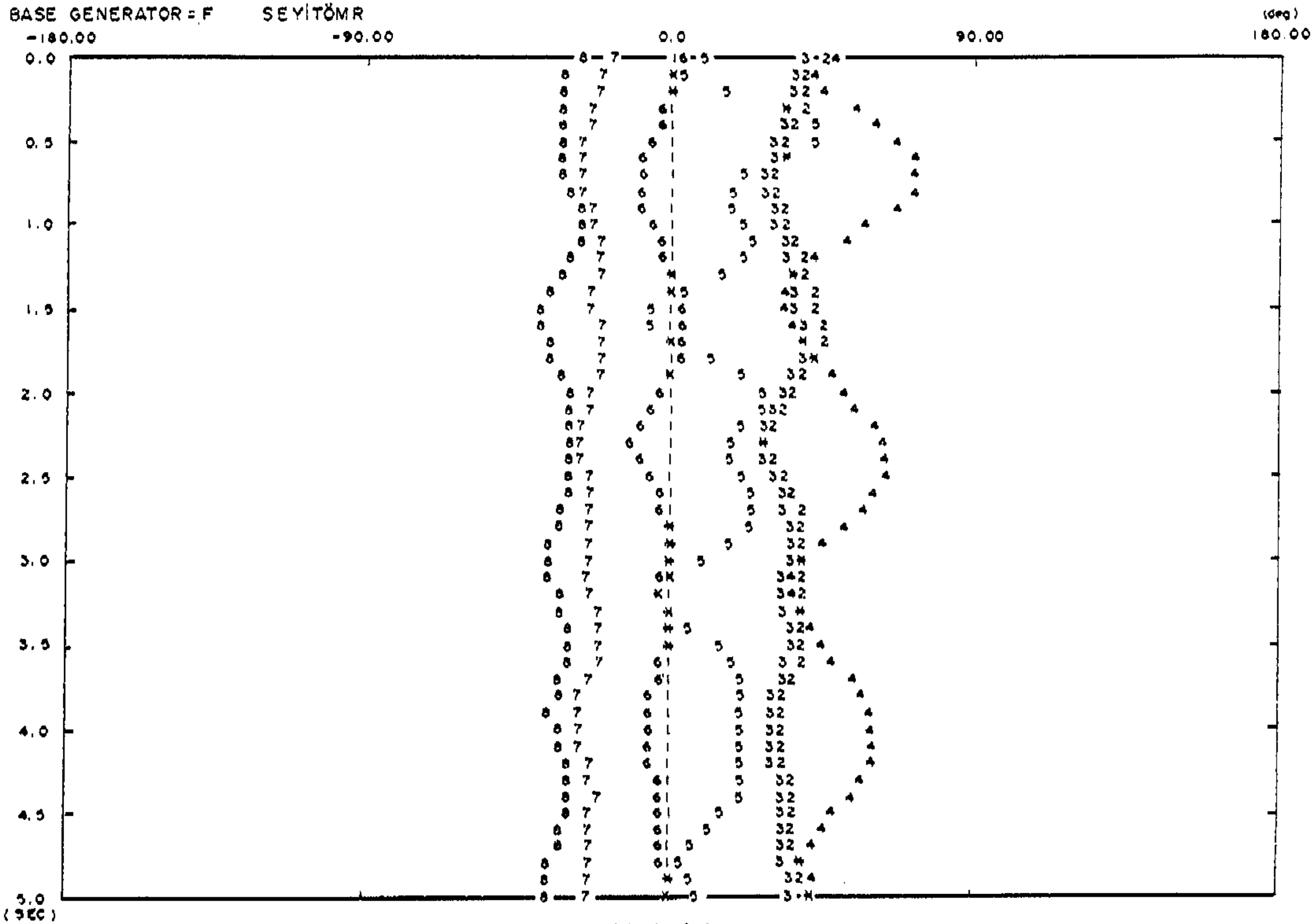


Fig. 10-8 Transient-stability Swing Curve following 3øG-fault (Pattern-E)



SYMBOL GND		SYMBOL GND		SYMBOL GND	
1=F	SEYİTÖMR	2=W	ILISU	3=U	ELBİSTAN
4=J	AKKUYU	5=M	BEŞKONAK	6=O	H.UĞURLU
7=A	BABAESKI	8=C	ALİAĞA		

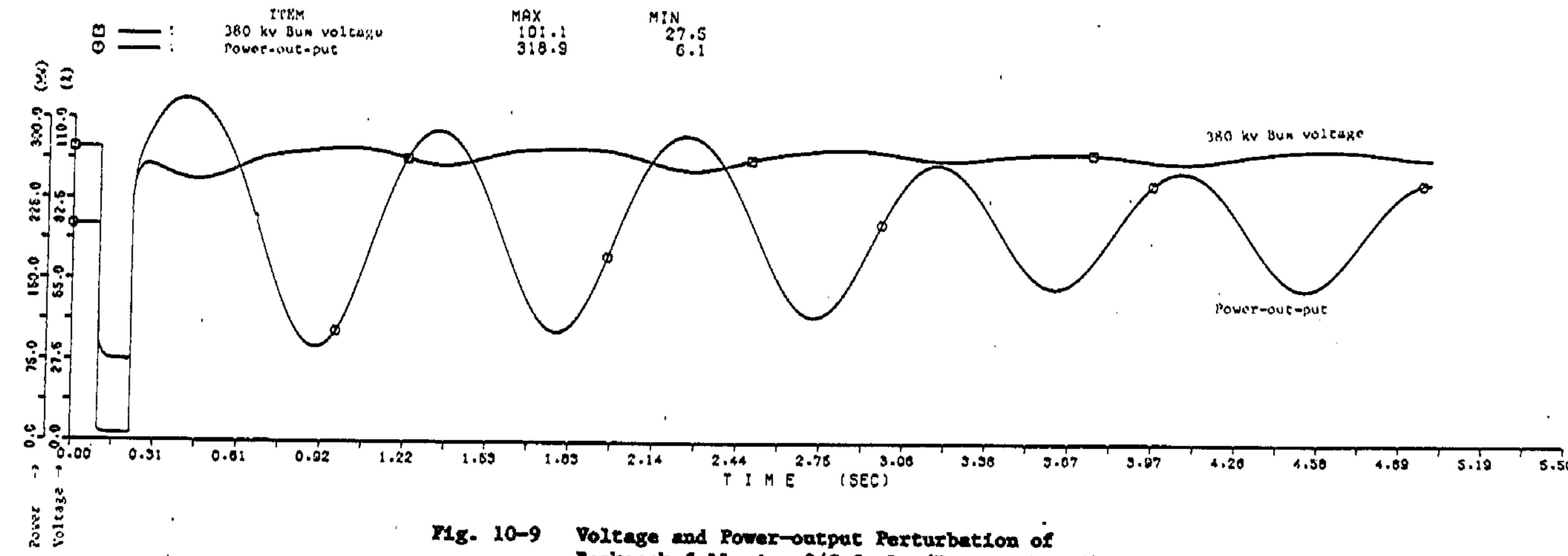


Fig. 10-9 Voltage and Power-output Perturbation of Beşkonak following 3ϕG-fault (Pattern-B, -C)

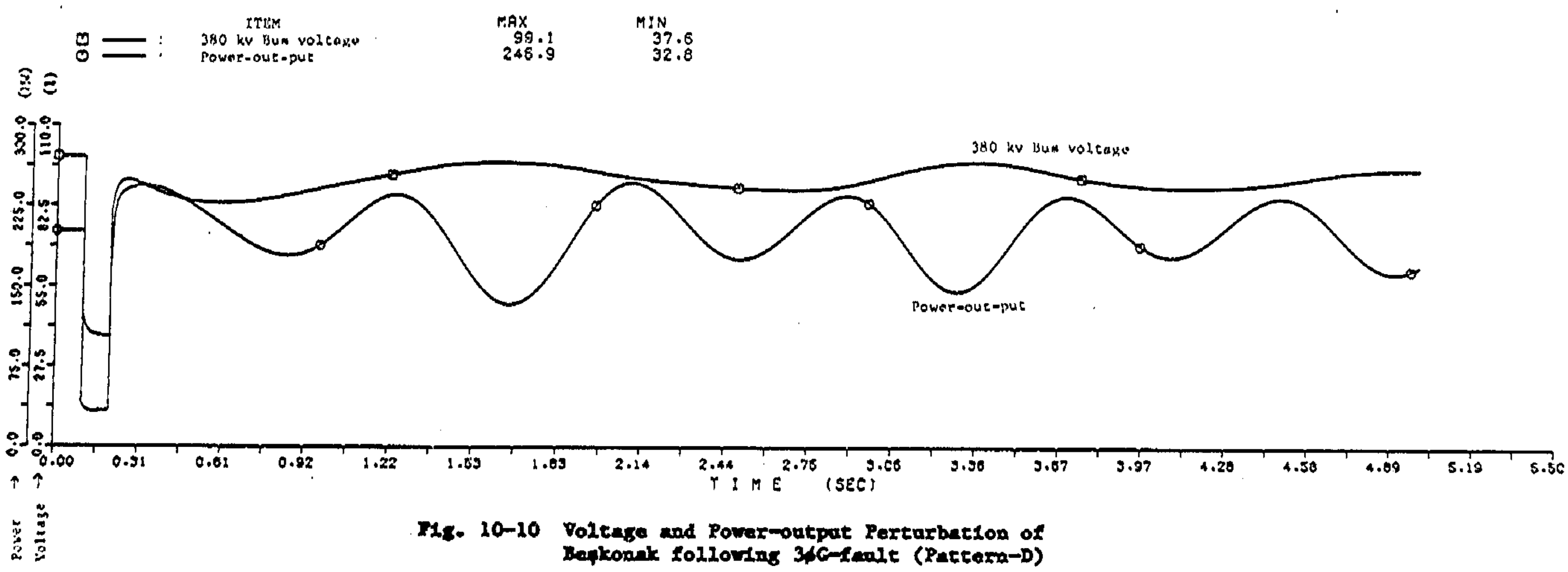


Fig. 10-10 Voltage and Power-output Perturbation of Beşkonak following 3ϕG-fault (Pattern-D)

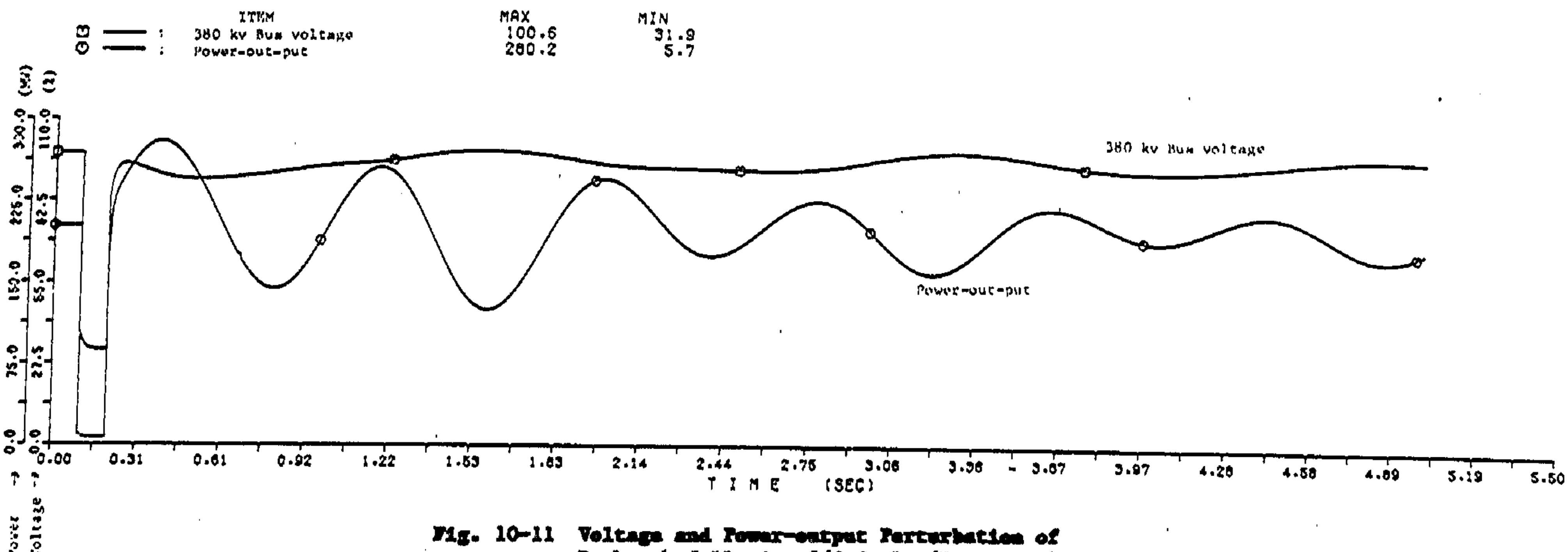


Fig. 10-11 Voltage and Power-output Perturbation of Beşkonak following 3ϕG-fault (Pattern-E)

(3) Short-circuit Current Calculations

Short-circuit current calculations were carried out for transmission Pattern-D only. In general, the short-circuit current is determined by the characteristics of transmission lines and generators connected to the network at the time of the short-circuit. Since there is a small differences in short-circuit currents according to variances in transmission patterns of the Project, Pattern-D was selected as representative of the five patterns.

The results of calculations are shown in Fig. 10-12. According to this figure,

380 kV bus of Kepez substation	:	6.8 kA (= 4480 MVA)
380 kV bus of Beskonak power station	:	6.1 kA (= 4020 MVA)

The power station closest to Beskonak power station is Oymapinar power station, but the supply of short-circuit current from the latter is small, and the short-circuit current of Beskonak is of low level compared with other power stations. Therefore, the interrupting capacities in selection of circuit breakers will not be of any problem in particular.

