STUDY AND DESIGN

OF

A DAM ON THE EUPHRATES RIVER

For Irrigation & Water Power

BY

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U N D E R G R A D U A T E T H E S I S

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CHAPTER 1

INTRODUCTION

The following work is an attempt to study and design a dam on the Euphrates River for irrigation and water power. It is not expected that this will be a complete work, since it requires a large experience and special investigations, which are not possible for a beginner. However, with the available data, it is desired to approach the problem and make the first step for the performance of this ~~big~~ project.

Before beginning such a work, it is important to find out the fundamental reasons which made of our Country in the past, an agricultural State, having in mind that the same results can be realised in the future, if an elaborate irrigation system is planned.

In ancient times, irrigation was practiced to a great extent in this part of the world. Traces of former systems are found in all our plains. Canals and dams were built up showing the great efforts which were given to overcome the difficulties of construction. Wells were dug and used for the development of ground water supply. All the technical methods, known at that time, were applied to use the available water.

The ruins, which are scattered in Mesopotamia and specially along the Khabour River, show the great effort, which was made to develop the agriculture. The Rulers of Assyria were proud that they had built canals to convey the water to Acade and to transform that desert into flourishing plains, providing water and food for the population. They had built more than ninety dams to raise the level of the water and increase the irrigated areas. Discharges and water levels were observed and amounts of rainfall precipitation were measured. Everything was recorded and studied carefully. In some of the inscriptions of Queen Semiramis, which were discovered by Alexander the Great, the following words were found:- "I obliged the streams to flow whenever I liked; I did not want them to flow but where it was necessary and I produced fertile plains after I irrigated them from my streams.[1]

Furthermore, if we study the history of this country under the Roman domination, we find that the water-works constructions were multiplied and that agriculture was extended to the desert. Kasr-El-Hir dam, situated midway between Kariateine and Palmyra, had a capacity of five million cubic meters. The flood water was collected during winter and then distributed during the dry seasons by an elaborate distribution system to 2500 hectares of cultivated land, where green gardens succeeded to those sterile plains. Homs Lake dam has still some traces; it had 850 m length by 5 m. height and allowing a reserve capacity of ninety million cubic meters. The flood water of the Orontes was impounded in the lake, when during the season of low water, it was fairly available for all the agricultural needs.[2]

Our ancestors were not satisfied only in irrigation projects, but also in water supply and its distribution in big cities.

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(1) From the Water Resources in Syria by Dr. Mazloum.

(2) In 1935 a new dam was built, having 2.5m higher than the old one and the new capacity was increased to 200 million cubic meters.

The construction of Heilam's canal near Aleppo and the distribution of its water to the different quarters of the city are good examples of the important works done in this field.

The remains of these different projects are numerous and the last discoveries have revealed a big number of those canals, dams and reservoirs which show clearly that the development of agriculture in this country has not been the result of a regular rainfall precipitation but rather a development of stream and underground water, and sometimes a collection of rainfall water.

In many historical books, it is pointed out that in ancient times, the valleys of Euphrates and Tigris, now almost a desert, were once densely populated. Millions of people were living in this country, having their own civilization based on the agriculture and development of irrigation. Large cities were built at several sites in those parts of the country where, to-day, we find barren lifeless sand heaps. No more those green and pleasant gardens; only bare soil and miles long stretches of sad and tragic lands. Very few people to-day, mostly nomads, are in search of some wilting yellow grass; nothing remains of that past beauty and wealth, so that the whole picture has changed.

The whole disaster may be attributed to one main reason: lack of water; the soil is still fertile, the riches are still in the earth but without water, life is impossible.

Hence, the desire to be of use to my country was one of the chief factors, which lead me to the choice of this problem. Omit

Another reason was the fact that our country has been and will always be an agricultural rather than an industrial state. And since the Euphrates constitutes a big source of water, and is surrounded by fertile plains, it would be wise therefore to study the possibility of building a dam and constructing a distribution system for irrigation. Many difficulties may be expected in the accomplishment of such a huge project, but all of them may be overcome, if all of us cooperate and have the willingness to help in the development of our country.

CHAPTER II

A- A BRIEF DESCRIPTION OF THE RIVER EUPHRATES

The Kara Sou and the Mourad Sou have their sources, the former in the Arzeroum region and the latter to the South-West of the Ararat mountain (5157m.); they meet to form the Euphrates after flowing respectively for about 400 and 500 Km. from the East to the West in the Armenian Plateaux(2000-5000m)

The Euphrates is first driven Eastward for about 100 Km. by the Anti-Taurus. It then follows the direction South-West keeping it for about 200 Km. At 50 Km. to the North of the Syrian Frontier- which it crosses at Djerablous- the river flows Southward.

