AN ANALYSIS OF ALTERNATIVE DESIGNS FOR ALMUS DAM

AND

CRITICAL DISCUSSION OF THE DAM DESIGN SELECTED BY D.S.İ.

THESIS

SUBMITTED TO ROBERT COLLEGE IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN

CIVIL ENGINEERING

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ROBERT COLLEGE ENGINEERING SCHOOL

BEBEK - ISTANBUL

MAY I96I

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INTRODUCTION

In the industrial and agricultural development of Turkey, Power Generation and Irrigation will play a major role. Indeed, in the last few years continuous research has led to the discovery and utilisation of many different project areas, one of these being the Almus site which will benefit considerably by the dam now envisaged.

The initial planning and study for the Almus dam was first performed by the D.S.I. and then improved by E.I.E. The report of this investigation was presented in 1943. A second and more complete study of the project was concluded by the D.S.I. in 1953.

I chose this as the subject of my thesis as I am especially interested in hydraulics and hope to specialise in this field as a practising engineer.

In my thesis I have taken the topographic map of the dam site and accepted the choice of the location of the dam axis from this report. I have used to a great extent the information given in the Preliminary Project Report of the D.S.I., assuming to be true the detail contained therein to which I have referred in footnotes. As the cost calculations for the earth dam are already given in the D.S.I. report, it was necessary for me only to adjust the given figures in order to bring them up to date and fit them into the suggested improvements. I made use of all the literature (referred to in the bibliography) I could find on the subject, as well as my class notes.

In my criticism of the D.S.I. design I have made several alterations to the structure of the dam which, in my opinion, will give it a higher safety margin. However, the D.S.I. people have also

./.

stated their intention to review the stability of the cross-section after material characteristics are fully studied. I have assumed normal materials and conditions, but due to my lack of practical experience, there may well be elements in my criticism of design which would prove to be impractical, although theoretically in order. However, I have tried to keep these to a minimum by taking dams already in existence as examples to supplement by theory.

I can by no means say that a fully complete analysis of the alternative designs of the Almus Dam is submitted, but I have done my best in the time at my disposal.

I should like to express my sincere appreciation of the helpful advice given to me by Dr Pfeifer who is the Professor in charge of my thesis.

GENERAL DESCRIPTION OF THE REGION

Location and area :

The Upper Yeşilırmak region is located in the North-East of Central Anatolia. The region is bounded on the North and East by the Kelkit valley :,in the south by Kızılırmak and on the West by the Çorum River.

Topography, Geology, and Earthquakes :

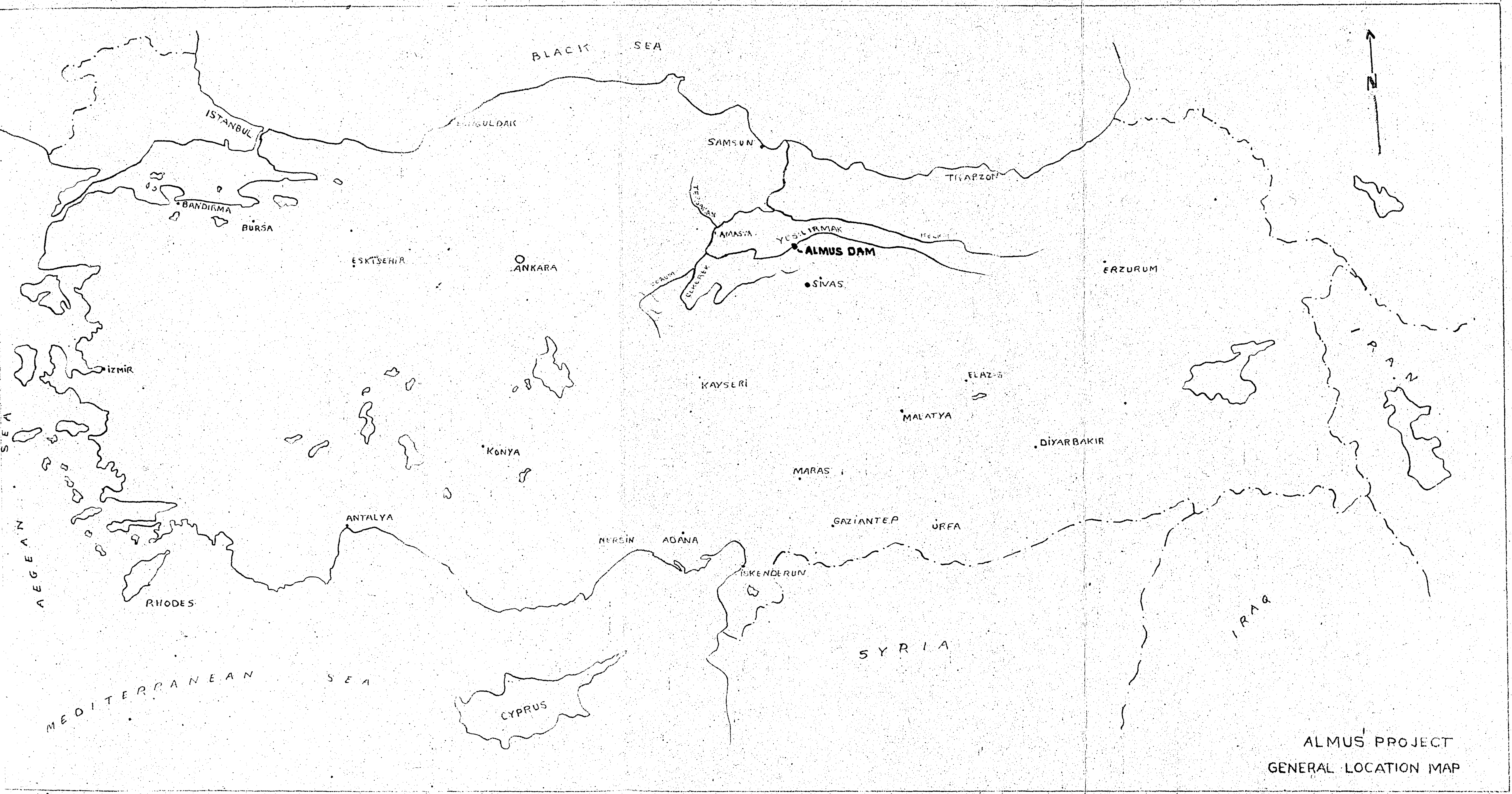
This is a fairly mountaneous region of Turkey although broad plains stretch between the hills. The area adjacent to the river is in general made up of fertile plains which receive relatively high precipitation. The most important plains within the area are Omala, Kazova, Turhal, Amasya, and part of the Geldingen plain.

The axis of the dam is on a very slight slope in a relatively wide gorge of andesite composition.

The project area has many earthquake zones. The section where the dam will be located lies within a zone rated as first degree of severity.

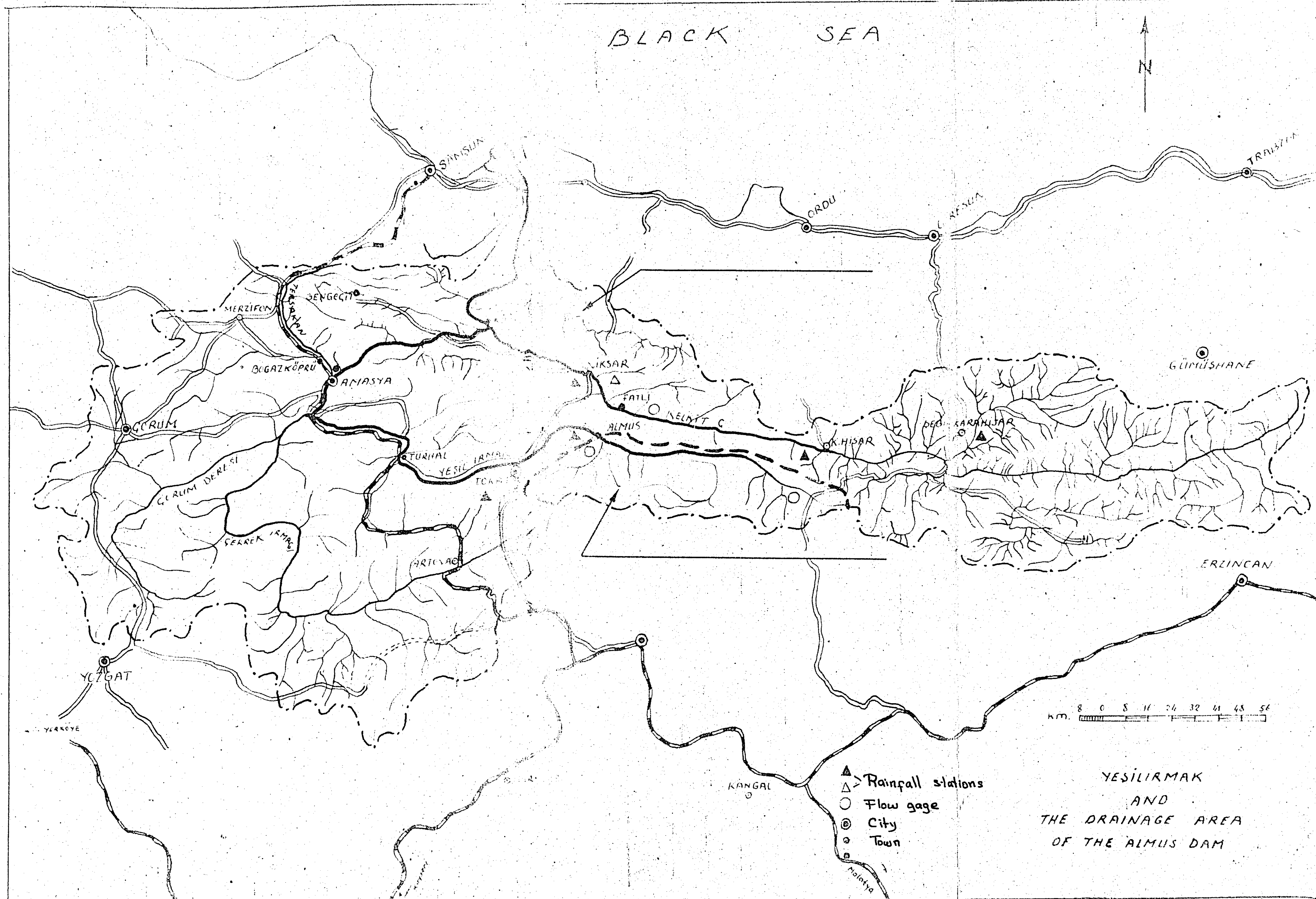
Meteorology :

Due to the lack of available stations that have the records of precipitation, one can not judge fairly the



ALMUS PROJECT

GENERAL LOCATION MAP


BLACK SEA
N
SAMSUN
ORDU
TRABZON
MERZIFON
BOGAZKÖPRÜ
AMASYA
ÇORUM
TURHAL
NIKSAR
FATLI
ALMUS
K.HISAR
KARAHISAR
GÜMÜSHANE
ERZINCAN
YOZGAT
KANGAL
ÇEKEREK IRMAGI
ÇORUM DERESI
YESIL IRMAK
ARTOVA
YERKÖYE
km. 8 0 8 16 24 32 40 48 56
Rainfall stations
Flow gage
City
Town
YESILIRMAK
AND
THE DRAINAGE AREA
OF THE ALMUS DAM

records not available

precipitation that falls on the Upper Yeşilırmak region. Records of precipitation are obtained from three stations (Tokat, Niksar, Fatlı) for the last fifteen years but the records from these stations do not indicate truly the precipitation that falls on the area that contributes to the Almus dam, as none of these stations are within the drainage area.

+2 more st... on map: Killisar and Almus

The preparation of an isohyetal map on this information will be meaningless because the map will not indicate the wide extremes that will make such averages. For example 24 years of record at Tokat shows an annual precipitation of 481 mm. but the range of extremes is from 198 mm. (1930) to 742 mm. (1939) .

The average annual isohyetals also would not indicate the fact that throughout that portion of the Yeşilırmak region below Tokat there are often periods of 30-60 days during the growing season when there is no measurable rainfall. hydrographs would show the...

From the short period of records available it is impossible to determine whether precipitation follows any sort of cyclic pattern.

why not those of all 5 st... shown on map

I include the rainfall datd at the Tokat station However I accept for my calculations an average rainfall of 900 mm. on the area as given in the D.S.I. report.

Why? On the basis of what reasoning?

Evaporation :

There is only one station for measurement of evaporation in the river basin. Average evaporation at this station ~~is~~ located at Tokat, covering a 17-year period, is approximately

RAINFALL DATA
TOKAT STATION (mm)

YEARS	MONTHS												
1945	54	54	76	71	40	51	5	6	28	29	9	22	441
1946	20	50	81	45	78	80	50	16	28	69	19	20	550
1947	76	50	49	9	49	16	7	1	38	28	78	32	403
1948	29	24	50	31	81	62	1	4	36	40	26	31	470
1949	57	79	29	114	58	68	22	2	29	30	21	61	542
1950	124	51	74	22	81	50	6	1	—	31	24	23	496
1951	34	40	42	49	98	30	7	68	22	62	30	58	528
1952	67	28	88	44	81	20	5	2	18	14	44	21	473
1953	49	68	45	39	63	41	8	8	29	29	60	31	463
1954	79	71	41	43	52	18	1	9	11	11	20	69	388
1956	28	34	41	80	30	3	21	8	12	30	38	79	418
1957	34	48	21	9	100	19	4	2	25	6	48	51	373
	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total

RAINFALL DATA
TOKAT STATION (mm)

YEARS	MONTHS											
1933	10	74	12	75	48	97	3	4	30	28	55	65	501
1934	38	56	14	77	50	57	6	4	6	53	15	17	389
1935	30	39	59	50	—	—	8	13	36	25	57	28	345
1936	40	71	39	70	96	102	60	30	9	20	50	47	634
1937	84	30	29	70	93	20	—	30	3	21	74	67	521
1938	20	8	30	71	19	7	19	3	41	8	32	86	344
1939	25	38	73	60	53	115	63	63	47	40	85	80	742
1940	58	73	61	77	107	80	4	8	9	58	18	99	642
1941	87	47	89	26	20	17	21	30	6	42	83	48	516
1942	54	17	41	82	68	21	17	6	32	103	104	37	642
1943	60	33	28	40	47	42	5	2	9	38	38	59	331
1944	68	48	60	36	50	47	12	4	12	30	58	16	481
	Jan	Feb.	Mar	Apr.	May	June	July	Aug	Sept	Oct	Nov.	Dec	TOTAL

How factors taken into consideration

9I5 mm. However I believe that the use of this figure can not be relied on, because I do not think that evaporation rate at the reservoir site on the lower river would be as high as those at Tokat.

Cities and population :

This — in this isolated manner — is meaningless

The Almus dam project is a multipurpose project and it would be useful to consider the number of people related to different purposes seperately.

People in the project area live scattered in small towns and villages. Number of people that will make use of the irrigation project is estimated to be 60,000.

Customers for electricity

After comparison of the previous population records, and by using the Uniform percentage growth method ,(1) I came to the conclusion that the population will be 72,000 in I975 and 88,000 in I990.

Seems very low. Population increase in Turkey at present at the rate of 2.9% per year. = ~ 90000 in 1975

Railroads and Highways :

Transportation ~~does not~~ is not cause a problem in this region, as the Samsun-Sivas railway passes through Amasya and Turhal and will afford the necessary facilities.

Sivas- Tokat highway is the principal highway that would be used for construction transportation ; the route has water-bound macadam and gravel surfaces.

Transportation is open to traffic all year. This will enable the use of the Samsun port.

Labor and Wages :

Sufficient unskilled labor is available for the project under discussion, and there are some skilled craftsmen in the area. I believe that when an earth dam is considered, however , additional skilled men will be required, particularlyeequipment operators and mechanics.

Current wages are :(2)

	T.L./day
Laborer	8–I2
Carpenter	I8– 25
Mechanic	26– 30
Driver	I8– 26
Foreman	27– 40
Bookkeeper	24– 35
Engineer	53– 80

no paid vac.
no unemployment insurance

In addition to daily wages approximately IO–I5% should be added to cover vacation, sick leave, accident insurance and workers' compensation.

Most of Turkey's requirement in gasoline, diesel oil and lubricants are being imported. All major equipment has to be imported and the majority of it will probably be coming from the United States and Germany. 2

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(1) Elements of Hydraulic Eng'n Linsley and Franzini
Mc. Graw Hill I955

(2) Information obtained from T.C. İŞ ve işci bulma kurumu

Meteorology from page 1/3 belongs here too

HYDROLOGY

DRAINAGE AREA :

The drainage area of Yeşilırmak, a map of the river with its tributaries on a scale of I:800,000 is taken from the preliminary project report of D.S.İ.