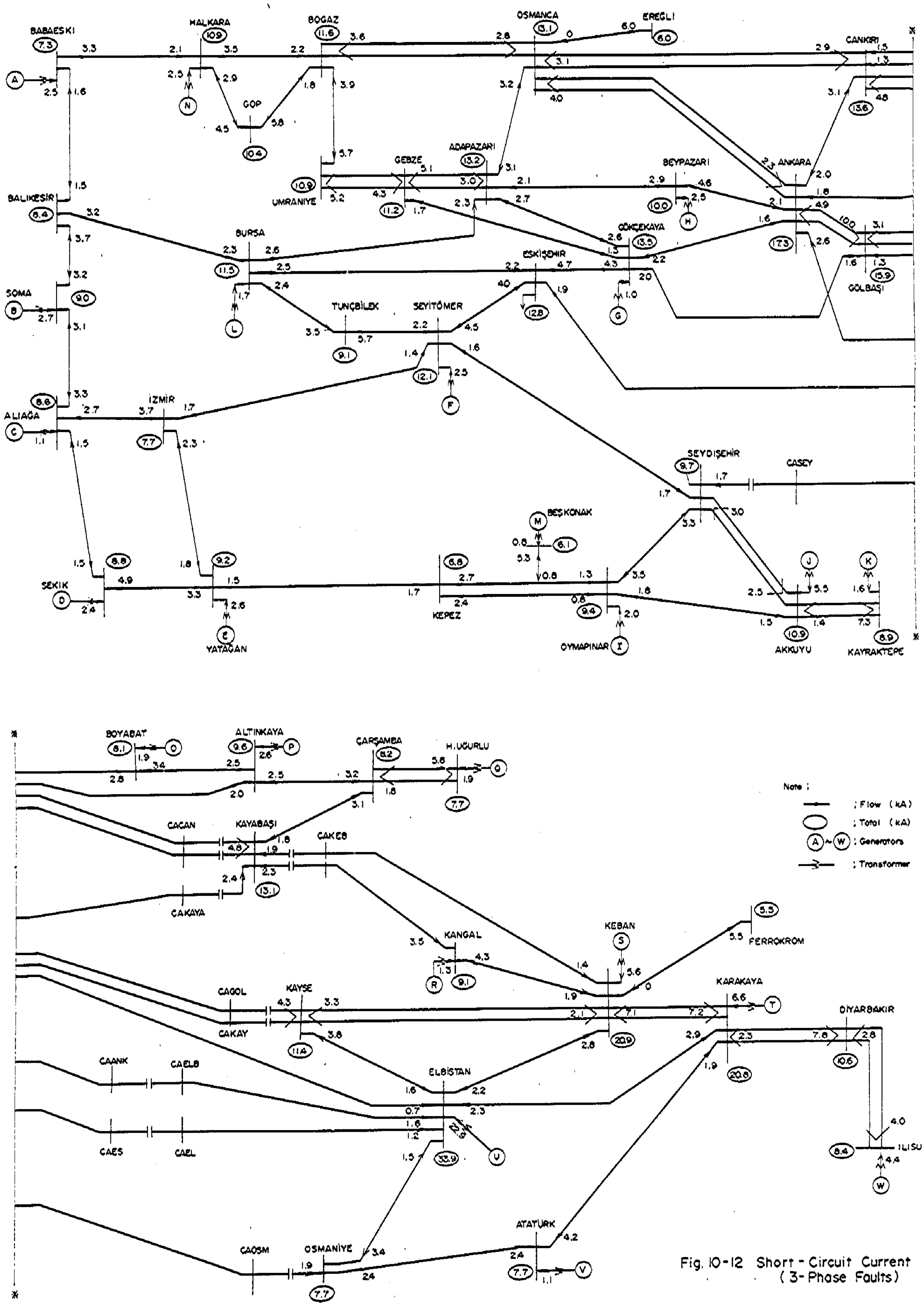


Fig. 10-12 Short - Circuit Current (3-Phase Faults)

10.3.3 Analysis of 380 kV Transmission System

The 380 kV transmission system consists of long transmission lines which are more than 1,000 km connecting the main power source area of the east with the load centers in the west. Here, system calculations are performed for the case of transmitting from the large power sources of the east, and the transient stabilities of generators including Beskonak were checked. The power flow conditions of the 380 kV transmission system were based on Pattern-1 and Pattern-2 described in 10.3.1(4). The calculation results by power flow pattern are shown in Table 10-4.

(1) Voltage Calculations

Since the transmission line length is extremely long, consumption of reactive power of the system is increased and voltage drop in the load centers (Western Turkey) is prominent. To compensate for this voltage drop, it is necessary to install static condensers equal to or greater than reactive power demand at substations in the vicinities of Istanbul and Izmir. The power demands and capacities of condensers for voltage compensation at 380 kV substations in Ankara city and to the west are shown in Table 10-4.

(2) Power Flows and Power Losses of Transmission Lines

The results of power flow calculations are shown by pattern in Fig. 10-13 (Pattern-1) and Fig. 10-14 (Pattern-2). On the whole, there are many sections of heavy power flows on the transmission lines, and as a consequence, transmission losses are large. The transmission lines where power flows exceed 1,000 MW on single-circuit sections and 1,500 MW on double-circuit sections are as indicated below.

Power Flow Pattern-1

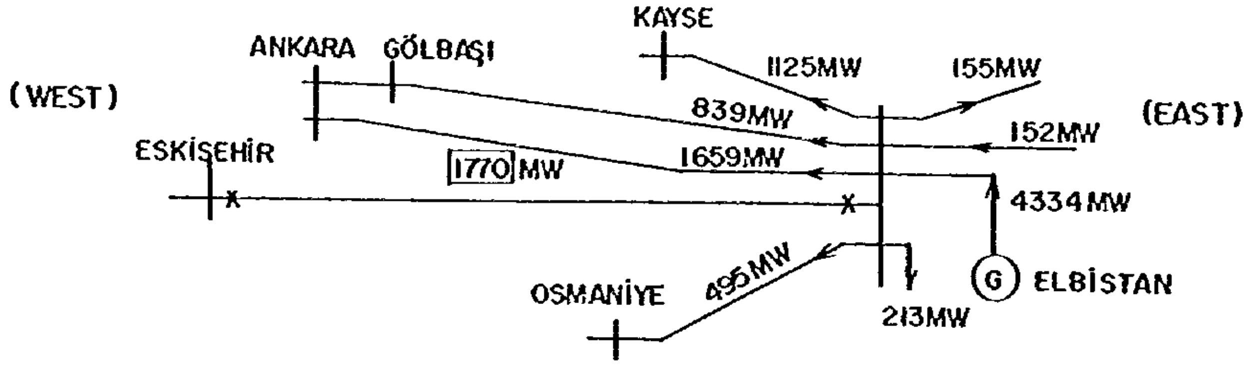
Eskisehir to Elbistan	:	1,316 MW/circuit
Ankara to Elbistan	:	1,098 MW/circuit
Bogz to Osmanca	:	1,506 MW/2-circuit
Osmanca to Cankiri	:	1,503 MW/2-circuit

Power Flow Pattern-2

Eskisehir to Elbistan	:	1,462 MW/circuit
Ankara to Elbistan	:	1,278 MW/circuit
Bogz to Osmanca	:	1,845 MW/2-circuit
Osmanca to Cankiri	:	1,764 MW/2-circuit
Gölbasi to Kayseri	:	1,552 MW/2-circuit

Power flow calculations were performed for cases of opening one circuit of heavy power flow transmission lines in order to judge allowances for power-carrying thermal capacities in the network. As a result, the transmission line influenced the most by opening one circuit is the line going west from Elbistan where heavy power flow transmission lines are concentrated. The power flow diagrams in case of opening one circuit are shown in the following.

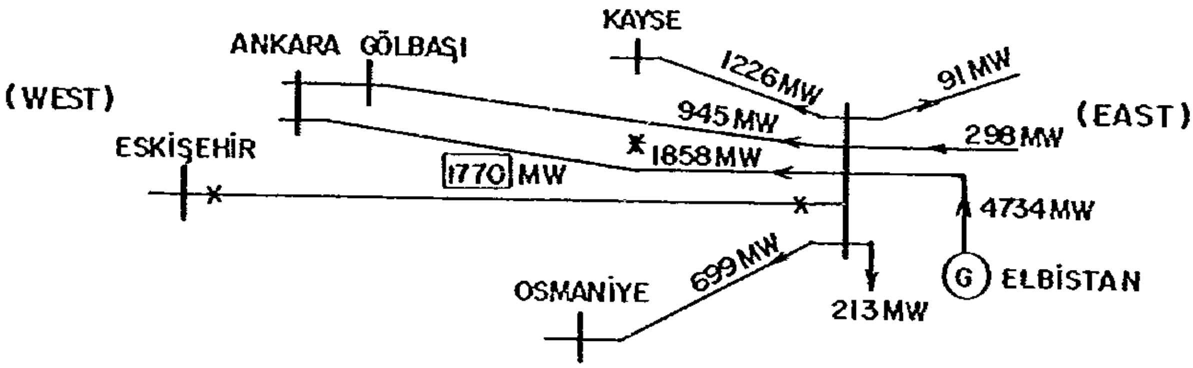
(a) Power Flow Pattern - 1



Note : $\boxed{1770}$: Thermal capacity of power carrying.

※ : Over the thermal capacity.

(b) Power Flow Pattern - 2



With power flow Pattern-1, there is ro transmission line which exceeding its thermal capacity at the time of opening one circuit.

With power flow Pattern-2, when the single-circuit section between Eskisehir and Elbistan is opened, the power flow on the single-circuit section between Ankara and Elbistan is increased to 1,860 MW, and that transmission line will exceed its thermal capacity of 1,760 MW by 100 MW.

The thermal capacities of 380 kV transmission lines computed based on TEK data are the following values:

Conductors	Thermal Capacity
954 MCM x 2B	1,170 MW/circuit
954 MCM x 3B	1,760 MW/circuit

The current-carrying capacity of transmission lines according to the working conditions set by TEK are given in Fig. 10-19.

Power losses in the transmission lines, as shown in Table 10-4, are increased with output of the eastern power sources, and its loss ratio of watt loss to peak load is increased from 5.3% in Pattern-1 to 6.7% in Pattern-2. This is a very high rate for the loss ratio of a primary network.

Table 10-4 Comparison with Power Flow Patterns of 380 kV Net Work

Items \ Power Flow Pattern	Pattern-1	Pattern-2
Demands in Turkey (Peak time in 1993)		
Active Power (MW)	16,500	
Reactive Power (MVar)	5,420	
Active Power at sending end	17,430	17,686
(breakdown)		
East of ANKARA	7,478	6,058
West of ANKARA	9,952	11,628
Transmission losses		
Active Power (MW) (loss ratio: %)	930 (5.3)	1,186 (6.7)
Reactive Power (MVar)	3,620	5,970
Demands in West of ANKARA		
Active Power (MW)	13,946	
Reactive Power (MVar)	4,580	
Static Condensers in West of ANKARA (MVA)	3,925	5,488
Transient-stability following 3ϕG-fault	Stable	Un-stable (ILISU is step out)

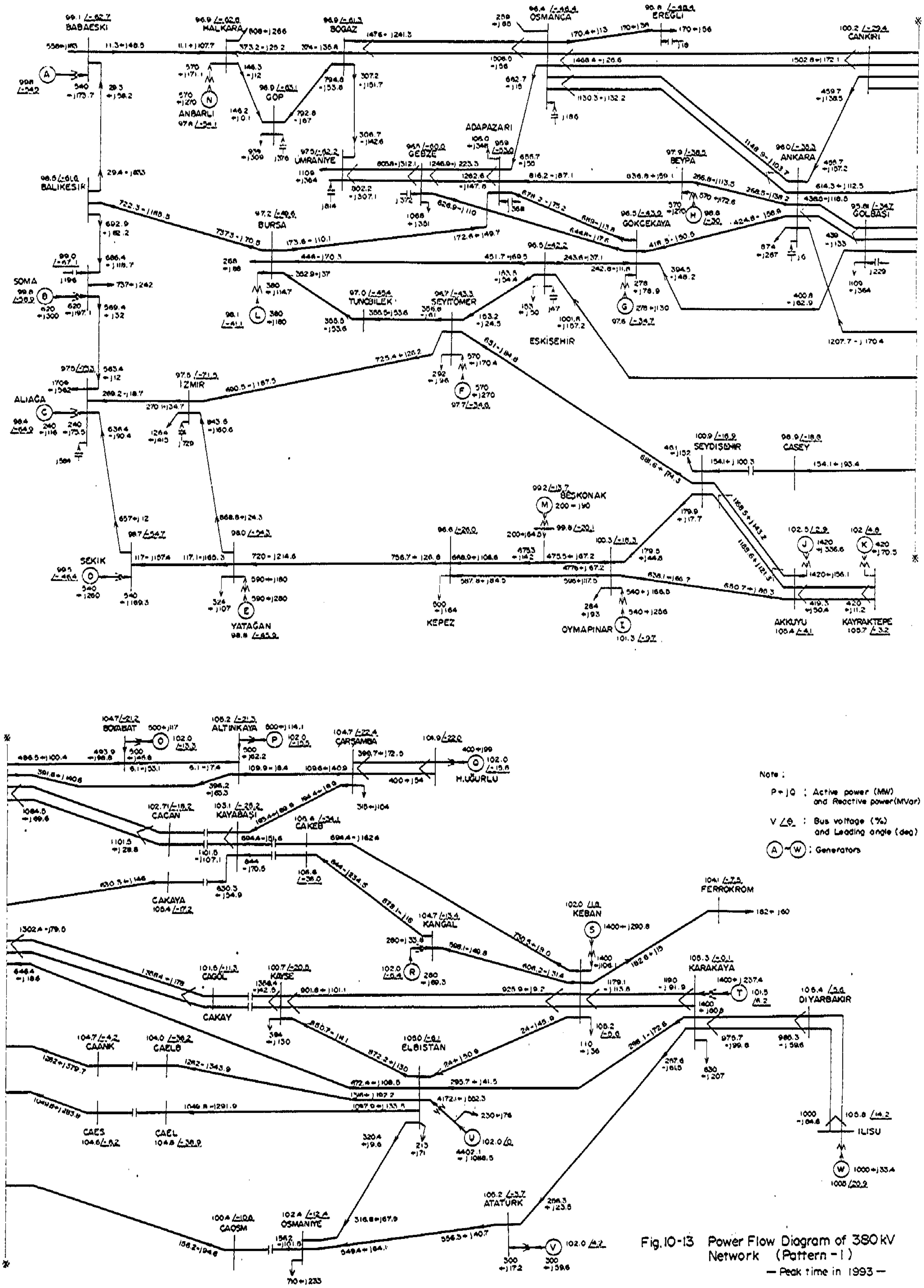


Fig. 10-13 Power Flow Diagram of 380 kV Network (Pattern - I)
— Peak time in 1993 —