The Kara Sou, the Mourad Sou and the Euphrates receive in the Turkish territory many affluents which drain the region of the Armenian Plateaux and the Anti-Taurus which they cut profoundly into.

Already, before Biredjik (Turkey), the valley widens and after it has passed the Syrian frontier, the neighbouring country falls to under 500m. above sea level.

Towards Meskeneh, these heights join the undulating plateaux of Chamieh and Djezireh through which the Euphrates henceforth flows.

Down to Meskeneh, the river keeps its North-South direction till it meets the Northern slopes of Djebel Bichri which turn it Eastward till Rakka. Then it takes gradually the South-East direction, specially after the pass of Halabie-Zalabie and keeps the direction till Abou-Kemal where it enters into Iraq.

In the Syrian territory, the Euphrates receives only three affluents, which all come from the North: on the right bank, near Tell-Ahmar, the Sadjour, and on the left bank, near Rakka, the Belik whose discharge, relatively meagre is, in the dry season, used up by irrigation. On the left bank also, the Khabour joins in near Bessireh, between Deir-Ez-Zor and Meyadine. It is much more important, having 35 to 40 cubic meters per second in Low Water. Its principal sources lie in the Ras-El-Ein region. It receives near Hassetche, the Jagh-Jagh river which has 6m^3 per second in low Water, almost completely used for irrigation. The Jagh-Jagh is formed by the union of many streams which drain 150 Km of the Southern slopes of the Taurus between Meyadine and the Tigris valley. The Jagh-Jagh receives also, though constituting a feeble amount, the runoff water of the Northern slopes of the Djebel Sindjar.

The catchment area of the Euphrates and its affluents in Turkish territory, where most of its water comes from, is about 120,000 square Kms. Its total length is 2330 Kms. 450 Km. are in Turkey, 680 Kms in Syria and 1200 Kms. in Iraq.

In this long course, the river valley rarely uses a single bed. It goes on forming unstable arms, almost changing its course during every flood, and leaving numerous sandy barren islands which are submerged during High Water.

The Euphrates enters into Syria at Djerablous at a height of 325.24m above sea level. Its slope constantly decreasing is 0.40 m/Km between Djerablous and Youssef Pacha (Km.77- Elevation 293.87m), 0.31 m/Km. between Youssef Pacha and Sabkha (Km.292-Elevation 227.80m.), 0.19 m/Km between Sabkha and Halabie (Km.383-Elevation 210.8.m) and 0.15 m/Km. between Halabie and Abou-Kemal (Km.680- Elevation 166.31m) where it enters into Iraq.

B-TEMPERATURE OBSERVATIONS

The following observations are taken from the monthly bulletin of the Meteorological service.

MONTHLY AVERAGE	Sept.	Oct.	Nov.	Dec.	Jan	Feb.	Mar.	April	May	June	July.	Aug.
1930-1931 Djerablous												
MAXIMUM	34.2	27.0	9.2	18.5	11.5	13.8	17.6	23.9	27.9	34.0	38.7	38.4
MINIMUM	27.2	11.2	6.0	9.7	4.2	6.2	6.2	10.9	13.0	18.4	24.1	21.9
AVERAGE	30.9	19.0	7.6	14.1	7.9	10.0	11.9	17.4	20.5	26.2	31.4	30.2
1931-1932 Djerablous												
MAXIMUM	36.7	27.7	17.1	10.9	8.0	10.9	18.9	25.4	30.0		40.4	
MINIMUM	19.4	9.7	4.9	2.7	0.3	2.1	5.1	9.7	13.7		23.0	
AVERAGE	28.1	18.7	11.0	6.8	3.9	6.5	12.0	17.6	21.9		31.7	
1932-1933 Djerablous												
MAXIMUM			18.2	11.3	8.7	12.1						
MINIMUM			7.6	3.2	1.5	4.0						
AVERAGE			12.9	4.1	3.6	8.1						
1930-1931 Kamechlie												
MAXIMUM	34.0		18.3	8.7	10.3	11.0	15.3	20.1	27.5	34.5	28.9	29.9
MINIMUM	21.1		11.4	4.5	3.5	4.5	7.0	10.6	15.1	21.5	25.7	25.7
AVERAGE	27.7		14.9	6.6	6.9	7.8	11.2	15.3	21.3	28.0	32.3	32.8