The total drainage area of Yeşilırmak is 36II4 km^2 but the drainage area that contributes to the Almus dam is 2337 km^2 and the average rainfall on this area is 900* mm.

FLOOD FLOW :

The maximum flood to be accomodated from a catchment area may be approximately determined by several different ways. I have used two methods, namely The Basic Stage method and the Yearly Flood method but I have also kept in mind the fact that a few stream flow records are not sufficient to be truly representative of average conditions and made a comparison of the record flows of other catchment areas of about the same size and characteristics. For comparison I have made use of the enveloping curves of Creager. A knowledge of the physical factprs affecting the magnitude of floods is necessary in order that a logical comparison can be made

The most important of these factors are:

a) Type of storms

b) Presence of lakes, swamps, and ground storage

c) Characteristics of the river bed and banks

d) Shape and slope of the drainage area

* DSI Report Estimation

b= I.85 P = 7 years

X = 325 m^3/ sec b = 3.5 t_p= 33.3 years

check point

X =290 m^3/ sec b = 2.72 t_p= I5.2 years

" The Gumbel method is based on sound statistical principles and has been checked with data from stations having long periods of records." (2)

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(1) This method is also called GUMBEL method.

(2) Elements of Hydraulic Engineering
Linsley and Franzini, Mc. Graw Hill I955

e) Forests and vegetation

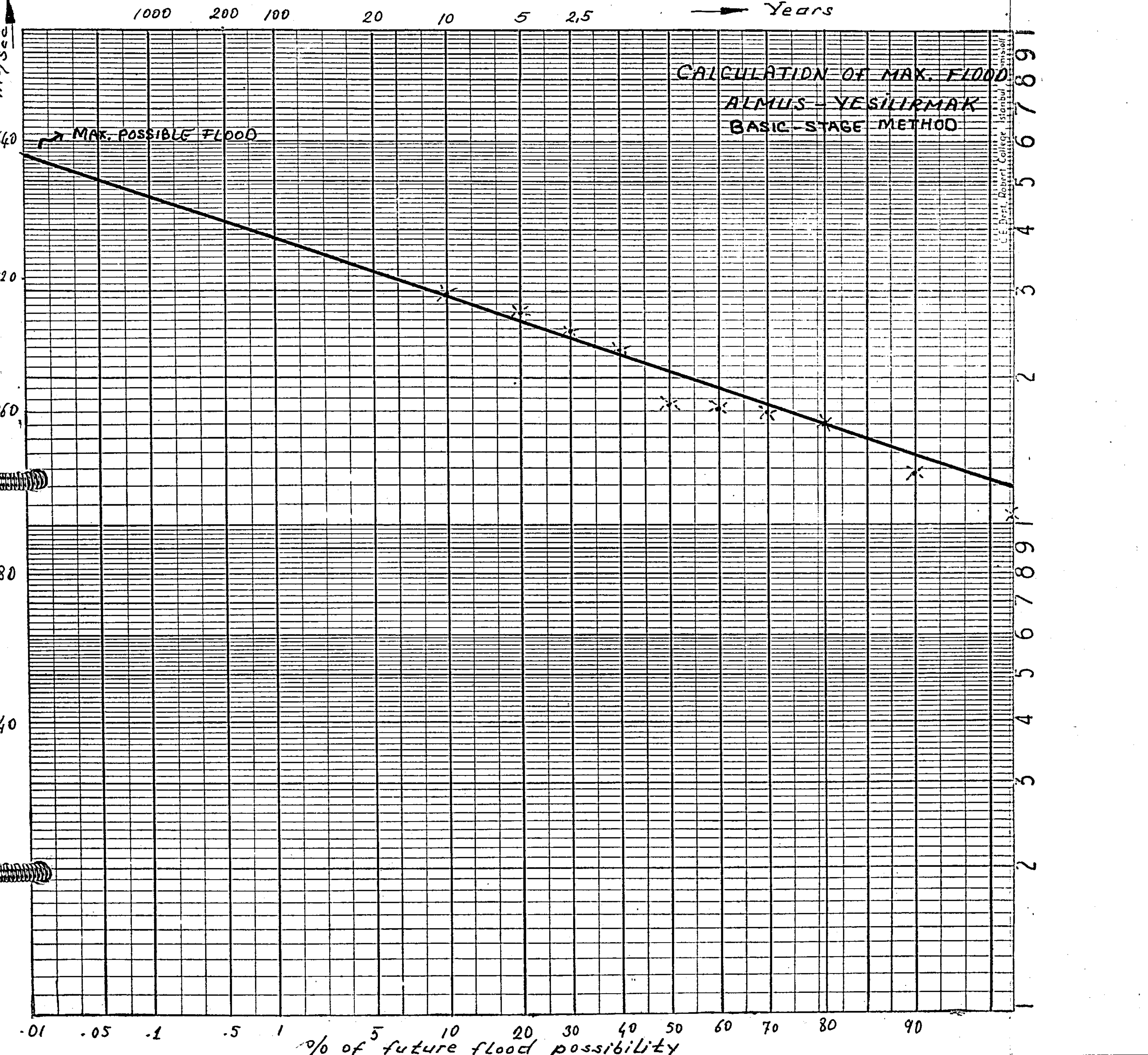
f) Freezing conditions of the soil

Physical indications of past floods would also be very helpfull but I could not find any data about the historical floods on this area.

BASIC STAGE METHOD

X m^3/sec	m	n	% of future flood possibility
102	1	10	100
130	1	9	90
156	1	8	80
176	1	7	70
180	1	6	60
182	1	5	50
200	1	4	40
225	1	3	30
275	1	2	20
300	1	1	10

P.S. Notation: same as is used in text.
Elements of Hyd. Eng'n
Linsley + Franzini



Years
1000 200 100 20 10 5 2,5
CALCULATION OF MAX. FLOOD
ALMUS – YEŞİLIRMAK
BASIC-STAGE METHOD
MAX. POSSIBLE FLOOD
40
20
60
80
40
.01 .05 .1 .5 1 5 10 20 30 40 50 60 70 80 90
% of future flood possibility

THE ANNUAL FLOOD METHOD (1)

Year	Peak Flow m^3/sec X	t_p	m	$\bar{X}-\bar{X}$	$(X-\bar{X})^2$
I953	300	I0.00	I	I07.4	II500
I952	275	5.00	2	82.4	6800
I955	225	3.33	3	32.4	820
I954	200	2.50	4	7.4	50
I948	I82	2.00	5	-I0.6	I20
I95I	I80	I.66	6	-I2.6	I50
I950	I76	I.43	7	-I6.6	280
I949	I56	I.25	8	-36.6	I340
I956	I30	I.II	9	-62.6	3900
I947	I02	I.00	I0	-90.6	8200
	I926 = ΣX				33I60 = $\Sigma(X-\bar{X})^2$

$\bar{X}$ = I92.6 m^3/sec

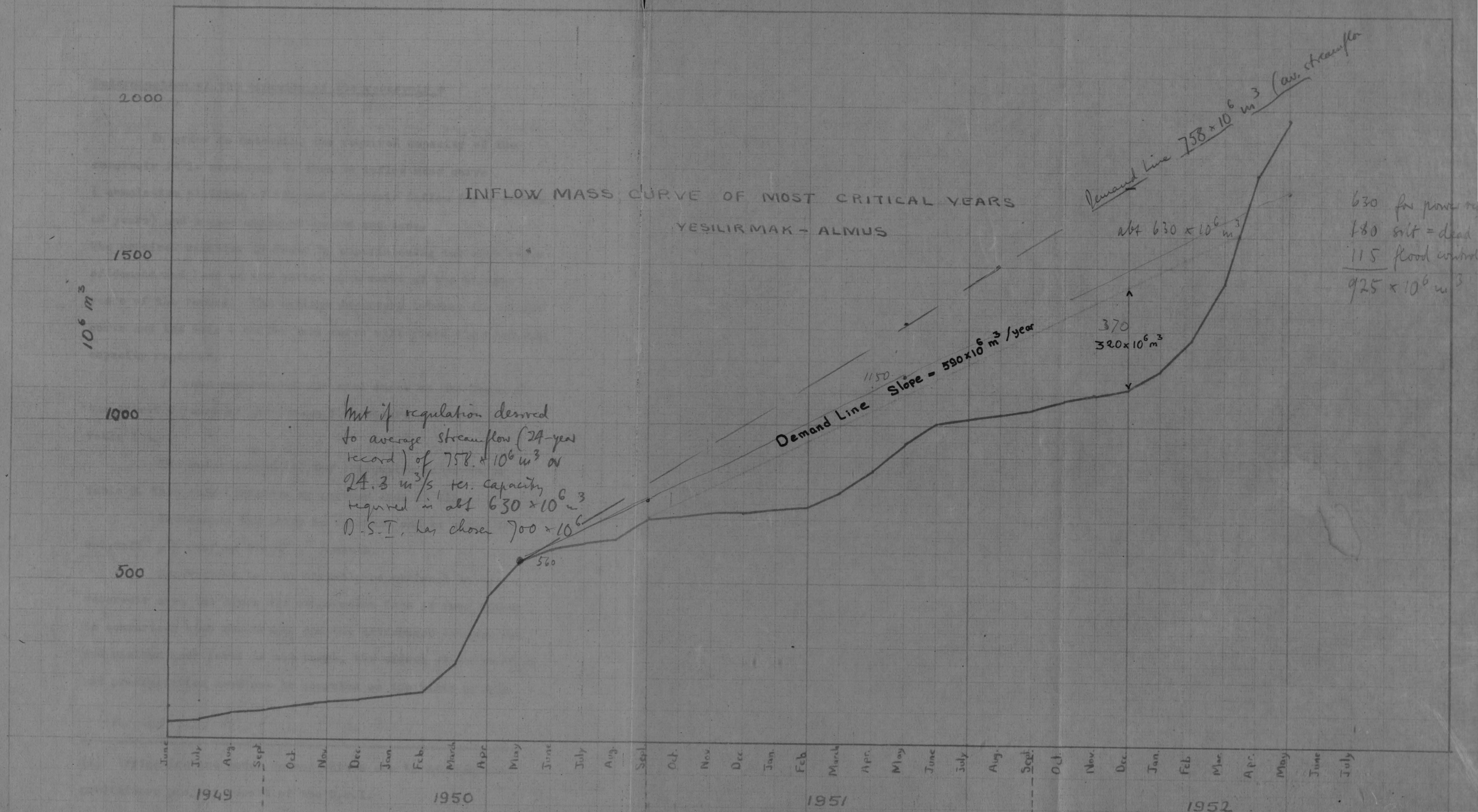
Standard deviation of the series $\sigma = \sqrt{\frac{\Sigma(X-\bar{X})^2}{N}}$ $\sqrt{3316}$ = 58

$b = \frac{I}{.78\sigma}(X-\bar{X}+.45\sigma)$ $\quad P = 1 - e^{-e^{-b}}$

X is the flood magnitude with the probability P

$\bar{X}$ is the arithmetic average of all floods in the series

when X = 250 m^3/sec



INFLOW MASS CURVE OF MOST CRITICAL YEARS
YESILIRMAK - ALMUS
10^6 m^3
2000
1500
1000
500
Demand Line 758 x 10^6 m^3 (av. streamflo
abt 630 x 10^6 m^3
630 for power re
180 silt = dead
115 flood control
925 x 10^6 m^3
Demand Line Slope = 590 x 10^6 m^3 /year
1150
370
320 x 10^6 m^3
But if regulation desired
to average streamflow (24-year
record) of 758 x 10^6 m^3 or
24.3 m^3/s, res. capacity
required is abt 630 x 10^6 m^3
D.S.I. has chosen 700 x 10^6
560
June July Aug. Sept Oct. Nov Dec. Jan. Feb. March Apr. May June July Aug. Sept Oct. Nov. Dec Jan. Feb March Apr. May June July Aug. Sept Oct. Nov. Dec Jan. Feb Mar. Apr. May June July
1949
1950
1951
1952

Determination of the capacity of the reservoir :

List and describe streamflow gaging station or statio[ns]
Compare records with those of rainfall gaging stations in a[rea]

In order to determine the required capacity of the reservoir it is necessary to draw an inflow mass curve (cumulative plotting of the net reservoir inflow for a period of years) and a mass curve of demand and loss.
The required capacity is found by superimposing the mass curve of demand and loss on the inflow mass curve of the driest years of the record. The maximum departure between the demand curve and the loss + inflow mass curve will yield the reservoir capacity required.

I have constructed the mass curve on the basis of the driest 3 years of the stream flow record submitted in Table I A.

The water demand (1) for irrigation is shown in Table B. This table includes an assumed loss of II%. — where?

The demand for power is constant and is equal to $589.3 \times 10^{6}\ m^{3}$/ year or $49 \times 10^{6}\ m^{3}$ / month.

Where from? how is this relat[ed] to average inflow of 758 ×10[...]

Evaporation loss is directly proportional to the reservoir area, but since the evaporation loss is very minor in comparison with the demand and the difference between max. and minimum pool level is not large, the effect of evaporation and precipitation need not be computed on the basis of the

I do not find this figure in the D.S.I. report, and cannot understan[d] how it w[...] make s[...] compared, [...]

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(1) Irrigation and power demand values are taken from the preliminary project report of the D.S.I.

TABLE A

Water Years	Months	Inflow	Cumulative Mass curve ordinates
1949	June	50.7	50.7
	July	19.3	69.0
	A	10.8	79.8
	S	10.3	90.1
1950	O	13.4	103.5
	N	11.3	114.3
	D	10.6	125.4
	J	12.3	137.7
	F	15.2	152.9
	M	91.1	243.0
	A	207.2	450.2
	M	113.1	563.3
	J	37.8	601.1
	J	17.6	618.7
	A	10.8	629.5
	S	69	698.5
1951	O	8.5	706.0
	N	7.7	713.7
	D	8.5	721.2
	J	8.7	729.9
	F	8.0	737.9
	M	51.0	788.9
	A	67.5	856.4
	M	89.1	945.5
	J	59.8	1005.3
	J	17.7	1022.0
	A	13	1035.0
	S	13.2	1048.2
1952	O	28.8	1077.0
	N	19.2	1096.2
	D	22.5	1118.7
	J	60.0	1178.7
	F	103.0	1281.7
	M	166.0	1447.7
	A	344.0	1791.7
	M	178.0	1969.7

Months	TABLE B	TABLE C	TABLE D
	IRRIGATION DEMAND	EVAPORATION	PRECIPITATION
Jan.		.1	.6
Feb.		.2	.6
March		.3	.7
April	4.8	.7	1.1
May	7.4	1.2	1.2
June	30.4	2.0	1.0
July	45.9	2.7	.2
August	22.5	2.4	.1
Sept.	22.5	1.9	.3
Oct.	8.2	1.0	.6
Nov.		.3	.7
Dec.		.2	.5
TOTAL	$161.7 \times 10^6\ m^3/year$	$13.0 \times 10^6\ m^3/year$	$7.7 \times 10^6\ m^3/year$

My calculations total to $177 \times 10^6\ m^3$

[illegible]

(18)

estimated water surface elevation for each month.
The values given in table C and D are found on the basis of an average constant reservoir area.
The gain through precipitation on the reservoir is also of little importance compared with the total demand, however I have included the precipitation figures. Table D

The monthly water requirement for power is greater than the maximum irrigation demand of any month, therefore the quantity of water necessary for irrigation demand is assumed to be used completely by the power plant first.

With the assumptions made the rate of demand may be considered uniform and equal to the demand for power.
Due to the very minor effect of the difference between evaporation and the precipitation this will not include any objectionable error.

Demand curves representing a uniform rate of demand are straight lines having a slope equal to the demand rate.

In order to provide the necessary amount of water for the driest three years of the record the mass curve shows that a useful storage of 320×10^6 m^3 of capacity is required. 370×10^6 m^3 see graph next page

The problem of sedimentation is very serious in every region of Turkey and the most advisable procedure in dealing with the sedimentation problem is to designate a portion of the reservoir as sediment storage.

The frequency curves constructed by two differet methods do not show a significant variation in results.

Maximum possible flood found by the basic stage method is $580 m^3/sec$ and by the annual flood method is $530 m^3/sec$.

Comparison with the enveloping curves of Craeger:

According to Craeger's enveloping curves, for a drainage area of $2337\ km^2$, the maximum flood flow may be as high as $2800-10,000 m^3/sec$

The values of Craeger's enveloping curves are too high compared with the values I found by the other two methods. Therefore it would be advisable to have a further check; the precipitation on the drainage area.