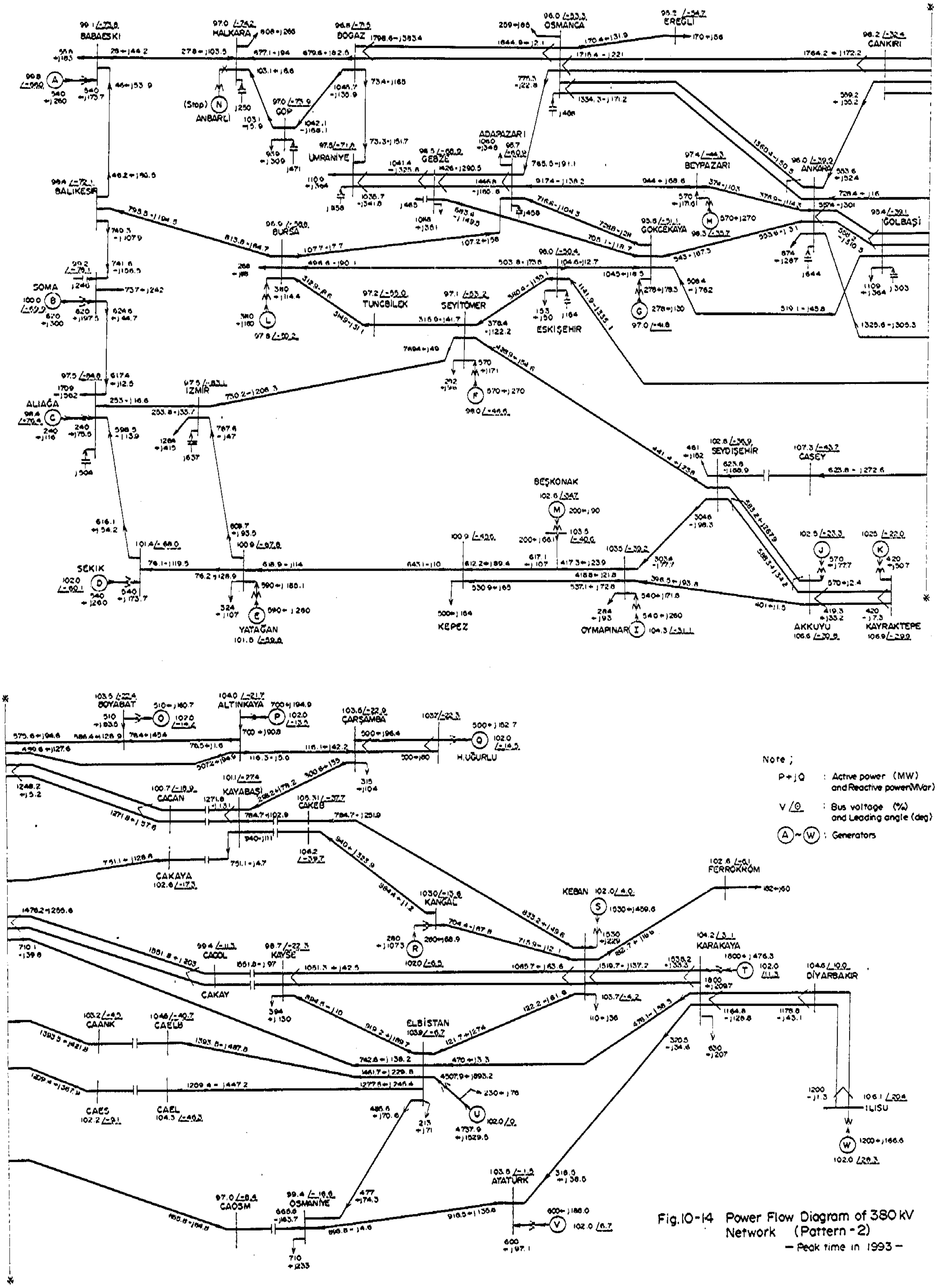
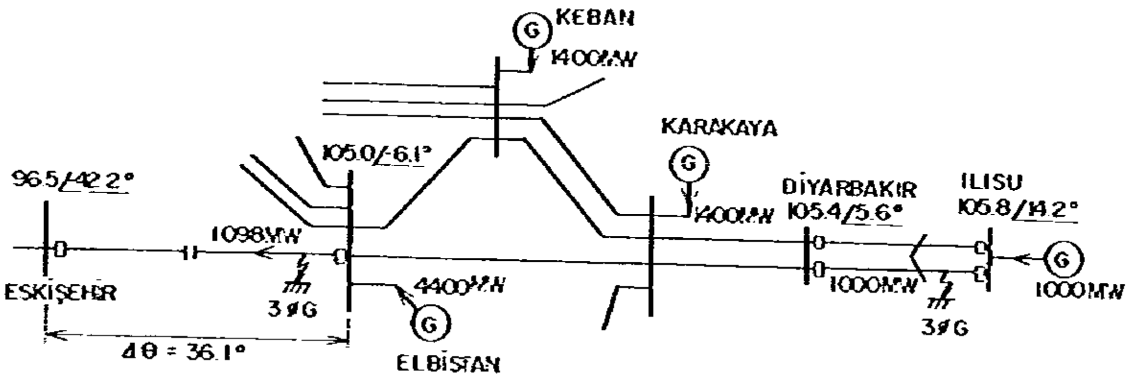


Fig.10-14 Power Flow Diagram of 380 kV Network (Pattern-2)
– Peak time in 1993 –

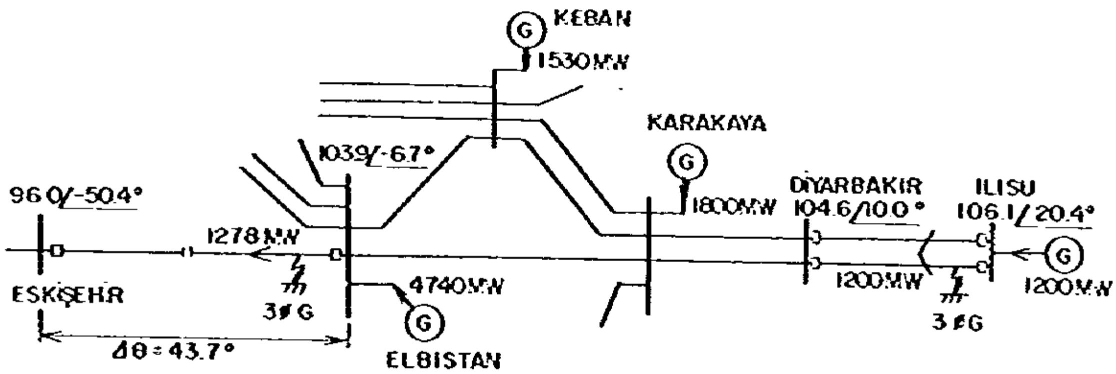
(3) Stability Calculations

The two transmission lines were selected as that of fault occurring. The one is Eskisehir-Elbistan section of heavy power flow and of largest phase angle of the two ends, and the other is the Ilisu power station site farthest from demand areas. The following diagram shows the points where 3-phase ground faults applied in transient stability calculations, the power flows of the disturbed transmission lines before faulting, and the outputs of power stations in the vicinity of Elbistan.

(a) Power Flow Pattern-1



(b) Power Flow Pattern-2



As a result of calculatins, the system stability can be maintained under power flow Pattern-1, but under power flow Pattern-2, the system is unstable because of that the generators of Ilisu power station will be step-out.

With a fault applied at the closest to Elbistan all generators will be stable. In case of power flow Pattern-1, the generator swing of Elbistan and Ilisu are great immediately after the fault, but Elbistan damps its swing more quickly compared with Ilisu (Fig. 10-15), since transmission lines in the vicinity of Elbistan are in the form of a mesh. In case of power flow Pattern-2, the swinging of generators in the east will be great and more than 10 sec. will be required until damping (Fig. 10-16).

On the other hand, with a fault closest to Ilisu, in case of power flow Pattern-1, the swing of the Ilisu generators is large, but all of the generators are stable (Fig. 10-17). In case of power flow Pattern-2, there will be step-out of the Ilisu generators in 1.5 sec. after the faulting (Fig. 10-18). This is because the stability of the system cannot be maintained as the phase angle difference of the generators continues to increase due to being too much power flow of the main trunk lines towards the west.

The stability of the Beskonak generators is stable under each power flow pattern, there is almost no difference due to the difference between power flow Patterns-1 and-2.

(4) Short-circuit Current

Since the network of Turkey is composed of long distance transmission lines, the short-circuit current is small throughout the system, while the breaking capacities of circuit breakers are of levels which will not be any problem in particular except for a part of the power stations in the east.

The short-circuit currents of 380 kV buses of all the substations and their flow are shown in Fig. 10-12. The largest short-circuit current in the network is 33.9 kA at Elbistan.

The locations where short-circuit currents of 380 kV buses exceed 15 kA are the following:

Elbistan ;	33.9 kA (= 22,310 MVA)
Keban ;	20.9 kA (= 13,760 MVA)
Karakaya ;	20.8 kA (= 13,690 MVA)
Ankara ;	17.3 kA (= 11,390 MVA)
Gölbasi ;	15.9 kA (= 10,470 MVA)

The other power stations and substations are all around 10 kA.

(5) Conclusion

According to the results of the above studies, as seen in the calculations for power flow Pattern-2, when the westward power flow (to Ankara) is increased more than 8,000 MW, there will be problems concerning the thermal capacity of the transmission line and the system stability with the present 380 kV transmission line plan.

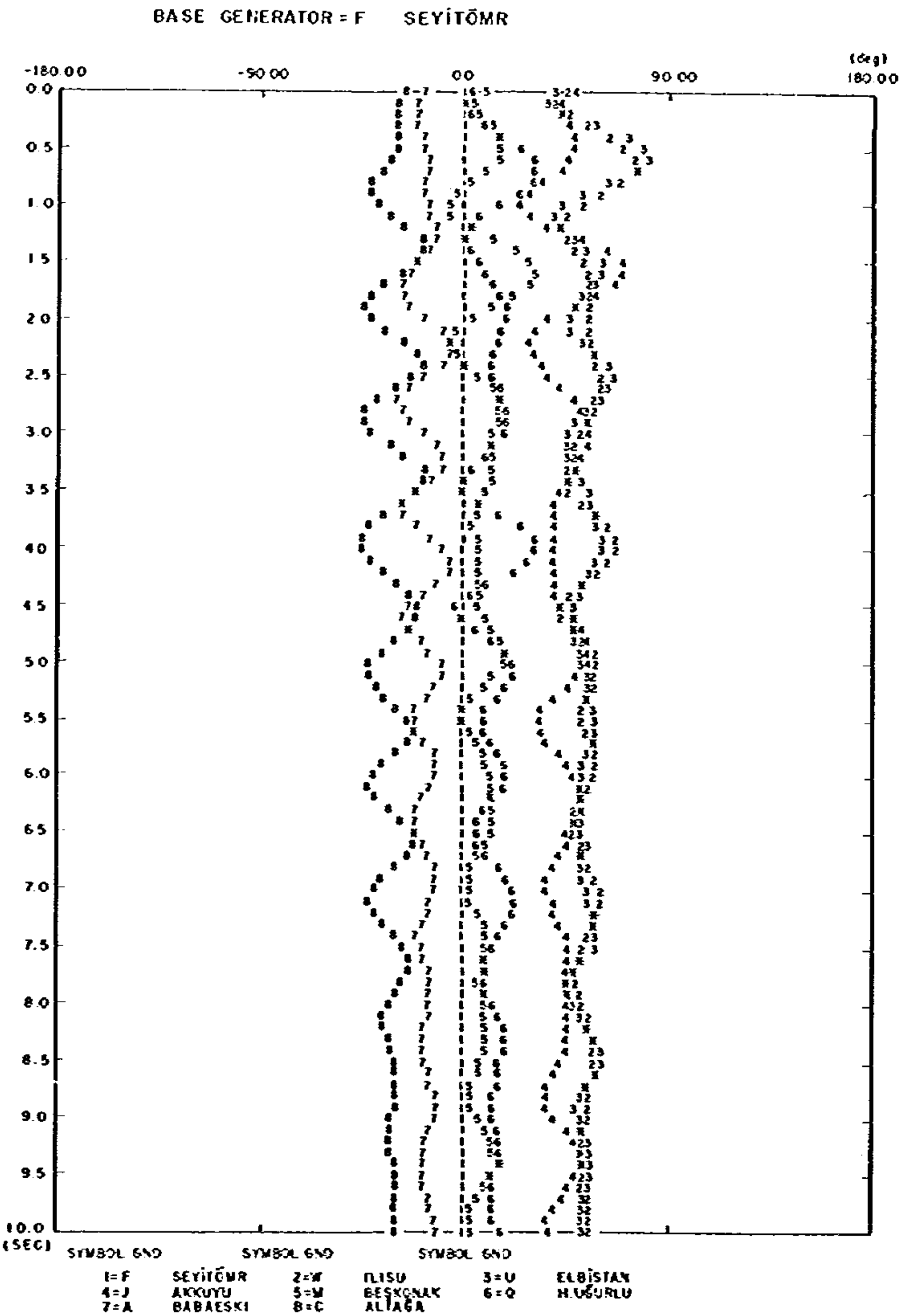


Fig.10 - 15 Transient-stability Swing Curve following 3ϕG-fault at ELBISTAN (Pattern-1)