MONTHLY AVERAGE	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	April	May	June	July	August
1931-1932 Kamechlie												
MAXIMUM					8.6	9.1	17.3	21.1	28.1	33.8	37.9	39.9
MINIMUM					2.2	2.5	7.9	9.6	16.2	20.0	24.2	24.9
AVERAGE					5.4	5.8	12.3	15.4	22.2	26.9	31.1	32.4
1932-1933 Kamechlie												
MAXIMUM	33.9	30.9	19.7			9.6						
MINIMUM	20.6	18.7	10.9			3.0						
AVERAGE	27.1	24.8	15.3			6.3						
1932-1933 Deir-Ez-Zor												
MAXIMUM									31.9	37.2	39.8	40.0
MINIMUM		14.3	6.3	2.3		3.4			16.4	18.4	23.0	22.2
AVERAGE									24.8	27.8	31.4	31.1

For these three years, the absolute minimum and maximum were as follows:-

Djerablous

MINIMUM 17/3/1931 ....3.6
17/2/1932 ....5.9
8/1/1933 ....9.9

MAXIMUM 18/8/1931 ....42.9
19/8/1932 ....45.8

Kamechlie

MINIMUM 25/1/1931 ....4.7
8/2/1932 ....9.1

MAXIMUM 18/8/1931 ....44.0
31/8/1932 ....42.5

Deir-Ez-Ezor

MAXIMUM 4/8/1933 ....43.0
MINIMUM

Rakka

MINIMUM 11/1/1933 ....11.0

MAXIMUM 24/8/1933 ....45.0

It is to be noticed: 1) that for the same month, differences between maximum, minimum and average temperature are not important, between the Northern region of the Euphrates (Djerablous), the Northern region of the Khabour (Kamechlie) and the rest of the Syrian Euphrates valley (Rakka and Deir-Ez-Zor type). Cold time and hot climate seem to be coïnciding.

2) That temperature, sometimes, falls very low in winter untill it reaches -10°C. Cold, as one could see from the daily observations, may persist for several weeks and even be rather late. But on the other hand, hot days are early and temperature in the middle of summer may be very high and even it reaches sometimes 45°C.

C- RAINFALL PRECIPITATION

There are only very irregular observations, since winter 1928-1929 in the following stations Kamechlie, Djerablous, Rakka and Deir-Ez-Zor. Due to the irregularity of such observations, not much profit can be expected, and the only thing which can be deducted, is that the Northern part of the basin seems to receive an average of not less than 250 m/m of rain per usual year and that the rainfall diminishes much from North to South and from West to East. The Syrian basin gives but little runoff water to the Euphrates, as was shown by other gauge readings in different stations.

The following records are those of the French Deleguation:-

STATION	Nov.	Dec.	Jan.	Feb.	March	April	May	TOTAL
Kamechlie								
1928-1929								
1929-1930								
1930-1931	58	85	81	76	66	57	8	431
1931-1932	3	0	64	24	11	71	0	173
Djerablous								
1928-1929	50	84	37	49	37	34	6	297

STATION	Nov. m/m	Dec.	Jan.	Feb.	March	April	May	TOTAL
Djerablous								
1929-1930	18	10	17	57		75		
1930-1931			121	91	14	66	4	
1931-1932	14	28	31	17	22	5		
1932-1933	18	0	36	54	23	23	0	154
Rakka								
1928-1929	37	31	7					
1929-1930								
1930-1931								
1931-1932	2	41	81	21				
1932-1933								
Deir-Ez-Zor								
1928-1929	21	25	9	6		14	2	
1929-1930	135	50	36	33		6	0	
1930-1931								
1931-1932			99	20	4	0	0	
1932-1933	20	3	28	4	1	22	0	78

D- DISCHARGE OF THE EUPHRATES AND ITS AFFLUENTS BETWEEN DJERABLOUS AND ABOU-KEMAL

Up till 1930, there were no records for the discharge of Euphrates; but a gauging station was established at Meskene by the end of January 1930. Since February 7, 1930, readings of scale were registered without interruption. A graph of the discharge as a function of the water level was determined with the help of 17 gaugings undertaken between June 16, 1930, and June 25, 1931. Though the river bed is not very ~~much~~ stable, this graph, as was proved in 1932 and 1933, gives acceptable results. See plate 4 where the monthly discharge of Euphrates is found.