The precipitation (average) is 900 mm. on an area of $2337\ km^2$. This will give a total precipitation of

$$.9 \times 2337 \times 10^{9} = 2.1 \times 10^{6}\ M^3/\ year$$

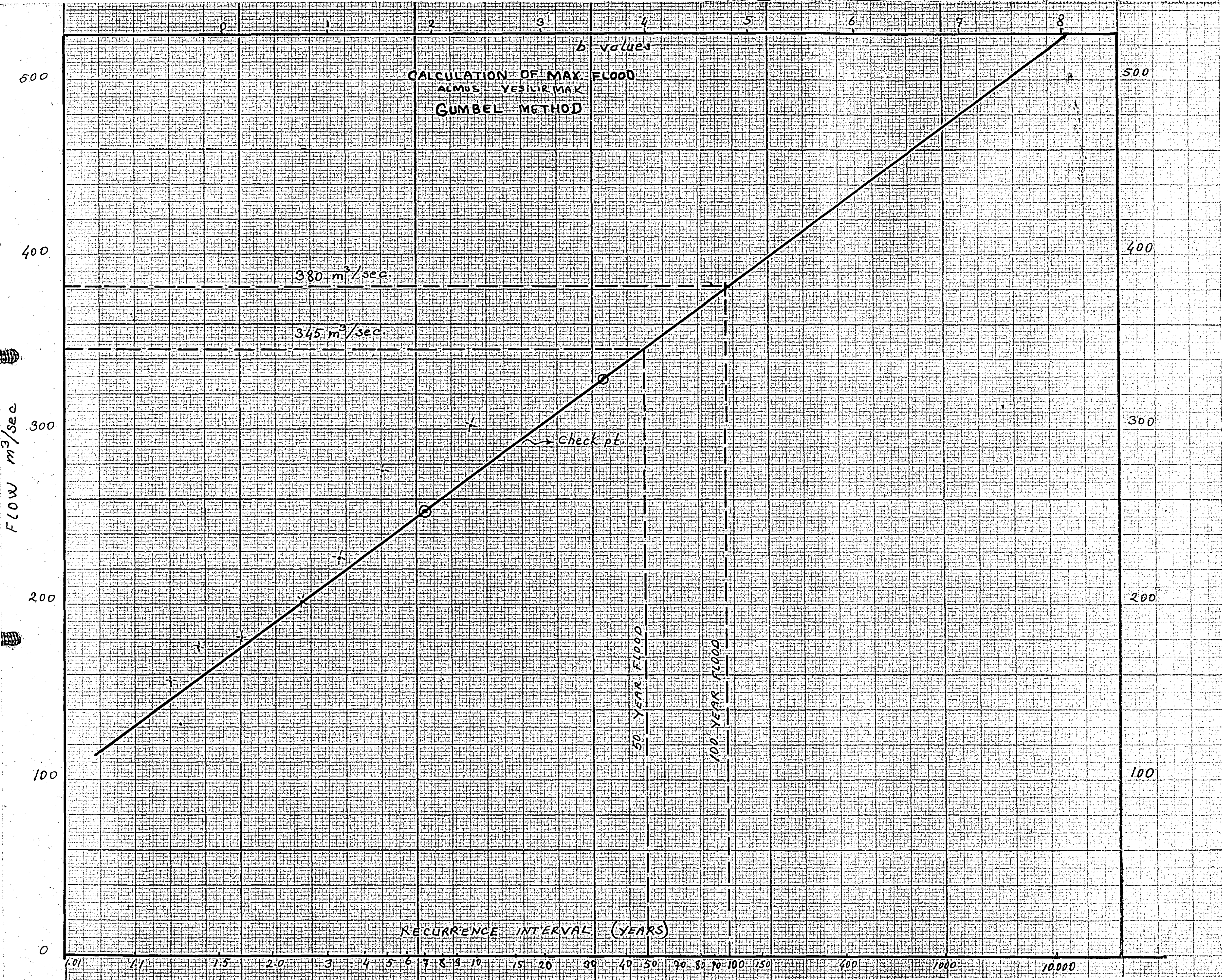
Assuming that a maximum flood will cause 10% of the total rainfall in 24 hours and 80% of this precipitation will constitude the streamflow, (This should be is an extremely safe conservative assumption) the maximum flood flow will be

$$\frac{2.1 \times 10^{6} \times .8 \times .1}{24 \times 3600} = 2,000\ m^3/\ sec$$

The preliminary project report of D.S.I. has assumed a max. flood of $2243\ m^3/\ sec$, But I will use $2,000\ m^3/\ sec$ in my design.

based on an actual record of a 91 mm rainstorm during 24 hours in 1955 (DSI report page 86) and construction of a flood hydrograph on a basis that I do not understand.

Believe this should be page 16 = flood flow



CALCULATION OF MAX. FLOOD
ALMUS - YESILIRMAK
GUMBEL METHOD
b values
0 1 2 3 4 5 6 7 8
FLOW m³/sec
500
400
300
200
100
0
380 m³/sec.
345 m³/sec.
Check pt.
50 YEAR FLOOD
100 YEAR FLOOD
RECURRENCE INTERVAL (YEARS)
1.01 1.1 1.5 2.0 3 4 5 6 7 8 9 10 15 20 30 40 50 70 80 90 100 150 400 1000 10.000

In the absence of the necessary data for the sediment inflow rate at the dam site the thing to be done is to assume a safe (conservative) value.

The inflow rate of sediment is assumed to be I% by weight of the total inflow. No measurements whatever? DSI report

A method is suggested by Linsley to find the probable life of a reservoir limited by the sedimentation. This method can also be used in determining the capacity required to designate as sediment storage.

Since the capacity-inflow rate is quite small I use a trap efficiency of 90% and assume that 25% of the silt will be washed through the sluices provided. Taking the specific gravity of the sediment deposits as I.4 , the silt deposits will cover a total volume of

$$\frac{758.8 \times 10^6 \times .01 \times .75 \times 50 \times .9}{1.4} = 182 \times 10^6 \ m^3$$

in 50 years. = Considerably larger than DSI, which is probably a good idea, but reason?

Accepting a dead storage of $180 \times 10^6 \ m^3$ and an additional reserve of $68 \times 10^6 \ m^3$, the total normal capacity of the reservoir is $570 \times 10^6 \ m^3$.

The elevation capacity curve shows that in order to maintain this capacity the normal pool level is to be at an elevation of 790 meters.

Allowing a volume of 100×10^6 m^3 for partial flood control the maximum capacity of the reservoir will be 670×10^6 m^3 at an elevation of 793 meters.

discuss how obtained

If the engineer in charge is informed of an approaching great flood 36 hours in advance by the meteorology station, the proper operation of the sluices will permit an additional flood control volume of

which? rely in gravity dam

(1)

$$\frac{33{,}000 \times 3600 \times 36}{35.3} = 122 \times 10^6 \ \text{m3}$$

hardly possible – the drainage area measures only 2337 km² or maybe 30 × 80 km. Flood flow will take only 2–10 hrs to reach dam site, depending on where storm center was.

Furthermore: this figure is meaningless without relation to expected total volume.

It is accepted practice not to rely on such gates in an emergency, because they may jam, and we must here protect an extremely vulnerable dam.

The total discharge through the sluices is found on page 57.

advance notice of approaching storm from other watershed

DISCUSSION OF THE SUITABILITY OF VARIOUS TYPES OF DAMS :

The choice of the type best suited to a particular location or use is a matter on which even very experienced engineers will differ considerably, and is to a great extent a matter of judgement and experience, however an intelligent study of existing conditions and requirements is of course the best guide for the choice.

Safety should be the main consideration in the design. The cost of the dam, affected by the availability and price of construction materials is also of prime importance.

I have made a comparison of the usual types of dams and stated my opinion of the suitability of the type under consideration.

The usual types of dams are classified as follows;

I.) Concrete arch dams

2.) Concrete hollow gravity dams

3.) Concrete solid gravity dams

4.) Earth and rock embankments

Concrete arch dam ;

The weight of the arch dam is not counted on to assist in the resistance of external loads. The strength of an arch dam depends on the sides of the valley which must be composed of good rock that will resist the end thrust.

Arch dams are only adaptable if the dam length is small in proportion to the height.

The design of an arch dam is ~~very~~ complicated, and many skilled workers are necessary for the construction.

Considering all these factors I do not believe that the arch dam will be a suitable choice because;

a) The shape of the gorge is so that the ratio of length to height is large.

b) The sides of the valley, though composed of rock, are formed in layers, and the rock is decomposed; therefore it will not be ~~very~~ safe without extensive grouting, which will be ~~very~~ expensive.

c) Though unskilled labor is available, training of additional skilled men will be required, which will take a long time and be very costly.

Concrete hollow gravity dam ;

Hollow dams, being lighter per square foot of the area, exert less unit pressure on the foundation than solid dams.

For this reason the hollow dam is sometimes adopted where a strong foundation for a solid dam is lacking.

Due to the high reinforcement required this type of dam requires a great number of skilled workers.

A study of the existing hollow dams shows that this type of dams is not practiced if the height of the dam is over 50 m.

The construction of a hollow ~~gravity~~ buttress dam in an active earthquake zone, is ~~quite dangerous~~ not recommended.

Since the dam will rest on a rock foundation, the unit pressuree on the foundation will not be a governing factor; on the other hand, steel and steelwork is very expensive, and the quality of concrete used in a hollow dam must be very good which requires careful inspection, therefore is costly.

On the whole I would think that a hollow dam will be ~~inconvenient and uneconomical to build~~ inadvisable for in this location.

Solid gravity dam ;

A solid gravity dam is adaptable to all localities ~~except~~ where the foundations consist of rock strong enough to resist the high pressures of the mass concrete, except where there is a great uplift possibility. ~~It is however necessary that this kind of dam be built on rock foundations.~~

Availability of cheap cement, plentiful rock for aggregates, and extensive sand deposits

A ~~cement factory, a good quarry and a convenient sand bank~~ near the location of the gravity dam are absolutely necessary in order that this type of dam may be considered.

In my opinion in ~~this~~ our case a concrete gravity dam may be considered as one of the alternatives, for:

a) ~~Rock foundation~~ Bed rock was found lies at an average depth of seven meters ~~under~~ below the alluvial soil of the valley bottom.

b)The railway station connecting the Sivas cement factory to the site is very near. The factory itself is only — km away.

c)A sandbank is only I.5 kms. away from the dam site.

d) A good quarry is available in the vicinity of the dam.

Earth and rock embankments ;

If there are sufficient materials available in the vicinity, earth and rock fill dams can be built at a low cost; however a knowledge of the flood flows of the stream on which an earth dam is proposed is of great importance. The overtopping of a non-overflow concrete dam may cause some damage, but overtopping of an earth dam is disastrous. Therefore, ~~a~~ the spillway for an embankment dam may have to be conservatively designed for considerably more capacity than that of a concrete dam, if accurate flood records are not available ~~of more suitable character is a necessary adjunct.~~

~~The question of spillway requirements is of paramount importance~~ This question may be of decisive importance in determining whether or not it is economical to construct an earth dam at any location.

In the neighborhood of the accepted dam site the necessary materials for the construction of an earth dam are available but the spillway problem may present a serious problem since we do not have a long record of streamflows.

A high freeboard must be allowed as well as a great spillway capacity to compensate the danger of overtopping.

To sum up, I would say that, the earth dam has some ~~very~~ distinct advantages over the concrete gravity dam for this site, for:

1.) Most of the material necessary for the fill is to be found only at an average distance of one kilometer.

2.) It ~~may~~ will not be necessary to excavate to solid rock ~~in case of an earth dam~~, which will reduce the cost.

3.) The earthquake problem will be less serious for an earth-dam than it would for a concrete dam.

4.) If a complete flood control is wanted, ~~this will result in a greater capacity and the earth dam will most likely be more economical, because the~~ the necessary capacity should be increased, and a gravity dam will become ~~very~~ uneconomic.

expensive owing to the great mass required to resist
~~heavy due to greater~~ uplift and earthquake, ~~effect.~~

On the other hand the gravity dam has the following obvious advantages:

1.) The seepage ~~loss~~ losses and danger will be practically eliminated by the construction of a concrete gravity dam.

2.) The ~~spillway will represent no problem.~~ provision of a spillway will represent no problem.

3.) Maintanence costs of a solid gravity dam are much less than ~~the~~ maintanence costs of an earth dam.

4.) The time required to build a gravity dam will be considerably shorter. ? depends on construction plant available

5.) Less construction equipment will be required in the construction, which means less expenditure of foreign currency.

5.) Sedimentation ~~will be washed~~ can be washed to some extent through the ~~provided~~ sluices provided in a concrete dam, which will result in an increase of the useful life of the reservoir.

We see that both the concrete gravity dam and the earth dam have some (very) distinct advantages over eachother and at this point it is impossible to come to a definite conclusion as to the choice of the ~~more~~ most suitable and economical type.

In the following pages I will make a preliminary design of a concrete dam in order to compare the cost of this dam with the cost of the suggested earth dam design of D.S.I. The suggested D.S.I. design is for a larger dam ~~with a~~ to provide for a maximum capacity of 950×10^{6} m^{3}, but I will be able to estimate the cost of an earth dam of ~~equal~~ smaller height with the use of the improved, up-to-date D.S.I. figures

After the cost calculations I will state which of the two I would prefer.

I will also include a critical discussion of the suggested D.S.İ. design.

FORCES ACTING ON THE CONCRETE GRAVITY DAM :

The forces acting on a dam consist of the following;

I.) External earth pressure

2.) silt Earth pressure

3.) Uplift (water pressure through foundation)

4.) Ice pressure

5.) Earth quake forces

6.) Wind pressure

7.) Wave pressure

These forces must be resisted and counterbalanced solely by the weight of the dam

~~8.) Weight of the dam~~

In the design of the concrete gravity dam I have not accounted for silt earth pressure, wind pressure, and wave pressure in the stability calculations for the following reasons.

sediment Earth pressures have a minor effect on the stability of the structure and since in the design I will give a place to the sluice construction in the lower part of the dam which will periodically flush, the depth of such deposits will be limited.

Ice pressure; The climate of the region where the reservoir is located is quite mild, and there is no reason to believe that the water will be frozen to a degree which will require attention in the consideration of the design.

Wind pressure; Wind pressure is seldom a factor in the design of any dam because dams are almost always in sheltered locations, and even if they are not, the maximum possible wind pressures are very small in comparison with the loads for which the dam is designed.

Wave pressure; The upper portion of the dam is exposed to the impact of waves, but the pressure exerted on the structure is of little importance since the fetch distance is small, however, provisions will be made for the wave height in the free-board consideration.

A. External water pressure;

The density of fresh water is 1000 kg per cubic meter. The total pressure of quiet water on a submerged rectangular area A is $\gamma AH/2$ where γ is the density of water. The force is normal to the surface.

B. Uplift

Uplift is the upward force exerted by the water that percolates through a dam. This occurs at all levels. At the heel of any section the pore water pressure corresponds to the head of water above the section. The result is a reduction in the effective weight of the structure above it, which is undesirable.

In 1952 a report has been published by a Sub-Committee of the American Society of Civil Engineers who reviewed the data available for masonry dams. Unfortunately, the evidence was not sufficient for the Committee to prepare a specific code for dealing with the effects of uplift. However the majority had the opinion that when designing a new dam it was advisable to assume that the uplift pressure affected the whole area of the dam at any level and it would be advisable to assume that this pressure varied linearly from full hydrostatic head at the heel to the actual pressure at the toe.

C. Earth quake forces;

I have carefully studied the local conditions and particularly the seismographical history of the region and found that the site is very close to the known active faults. Therefore the dam must be so designed that it can safely resist the inertia effects caused by the sudden movements of the earth's crust.

" In regions of known earth quake activity, accelerations of .Ig are usually assumed to act on the dam. This value of .Ig seems to be standard for dams in seismically active regions"(1) The horizontal force Em on the concrete block acts at the center of gravity of the block and is equal to CW.

In the " Discussion of pressures on a dam during earthquakes Trans. ASCE Vol. 98 , 1933. T. Von Karman suggests that the increase in the hydrostatic pressure on the dam be computed from $Ew=.555\ kH^2$ where k is the ratio of acceleration caused by an earthquake to that of gravity. The force Ew acts at the distance $4H/3\pi$ above the bottom of the reservoir.

"An earthquake movement may take place in any direction, but for a gravity dam with reservoir full the most unfavorable direction is upstream normal to the axis.