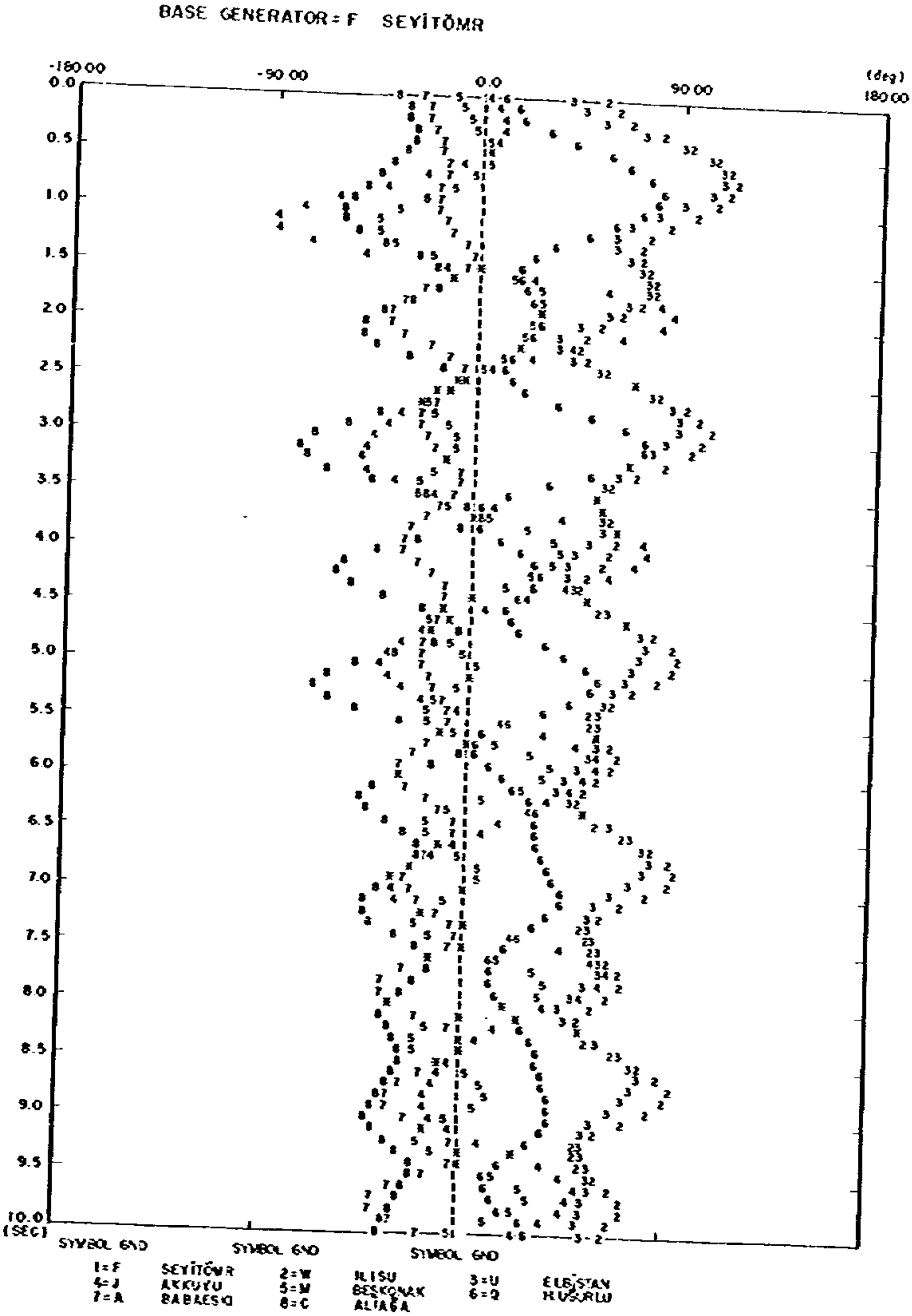
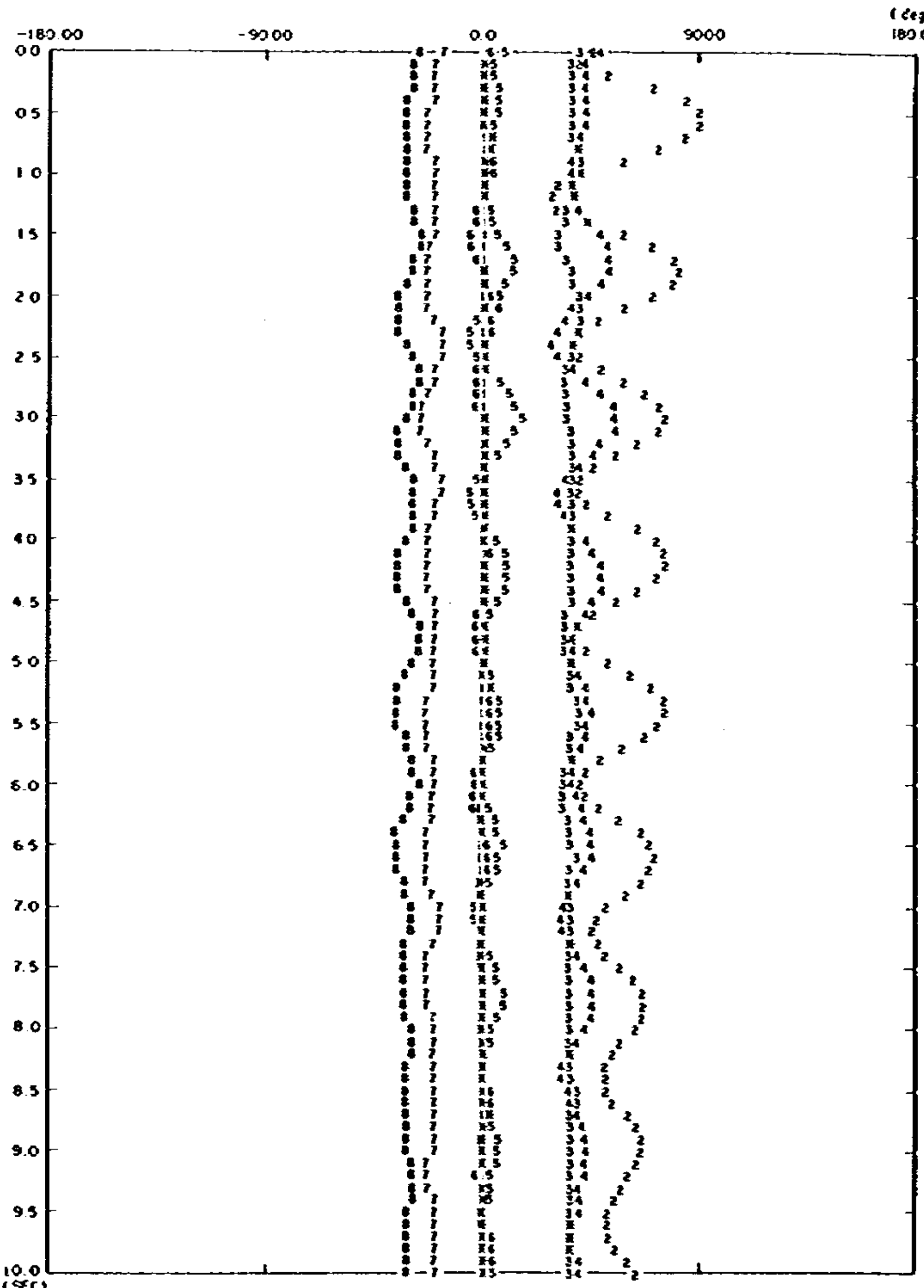


Fig.10-16 Transient-stability Swing Curve
following 3ϕG-fault at ELBISTAN (Pattern-2)

BASE GENERATOR = F



SYMBOL GND SYMBOL GND SYMBOL GND
1=F SEYITÖMER 2=# ILISU 3=U ELBISTAN
4=J AKKUYU 5=M BEŞKONAK 6=O HURGELD
7=A BABAESKI 8=C ALTAŞA

Fig. 10-17 Transient-stability Swing Curve following 3ØG-fault at ILISU (Pattern-1)

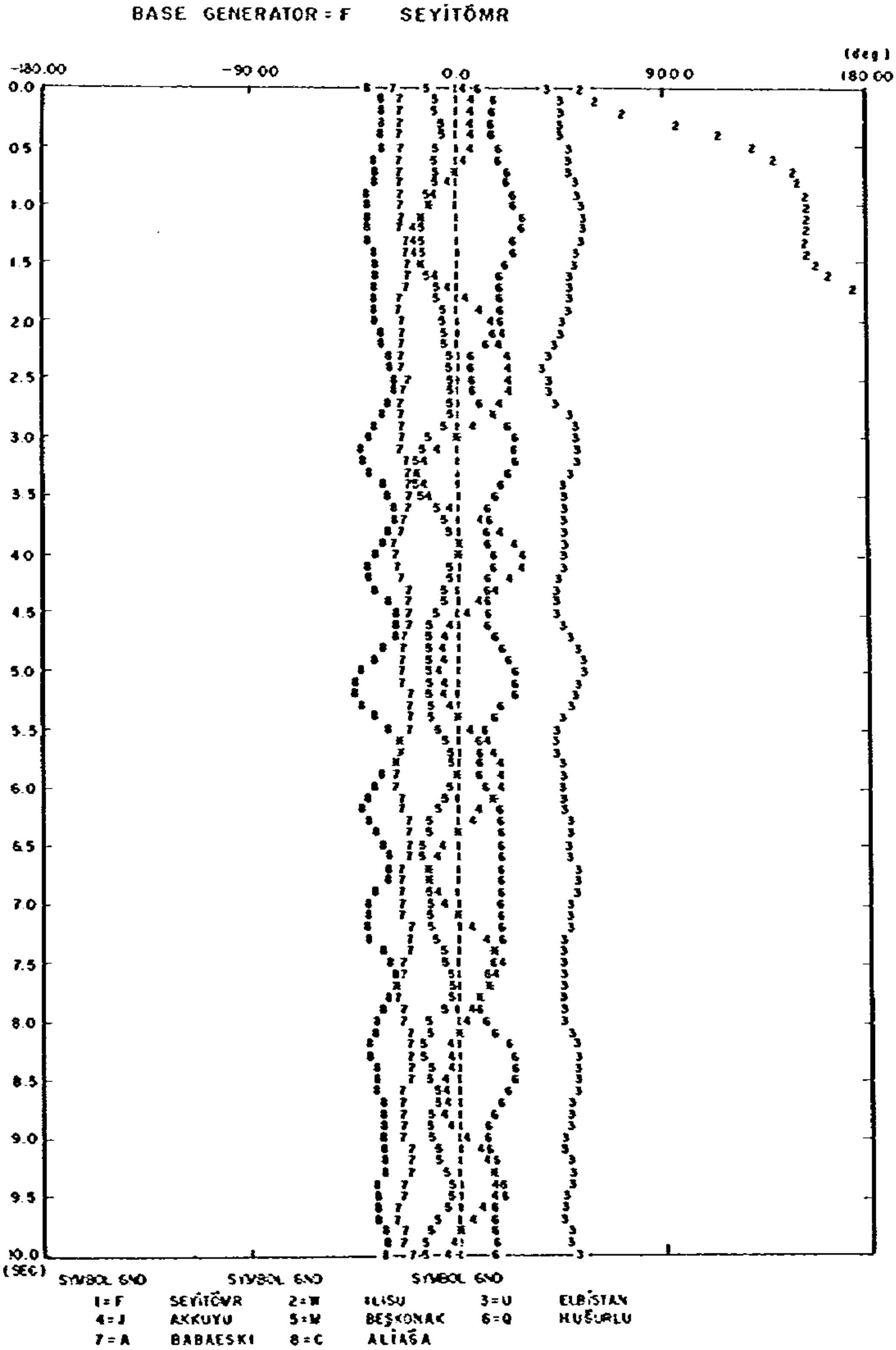
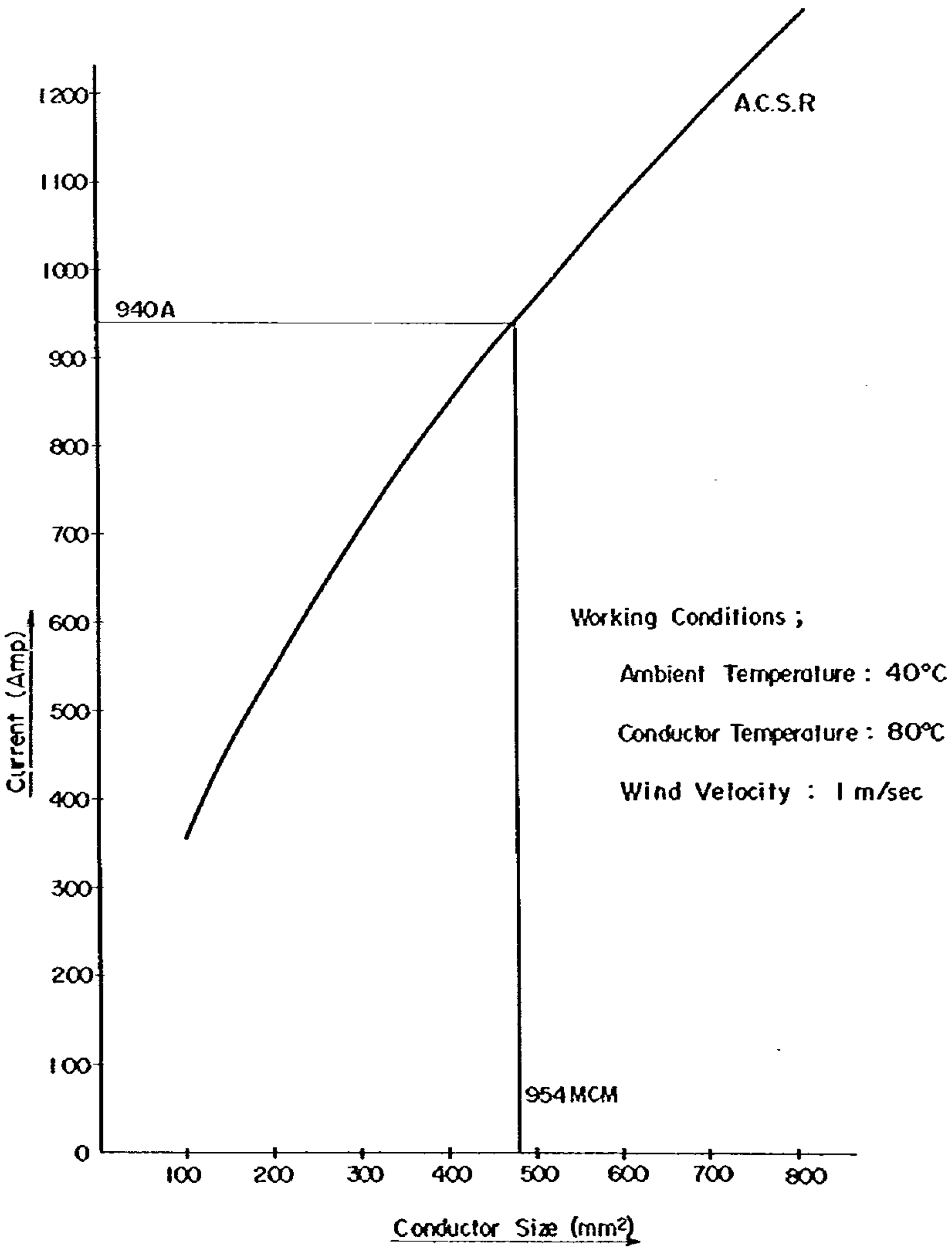


Fig.10-18 Transient-stability Swing Curve following 3ϕG-fault at ILISU (Pattern-2)

Fig. 10-19 Current - Carrying Capacity



10.4 Study of Economics

The results of economic comparisons of the five patterns of power transmission from Beskonak power station are shown in Table 10-1. The economic comparisons were made regarding construction costs, annual costs and transmission losses for the transmission lines and the facilities of the power station and the substation.