DISCHARGE AT MESKENE

	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	Jul.	Aug
1930												
Cubic meters/sec.	163	192	235	358			389	572	514	328	312	175
millions of m^3/month	430	503	620	945			1021	1510	1354	865	560	462
1931												
Cubic meters/sec.	215	204	218	234	491	440	842	2087	1803	1116	528	310

DISCHARGE AT MESKENE (Continued)

1931 (Cont.)	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	Jul	Aug
millions of m^3/month	567	538	575	617	1290	1160	2230	5500	4750	2840	1390	818
1932												
Cubic meters/sec.	192	189	205	205	227	306	657	1239	1368	649	301	216
millions of M^3/month	503	498	540	540	598	809	1730	3260	3610	1710	782	570
1933												
Cubic meters/sec.	192	208	234	205	189	250	379	559	1588	884	342	229
million of m^3/month	503	550	617	540	498	660	995	1470	4000	2360	9 00	604

It should be noticed that the discharge in millions of cubic meters per month was obtained by multiplying the average monthly discharge in cubic meters by the duration in seconds of an average month:-

$$\text{i.e.} \quad \frac{86400 \times 365}{12} = 2.63 \text{ million seconds.}$$

Seven gaugings of the Euphrates were made at Meyadine in September, October and November 1931, then 13 new gaugings from October 1932 to October 1933 near Deir-Ez-Zor and in the stations of Djaffra and Beghelie, to control the discharge given by the graph of Meskene and to establish a new graph of discharge as a function of water levels as read in Deir-Ez-Zor.

E- THE REGIME OF THE EUPHRATES

Two seasons of flood distinguish its regime of flow: One season in winter, in which floods begin to occur due to rainfall precipitation and a season in Spring in which floods are the result of the melting of snow in March.

The first increments which may be noticed as early as in December are of feeble amplitude and are followed by falls of Water Level. But due to the different factors affecting the melting of snow, the water level in the Euphrates will greatly vary from one year to the other. The general form of the water level graphs is itself modified.

The biggest difference in height registered from 1925 to 1932 reached 7 m. at Djerablous; and it was that between the lower Water Level of August 1928 and the highest Water Level of April 1929. This difference must have been exceeded in the narrow passes.

The lowering of water level takes place gradually till towards the end of July when Low Water Level is reached. This latter lasts till December unless some exceptional precipitation happens in November.

VELOCITY OF WATER

The biggest average velocity registered during the gauging made by the "Regie Des Etudes Hydrauliques" was 1.72m/sec. It corresponds to 2.95m on the surface and to a discharge of 2100 m^3/sec. on April 21st, 1931.

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CHAPTER 111

A- ACTUAL EXPLOITATION

It may be seen from the table given above, that the rainfall precipitation is insufficient for winter crops, except in the hilly region between Meskene and the Turkish frontier; on the other hand, the spring precipitation being insufficient too, the only cultivable lands are the cultivatable irrigable ones.

These irrigable fields are limited on every bank to a ribbon, the width of which, not exceeding usually 200 to 300m., is determined by the height to which the farmers can lift the river water using the rudimentary means usually at their disposal.

Different means are used to get water, but the typical installation is composed of two buckets pulled by two mules, which raise water to 5 or 6m., the cultivable lands being in general at least 4m. above Low Water Level.

In the Deir-Ez-Zor region alone, some 450 similar installations existed in 1932-1933, their number is even increasing. Their individual output is about 20m^3 per hour. i.e. 5 or 6 litres per second. Each one irrigates as an average 11 hectares.

However, some small pumps, which are run by motors, working on gas oil, were used instead of these buckets. Their number, which has exceeded 200 pumps, is increasing from day to day, due to the fair results that they give. The motors, which run them, have a total power of approximately 7000 HP. They give the pumps the necessary energy to raise the water to a height varying between 5 and 15m., and make possible the irrigation of 35000 hectares of cultivable lands.

Irrigation is carried on by submersion: the land being divided with help of small heaps, to rectangles, 5m by 15m., which are successively inondated, by opening and closing holes between the heaps.

B- POSSIBLE FUTURE EXPLOITATION

The amount of water, which flows in the valley of the Euphrates in the different seasons, is estimated by specialists, to be fairly sufficient for the irrigation of one and a half million hectares of cultivable land, if the duty of water is be half a liter per second per hectare.

But only a portion of this amount is being used for irrigation both in Iraq and in Syria: 625,000 hectares are being irrigated in Iraq and 50,000 hectares in Syria. However, the area of the land in Syria, which might be irrigated, if the level of the water is raised 10m. higher than the main water level, is estimated to be 250,000 hectares; it may reach 340,000 hectares, if the water level is raised 10m. more. why 10m?