---

(1) Elements of Hydraulic Engineering, Linsley and Franzini Mc. Graw Hill 1955

The corresponding force acts downstream. For reservoir empty a downstream acceleration is more unfavorable. Upstream

A vertical acceleration changes the weight of the concrete and the water in the same ratio. Considering these elements alone the resultant is not displaced from the position it would occupy if there were no earthquake. However the stresses are changed. If the acceleration is upward the stress is equal to the no-earthquake stress multiplied by $(I+\alpha)$. which is generally less than the stress for an equal horizontal acceleration. If the acceleration is downward, the multiplier is $(I-\alpha)$. For small deviations from the horizontal the maximum stress may be slightly greater than for a horizontal acceleration of equal value but the uncertanities in the value of α is greater than the difference in the stresses." (1)

In the design, in order to be on the safe side, I will adopt an earthquake acceleration of .I5g and also assume that the earthquake and the highest water act at the same time.

I will also allow a I0% increase in the maximum existing stresses for compensation of a deviation from the horizontal acceleration.

D. <u>The weight of the dam;</u>

The unit weight of the concrete varies slightly depending on the ingredients but in the absence of exact information I will assume 2400 kg./ m^3 for the design weight of concrete, which conforms to modern practice.

---

(1) Engineering for Dams, Creager Vol. 2 John Wiley and sons, I945

E. Reaction of the foundation

If we let ΣV be the resultant of all vertical forces acting on the dam above the foundation and ΣH , the resultant of all horizontal forces, the resultant of ΣV and ΣH will represent the resultant of forces acting.

In order that static equilibrium is established this resultant R must be balanced by an equal and opposite reaction of the foundation.

Therefore the total vertical reaction of the foundation will be ΣV and the total horizontal shear or friction equal to ΣH.

DESIGN ASSUMPTIONS :

I. All loads are carried by the gravity action, or weight, of vertical, parallel-side cantilevers which receive no support from the adjacent elements on either side.

2. Unit vertical pressures, or normal stresses on horizontal planes, are assumed to vary uniformly as a straight line from the upstream face to the downstream face.

3. Horizontal shear stresses are assumed to have a parabolic variation across horizontal planes from the upstream face to the downstream face of the dam.

REQUIREMENTS FOR STABILITY :

The principal factors in safeguarding gravity dams are to ensure, under all kinds of possible loading ,

1. The principal vertical and inclined stresses at anypoint in the dam or the foundation shall not exceed the safe value.

2. At no point in the structure, tensile stress shall be allowed. (Since this criterion means that the resultant force within the middle third of the base, the danger of overturning is automatically eliminated.)

3. The tangent of the angle between the vertical and the resultant of all forces acting on the dam above any horizontal plane shall not exceed the allowable coefficient of friction at that plane.

4. All assumptions of forces acting on the dam shall be on the safe side and unit stresses will provide ample margin against rupture.

With regard to overturning, a gravity dam will be stable against overturning if stabilty is obtained against sliding, if the stresses are within allowable limits, and if adequate measures are taken to secure a stable foundation. For this reason the computation of an overturning factor of safety is unnecessary.

CRITERIA FOR DESIGN :

Allowable working stresses:

" Under normal loading conditions an allowable compressive stress of 400-600 psi (280-420 T/m^2) is generally specified for massive concrete dams. The max. allowable shear stress is generally specified as 200-300 psi (I40-210 T/m^2) or 50% of the allowable compressive stress." (1)

The allowable stresses that I will use in the design will be conservative in order that safety and permanence is secured. An allowable compressive stress of 280 T/m^2 and an allowable shear stress of I40 T/m^2 are specified.

" For dams built on poor rock, the sliding factor is generally used as the criterion for sliding. Since it includes no resistance other than friction this factor should not exceed.8"(1)

Foundation pressure:

" In general, a value of 2.0 or more is a desirable ratio of foundation strength to allowable bearing pressure. Factors of safety for pressures of conrete dams on rock foundations are of large magnitude.

The average compressive strength of rock foundations range from I000-20,000psi "(2)

The rock foundation of the Almus dam is relatively poor, but even the lowest allowable compressive strength value will provide the desirable ratio of 2, as the max. compressive stress allowed in the concrete will not surpass 400 psi.

Test values ?

---

(1) "Working stresses for axially loaded concrete"

Lab. Report No C-277 U.S.D.I.B.R. Denver, Colorado

(2) Earth Manual, U.S.D.I.B.R. Denver, Colorado I95I

SYMBOLS USED :

NOTATIONS :

H.......Horizontal component of the reservoir load.

U.......Total uplift force on horizontal section.

W.......Dead load weight above section

Ew......Increase in unit water pressure caused by earthquake

Em......The inertia force caused by the earthquake on the dam

ΣV.....Algebratic summation of the vertical components of all forces, including uplift but exclusive of the reaction of the joint.

ΣH.....Resultant horizontal force above section

Tan θ ..Angle of inclination of the resultant R of the forces with the vertical

e.......Eccentricity, distance from the center of gravity of a section to the resultant force

γ_w ...Unit weight of water 1T/m^3

γ_c ...Unit weight of concrete 2.4 T/m^3

$\bar{X}$......Distance, in meters, of the resultant of all forces acting on a section, from the lowest pointof the downstream face of the section

$\bar{\bar{X}}$..... Distance , in meters , of the centroid of the dam from the lowest point of the downstream face of the section

Σ-M...Summation of the overturning moments

Σ+M...Summation of the righting moments

σ....Normal stresses at the extreme fibers

σ'....Stresses parallel to the face of the dam

τ ....Average shearing stress along the plane between the blocks

FREEBOARD :

Freeboard is the vertical distance between the crest of the dam and the maximum reservoir elevation that would be attained during spillway design flood.

The amount of freeboard is generally determined after a consideration of the following factors:

I. Maximum wave height

2. Relation of spillway capacity to flood runoff

3. Steepness of upstream face.

Since the upper portion of the upstream face that will be exposed to the wave action is vertical and the design capacity of the spillway is very conservative, these factors are of little importance.

The height of waves depend on the reservoir fetch and wind velocity. The maximum wave height corresponding to an assumed wind velocity of 60 mi/hr and a reach of 4 miles is calculated by the Stevenson-Molitor formula

$$Z_w = .17\sqrt{V_w L} + 2.5 - \sqrt[4]{L}$$

where V_w is the wind velocity in mi/hr

L is the fetch

Z_w is the wave height in feet

$$Z_w = .17\sqrt{60 \times 4} + 2.5 - \sqrt[4]{4} = 3.83 \text{ FEET} = 1.15 \text{ meters}$$

A freeboard of 2 meters is accepted in order to get beyond the reach of waves, appearence and for other incidental purposes. This is provided by ~~one meter of paraphet wall and one meter of the~~ extending extension of the dam section one meter above the high water level and building a parapet wall of one meter height on top of the crest of the dam.

HEIGHT OF THE DAM :

The crest of the dam including the freeboard allowance is at an elevation of 795 m.

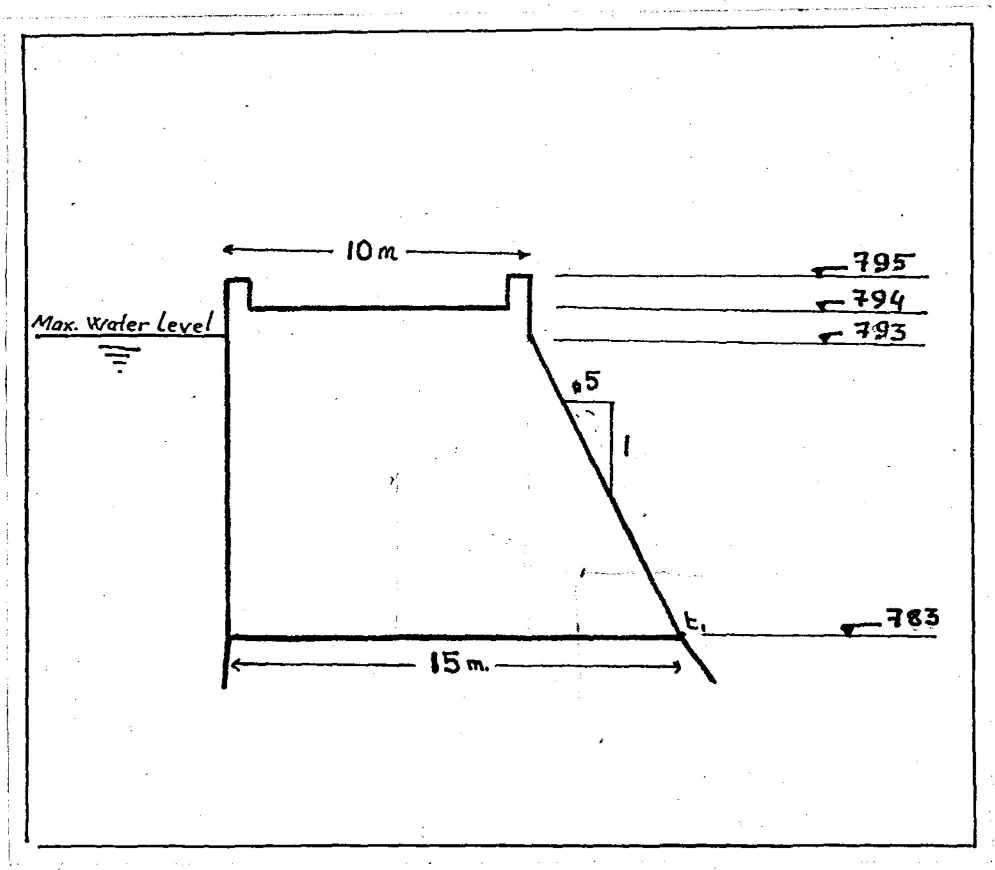
A study of the profile of the dam site will reveal the fact that the maximum height of the station 0+231 is 55 meters above the original foundation level and the dam will extend an additional 17 meters below the original soil level so that it will rest on a reasonable sound rock layer.

very deep excavation Is that necessary ?

TOP WIDTH :

The width of the crest of gravity dams of moderate heights vary between 10%-15% of the height. Though a heavy top section is a disadvantage when earthquake forces are involved, after some trials I had to choose a top width of 10 meters (.14H) in order to provide the necessary width for the lower sections.

BLOCK I



$$\bar{x} = \frac{\frac{250}{3}\times 2 + 110\times 10}{135} = 9.8 \text{ m.}$$

2 8.7 2

$$\bar{y} = \frac{25\times 3.33 + 110\times 5.5}{135} = 5.05 \text{ m.}$$

downstream

Reservoir full. Earth Quake ~~upstream~~ normal to the axis.

Vertical forces:

		Forces Tons	Lever Meters	Moment Meter- Tons Resisting M	Overturning Moment
W	2.4x135	324	9.3	3,000	
V	-	-	-	-	
U	$\frac{1}{2}$x10x15	-75	10.0	- 750	+750

$\Sigma V = 249$ $\Sigma + M = 3,000$

This is confusing!

Horizontal forces:

H	$\frac{1}{2}$x10x10	50	3.33	-167
Em	.15x324	48.5	5.5 ~~5.05~~	-245
Ew	.555x.15x10^2	8.4	4.25	- 35.6

$\Sigma H = 106.9$ $\Sigma - M = 1197.6$

$$\bar{X} = \frac{3,000 - 1196.5}{249} = 7.3 > \frac{15}{3}$$ Resultant within the middle third.

$$\text{Tan}\,\theta = \frac{106.9}{249} = .69$$

At this height inclined and vertical compressive stresses need not be checked since they can' not be critical.

down

Reservoir full, Earth quake upstream normal to the axis, NO UPLIFT

$$\bar{X} = \frac{3{,}000 - 447.6}{324} = 7.9 > \frac{15}{3}$$

Resultant within the middle third.

$$\text{Tan}\,\theta = \frac{48.5}{324.0} = .15$$

upstream

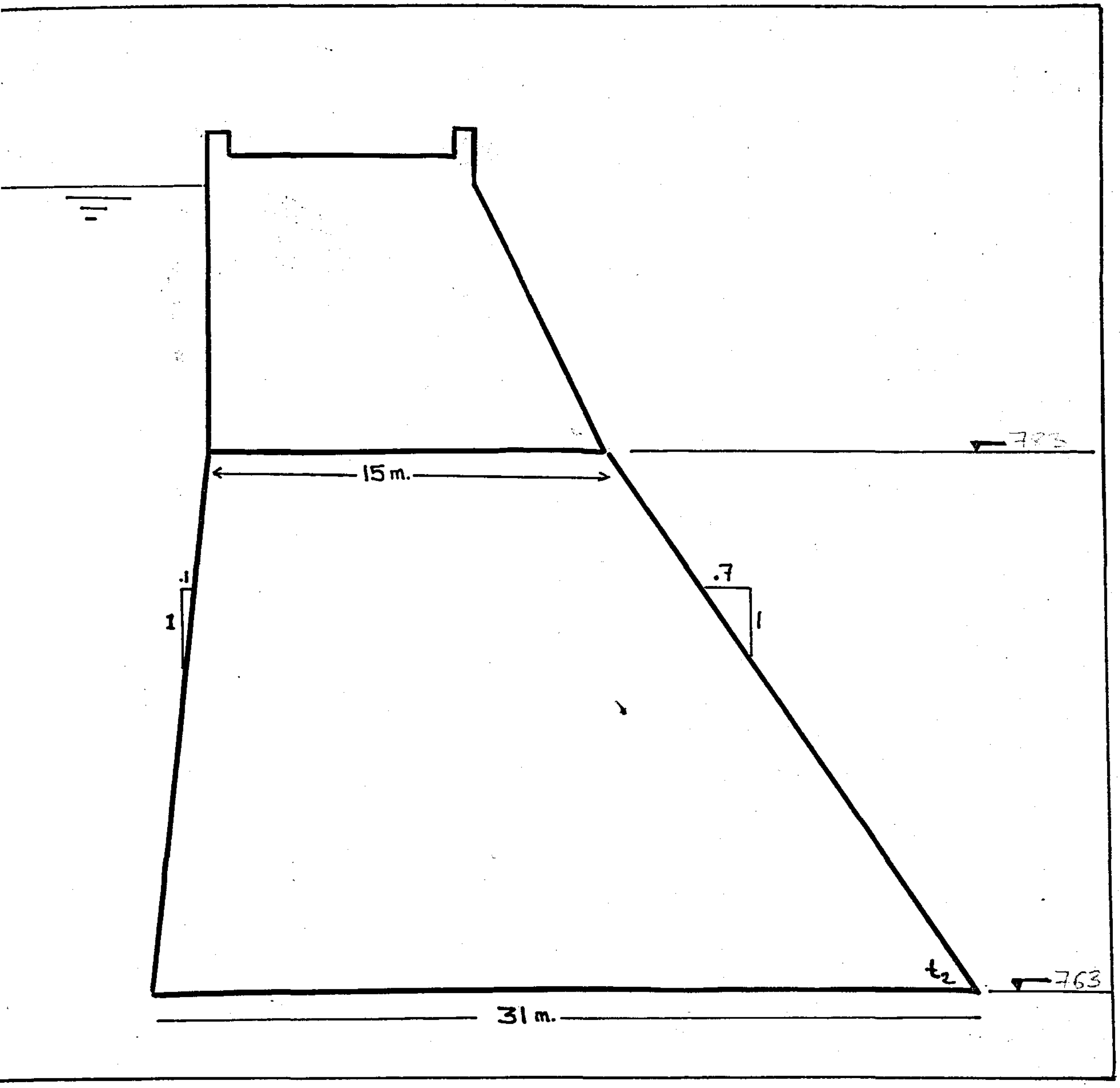
Reservoir empty, Earth quake downstream normal to the axis.

$$\bar{X} = \frac{3{,}000 + 245}{324} = 10 = \frac{15\text{x}2}{3}$$

Resultant at the middle third.

$$\text{Tan}\,\theta = .15$$

BLOCK II



$135 \times 22.8 = 3160$
$300 \times 21.5 = 6450$
$20 \times 29.7 = 600$
$140 \times \frac{28}{3} = 1310$
$595 \qquad 11520$

$\bar{x} = \frac{11520}{595} = 19.4$ m. from t_2.