As a result, in the comparisons of construction costs, Pattern-A (154 kV, 1-cct proposal) is found to be the lowest, and Pattern-E (380 kV, 2-cct proposal) the highest. On the other hand, in comparisons of annual costs including transmission losses, Pattern-D (380 kV, 1-cct proposal) is the most economical and Pattern-A the most uneconomical.

10.5 Conclusions

Pattern-D is recommended as the transmission method for the Project. The transmission line facilities of Pattern-D are as listed below.

Transmission voltage	:	380 kV
Number of circuits	:	1
Length	:	Approx. 25 km
Expansion section	:	A section of minimum length from Beskonak power station to the projected 380 kV transmission line (Oymapinar - Kepez)

The merit and demerit of this transmission pattern are as follows:

(1) This is the most economical compared with other transmission patterns.

(2) The system reliability is lower compared with the double-circuit transmission line proposal, since it is a single circuit.

Regarding the system reliability, since the output of the power station is 200 MW and the ratio to the demand of the Turkish power system of 16,500 MW (in 1993) is an extremely low 1.2%, the reduction in service capacity due to isolating of this power station caused by transmission line faulting will hardly be a problem. Therefore, the economically advantageous Pattern-D is recommended.

10.6 Proposition for System Analysis

In proceeding with the transmission line expansion plan suiting to the electric power development program for the eastern part of Turkey, the transmission facilities must be in harmony with the development scales and development order of power sources, and with the demand distribution, and those facilities must be high in the system reliability and economically advantageous.

As shown in the results of analysis on the 380 kV transmission system of 10.3.3, as the demand is enlarged, complex problems latent in the power system will come into the open. Because of this, in the expansion program for the electric power system, the problematic points existing in the power system must be extracted and also their solution must be aimed for on each occasion.

The following item may be listed as the problems which can be imagined for the 380 kV transmission system in 1993 or later.

- Heavy power flow and voltage adjustment
- Transmission losses
- System stability

In order to resolve these problems it is considered fundamental to carrying out system analysis by TEK based on its system expansion policy.

It is proposed the following items should be added to the study themes in step with the power source development of the eastern part of Turkey.

- The system scale which can be met with a 380 kV transmission system and the time when a limit will be reached

- Introduction of the next higher voltage (e.g., AC 750 kV)

- Introduction of a direct-current transmission system

CHAPTER 11

PRELIMINARY DESIGN

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CHAPTER 11 PRELIMINARY DESIGN

11.1 Design Conditions

Preliminary design was performed in accordance with the conditions described below.

(1) The crest elevation of the Beskonak dam is to be EL. 160.00 m, taking into consideration design flood water level, wave height, etc., with the high water level of EL. 155.00 m studied in 9.3.

(2) The spillway is to possess a capacity to safely discharge the design flood flow of 4,500 m^3/sec (PMF) calculated in 6.8, while a flip bucket system is to be adopted for energy dissipation in consideration of the downstream river configuration.

(3) For the design discharge of the diversion tunnel, the 5-year return period flood of 1,250 m^3/sec is to be adopted in consideration of dam type (concrete) and its construction period.

(4) A secondary dam is to be constructed at the saddle of Hortu Creek.

(5) The maximum output of the power station is to be 200 MW, with 6-hour peaking power generation as standard. The facilities are to be such that it will be possible for power discharge of 30 m^3/sec to be made 24 hours a day during June-September in order to secure irrigation water for the downstream area.

(6) Each structures were designed at standard preliminary level, and at final design stage they should be designed in detail.

11.2 Dam and Appurtenant Structures

11.2.1 Beskonak Dam

(1) Selection of Dam Site

The Beskonak dam is planned at the upstream part of Beskonak gorge on the Köprücay River. This gorge, as shown in Dwg. 1-2, has an extremely narrow river width of about 20 m for a section of approximately 600 m, where both banks are almost vertical cliffs of the Köprüçay Conglomerate. The left bank side opens up approximately 200 m downstream from the entrance of the gorge, and the ridge there is lower.

Therefore, the dam site would be limited to a section of approximately 100 m from the entrance. Comparison studies of dam sites were carried out for three cases in this section indicated in Fig. 11-1. Table 11-1 gives comparisons of these cases.

Fig. 11-1 Comparison of Dam Location

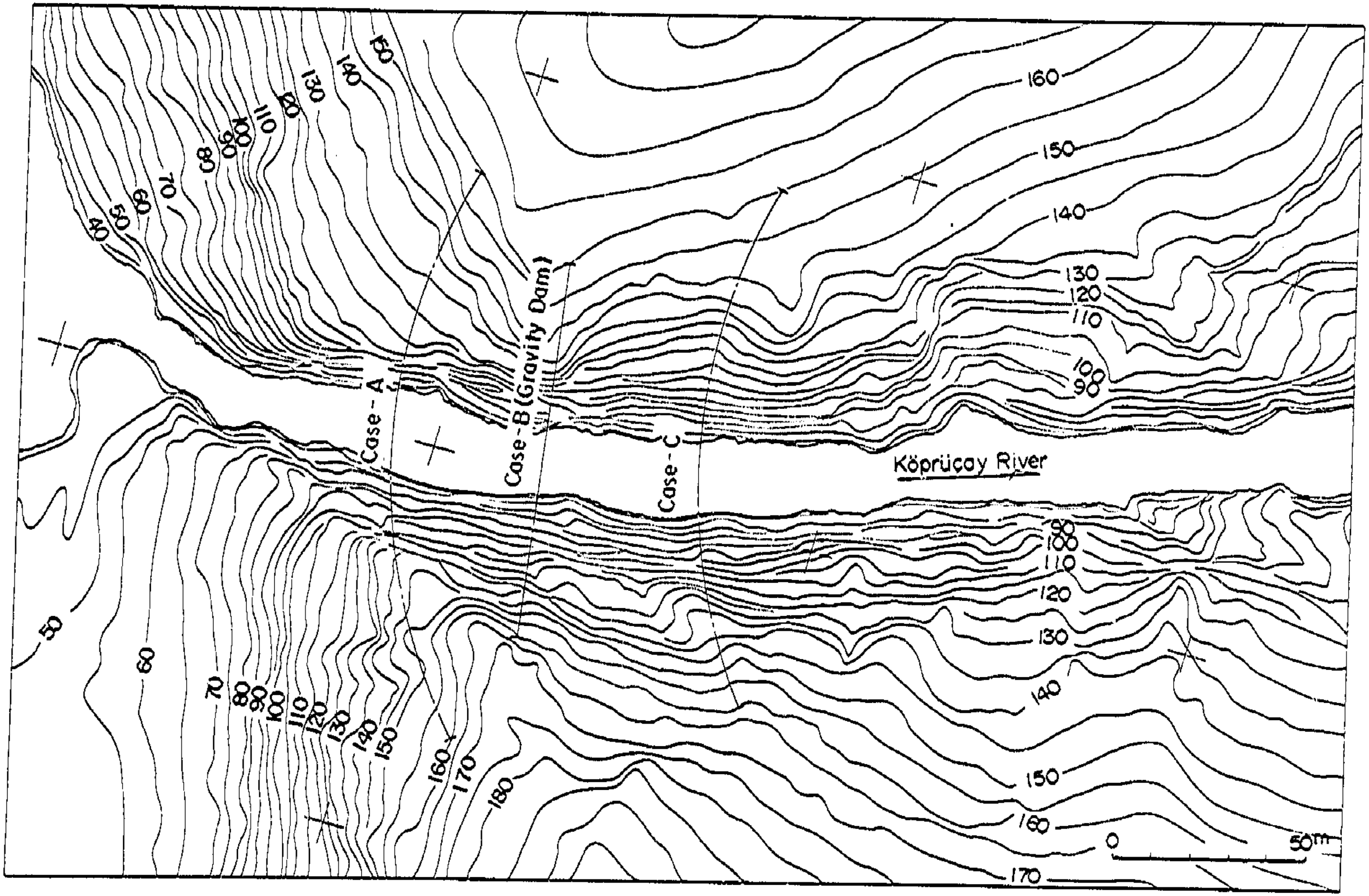


Table 11-1 Comparisons of Dam Sites

Item	Case A	Case B	Case C
Length of diversion tunnel	Shortest	Slightly longer than A	Longest
Dam type	Concrete curved dam	Concrete straight-line gravity dam	Concrete curved dam
Dam volume	Slightly more than C	Largest	Slightly less than A
Spillway	Spillway layout advantageous	Requiring extensive excavation for spillway	Spillway layout difficult
Construction works	Upstream space wide and excavation and access easy	Valley width and excavation and access difficult	Same as Case B
Dam concrete placement	By jib crane	By cable crane	By cable crane

As a result of studies on the three cases above, the location of case A, which was superior with regard to structural design of the dam, layout of the spillway and easily execution of the work, was selected as the dam site.

(2) Selection of Dam Type

The projected Beskonak dam site is topographically suited for a concrete dam, and studies were carried out for both an arch dam and an arch-gravity dam.

With an arch dam, the dam itself would have a small concrete volume and be economical, but because the valley has extremely small width in the downstream of the dam, there are many disadvantageous aspects from a structural standpoint. With respect to the spillway design, it is feared that it would adversely affect the stability of dam itself to overflow the design flood discharge of 4,500 m^3/sec at the center of dam crest directly onto both downstream banks. Therefore, a tunnel type would be suitable for the spillway of the arch dam. In this case, it is conceivable for spillway tunnels of 12 m diameter to be provided at both banks. The streams of the two are to be made to collide with each other in the air downstream of the dam for energy dissipation, but the energy dissipation mechanism would be extremely complex.

On the other hand, with an arch-gravity dam, the concrete volume would be 2.2 times that of an arch dam, but it would be possible to make the spillway an overflow type of the dam crest, while a simple flip bucket type can be adopted for energy dissipation.

A comparison by construction costs of the two cases including spillways will be follows:

Arch dam	3,816.4 million TL
Arch-gravity dam	3,647.1 million TL

As described above, the arch-gravity dam is more advantageous from the technical and economic aspects of the dam structure including the spillway, and therefore, this was taken as the optimum proposal.

The arch-gravity dam, which is the optimum proposal, is shown in Dwgs. 11-4 and 11-5, and the arch dam in Dwgs. 11-11 to 11-13.

(3) Spillway Gates

Spillway Gates were planned to be four sets of radial gates of 12 m wide and 10 m high. These gates are usually operated with wire rope type hoists by the electric power of the Beskonak power station. In an emergency it is desirable to use the power of diesel generators.

It should be favorable to reexamine spillway gates in detail at the final design stage.

(4) Outlet

It is desirable for outlet to be provided in the dam body other than the spillway. The outlet is to be used to adjust the water level when impounding water in the reservoir at the time of completion of the dam, and to be used in case it is necessary to lower the reservoir water level below low water level in an emergency. Details of the outlet should be reexamined at the time of the final design.

(5) Foundation Treatment

Foundation treatment of the dam is to be executed by consolidation grouting and curtain grouting.

Curtain grouting was planned, shown in Dwg. 8-1, in order to prevent the leakage from the dam foundation and the reservoir as described in 8.3.