Hence, there should exist a dam, to raise the level of the water and canals, to convey this water to the fields. This dam will help in providing a reservoir in which flood water is stored and then distributed to the fields in the season of Low Water Level; it will serve at the same time as a regulator to the flow of the River water.

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CHAPTER 1V

A- LOCATION OF THE DAM

The selection of the site of the dam requires a long geographical and geological study. The first step should be the preparation of an accurate topographic map of each possible site with the details which should be sufficient to permit close visual indentification on the ground of topographic features shown on the maps. The second step should be a field examination by a structural geologist. This examination should be directed primarily toward a determination of conditions that might render unusuable one or more of the possible sites, such as the presence of faults, or the absence of rock of adequate strength.

foundation is [illegible]

The conditions favorable to a good site for a dam are the followings:-

1- Where there is room for the construction of canal headgates and diversion line without the necessity of expensive construction such as tunnel work, retaining wall sections etc..

2- Where the canal headgates can be placed at right angles to the weir, so as to maintain a clear channel in front of the gates.

3- Near suitable building materials.

4- Where good foundation and permanent banks can be obtained.

These are the cond. for the headworks, not for the dam.

5- Where an impervious stratum is at or near the surface of stream bed.

6- Where the grade of the stream is steep enough so that with a low weir the canal and stream shall be near the same level only for a short distance.

7- Where the construction of the dam will not cause flooding of valuable lands above or tend to change the stream channel.

8- Where the velocity in the stream shall be preferably less than the velocity in the canal, to prevent silt deposits in the canals.

9- Where the stream channel is straight with uniform velocity and regular cross-section.

After this necessary exposure of items which must be kept in mind in the selection of a good site, the first step is to examine the different sites, which were proposed by the engineers, who were charged by the Syrian Government to study the problem, and to choose the best one among them, which satisfies more of the above mentioned requirements.

From all the reports which were presented to the Government, it may be concluded, that there was a general agreement, about the possibility of constructing the dam in three different points, each one having its own advantages and disadvantages. These three sites are: - Kalaât Neim, Halabie and Yossef Pacha.

Kalaât Neim is a site situated at 20Km. from the Turkish frontier. The construction of a dam at this point does not allow a place for a big reservoir and therefore the amount of water necessary for irrigation would not be sufficient. On the other hand, the soil does not resist the weight of the dam, and since the foundation is an important problem which must be studied very carefully in the design of the dam, it is not possible to build a dam safely and economically at that place. Another disadvantage is that the stream channel is not straight and the velocity is not uniform, due to the shape of the valley and the presence of water currents.

The storage of water in Halabie place is possible, but a vast cultivated area would be subjected to submersion, if such a project would be performed ; and therefore there is a loss of fertile and valuable lands which cannot be compensated by other means. Moreover, the grade of the stream at this point is steep enough so that with a low weir, the canal and stream shall be near the same level and no high cultivated areas may be irrigated. Its main other disadvantage too, lies in the impossibility of having a good foundation, because of the perviousness of the soil.

Therefore, the pass of Youssef Pacha is the most suitable for such a construction, because it fulfills most of the necessary requirements of a good site. One of the conditions, which may render it the best site, if all the others would be neglected, is the kind of soil, which was encountered after the geological examination of some engineers who were asked to carry on the work. It was found that a basaltic rock constitutes the soil of that place, and has an adequate strength to support the dam. Moreover, the construction of this dam at that site will cause the flooding of 13000 hectares from which only 5000 hectares are cultivable lands. This loss of land is not too much, compared with that resulting when the dam is built in any other site. A third reason, which must be considered, is that permanent banks are obtained in this site since the river bed is changing almost continuously in all its course due to floods.

B- DIMENSIONS AND TYPE OF DAM

The length and height of the dam are limited respectively by the cross-section of the river bed and the height to which the water can be raised without causing much damage to the surrounding cultivable lands; and therefore, an economical problem arises here. And according to Dr. Mazloum's plan, the dam will have to be 700m.(2300ft.) in length and 25m.(82ft.) in height.

The type to be selected will in general be based on considerations of the availability of suitable construction materials. The following reasons justify the selection of an overflow concrete dam;-

1- Lack of good construction materials which have to be of a uniform specific weight.

2- The dam site being convenient to transportation facilities, a concrete dam is more desirable from an economical point of view.

3- The question of uplift is to be studied more carefully in the case of a masonry dam than when it is a concrete dam.

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CHAPTER V

A- FORCES ACTING ON THE DAM

The investigations of the forces acting on the dam is carried out on a vertical strip 1m wide cut from the dam at the point of maximum height. The exterior forces acting on this strip are as follows: The water pressure, the pressure due to the deposited silt and the uplift along the foundation joint.