$135 \times 25 = 3380$
$300 \times 10 = 3000$
$20 \times 6.67 =$
$140 \times 6.67 = \} 1065$
7445

$\bar{y} = \frac{7445}{595} = 12.5$ m. from base

Reservoir full, Earth quake upstream normal to the axis

Vertical forces

		Forces Tons	Lever Meters	Moment Meter- Tons
W	595x2.4	1422	19.4	27620
V	$2\times10 + \frac{1}{2}\times10\times20$	30	31.2	936
U	$\frac{1}{2}\times30\times31$	465	20.5	-9440
		$\Sigma V = 987$		$\Sigma+M$ 28556

Horizontal forces

		Forces Tons	Lever Meters	Moment Meter- Tons
H	$\frac{1}{2}\times30\times30$	450	10.0	-4500
Em	.15x1422	213	12.5	-2660
Ew	$.555\times.15\times30^2$	75	12.7	- 950
		$\Sigma H = 738$		$\Sigma -M$ 17550

$$\bar{X} = \frac{28556-17550}{987} = 11.3 > \frac{31}{3}$$ Resultant within the middle third.

$$\text{Tan}\,\theta = \frac{738}{987} = .75$$

$$\sigma_{1,2} = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{987}{31}\left\{1 \pm \frac{6\times4.2}{31}\right\} =$$

$$\sigma_{TOE} = 57.2\ T/m^2 \qquad \sigma'_{TOE} = \frac{57.2}{.675} = 85\ T/m^2$$

$$\sigma_{HEEL} = 5.9\ T/m^2 \qquad \sigma'_{HEEL} = \frac{5.9}{.96} = 6.15\ T/m^2$$

Not checked
from here to page 53

Reservoir full, Earthquake upstream normal to the axis, no uplift assumed.

$$\bar{x} = \frac{28556-8110}{1452} = 14.1 \text{ m.}$$ Resultant within the middle third.

$$\tan\theta = \frac{738}{1452} = .51$$

The inclined and the vertical stresses at the toe are obviously less than the previous case therefore need not be calculated.

$$\sigma_{HEEL} = \frac{1452}{31}\left(1 \pm \frac{6\times1.4}{31}\right)$$

$$\sigma_{HEEL} = 33.5\ T/m^2$$

$$\sigma'_{HEEL} = 35\ T/m^2$$

Reservoir empty, Earthquake downstream normal to the axis.

$$\bar{x} = \frac{27620+2660}{1422} = 21.3 > 20.7$$ The resultant falls ~~many~~ slightly out of the middle third.

$$\tan\theta = \frac{213}{1422} = .15$$

$$\sigma_{TOE} = \frac{1422}{31}\left(1 - \frac{6\times5.8}{31}\right)$$

$$= -5\ T/m^2$$

$$\sigma'_{TOE} = -7.4\ T/m^2 = -3\text{psi}$$

I believe that under the worst conditions such a small tension especially at the toe may be neglected. Therefore I do not make another trial.

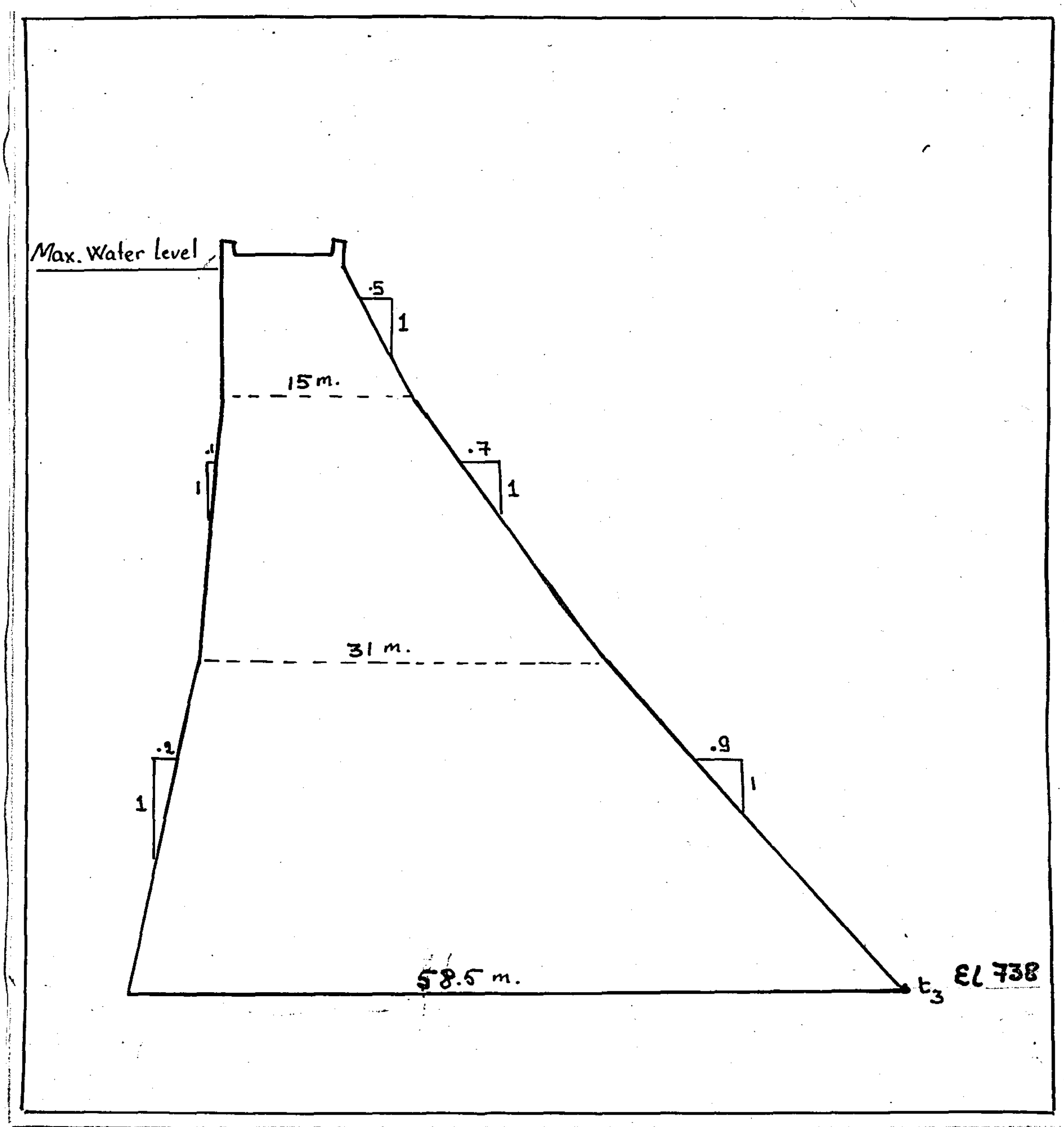
$$\sigma_{HEEL} = \frac{1422}{31}(1+1.12)$$

$$= 97.5\ T/m^2$$

$$\sigma'_{HEEL} = 101.4\ T/m^2$$

$$\tau_{max} = .7\times97.5 = 68\ T/m^2$$

BLOCK III



$$595 \times 42.4 = 25200$$
$$775 \times 38.0 = 29300$$
$$281 \times 15.0 = 4230$$
$$62.5 \times 55.2 = 3450$$

1714 $\qquad$ $62,180$

$$\bar{x} = \frac{62180}{1714} = 36.3 \text{ from } t_3$$

$$595 \times 37.5 = 22300$$
$$775 \times 12.5 = 9700$$
$$344 \times 8.3 = 2850$$

34850

$$\bar{y} = \frac{34850}{1714} = 20.3 \text{ from base of block } \underline{III}$$

Reservoir full, Earthquake upstream normal to the axis.

Vertical forces :

		Forces Tons	Lever Meters	Moment Meter- Tons
W	1714x2.4	4125	36.3	149,500
Vw	170 82.5	252.5	54.7	13,800
U	$\frac{1}{2}$x55x58.5	1600	38.9	-62300
		Σ V= 2777.5		Σ+ M = 163,300

Horizontal Forces:

H	$\frac{1}{2}$x55x55	1520	18.3	-27,800
Em	.15x 4125	618	20.3	-12,500
Ew	.555x.15x55x55	252	22.9	-5,800
		Σ H = 2380		Σ- M = 108,400

$\bar{X} = \frac{54900}{2778} = 19.7 > \frac{58.5}{5}$ Resultant within the middle third.

$\tan \theta = \frac{2380}{2777.5} = .85$

$$\sigma_{T,H} = \frac{2777.5}{58.5}\left(1 \pm \frac{6 \times 9.6}{57.5}\right)$$

$\sigma_{Toe} = 95\ T/m^2$ $\sigma'_{Toe} = 142\ T/m^2$

$\sigma_{Heel} = 1.2\ T/m^2$ $\sigma'_{Heel} = 1.3\ T/m^2$

Reservoir full, Earthquake normal to the axis, no uplift assumed

$$\bar{x} = \frac{117,200}{4378} = 26.8 \text{ m.}$$ Resultant within the middle third.

$$\tan\theta = \frac{2380}{4378} = .54$$

$$\sigma_{HEEL} = \frac{4378}{58.5}\left(1 - \frac{6\times 2.5}{58.5}\right)$$

$$\sigma_{HEEL} = 56.2 \text{ T/m}^2$$

$$\sigma'_{HEEL} = 58.5 \text{ T/m}^2$$

Reservoir empty, *Earthquake downstream normal to the axis.

$$\bar{X} = \frac{149,500+12,500}{4125} = 39.6 = 58.5 \times \frac{2}{3}$$ Resultant at the middle third

$$\tan\theta = \frac{618}{4125} = .15$$

$$\sigma_{\substack{TOE\\HEEL}} = \frac{4125}{58.5}(1 \pm 1)$$

$$\sigma_{HEEL} = 142 \text{ T/m}^2$$

$$\sigma'_{TOE} = 0$$

$$\sigma'_{HEEL} = 148 \text{ T/m}^2$$

$$\tau_{max} = .7\times 142 = 99.5 \text{ T/m}^2$$

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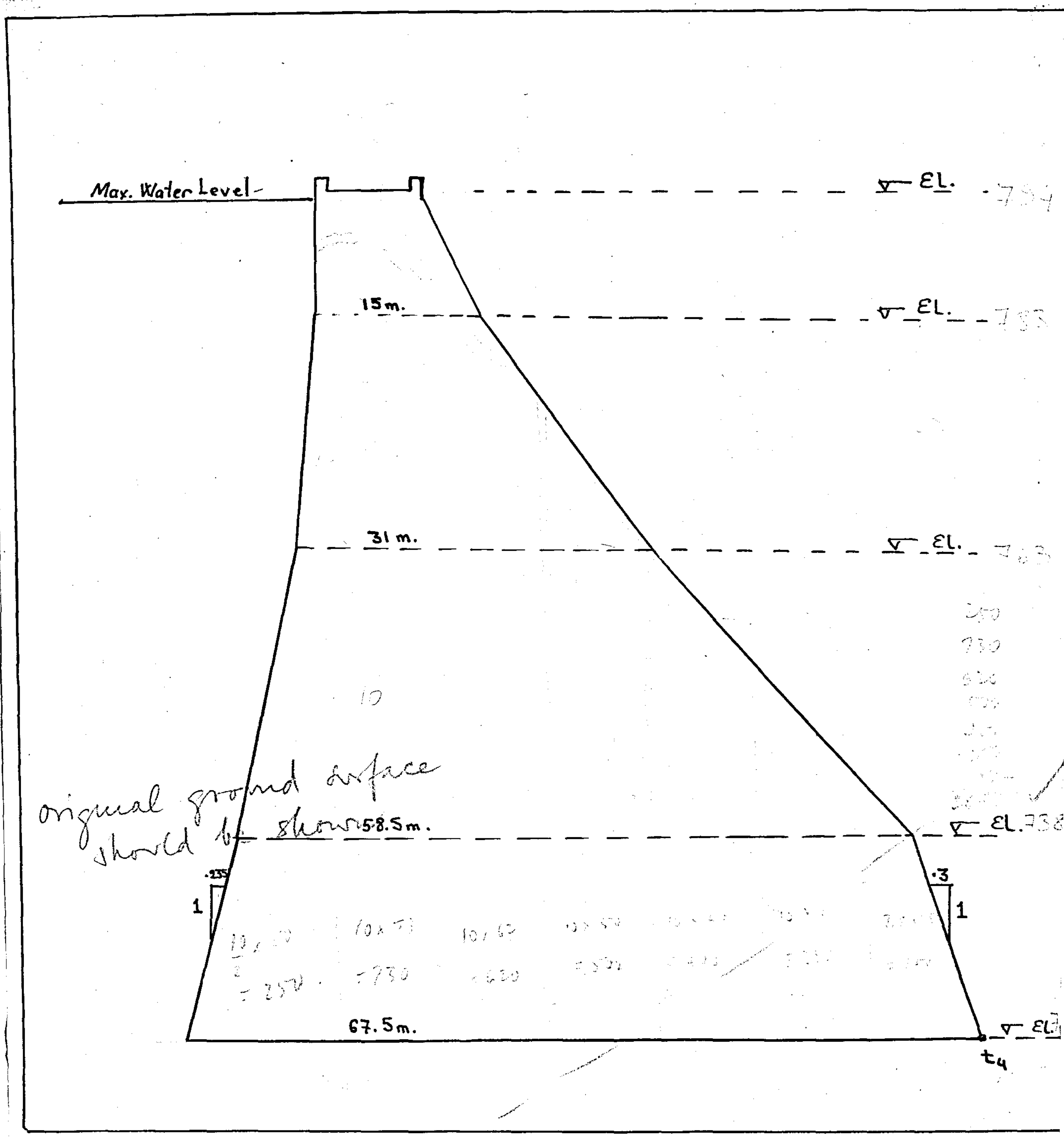
* The minimum water level could be assumed but ~~xxxxx~~ this assumption is on the safe side and faciliates the problem.



$1714 \times 41.3 = 71{,}000$
$1000 \times 34.3 = 34{,}300$
$36 \times 68.8 = 2{,}550$
$45 \times 3.3 = 150$
$2795 \qquad 108{,}000$

$$\bar{x} = \frac{108{,}000}{2795} = 38.8\text{ m. from } t_4$$

$37.3 \times 1714 = 64000$
$8.5 \times 1000 = 8500$
$81 \times 17/3 = 500$
73000

$$\bar{y} = \frac{73000}{2795} = 26\text{ m. from base}$$

Reservoir full, Earthquake upstream normal to the axis

Vertical forces:

		Forces Tons	Lever Meters	Moment Meters- Tons
W	2795x2.4	6700	38.8	260,000
Ww		452	62.5	28,300
U	$\frac{1}{2}$x62.5x55	1720	45.0	-77,400
		Σ V = 5,432		Σ+ M 288,300

H	$\frac{1}{2}$x55x55	1510	35.3	-53,200
Em	.15x6700	1000	26.0	-26,000
Ew	.555x.15x 55^2	252	39.9	-10,000
		Σ H 2752		Σ- M = 166,600

$$\bar{x} = \frac{121,700}{5,432} = 22.45 \approx 22.5$$ Resultant at the middle third.

not so good

$$\tan\theta = \frac{2752}{5432} = .5$$

$$\sigma_{heel} = 0 \qquad \sigma'_{heel} = 0$$

$$\sigma_{toe} = \frac{5432}{67.5}\left(1+1\right) = 161 \ T/m^2 \qquad \sigma'_{toe} = 178 \ T/m^2$$

Reservoir full, Earthquake normal to the axis, no uplift assumed

$$\bar{x} = \frac{199,100}{7,152} = 27.9 > 22.5 \text{ Resultant within the middle third.}$$

$$\tan\theta = \frac{2,752}{7,152} = .39$$

$$\sigma_{HEEL} = \frac{7152}{67.5}\left(1 - \frac{6 \times 5.35}{67.5}\right)$$

$$\sigma_{HEEL} = 56 \ T/m^2$$

$\sin^2\alpha_u = .955$

$$\sigma'_{HEEL} = 53.5 \ T/m^2$$

Reservoir empty, Earthquake downstream normal to the axis.

$$\bar{x} = \frac{288,000}{6,700} = 42m. < 45m. \text{ Resultant within the middle third.}$$

$$\tan\theta = \frac{1000}{6700} = .15$$

$$\sigma_{TOE} = \frac{6700}{67.5}\left(1 - \frac{6 \times 8.75}{67.5}\right)$$

$$\sigma_{TOE} = 21.8 \ T/m^2$$

$$\sigma'_{TOE} = 23.9 \ T/m^2$$

$$\sigma_{HEEL} = 177 \ T/m^2$$

$$\sigma'_{HEEL} = 186 \ T/m^2$$

The max. shearing stress $= .7 \times 177$
$= 124 \ T/m^2$

This is well within the allowed value.