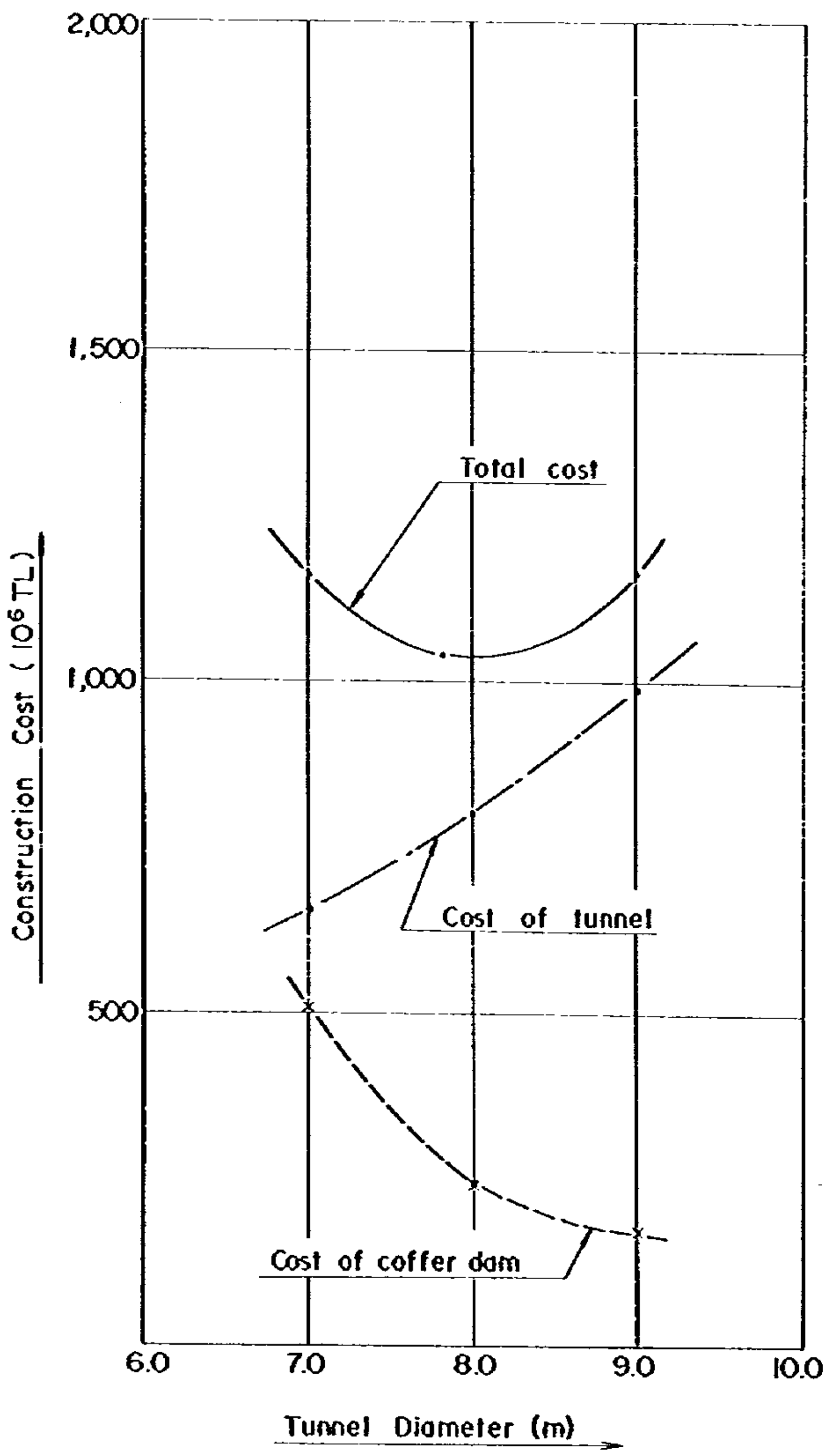
Further, caves and soft foundation portions, if any, are to be treated by replacement with concrete or by concentrated grouting.

11.2.2 Diversion Tunnels

The routes of the diversion tunnels were selected at the right bank side shown in Dwg. 11-2 taking into consideration the topography and the access road for the construction work of the dam.

The cross sections of the diversion tunnels were selected so that the construction cost of the upstream cofferdam and the tunnels would be a minimum for the design flood discharge of 1,250 m^3/sec. and as shown in Fig. 11-2, two tunnels of economical cross section with D = 8.00 m were planned.

Fig. II-2 Economical Diameter of Diversion Tunnel



11.2.3 Cofferdams

The upstream cofferdam was designed as crest elevation of EL. 60 m based on the capacity calculations of the diversion tunnels. This dam, shown in Dwg. 11-3, is to be arranged at the upstream side so that an access road for the river bed excavation for the main dam can be secured, and a rockfill type was adopted to be banked on river-bed sand-gravel of approximately 25 m in thickness. Foundation treatment of the river-bed sand-gravel is planned to be achieved by a grout curtain.

The downstream cofferdam is to be a rockfill dam provided approximately 260 m downstream of the main dam.

11.2.4 Secondary Dam

A secondary dam was planned at the saddle of Hortu Creek approximately 1,300 m east of the Beskonak dam.

Judging by topographical and geological conditions, it is thought suitable for the secondary dam to be a rockfill type shown in Dwg. 11-6.

The crest elevation of the secondary dam was made at EL. 161.00 m adding an allowance of 1 m to the elevation of the Beskonak dam since the former is to be a rockfill dam.

11.3 Waterway and Power Station

11.3.1 Outline

Designing of the waterway and power station was done on two proposals, the optimum proposal selected in 9.4 and an alternative.

The optimum proposal is that which plans the Beskonak project independently without a regulating pond provided downstream. The power station is to accommodate two main units, large and small, of 155 MW and 45 MW, taking into consideration the supply of irrigation water during June - September. In effect, irrigation water is secured in the period (June - September) through the discharge of 24-hour power genration of the smaller main unit.

On the other hand, as an alternative, a study was made of the Project in case a regulating pond is provided at the Kisik site shown in Dwg. 15-1. This alternative is a proposal which would be valid in case the Kisik project is simultaneously developed with the Beskonak project. In case of the alternative, Beskonak power station would perform only peaking power generation with two main units of 100 MW each, and irrigation water for the downstream area would be discharged after storage and regulation at Kisik dam.

11.3.2 Power Intakes

The location of power intakes was selected at a gentle slope at the left bank. Sloped intakes were adopted in consideration of the topography shown in Dwg. 11-8. The intake sills were selected at EL. 105.00 m in order to secure a depth that intake could be done safely at low water level of 134.50 m.

The maximum intakes of these facilities are to be 167 m^3/sec for No. 1 and 50 m^3/sec for No. 2.

The intakes of the alternative are shown in Dwg. 11-15.

11.3.3 Headraces and Penstocks

Two tunnels of diameters 4.3 m and 7.3 m were planned as headraces shown in Dwg. 11-7. The cross sections of the tunnels were decided in Fig. 11-3 on calculation of the econoaical cross sections. The headrace tunnels are to be in a section of thick rock cover by natural ground at the upstream part of the waterway, and are to be concrete-lined.

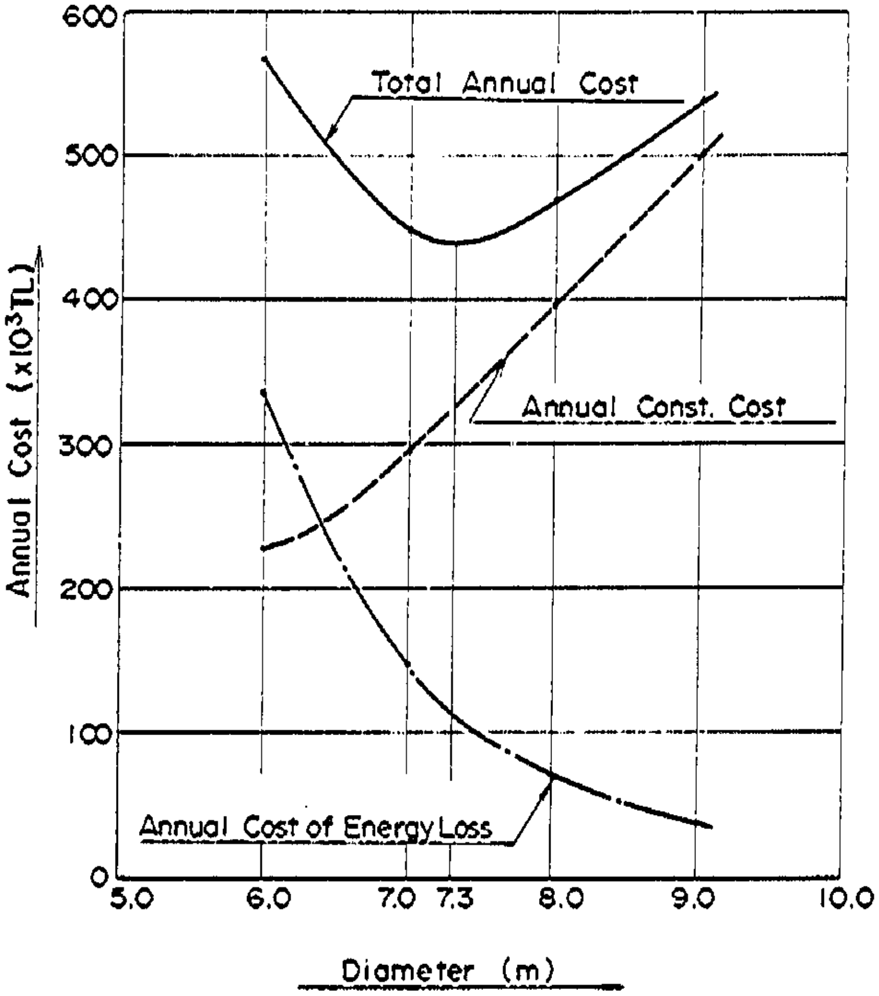
The downstream part of the waterway will have only a thin rock cover by natural ground, and therefore, is to be protected by steel lining. This section to the power station was planned to take the form of penstock lines. These penstocks were planned as buried lines over their entire lengths in view of the topography.

A surge tank is not to be provided. The total length of the waterway is short at approximately 600 m, the total head is small, etc., so that water hammer pressure will be low and there will be no hindrance to functioning of the turbines. Since the downstream part of the waterway would be protected by steel lining, there is little to expect in the way of reduction in construction cost of the waterway by provision of a surge tank.

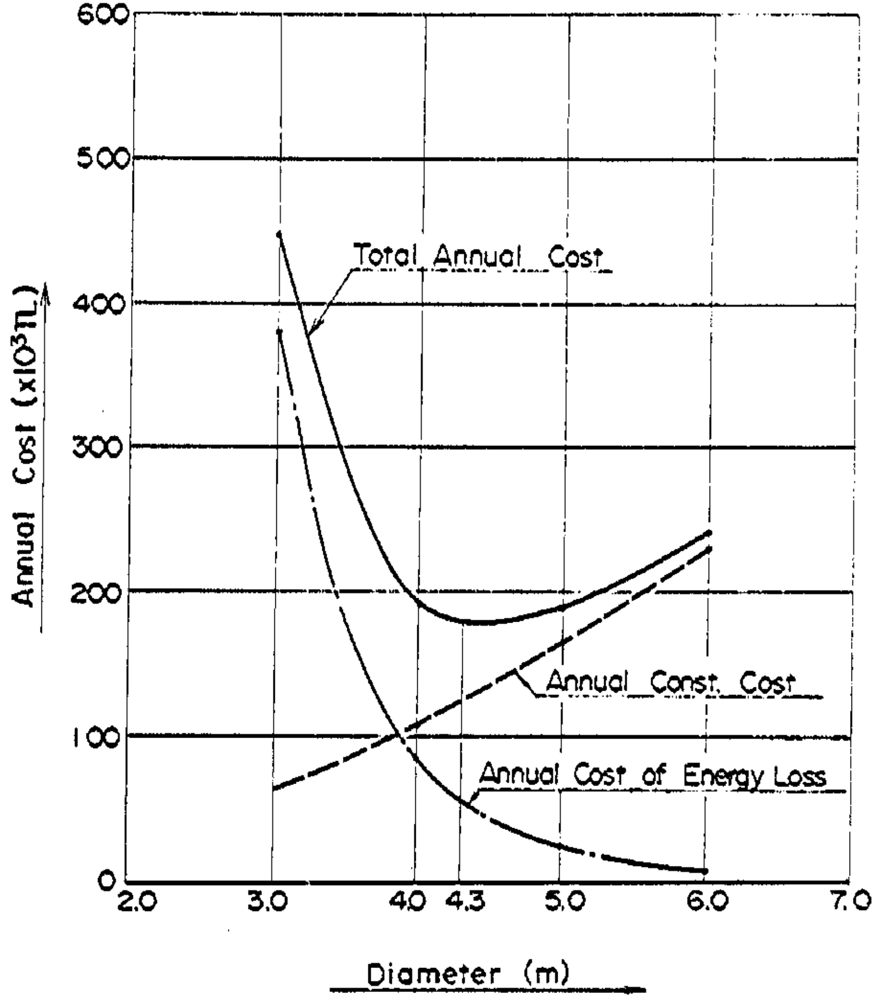
For the alternative proposal, the waterway in general is shown in Dwg. 11-14.

Fig.11-3 Economical Diameter of Headrace Tunnel

NO.1 Tunnel
(Q = 167 m³/s)



NO.2 Tunnel
(Q = 50 m³/s)



11.3.4 Power Station and Switchyard

The location of the power station, shown in Dwg. 1-2, was selected as the confluence with the Hortu Dere approximately 600 m downstream from the dam site.

This power station, as described in 11.4, will have two main units, large and small, and was planned as a peaking power station with the installed capacity of 200 MW. The outline of the power station is given in Dwg. 11-9.

As for the power station of the alternative proposal (two main units of 100 MW), the outline is given in Dwg. 11-16.

The switchyard is to be provided immediately downstream of the power station, excavating and developing a gently sloped lot at the left bank.

The access road to the power station and the switchyard, shown in Dwg. 1-2, is to be a length of approximately 1.5 km branching from a existing road downstream of the secondary dam.

11.4 Electro-mechanical Equipment

11.4.1 Selection of Units

This power station is planned for normal effective head of 105 m, maximum available discharge of 217 m^3/sec, and installed capacity of 200 MW. In consideration of both the development scale and the irrigation discharge during the dry season, a proposal for two main units of differing capacities was adopted.

The No. 1 unit is to consist of a 158 MW vertical-shaft Francis turbine and a 172 MVA synchronous generator, and the No. 2 unit of a 47 MW vertical-shaft Francis turbine and a 51 MVA synchronous generator. The compositions of the units are given below.