WATER PRESSURE: The water pressure due to the water in the reservoir is best determined graphically; it is of course always perpendicular to the surface against which it acts. The pressure on a surface of 1 sq.m. and at a depth of h meters below the water surface is PH metric tons, where P is the unit weight of water in metric tons per m^3. In the determination of the water pressure, the surface is assumed to be at its highest possible position. In the statical analysis of the dam, which is represented on plate 3, the water pressure acting on horizontal slices of the dam were determined, and for this purpose the area to the right of the curve of water pressure is divided into a series of contiguous trapezoids as shown. The line of action of the total water pressure acting on a zone passes through the center of gravity of the corresponding trapezoid.

EARTH PRESSURE: Earth pressure should be taken into account because of the big amount of silt which may be deposited after some time, against the upstream face of the dam. The unit weight of this silt when dry i.e. when the reservoir is empty, is about 1.8 metric tons per

m^3 (112 lb per ft^3). When the silt is under water, i.e. when the reservoir is full, its unit weight is only about 1.2 metric tons per m^3 (75lb per ft^3), due to the buoyancy of the water. The determination of this silt pressure may be found by assuming it acting as a liquid without decreasing materially the stability of the dam.

UPLIFT: Generally, if the gravity dam is designed so that tensile stresses cannot occur, no cracks, in which water might penetrate, can form. However, regardless of measures taken to prevent this percolation, some water under pressure will find its way between the dam and the foundation, along poor joints in the dam or in horizontally bedded joints under the foundations. Wherever this occurs, part of the weight of the dam is supported by water and the direct foundation reaction is correspondingly reduced. But this upward pressure, which is termed uplift, may be effective only in those portions of the foundation joint, in which it exceeds the pressure due to the weight of the dam. In the remaining parts of the joint, this upward water pressure takes the place, partially or completely of the foundation reaction, since it is immaterial as far as the stability is concerned, whether the reaction along parts of the base is due to the foundation itself, or to the water under pressure.

The effective uplift to be considered depends upon

on the assumed distribution of the upward water pressure. The assumption of this distribution is very important, as a ~~glance on~~ look at plate 2, where the maximum observed uplift pressures under existing gravity dams are found, will show. Furthermore, it should be kept in mind, that the condition of uplift, along the entire base of the dam, can never occur; because, for the latter to be possible, the whole rock foundation would have to be pervious, and that is not the case of this dam which is to be built on basaltic rock.

Therefore, the pressure distribution diagram may be assumed as a triangle with a pressure at the heel corresponding to three-fourths of the depth of water in the reservoir and that at the toe equal to zero. This value of the pressure at the heel is the most suitable, since the dam would be more economical than when it is assumed to be corresponding to the total depth , and safer than when it is assumed to be corresponding to one half of the total depth.

B- DETERMINATION OF THE PROFILE OF THE DAM

A direct determination of the profile of the dam is not possible. The cross-section is first assumed, and then investigated to make sure that the allowable working stresses are not exceeded and that no tensile stresses occur. If the working stresses are not attained, the profile is changed so as to give a better utilization of the masonry. The statical analysis is made for the conditions of reservoir full and reservoir empty.

Generally, when the dam has medium heights, the most economical cross-section is a triangle with vertical upstream side and vertex at the maximum reservoir stage. In such a section, there can be no tensile stresses anywhere on the upstream side, even when the unit stress at the upstream edge of the joint is zero. The lines of resistance, reservoir full and reservoir empty must lie within the middle third. If the masonry is to be utilized to the fullest extent possible without permitting tensile stresses to occur anywhere, the lines of resistance must intersect the base in the extremities of the middle third.

If the unit weight of the concrete is Pm, the weight of the dam, which has a base of b units and a height of h units, per unit length, will be $Pm\,\frac{bh}{2}$; and for equilibrium it is necessary that $Pm\frac{bh}{2} \times \frac{b}{3} = Po\frac{h^2}{2}\cdot\frac{h}{3}$ where Po is the unit weight of the water. Therefore

$$b = \frac{\sqrt{Po}}{Pm} \quad .$$

But this ideal profile is of course not practical, since a sharp crest is impracticable, and it is usually required that the top of the dam serve as a road. impract?
Furthermore, from a certain elevation downward, the stresses in such a profile exceed the working stresses. For these reasons, some deviation from the triangular profile is necessary.