ABOUT THE CENTER CROSS SECTION :

I have to accept that I ended with a heavier cross section than I expected. However the specifications to which I conformed the design are more restricted than the assumptions made in the design of the existing dams.

I. Uplift in most of the existing dams has been assumed to vary from .5-.67 of the full hydrostatic pressure from heel to the toe. But recently The U.S. Bur. of reclamation has suggested the use of full hydrostatic pressure rather than a percentage. Therefore I had to consider full hydrostatic pressure.

2. The accepted acceleration of the earthquake (.I5g) is also greater than the assumption made in the design of most of the existing dams, however I believe that this is not an oversafe assumptionfor the site.

3. The assumption of the earthquake and the maximum water acting at the same instant is also very conservative but requires a heavy section.

A study of ~~the~~ existing dams in the U.S. shows that the vertical and the inclined compressive stresses allowed in this design is very near to the allowed stresses in the dams of equal height.

For the sake of comparison the stresses allowed in the design of
~~I have included the allowed stresses~~ in Norris dam
are shown in the following tabulation, all calculations
~~and for the sake of comparison I have converted the units to~~
converted to
~~the~~ British units.

		Maximum compressive stresses			
		Vertical		Inclined	
	Height	Heel	Toe	Heel	Toe
NORRIS	242	33,700	28,000	35,000	41,000
ALMUS	243	36,000	32,500	38,300	36,200
10% increase		39,600	35,750	42,100	39,800

Why?

See page

The units are in psi

Friction factor Tan θ, allowed in the design is .85. Usually a friction factor of .75 is assumed and my assumption may not be considered ~~very~~ safe, but this figure is solely for the earthquakes. Again a study of the existing dams reveal that in the design of Shasta dam at about 200ft. above base Tan θ =.90 is allowed. Elephant Butte dam is designed with ~~respect to~~ a friction factor of .85.

No tension is allowed in my design of a concrete dam for Almus, ~~the concrete body~~ except at an elevation of 763m. at the toe a tension of 3 psi exists when the reservoir is empty. This is a very slight value and anyway the practical dimension of the cross section will deviate to some extent from the theoretical cross section designed, also the assumption of the minimum water level instead of empty reservoir assumption would diminish this tension.

Since the dam is placed quite deeply into the foundations I do not find it necessary to provide a key. ?

SPILLWAY AND SLUICEWAYS :

The purpose of a spillway is to provide a means for the passage of flood flows without damage to the dam or its appurtenant structures. Therefore a spillway must have the capacity to discharge the max. flood estimated. However sluiceways may help to share the max. load.

Sluiceways for a concrete dam pass through the structure. It is advantageous to have a number of sluiceways rather than a single large capacity sluiceway, both for structural reasons and ~~the~~ for facility of ~~the~~ control of discharge quantity ~~of discharge. and~~ ~~It also permits the regulation of outflow with the valves wide open.~~

I ~~find it usefull to use~~ propose to provide eight sluices (circular, because square-edged entrances may cause turbulence & cavitation.) located at two different levels. These sluices will also permit discharge of the fine sediments.

The max. discharge (in case of flood) through the sluices is found by the formula

$$Q = .7A\sqrt{2gh}$$

The diameter of ~~the~~ each sluiceway ~~s~~ is 3 meters and the average head is 25 meters.

Assuming that three of these sluices become inoperative, to be on the safe side though trash-racks are provided, the total flow is $5(\ .7X76\ \sqrt{2X32.2X82}\ ) = 33{,}500\ ft^3/\ sec.$

The discharge of an overflow spillway is given by the formula

$$Q = Cw\ L\ h^{3/2}$$ + velocity of approach

where Q is discharge in $ft^3/$ sec.

L is the length of crest in feet

H is the head on the spillway in feet

Cw is a coefficient which varies from 3 at low heads to 4 at high heads.

The max. flood was estimated to be 70,500 $ft^3/$ sec (2000 $m^3/$sec.)

$Q_s = 70{,}500 - 33{,}500 = 37{,}000\ ft^3/\ sec$

Assuming Cw = 3.5 and a max. head of 3 meters (9.9 ft.) the required length of the crest is

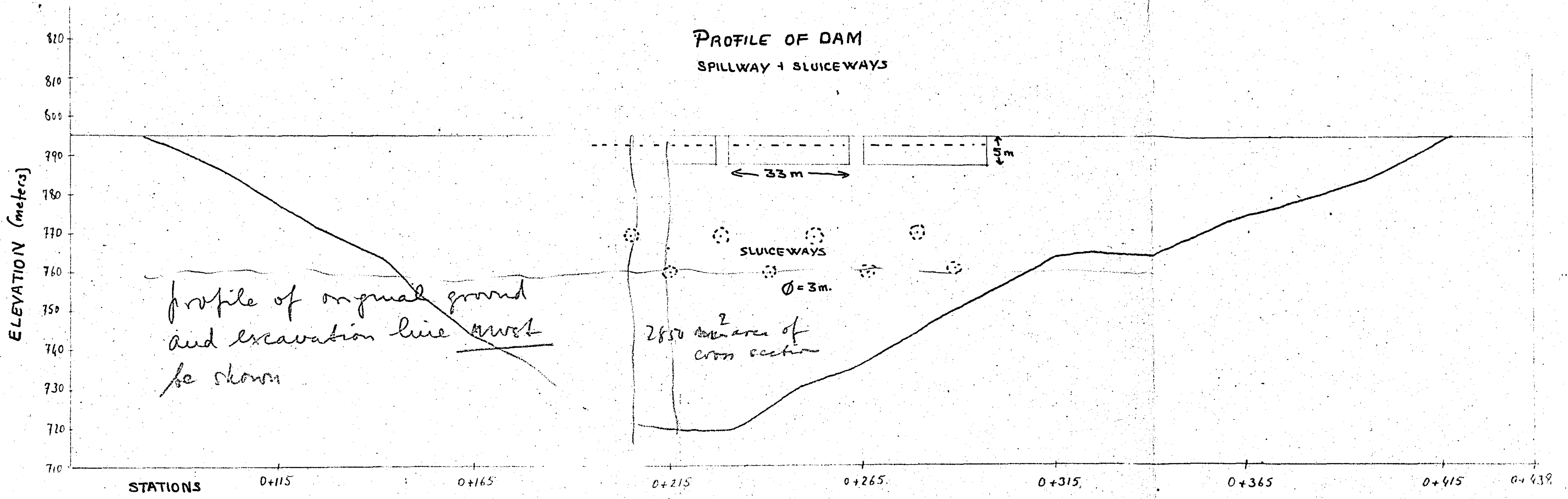
$$L = \frac{Q}{Cw\ h^{3/2}} = \frac{37{,}000}{3.5x\ 9.9^{3/2}} = 324\ ft$$

The choice of gate :

In general, the volume of the concrete in the crest of a spillway and the cost of the gates and their operating mechanisms increase as the discharge head increases. This indicates that the spillway should be as long as possible in order to effect a reduction in head.

The most common types of gates are: radial gates, vertical-lift gates, drum gates, and roller gates.

Radial gates and vertical-lift gates are simple in design but these gates are for relatively short spans while the drum gate offers a long unobstructed crest .



$$2850 \times \frac{450}{2-3} = 400,000 - 500,600\ m^3$$

I believe that it would be advantageous to use three I08 ft span drum gates instead of providing 6 or 7 shorter spans and using radial or vertical-lift leaf gates. far too expensive in first cost

Another advantage of the drum gate is that it is operated and entirely by ~~the~~ hydraulic pressure. This feature introduces an ~~a~~ additional factor of safety in that its operation is assured in the event of failure of a station power system. construction

A drum gate also conform to the shape of the ogee crest when lowered, but design of crest becomes complicated & heavy

Trashracks:

Trashracks are used to keep out of the conduit any objectionable large debris which may result in rendering the sluices inoperative. The load on a trashrack structure is normally very small.

Trashbars will consist of flat steel bars set on edge and spaced I5 cm. apart. The bars will be assembled into sections and held in place by guides in the trashrack structure.

FOUNDATION TREATMENT :

~~The ultimate goal for the~~ The purpose of foundation treatment is the formation of a sound, impervious, and consolidated base capable of withstanding all ~~of the~~ stresses induced by the superimposed ~~loaded~~ structure under load.

~~These operations on the site~~ Proposed foundation treatment may be divided into two parts:

A. The Excavation ~~operation~~ of overburden, consisting of the removal of an average depth of 8 meters of alluvial soil and decomposed rock above ~~the reasonably~~ sound rock bed. ~~foundation.~~

B. ~~The~~ Subsequent ~~operations consisting of~~ grouting and provision of drainage, ~~treatments.~~

The excavating operation must be conducted in such a manner that the underlying sound rock will not be damaged.

~~The boring test results show~~ Core drilling showed that some parts of the rock is cracked and contains seams and faults. In order to clean out the seams as well as the rock surface, it will be necessary to ~~use~~ apply jets of a mixture of air and water under high pressure. ~~Such a jet will also clean the rock surface.~~

A clean rock surface is necessary so that the concrete dam shall ~~have the maximum possible~~ be in perfect bond with the foundation.

Narrow seams and faults must be grouted after ~~washed, in~~ washing; in case of wide seams, broken rock which fills them must be excavated and the seams must be refilled with concrete.

The principal objective of foundation grouting is to establish an effective barrier to seepage, thereby reducing hydrostatic uplift under the structure. A secondary objective is to consolidate the rock under the structure and thus secure a monolithic foundation. Proposed depth?

The final treatment of the foundation is to provide drainage. Although a well executed grouting program will considerably reduce seepage ~~some~~ means must be provided to intercept ~~the~~ percolating water which ~~will percolate through and~~ if not removed, may build up prohibitive hydrostatic pressures under ~~on~~ the base of the structure.

Drainage is provided by drilling horizontal? ~~some lines of~~ holes near the downstream face. The holes are 3 inches in diameter at a spacing of I.5 meters and the depth of the holes is about 30% of the hydrostatic head. (1)

In order to provide drainage for water that may percolate ~~percolating~~ through the upstream face or seep ~~seeping~~ through the foundation, and also to provide access to the interior of the structure for observing its behavior after completion, galleries are constructed in the structure. The foundation gallery extends the length of the dam near the rock surface, in plan it is near and parallel to the axis of the dam.

A supplementary drainage gallery is located further downstream. These galleries also serve as inspection galleries.

Steel reinfocement is required in the vicinity of the galleries since stress concentrations will be induced. The amaunt of the reinforcement must be based on the experimental determination of stress concentrations obtained by model tests.

---

(1).... This is the suggestion of the U.S.Dept. of the interior Bureau of Reclamation.

GENERAL FEATURES OF THE CONCRETE GRAVITY DAM

General Information

Purpose: Storage for irrigation, power generation and partial flood control

Drainage area: 2337 km^2

Concrete Volume: 356,490 m^3

how has this been so accurately calculated ?

Reservoir capacity: 670×10^6 m^3

Irrigation and power	386,000,000 m^3
Dead storage	180 000,000 m^3
Flood control	104,000,000 m^3

Dam

Foundation : Andesite

Type : Straight concrete gravity

Dimensions :

Structural height	74 m
Hydraulic height	55 m
Max. base width	67.5 m
Top width	10 m
Crest length	425 m
Freeboard	2 m

Power plant

Capacity: 27,000 kw ? Why ?

at average head of about 45 m and Q = 19 m^3/s as assumed in this design such a large capacity is ~~not~~ warranted

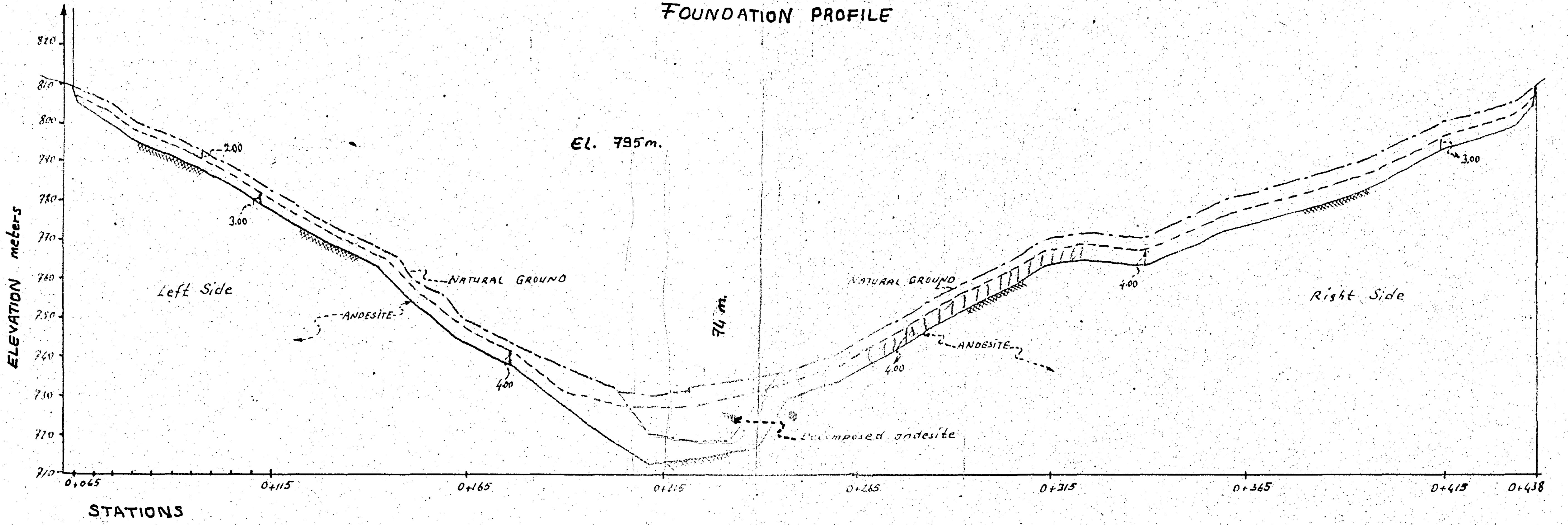
Spillway: Gate controlled over-flow section near the center of the dam with 3 automatically controlled drum gates.