Electro-mechanical Equipment of Power Station

- Outline Specifications -

Installed Capacity: 200 MW

No. 1 Unit:

Turbine;

Type	Vertical-shaft Francis turbine
Number	1 unit
Normal effective head	105 m
Maximum discharge	167 m^3/sec
Standard output	158 MW
Revolving speed	167 rpm

Generator;

Type	3-phase, AC, synchronous generator
Number	1 unit
Capacity	172,000 kVA (power factor 0.9, lagging)
Frequency	50 Hz

No. 2 Unit:

Turbine;

Type	Vertical-shaft Francis turbine
Number	1 unit
Normal effective head	105 m
Maximum discharge	50 m^3/sec
Standard output	47 MW

Revolving speed	300 rpm

Generator;

Type	3-phase, AC, synchronous generator
Number	1 unit
Capacity	51,000 kVA (power factor 0.9, lagging)
Frequency	50 Hz

Main Transformer:

Type	Outdoor type, single-phase, oil-immersed, forced-oil-cooled, with forced-air-cooled type
Number	3 units
Capacity	75,000 kVA
Voltage	380//3/14.4 kV

Switchyard:

Type	Conventional
Number of transmission line	1 cct

11.4.2 Power Station

The power station is to be indoor type of 24.50 m wide and 66.00 m long equipped with two units of turbines and generators, large and small. In addition, one overhead travelling crane, one hauling crane, and auxiliary equipment are to be accomodated.

A layout of the power station is shown in Fig. 11-5.

11.4.3 Main Circuit and 380 kV Switchyard

The so-called central system is adopted, where two generators are connected to one main transformer, the generators and the main transformer installed outdoors being connected by enclosed bus.

The electric power produced is stepped up to 380 kV by the main transformer and sent to the 380 kV switchyard by aerial bus.

The switchyard is located at the downstream left bank side approximately 200 m from the power station. The switchgear is planned to be of conventional type.

The single line diagram of the main circuit and the switchyard equipment layout are shown in Figs. 11-4 and 11-6.

Fig. 11-4 Single Line Diagram

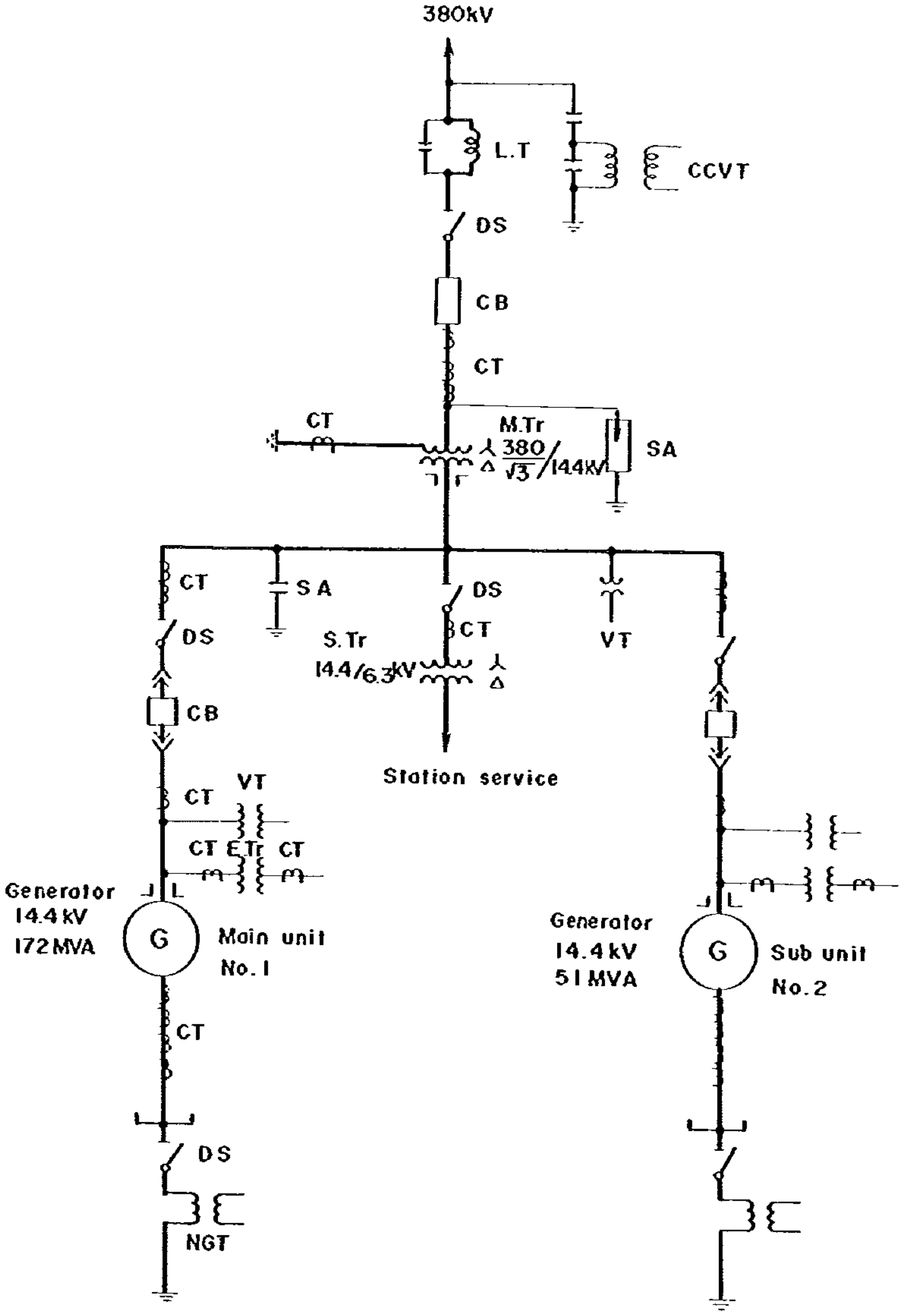


Fig. 11-5 Plan of Power Station

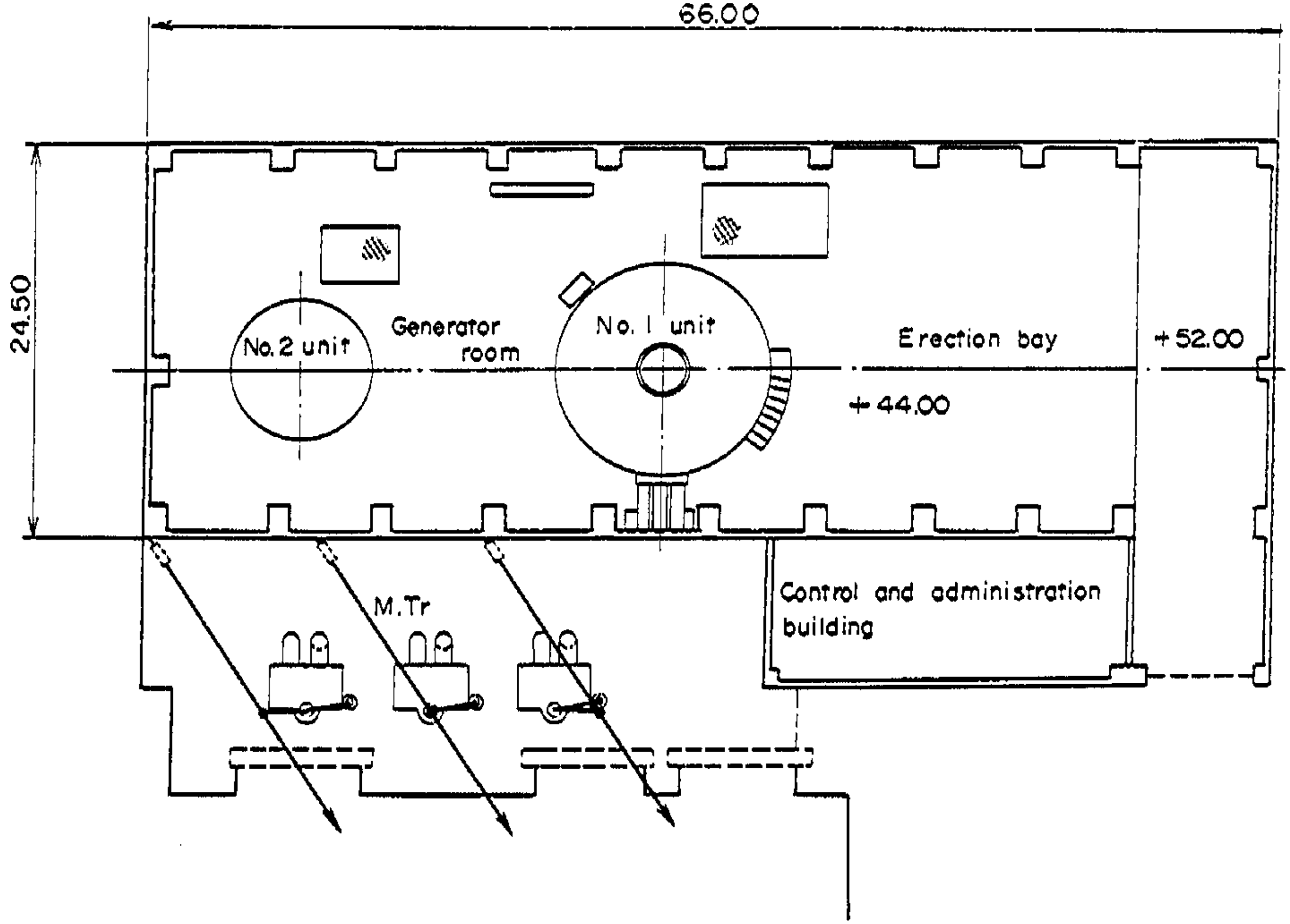
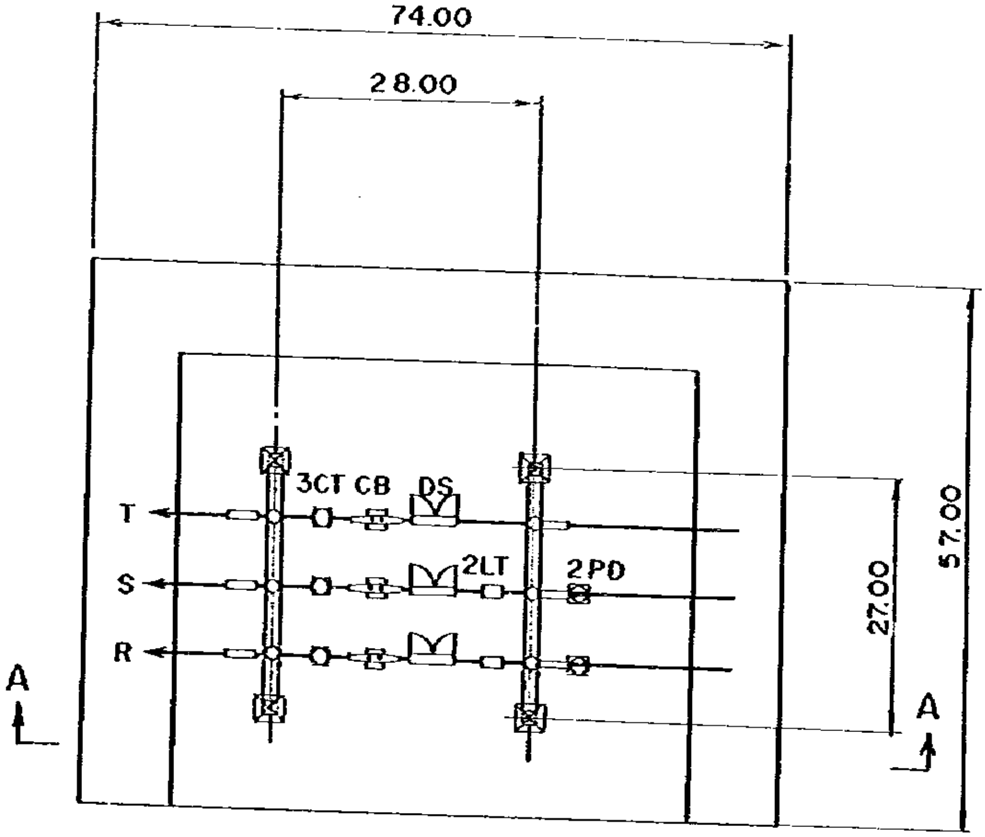
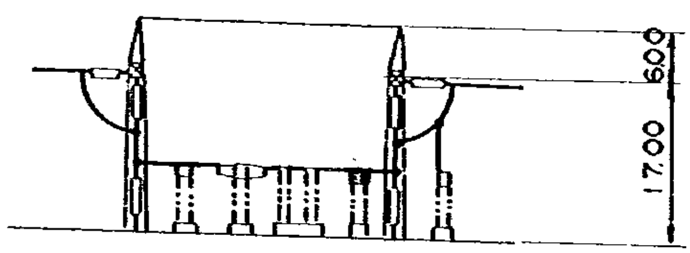


Fig. 11-6 380kV Switchyard Layout

PLAN



SECTION A - A



11.5 Construction Schedule and Planning

11.5.1 Basic Conditions

The structures to be constructed in this Project consist of an arch-gravity dam of 165 m high as main, and power intakes, headraces, penstocks and a power station. Further, there is the most important work consisting of grout curtain of the reservoir amounting to an area of approximately 380,000 m^2. The quantities of the main civil works are given in Table 11-2.

An outline of the matters affecting the construction schedule and planning of the Project is described below.

(1) Meteorology

Concerning the meteorological conditions at the project site such as temperature, precipitation, the number of rainy days a year, it is already described in Chapter 6.

The meteorological conditions at the project site are extremely favorable for placing concrete and banking a fill dam, and the construction schedule was formulated assuming that construction work can be performed throughout the year.

(2) Transportation

The road situation to the project site is very good with paved roads from Antalya to the vicinity of the dam site. However, in delivery of principal equipment, it will be necessary for access roads to be constructed to the dam and the power station.

The Antalya port is the nearest one from the project site. This port is being used for unloading materials and equipment for the Oymapinar Project presently under construction on an adjacent river, and cargo handling facilities of this port are adequate.

(3) Construction Materials

(a) Cement

It is desirable for moderate heat cement to be used for construction of the dam. The cement mill at Isparta 220 km apart from the project site will be the main plant for supplying cement. Further, if fly ash can be obtained, it will be possible to reduce heat of hardening by replacing 20 to 30 % of the cement with fly ash, and will be helpful in strenghtening concrete.