To serve as a highway, the crest must have a width of between 3 to 6 meters (10 to 20ft). The provision of such a crest amounts virtually to the addition of a triangular area to the top of the profile: this addition which is called the crest triangle, increases the load at the heel of the dam. Below this triangle, the downstream face is rounded off with additional masonry. The additional load of the crest triangle makes it necessary to increase the width of the dam on the upstream side in order to prevent tensile stresses on the downstream side when the reservoir is empty.

After the study of the different dams which were built in the last thirty years, all over the world, it was found that a good average to take for the base of the dam is 0.65 times the height and we may write that:

$$b = 0.65 \times 25 = 16.25 \text{ m } (53.5 \text{ ft})$$

The crest is assumed to be taken as 3.5 m (11.5ft) to serve as a highway where only one car may go on, and the slope of the upstream face of the dam, according to Mr. Link's Rule, is taken as 1:30

The statical analysis is best done graphically. The profile is assumed to be cut by horizontal planes called joints about 5m (16.4ft) apart; and for each joint the resultant of all forces acting on the part of the dam above it, is determined both for the condition of the reservoir full and that of reservoir empty. The line connecting the points of intersection of these resultants with the joints is called the line of resistance and must everywhere lie within the limit of the middle third.

The profile having 25 meters height will have 5 parts whose weights are W1, W2, W3, W4, W5 respectively. The values of these weights are found as follows:-

$$W1 = \frac{3.45 \times 5}{2} \times 1 \times 2.2 + \frac{3.25}{2} \times 1 \times 2.2 = 37 \text{ tons(metric)}$$

$$W2 = \frac{3.45 + 6.9}{2} \times 5 \times 1 \times 2.2 = 57 \text{ tons(metric)}$$

$$W3 = \frac{6.9 + 10.3}{2} \times 5 \times 1 \times 2.2 = 95 \text{ " " }$$

$$W4 = \frac{10.3 + 13.75}{2} \times 5 \times 1 \times 2.2 = 133 \text{ " " }$$

$$W5 = \frac{13.75 + 17.1}{2} \times 5 \times 1 \times 2.2 = 170 \text{ " " }$$

The water pressures which act on these portions of the cross-section are P1, P2, P3, P4, P5, respectively and are found according to the known principles of Hydraulics. If the weight of $1m^3$ of water is 1 metric ton we have that:-

$$P1 = \frac{5 + 5}{2} \times 1 = 12.5 \text{ tons (metric)}$$

$$P2 = \frac{5 + 10}{2} \times 5 \times 1 = 37.5 \text{ tons (metric)}$$

$$P3 = \frac{10 + 15}{2} \times 5 \times 1 = 62.5 \text{ " " }$$

$$P4 = \frac{15 + 20}{2} \times 5 \times 1 = 87.5 \text{ " " }$$

$$P5 = \frac{20 + 25}{2} \times 5 \times 1 = 112.5 \text{ " " }$$

The weight 1 m^3 of concrete is assumed to be 2.2 metric tons and a strip of 1 m width is taken.

Therefore the force polygons for the weights and the water pressures are drawn, from which the ppoints of application, the directions and the magnitudes of the resultant forces which act on every portion of the cross-section of the dam, are found, for both conditions of reservoir full and empty. The values of these resultant pressures for the case of full reservoir, are found graphically to be respectively:-

R1 = 40 tons (metric)

R2 = 108 " "

R3 = 224 " "

R4 = 386 " "

R5 = 594 " "

The lines joining the points of application of these resultant forces for both conditions, constitute the lines of resistance. The line of resistance, reservoir empty, is found to pass within and at a distance of 22 cm of the upstream extremity of the middle third and it passes, for the case of a full reservoir, within the middle third and at a distance of 42 cm from its downstream extremity.

The results which were obtained above are quite reasonable, and the assumed cross-section is acceptable. The graphical investigation of the dam is shown on plate 3.

C- INVESTIGATION OF THE EXTREME FIBER STRESSES

The investigation of the extreme fiber stresses is also done graphically by means of the well known method as shown on plate 4.

For the condition when the reservoir is empty, the vertical component V of the resultant force is found to be: $37 + 57 + 95 + 133 + 170 = 492$ (metric tons) and $\frac{V}{L} = \frac{492}{17.1} = 28.75$ tons/m^2. The diagram is drawn and the stresses are

$Su = 5.5$ Kg/cm^2 at the upstream face

$Sd = 0.22$ Kg/cm^2 at the downstream face.