Sluiceways: 8 circular sluices located at two different levels

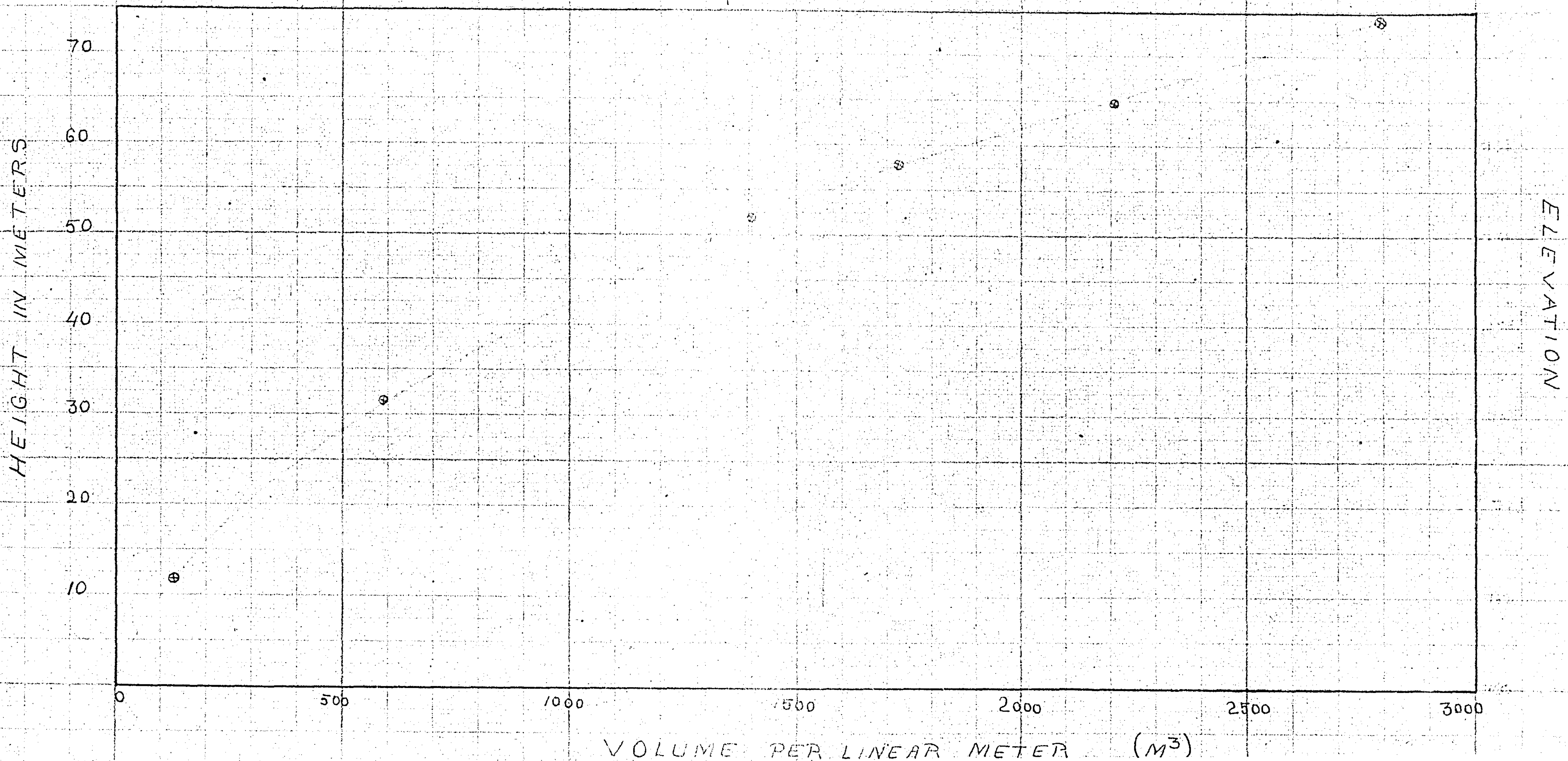
Items of interest

An earthquake intensity of .15g assumed in the design

FOUNDATION PROFILE



line of excavation
should be shown


HEIGHT IN METERS
70
60
50
40
30
20
10
0
500
1000
1500
2000
2500
3000
VOLUME PER LINEAR METER (M³)
ELEVATION

Station	Elevation of ~~Soil~~ foundation ?	Average Volume per linear ft. M m^3	Distance m	Volume m^3
0 - 080	795			
0 -085	792	24	10	240
0 - 995	788	60	10	600
0 - 105	784	100	10	1000
0 - 115	777	240	10	2400
0 - 125	772	380	10	3800
0 - 135	766	540	10	5400
0 - 145	762	680	10	6800
0 - 155	753	1000	10	10000
0 - 165	743	1410	10	14100
0 - 175	738	1700	10	17000
0 - 185	733	2050	10	20500
0 - 195	725	2550	7.5	20100
0 - 200	723	2700	5	13750
0 - 205	722	2750	5	13750
0 - 210	722	2750	5	13750
0 - 215	722	2750	5	13750
0 - 220	721	2800	5	14000
0 - 225	721	2800	5	14000
0 - 230	721	2800	5	14000

Station	Elevation of soil	Average Volume per linear (m). m^3	Distance m	Volume m^3
0 + 235	725	2550	5	12,750
0 + 241.25	727	2450	7.5	15,900
0 + 250	730	2300	10	23,000
0 + 260	735	1850	10	18500
0 + 270	739	1650	10	16,500
0 + 280	743	1400	10	14,000
0 + 290	753	1000	10	10,000
0 + 300	758	740	10	7,4000
0 + 317.5	763	600	25	15,000
0 + 335	763	600	10	6,000
0 + 345	770	410	10	4,100
0 + 355	772	380	10	3,800
0 + 365	772	380	10	3,800
0 + 375	776	290	10	2,900
0 + 385	780	190	10	1,900
0 + 395	782	150	10	1,500
0 + 410	790	40	10	400
0 + 415	795			
		Total Volume		356,490 m^3

Estimation of the total amount of cement required:

The measuring of the quantities of the constituent materials of concrete is done either by volume or by weight. The latter method is much more accurate and will be used here. For the interior of the gravity dam a lean mixture of I:3:6 will be used (240 kg of cement/ m^3 of concrete) .
For the faces of the dam a richer concrete of I:2.5:4.5 will be used (300 kg of cement/ m^3 of concrete) .

Assuming that I/4 of the structure will be constructed with richer concrete an average of

$$\begin{array}{rl} 240\text{x}3 = & 720 \\ 300\text{xI} = & \underline{300} \\ & \text{I020} \;/4 = 255 \text{ kg of cement/ } m^3 \end{array}$$

will be used in the construction.

This will amount to a total of 356,000x255 = 9I,000 tons of cement. Assuming an additional cement consumption of %20 of the estimated figure, for foundation treatment, waste, and spillway requirements, the total consumption of cement is I09,300 tons.

Cost of cement:

The price of cement is I40 T.L./ ton. The total cost of cement is I09,200xI40 = I5,250,000 T.L.
Cement will be transported from Sivas, where the cement factory is, to Artova by freight train and from Artova to the dam site by trucks.

Aggregate ?

Required equipment and machinery:

Item	capacity	number	Price $ unit	total
Concrete Lab. equip.				
Carpenter shop				
Concrete mixers		4	I000	4000
			these would be extremely small ones	
Concrete enjectors		5	4000	20000
vibrators		I0	300	3000
Wagon drill	I00 kg	2	5000	I0000
Conveyor belts	$50m^3/hr$	3	4000	I2000
Trucks	I0 T	I0	22000	220 000
	5 T	5	I8000	90 000
	3 T	3	I2000	36 000
Bulldozers		2	30000	60 000
Powershowel		2	25000	50 000
Cranes		2	32000	64 000
High level mixing plant			80000	80 000
Low level mixing plant			45000	45 000
Cableways			30000	30 000
Dumping buckets		6	500	3 000
Station wagons		4	5000	20 000
Jeeps		6	4000	24 000
Busses		3	I0000	30 000
				80I 000
Unforeseen 20% of the expected				I60 200
			TOTAL	$ 96I 200

The total cost of the necessary equipment and machinery is estimated to be $ 961,000 or T.L. 8,650,000 .

Assuming an average useful life of 7 years , the direct cost of the machinery is the number of years the dam is under construction multiplied by

$$\frac{8,650,000}{7} = 1,235,800 \text{ T.L.}$$

(Annual depreciation) + [illegible] 15%)

Cost of the spare parts will amount to an estimated 30% of the annual depreciation.

[illegible] ~ 2,500,000

Estimation of personnel required and total payment:

	Number	Average daily payment	Total daily payment
Engineers and Administrators	I2	75	900
Superindendents and Foremen	40	40	I600
Skilled workmen	I20	25	3000
Equipment operators	80	28	2240
Mechanics	24	28	673
Unskilled laborers	400	II	4400
Physicians	3	60	I80
	679		I2993

In addition to gross daily wages, I2% is added to cover vacation , sick leave, accident, insyrance and workmans compensation.

Total payment to the personnel is estimated to be I4,880 T.L./ Day.

Estimation of the total construction time:

The time required to construct a high concrete dam is usually long because the maximum height of a single pour is about I.5 meters and each block ~~is to~~ must be permitted to ~~stay~~ set for a few days before another is poured next ~~to it~~ or on top of it. ~~But in this~~ In our case this will not be a speed limiting problem since the dam is relatively long and not very high.

If, including the foundation treatment it is assumed that the daily average use of cement is about 200 Tons (1), the time required to construct the body of the dam will be

$$\frac{109,300}{200} = 547 \text{ days}$$

assuming that the work will be continuous.

However an additional time due to ~~camping~~ camp preparations, placing the necessary machinery on the site and the excavation to solid rock need be taken into consideration. far too short

Considering all these factors my estimation is that the total construction time will be 2 years. allow 3 years

---

(1) Average daily use of cement Of the Sariyer dam was about I50 tons. Howevr due to insufficient transportation the work had stopped for quite a few times.

Direct rough cost estimate
400,000 m³ @ T.L. 200 = T.L. 80,000,000

TOTAL FIRST COST OF THE CONCRETE GRAVITY DAM

	T.L.	
Cost of cement	I5,250,000	
Transportation of cement by freight	16,000,000 I,343,000	(1)
Reinforcement Steel	I,525,000	(2)
Depreciation of equipment and machinery	2,470,000	low
Spare parts	740,000	
Total payment to the personnel	I0,850,000	
Cost of water	I,900,000	
Camping expences	800,000	low
Gates	600,000	(3)
Cost of fuel	4,I00,000	
Unforeseen charges I0%	3,957,800	
ESTIMATED TOTAL FIRST COST	43,535,300	T.L.
say	65,000,000	
Interest during constr.	10,000,000	
Engineering + adm.	5,000,000	
	T.L. 80,000,000	

Aggregate
800,000 ton = 180,000 m³ @ 50.–
quarry rock, blasting, crushing, transport
T.L. 8,000,000
800,000 ton
including quarrying, washing, screening, transport
8,000,000

~ 600,000 m³ of sand @ 10.–

(1) Freight rate for cement = .0975 T.L./T/Km
Distance from Sivas to Artova = I26 Kms
I26x.0975xI09,200 = I,343,000 T.L.

(2) Assumed to be I0% of the cost of cement

(3) No basis for estimation

(4) Correlated from Sarıyar dam

(5) A distance of I,585,000 km must be covered by trucks to carry cement from Artova to the dam site. 77 km. Appr. 635,000 T.L. of fuel is used for this purpose. The rest is used for hauling materials and all other sorts of transportation.

GENERAL FEATURES OF THE EARTH AND ROCK-FILL DAM SELECTED BY D.S.I.

General Information

Purpose: Storage for irrigation, power generation and partial flood control

Drainage area: 2337 km^2

Earth and rock-fill volume 2,670,000 m^3

Reservoir capacity:	950,000,000 m^3
Irrigation and power	700,000,000 m^3
Flood control	115,000,000 m^3
Dead storage	135,000,000 m^3

Dam

Foundation : River sand and gravel

Dimensions :	Structural height	79.50m
	Hydraulic height	68.50
	Max. base width	383. m
	Top width	10 m
	Crest lenth	342 m
	Freeboard	5 m

Type : Earth and rock-fill

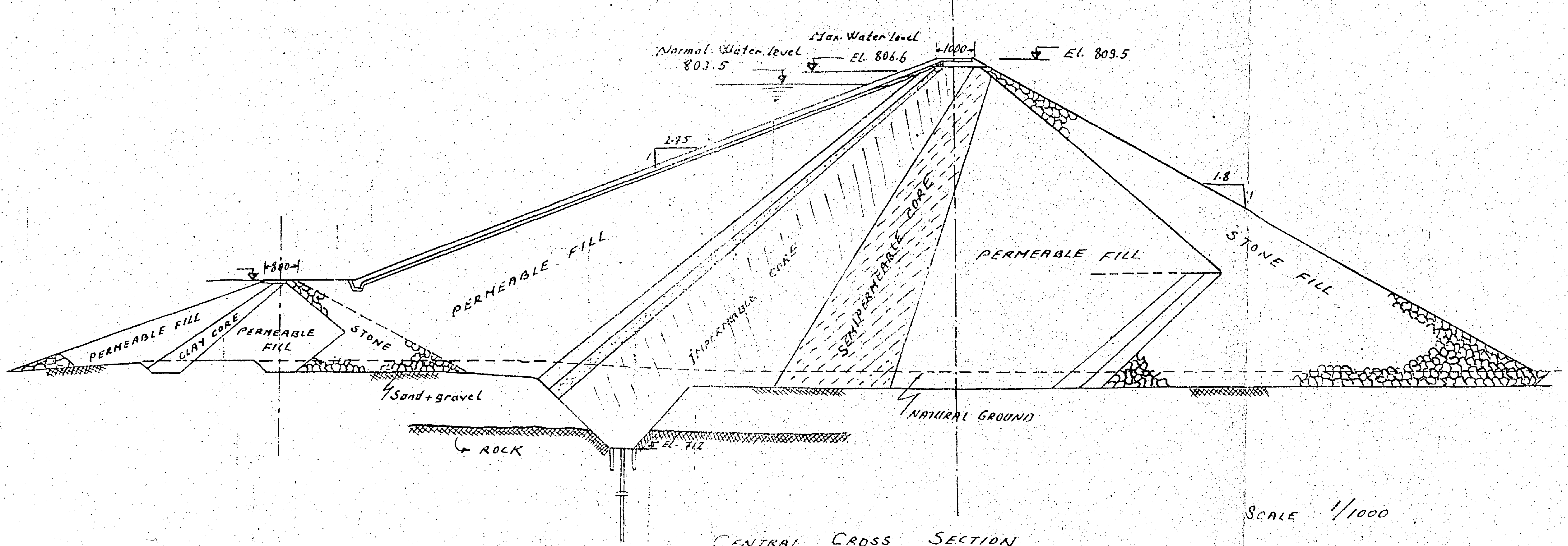
Face slopes : Upstream 2.75: I cofferdam 3: 1
Downstream I.8: I

Cut-off trench :	Depth below streambed	I5m
	Side slopes	I:I
	Bottom width	6m

Power plant

Location: Left bank of river

Capacity: 27,000 kw



CENTRAL CROSS SECTION

MAX. CROSS SECTION
DAM DESIGN SELECTED
BY D.S.İ.

CRITICAL DISCUSSION OF THE DAM DESIGN SELECTED BY D.S.I.

The suggested D.S.I. design is an earth and rockfill embankment. (Maximum cross section ~~submitted~~ shown in figure P.74). The use of the rockfill makes it possible to steepen the slopes of the ~~underlying permeable~~ inner layers of semi-permeable earth and ~~the~~ impermeable core, ~~and therefore~~ which results in added stability and economy, since a plentiful and inexpensive supply of suitable rock is available in the neighborhood, while the material for the impermeable core is to be hauled from a distance of 2 miles. The use of the rock-fill is governed by considerations of economy and stability besides slope protection.

The designer of an earthdam can not, to the same degree as for a concrete structure, rely upon the application of mathematical analyses or formulae to determine the cross section. Soils and rock occur in nature with infinite combinations of size, gradation and composition. Present practice in determining the required cross section of an earth dam consists largely in designing to the slopes and characteristics of existing dams of similar foundation characteristics.

Downstream Slope:

The downstream slope of the suggested design is chosen as I.8 horizontal to I vertical. However, In ~~the~~ existing dams, ~~where~~ using a downstream loose rock-fill ~~is used~~, the outer rock slope is generally established as 2:I or 2.5:I . In the table below I list some successful existing dams in U.S.A. of various heights that have similar foundation characteristics.

SLOPES

DAM	HEIGHT	DOWNSTREAM rock-fill	UPSTREAM cohesionless	FOUNDATION
Echo-Utah	I25'	2:I	3:I	River sand, gravel
I930		6:I	5:I	
Deer Creek	I60'	2.25:I	3:I	Sand and gravel
Utah I94I		6:I	5:I	
Green Mountain	274'	2.25:I	3:I	Sand and gravel
Colorado I94I		5:I		
Anderson Ranch	344'	2:I	3:I	Sand and gravel
Idaho, I947		2.5:I	3.5:I	
Granby	232'	2.25:I	3:I	Sand and gravel
Colorado I950		5:I	5:I	

slopes at heel and toe

Tabulated results of the Glover-Cornwell[1] method of stability analysis show that the steepest slopes for the stability of dry cohesionless fill with a coefficient of friction of .75 and with an earthquake consideration of .I5g is I.85 horizontal to I vertical. ~~Also,~~ + the existing dams, however though they are not located in ~~at~~ such ~~a~~ seismically active zones, are designed to have flatter slopes. even

Bearing all these in mind, I doubt ~~do not think~~ that the choice of the downstream slope is safe enough ~~conservative~~. I would think that the maximum slope of the downstream face should not be flatter than ~~not exceed~~ 2:I .

---

(1) Treatise on dams

United States Dept. of the interior Bureau of Reclamation

Chapter 8, Earth dams (pp. I70-205)

The earth slope underlying the rock downstream face is designed as I.I horizontal to I vertical, but the maximum slope of the repose for loose and dry material is about I.5 and it is not good practice, even though the material is under a heavy stabilizing rock-fill, to surpass the slope of repose – therefore I believe that it would be advisable not to exceed this limit.

The upper part of the downstream lower of perneable earth ~~underlying the rock cover~~ apparently is not provided with a filter. and a filter must be provided ~~This must not be allowed~~ because the rainwater that percolating ~~percolates~~ through the rock lower ~~surface~~ can easily wash ~~away~~ the sand ~~that is on a steep slope~~ into the rock and cause internal instability, in particular since the slope of the dam is very steep.

Upstream Slope

The upstream face of the dam is designed to have a slope of 2.75 horizontal to I vertical. ~~However, the same reasoning for the downstream slope holds true for the upstream slope.~~ This slope also is steeper than generally practised.