(b) Aggregates

The sand-gravel deposited at both banks of the Sagirin River, approximately 10 km downstream from the dam site, will be used for concrete aggregates. The physical properties of the aggregates are described in 7.5. The total volume of concrete required for the Project is expected to reach approximately 630,000 m^3, and all of the aggregates needed for this concrete is to be collected from the above deposit.

(c) Embankment Materials

Of the embankment materials to be used for the cofferdam and the secondary dam, the impervious core materials is to be taken from a borrow area adjacent to the aggregate deposit, while for filter materials, river-bed gravel upstream of the dam site, and for rock materials excavation muck diverted from the intakes, power station, etc., are to be used.

(4) Electric Power for Construction

It is considered that approximately 3 MW will be required as electric power for construction. This electric power is to be supplied branching off from the 34.5 kV transmission line running through Beşkonak village.

(5) Construction Facilities

An aggregate plant is to be installed at the aggregate deposit site. The capacity needed for the aggregate plant, in consideration of both the concrete volume required and the construction period, will be of 250 ton/hr. class.

A batching plant is to be provided at the left bank side of the dam. The nominal capacity is to be 140 m^3/hr. Placing of dam concrete is to be executed by jib cranes with two cranes of 13.5 ton class and one crane of 9 ton class to be provided.

Cement silos, a cooling plant, etc., are to be provided in the vicinity of the batching plant. The construction facilities including the above are mostly to be procured indigenously by the contractor, but large sized cranes for placing concrete and special equipment such as the many boring and grouting machines might be imported from abroad.

11.5.2 Construction Schedule and Planning

The construction work of the Project is thought to require a period of 72 months including preparatory works, in consideration of the scale of the Project, the layout of structures and regional conditions. Commissioning of the Project is targeted to be at the end of the sixth year after start of construction, and the construction schedule was set up shown in Fig. 11-7. In this Project, the work quantity of curtain grouting of the reservoir will be especially large, and this work will be the factor with the greatest amount of influence on the work schedule.

The outlines of the construction schedule and planning are described below.

(1) First Year

Simultaneously with start of construction, preparatory works will be commenced such as access roads, temporary bridges, water and air supply facilities, electric power for construction facilities and temporary buildings for construction. Following these works, in the latter half of the year, excavation works of both of the diversion tunnels will be simultaneously started.

(2) Second Year

Preparatory works will be continued. Preparation of installation of the aggregate plant and the concrete plant will be carried out. Excavation and concrete lining of the diversion tunnels will be completed and river diversion done before the autumn flood season. A small amount of concrete at the initial stage will be made with handy facilities.

Simultaneously with diversion of the river, excavation and grouting of the cofferdam foundation will be carried out and the embankment work started from the end of the year. Tunnel excavation for the gallery to perform curtain grouting will be started from four work adits on the right bank side and two adits on the left bank side.

(3) Third Year

Temporary works such as on the aggregate plant, concrete plant, concrete placing cranes, etc., will be carried out. Banking of the cofferdam will be at its busiest and at the same time excavation for the dam will be started. Foundation grouting will be begun as soon as excavation at the river bed is finished, and placement of dam concrete will be commenced. The placement of concrete is taken to require 37 months in consideration of the lift schedule and crane capacities. Meanwhile, the grouting gallery will continue to be excavated, concrete lining provided, and part of the grouting work started. The grouting will be performed divided into right and left banks and is planned to require approximately 36 months. Excavation of the intakes will be started at an early date, and the muck will be diverted for use as rock material for the cofferdam. Further, manufacturing of electro-mechanical equipment will be started.

(4) Fourth Year

The construction work will be at its peak, and placement of dam concrete, performance of curtain grouting and banking of the secondary dam will be started. Rock materials for the secondary dam will be diverted muck from the power station excavation as much as possible. Also, manufacture of hydraulic equipment and transmission line will be started.

(5) Fifth Year

In the fifth year, concrete placing and curtain grouting will be at their peaks. As for the intakes and headrace tunnels, concrete and grout works will be chiefly performed. Excavation for the penstocks will be completed, and steel pipe installation and concrete filling work will be started.

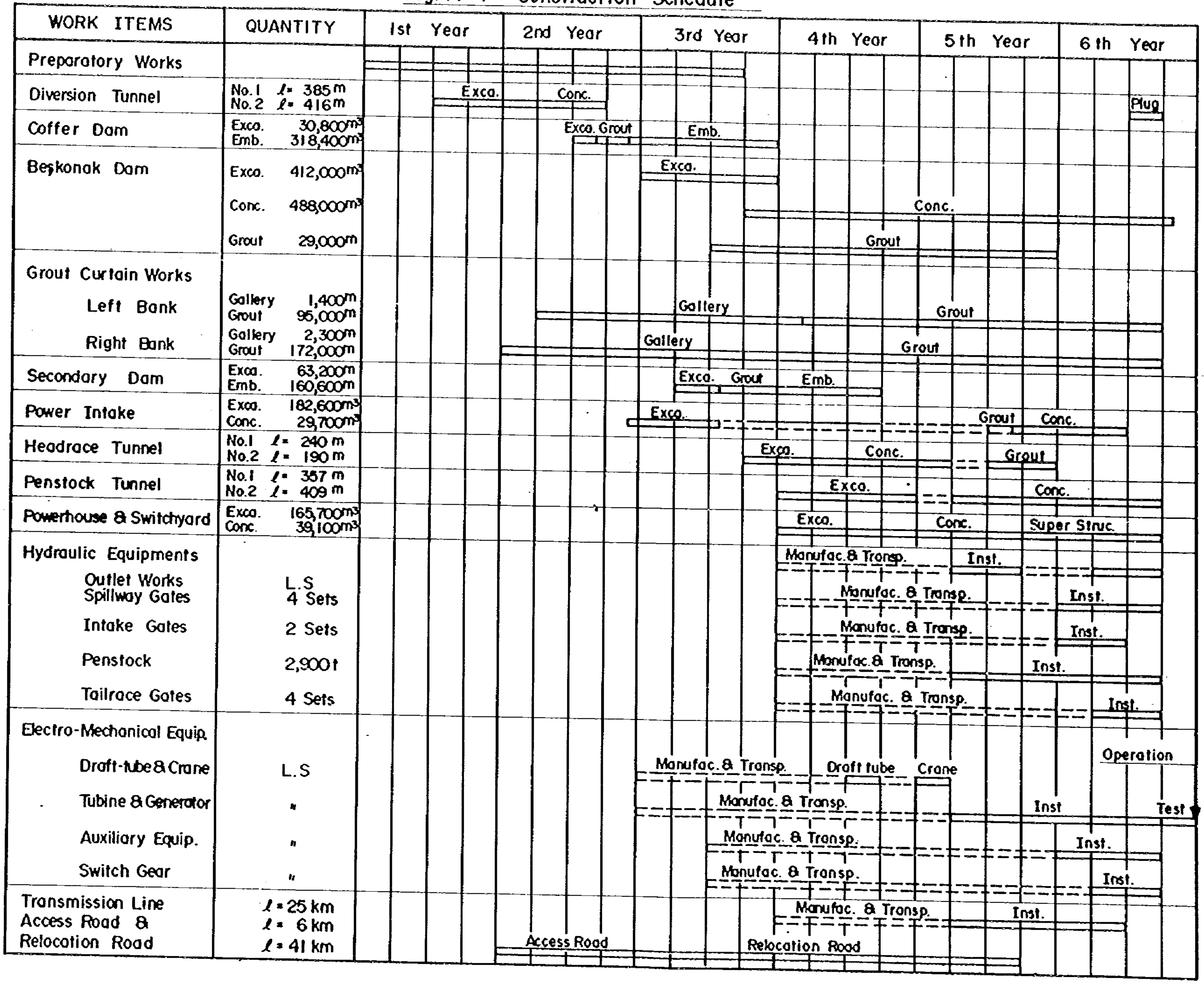
(6) Sixth Year

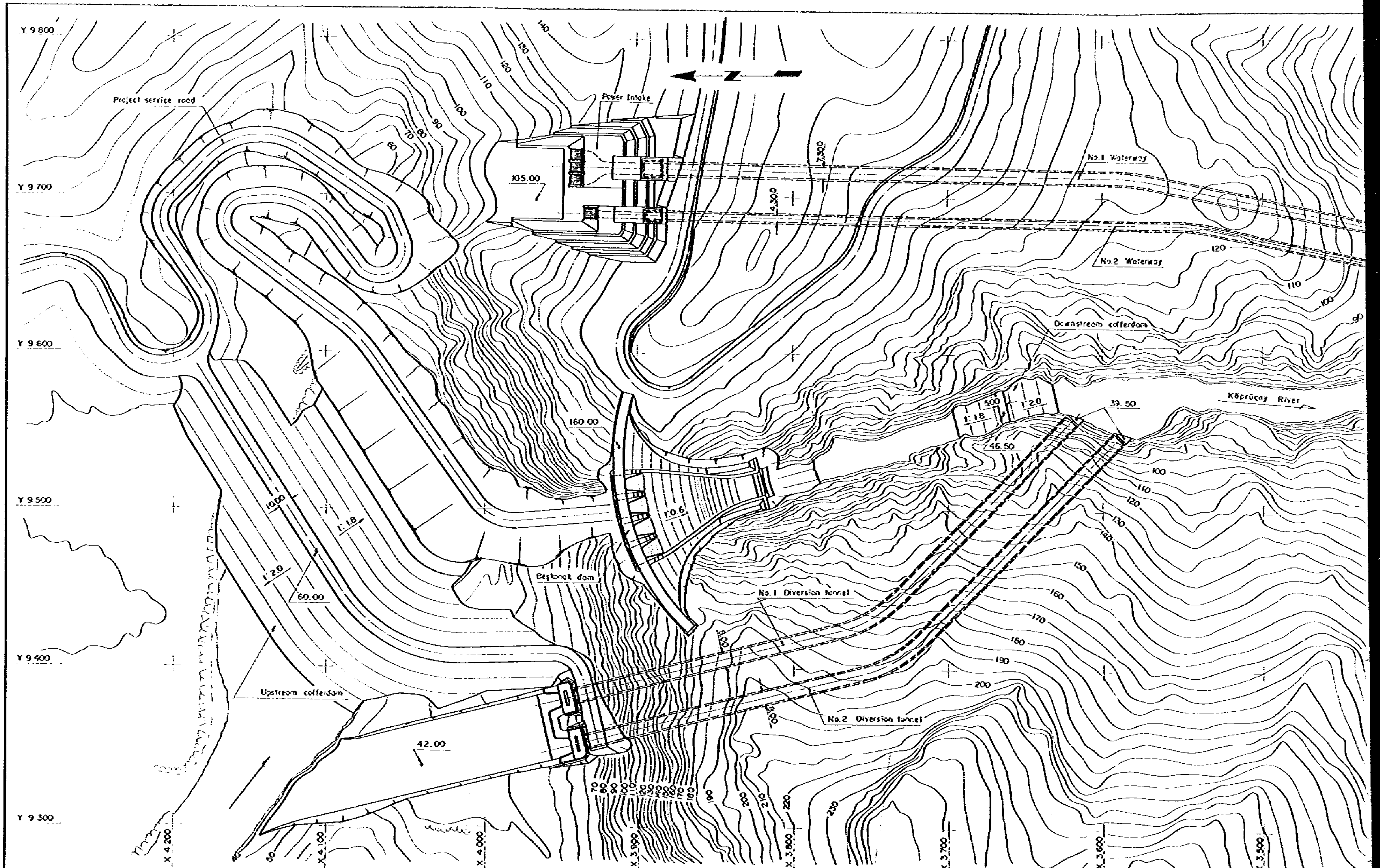
Construction will be at its final juncture, and the stage of finishing the dam concrete and grout curtain works will be entered. Installation of gates and penstocks will be at the busiest. The dry season in the summer time is selected for closing the diversion closure gates, and simultaneously with starting water impoundment, plug concrete for the diversion tunnels will be placed. Installation works of turbines, generators, auxiliary equipment, switchgear, etc., will be completed, and overall adjustments started in the autumn. Construction of the transmission line will be completed in the first half. Wet tests will be performed on awaiting rise in reservoir water level, and operation will be started at the end of the year.

Table 11-2 Quantity of Main Civil Works

	Open Excavation (m^3)	Tunnel Excavation (m^3)	Concrete (m^3)	Embankment (m^3)	Grouting (m)	Remarks
Main Dam	514,600	68,100	520,700	318,400	15,900	Including diversion tunnel and cofferdam
Secondary Dam	63,200	-	-	160,600	8,300	
Curtain Grouting	-	40,500	22,600	-	290,000	Including dam site, right bank and left bank
Waterway	182,600	50,300	49,800	-	5,700	Including power intake, headrace and penstocks
Power Station & Switchyard	165,700	-	39,100	-	-	
Total	926,100	158,900	632,200	479,000	319,000	

Fig. 11-7 Construction Schedule





Project service road
Power Intake
105.00
No.1 Waterway
No.2 Waterway
Downstream cofferdam
Köprüçay River
33.50
45.50
160.00
Beşkonak dam
No.1 Diversion tunnel
No.2 Diversion tunnel
Upstream cofferdam
42.00
60.00
10.00
1:1.8
1:2.0
1:0.6
Y 9 800
Y 9 700
Y 9 600
Y 9 500
Y 9 400
Y 9 300
X 4 200
X 4 100
X 4 000
X 3 900
X 3 800
X 3 700
X 3 600
X 3 500


Power Intake
No.1 Waterway
No.2 Waterway
Access Road
Switchyard
Powerstation
Downstream cofferdam
Köprüçay River
No.1 Diversion tunnel
No.2 Diversion tunnel
Beşkonak dam
52.00
60.00
31.00
39.50
46.50
160.00
Y 9800
Y 9700
Y 9600
Y 9500
Y 9400
X 3,900
X 3,800
X 3,700
X 3,600
X 3,500
X 3,400
X 3,300
X 3,200
BEŞKONAK PROJECT
BEŞKONAK DAM
AND POWERSTATION
GENERAL PLAN
DWG. 11-1
Nov, 1983