Referring again to the Glower-Cornwell method, we see that for cohesionless materials on the upstream face, for stability of fills with friction coefficient of .7 saturated and draining, the steepest slope must be not less than 2.82 horizontal to I vertical, ~~for the fill possessing a friction coefficient of .7. (This is~~ The accepted assumption for the coefficient of friction of cohesionless of 0.7 materials).

Tabulated calculations show that the steepest upstream slope permissible is 3.29:1, If an earthquake of .I5g intensity is assumed to occur at the same time with the drawdown, the tabulated results show that ~~the steepest slope for the upstream face for stability is~~ 3.29:I (1) and for an earthquake of .Ig intensity the steepest

(1) Treatise on Dams, U.S.D.I.B.R. Chap.8 p. I80 Appendix C

3.01 ?

slope to be allowed is 3.91:1.

My opinion, after studying these figures, is that the slope of the upstream face must not be steeper than 3.3:1, whereas in the D.S.I. design this slope is accepted as 2.75:1 .

The cofferdam is later made use of as an integral part of the main structure and this has made possible an economy of materials, but had the cross section of the cofferdam been designed differently, it could have increased the path of percolation or creep distance in the foundation.

Water can freely percolate through the permeable fill and the rock to the foundation even at a distance of 80 meters away from the heel. This could be very much improved by the use of semipermeable material for the downstream part of the cofferdam and the continuation of the impervious section below the upstream zone, as shown in the improved cross section.

I find it much more usefull to distribute the semipermeable material as shown on the figure. This change will result in lengthening of the path of percolation without a significant additional amount semipermeable material required.

However the impervious blanket below the upstream face will require an additional amount of impervious material, but the path of percolation will be doubled resulting in a considerable decrease in seepage through the foundation. Therefore I believe that this change is well justified.

In the original construction of the cofferdam, a rock toe drain must be provided in order to allow an easy escape of seepage water, and thereby prevent saturation and consequent reduction in stability of the material in the downstream toe of the cofferdam. ~~but,~~ As the construction of the main dam proceeds, this rock drain will be replaced by impermeable and semipermeable material as shown in the figure.

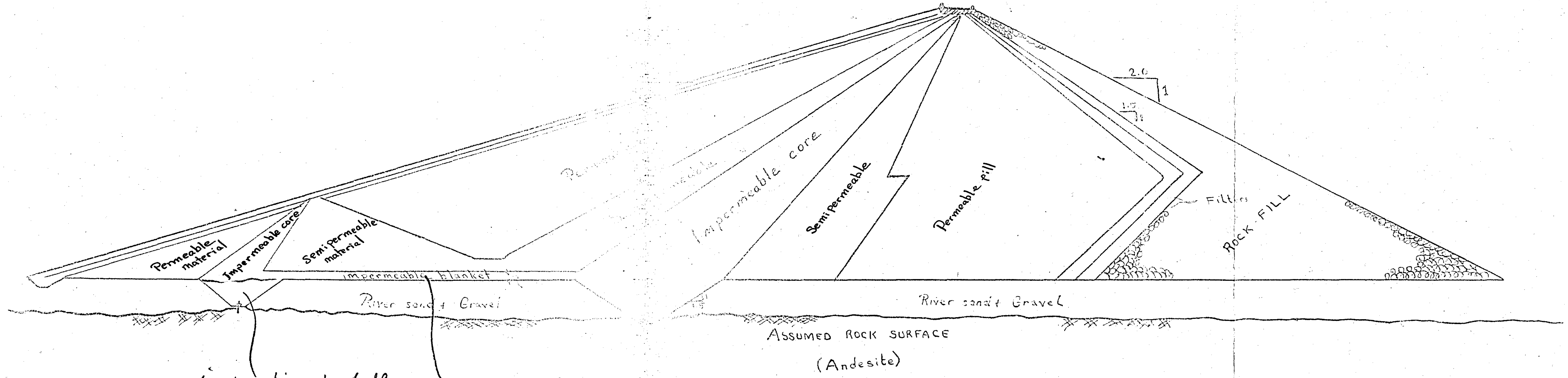
Cut-off trench:

The ~~sides~~ slopes of the cut-off trench ~~is~~ are chosen as I:I. Since the foundation material is a mixture of sand and gravel, ~~in order to facilitate~~ proper compaction of the back-fill in contact with the slopes would be facilitated by using ~~, I believe that it would be usefull to allow~~ flatter slopes. This would also increase the width of the impermeable ~~layer~~ core and result in a reduction of seepage with a minor increase in the consumption of impermeable material required.

It is also advisable to provide a second smaller cut-off trench under the cofferdam that will reduce the seepage while the main dam is under construction and will be of great help in reducing the seepage later on when the structure is completed.

Grout curtain :

A grout curtain varying from 20 meters to 40 meters in depth is given place in the design of D.S.İ. The drilling of the many holes required This is an extremely expensive and time consuming ~~process, since the rock foundation must be drilled to such a depth~~, and also the chances are ~~very~~ high that, while drilling, the water table will be reached in which case it would ~~and it will~~ be a very difficult problem to pump



in practice probably impossible to provide (without a cofferdam for the cofferdam)

probably also impossible to provide (the river is flowing!)

MAX. CROSS SECTION
AFTER PROPOSED CHANGES
ARE MADE

SCALE 1: 1000

the water so that ~~concrete~~ grout can be ~~poured.~~ injected.

The heavy reinforcement necessary for the grout curtain will be another source of expense that must be considered.

I ~~find it questionable to spend such~~ doubt that spending a great deal of money and time for a deep grout curtain ~~in a spot~~ is warranted in a location where earthquakes are very probable, because the rigid body of the high grout curtain, though buried in rock, will be very sensitive to even the slightest earth movements.

Much time and money can be saved by the use of a careful grouting system in two different places, under the cofferdam cut-off trench and the main cut-off trench, instead of the deep grout curtain. not possible

Dwg. No. makes believe DSI recommend a real, continuous curtain

Dead storage:

In the D.S.I. design a dead storage of $135 \times 10^6\ m^3$ is allowed. Annual Inflow to the reservoir is $758.8 \times 10^6\ M^3$. Since the useful life of the dam is estimated to be 50 years, the assumed inflow rate of silt is

$$\frac{135 \times 10^6}{50 \times 758.8 \times 10^6} = .00356 \text{ or } .356\%$$

by volume of the total inflow.

In view of apparent experience with reservoirs already built and the high degree of ~~This is a very optimistic assumption, because in Turkey~~ erosion in Turkey, such an ~~is very high, and this~~ assumption ~~will not~~ appears ~~be permissable.~~ must appear as too optimistic. I would recommend that ~~A~~ a minimum dead storage of $270 \times 10^6\ m^3$ must be allowed, ~~this~~ which corresponds to an assumption of .712% by volume or 1% by weight of the total inflow. Fr. actual measurements in similarly eroded watersheds.

In the design of the Sarıyar dam .3% by volume of the total inflow was assumed to be silt, but this assumption is found highly unsatisfactory. Certain Experts have reported that the useful life of the Sarıyar dam will be only 20-25 years or less due to sedimentation as against to an estimated life of 50 years.

<u>Parapet wall:</u>

It will be more economical to secure one meter of freeboard by the use of a parapet wall of masonry, since suitable rock is available.

The additional placement of embankment materials and slope protection for a crest one meter higher will be more expensive than a parapet wall one meter high.

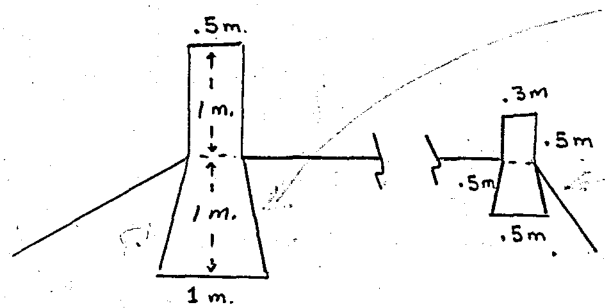
excavation would cost quite some money too

The volume of earth saved= 330xI0xI = 3300m^3

The volume of masonry required if the suggested parapet wall shown below is used = 330 (.5+.3+.I5+.2) = 380 m^3.

Besides being economical this wall will also provide a finished treatment for the crest.

Also danger of cracking from settlement



Capacity of the reservoir :

In determining the capacity of the reservoir I have based my calculations on the possibility that the inflow at the dam site during the three driest years of the past 30 (1) years of record will recur ~~reoccur~~ in the following 50 years. However I have allowed a margin of $68\mathrm{x}10^{6}$ m^{3} volume of water to tide over ~~compensate~~ a period of slightly drier character.

The D.S.İ. design has assumed 5 consecutive dry years of the same intensity . This has almost doubled the required capacity of useful storage.

I believe that this is a too-conservative ~~an over safe~~ assumption, which ~~and it~~ has considerably increased the cost of the project ~~structure~~.

? See mass curve.

The choice of the lower capacity will also permit the preservation ~~use~~ of 600- hectares of fertile land for agriculture, (refer to The elevation vs area curve p.24) which will increase the gross national income appr. by 600,000 T.L. per year and reduce the amount paid for expropriation which is estimated to be 14,000,000 T.L.

The construction of the Hirfanlı dam is very strongly criticized because thaushands of hectares of fertile land have ~~is~~ covered by water. X

(1) Records of inflow are given in the D.S.İ. report for the years 1930-1954, and for the last 6 years after 1954 no consecutive dry years are recorded any where in Turkey.

X That is criticism by non-professionals. All reservoirs will flood areas of more or less fertile valley land, but the benefits of all sound projects outweigh the damages

COST FIGURES OF THE EARTH AND ROCK-FILL DAM brought up to date

Cost calculations of an earth and rock-fill dam were redone by D.S.I. in 1958, after the devaluation of the Turkish Lira. The figures that I give in the following page are taken from the D.S.I. report.

These figures, however, are for a dam of 79 meters height and a capacity of $950 \times 10^6 m^3$, and therefore need to be adjusted as follows:

(1) An increase in cost due to the improvements suggested.

(2) A reduction in cost due to the difference in volume between a dam of 79 mts height and one of 66 mts* height.

(3) A change in cost values due to the substantial reduction of ~~difference in~~ the cost of cement and ~~an~~ the increase ~~due to the changes in~~ of wages between from 1958 to 1961.

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* The hydraulic height of the earth dam is 55 mts and at the maximum cross-section an excavation of 6 mts is found to be necessary by D.S.I. Also the freeboard of the earth dam will be 5 mts so $55 + 6 + 5 = 66$ meters.

COST FIGURES OF THE SELECTED EARTH AND ROCK-FILL DAM

OF 79 METERS HEIGHT

Quantities must be shown

Excavation of earth for the main dam	TL 12,600,000
Excavation of rock for the main dam	4,452,000
Compaction of soil	2,968,000
Transportation	20,600,000*
Water used in washing foundations	1,000,000
Water for materials	2,520,000
Spillway Excavation costs	3,000,000
Cement	3,234,000
Reinforcement Steel	2,316,000
Foundation Treatment	875,000
	TL 53,565,000

* This figure includes depreciation of machinery, fuel, and labor involved in transportation of materials.

(1) Adjustment to the suggested improvements.

The volume of the Earth and Rock-fill dam is 2,670,000m^3. With the suggested improvements the cross-section of the dam will be:

$$\frac{\frac{79}{2} \cdot 10 + (10 + 2 \times 79 + 3.3 \times 79)}{\frac{79}{2} \cdot 10 + (10 + 1.8 \times 79 + 2.9 \times 79)} = \frac{438.7}{391.3}$$

or 1.121 times as great.

Assuming roughly that the volume of the dam will increase in proportion with the maximum cross-section, and that the total cost will be proportionate to the volume, the total first cost of the improved D.S.I. dam of 79 meters height is:

$$1.121 \times 53{,}565{,}000 = \underline{\underline{60{,}046{,}365}} \text{ T.L.}$$

(2) If the dam is to be only 66 meters high, under the same assumptions the cost of the dam will be:

$$\frac{\frac{66}{2} \cdot 10 + (10 + 2 \times 66 + 3.3 \times 66)}{\frac{79}{2} \cdot 10 + (10 + 2 \times 79 + 3.3 \times 79)} = \frac{24406.8}{34657.3}$$

or .704 times the cost of the higher dam.

or $.704 \times 60{,}046{,}365 = \underline{\underline{42{,}272{,}641}}$ T.L.

(3) Statistics show that in Turkey in the last three years the wages and salaries have gone up about 12%, but the price of cement has very much decreased (from TL 231/ton in 1958 to TL 140/ton in 1960). The price of fuel has not altered since

(3) Continued:

1958 and the value of foreign currency in relation to Turkish liras remained unchanged.

Due to the reduction in the cost of cement the total cost will reduce to an amount of:

$$3,234,000 \times \frac{140}{230} = 1,970,000$$

a decrease of

$$3,234,000 - 1,970,000 = 1,264,000$$

I assume that an increase of 12% in the wages will result in an increase of 2% in the total first cost of the dam: so the total first cost of the dam will be:

then the wage share of the tot cost would only 16 – 17

in an earth dam probably nearer 40% of total

$$(42,272,641 - 1,264,000)1.02 = \underline{\underline{41,828,814}} \text{ T.L.}$$

COMPARISON OF THE TWO ALTERNATIVES :

The results of the cost calculations for the concrete gravity dam and the earth and rock-fill dam show that the first cost of the concrete gravity dam is about I,707,000 T.L. or 4% higher than that of the earth and rock-fill dam. which is within estimating tolerances

However the difference in the first costs is so small compared to the total first costs that when the higher maintanence cost of the earth and rock-fill dam ~~in comparison with the concrete gravity dam~~ are taken into consideration ~~for the 50 year life of the~~ dams, the concrete gravity dam will ~~definitely~~ probably be found more economical. Doubtful

In I958, when the final cost calculations of the Almus dam were made, the price of cement was 23I T.L./ T. (1) This price decreased to I40 T.L. in I960. The drop in the price of cement reduced the cost of the concrete gravity dam by I0,000,000 T.L. Probably the most important factor in the choice of an earth and rock-fill dam instead of a concrete gravity dam by D.S.İ. was due to this extra I0,000,000 T.L.

No — probably the reason is the [illegible] much lower dam with smaller reservoir was never seriously considered and a concrete dam for the greater height would become prohibitively [illegible] expensive

There are also some other important factors that favor the choice of a concrete gravity dam rather than an earth and rock-fill dam, of which I can list the following :

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(1) Please refer to the cost figures of the D.S.İ. design

A.) There is practically no seepage loss through the concrete body, whereas even the most carefully constructed earth dam will suffer some seepage loss. The earth and rock-fill dam will rest on a foundation of sand and gravel and even though cut-off trenches are provided the seepage loss will be greater than that of the seepage loss through the rock foundation underlying the concrete gravity dam.

B.) Flood control capacity of the concrete gravity dam can be increased by allowing discharge through sluices in advance, as has already been discussed on page 23. To provide ~~a means for this~~ similar storage through a fill dam would be inadvisable ~~will be very costly in case of the earth and rock-fill dam.~~

C.) Total first cost of the earth and rock-fill dam includes an item of I,880,000 (1) dollars for equipment to be used in the construction which will be imported. For the concrete gravity dam this item is found to be 96I,000 dollars.
More fuel will be consumed in the construction of the earth and rock-fill dam, ~~which is also to~~ and fuel must also be imported.
Therefore with the choice of the concrete gravity dam a considerable amount of foreign currency will be economised.

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(1) Figure taken from the D.S.İ. report. Only the machinery that will be used in the construction of the main dam is considered.

D.) ~~Provided~~ The sluices provided (in the concrete dam) will be ~~usefll in~~ for washing away some silt and reduce sedimentation of the reservoir, but since it is not good practise to allow pipes running through the earth body no sluices can be provided in the earth dam and sedimentation will represent a more serious problem.

The estimated time ~~to~~ for construct ion of the gravity dam is about one year less than the required time for the construction of the earth and rock-fill dam, however I am not justified to make a benefit analysis, for; the construction of the transmission lines, power house and irrigation channels must be assumed to be completed before the benefit items can be considered.

The possibility of constructing the power house as an integral part of the concrete gravity dam must be studied, because ~~than~~ tunnel and ~~probable~~ surge tank ~~expences will~~ can probably be eliminated in that case.

As a conclusion I may say, that, at the chosen capacity the concrete gravity dam is more economical and advantageous, However, if it is insisted that a dam of greater ~~capacity~~ height be built, than ~~the~~ a earth and rock-fill dam may be found more economical, since there will not be a great difference in ~~in~~ the costs of the spillway construction and foundation treatment which are the most costly items of the total cost of the structure.

Should be separate page under heading: CONCLUSIONS

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