These values may be found algebraically as follows:-

$$Su = \frac{492}{17.1} + \frac{492 \times 12 \times 2.62 \times 8.55}{(17.1)^3} = 55 \text{ tons}/m^2 \text{ or } 5.5 \text{ Kg}/cm^2$$

$$Sd = \frac{492}{17.1} - \frac{492 \times 12 \times 2.62 \times 8.55}{(17.1)^3} = 2.2 \text{ tons}/m^2 \text{ or } 0.22 \text{ Kg}/cm^2$$

Where 2.62 m. is the distance of V from the center and 8.55 m. is that of the extreme face from the same point.

The same thing, if the reservoir is full would give:

$Su = 5$ tons/m^2 or 0.5 Kg/cm^2

$Sd = 54.2$ " " 5.42 Kg/cm^2

This value of $Su = 0.5$ Kg/cm^2 is frequently specified in order to prevent the occurence of tensile stresses on the upstream side; because the fulfilment of the condition that the line of resistance, reservoir full shall pass through the middle third is not sufficient to insure the non-occurence of tensile stresses.

D- STABILITY AGAINST EXTERNAL FORCES

STABILITY AGAINST OVERTURNING

The horizontal forces modified by the vertical forces due to uplift tend to overturn the dam, being resisted by the vertical downward forces. All of the horizontal forces may be reduced to a simple force H which is equal to 312.5 metric tons acting at the center of gravity of all of these forces which is at a height h = 8.33 m. above the base of the dam.

The vertical forces consist of the weight of the dam which is equal to 492 metric tons and which is acting at 11.2 m from the toe; and of the weight of the amount of water against the inclined upstream face which is equal to 15 metric tons and is acting at a distance of 16.82m. from the same point.

The distribution of the uplift being supposed to be in a triangular shape and having a stress of:

$$0.75 \times 25 \times 1 = 18.75 \text{ metric tons}$$

at the heel and 0 tons at the toe, the total force may be taken as equal to:

$$\frac{18.75 \times 17.1}{2} = 160.31 \text{ metric tons}$$

acting at a distance of $17.1 \times \frac{2}{3} = 11.4$ m. from the toe.

Therefore the factor of safety against overturning which is the ratio of the righting moments to the overturning moments about the toe is found as follows:-

	FORCE	MOMENT ARM	RIGHTING MOMENTS
	Metric Tons	In Meter	In M.-Tons
Weight of dam -	492	11.2	5510.4
Vertical weight of water -	15	16.82	252.3
			5762.7

	FORCE	MOMENT ARM	OVERTURNING MOMENTS
	Metric Tons	In Meters	In Meter-Tons
Horizontal water Pressure -	312.5	8.33	2603.12
Vertical force of uplift -	160.31	11.4	1827.53
			4430.65

Factor of safety against overturning is $\frac{5762.7}{4430.65} = 1.30$

STABILITY AGAINST SLIDING

If the coefficient of friction be denoted by μ, therefore if the dam is to be safe against sliding, the value of the horizontal pressure of water must be less than the product of the vertical forces by this coefficient μ.

Assuming that $\mu = 0.70$ with a big factor of safety since the soil on which the dam is to be built is basaltic, we have that:

Horizontal forces - Water Pressure 312.5 tons

Vertical forces - (Weight of dam 492 tons
(" of water 15 tons

The vertical forces are 492 + 15 = 507 metric tons.

Therefore: 0.7 x 507 = 354.9 metric tons

This is a horizontal force opposing in direction the water pressure force and having a higher magnitude.

Hence, from the computations given above, it is shown that the dam is safe against overturning and against sliding.

E- CONTRACTION JOINTS AND WATER STOPS

The volume of concrete in a dam tends to change with variations in temperature and variations in water content. This shrinkage if unrestrained, amounts to 0.0003 to 0.0005 of the length of each dimension of the mass during the curing process, which corresponds to a drop in temperature of from 50 to 83°F. If restrained the stress caused by the tendency to shrink, assumed to be uniform throughout the cross-section under consideration, is identical with that to a force causing an equal deformation. If no provisions are made to withstand such shrinkage, cracks will be formed in the concrete. These cracks usually are irregular and unsightly and also provide means for water to seep through the structure.

In order to avoid these cracks, it is necessary to construct the dam in independent blocks of suitable lengths. Other possible methods are the use of cements of low setting heat and the dissipation of heat in the concrete by forcing cold water through a network of pipes distributed throughout the mass, thus preventing the concrete from reaching a high temperature and hastening the reduction to a degree at which it is permissible to pour the adjacent blocks. This last method being difficult in the case of this dam, contraction joints are more easily used. The distance to be provided between two independent blocks, each one having a length of 30m. is equal to:

$$0.000006 \times 30 \times 30 = 0.0054 \text{ m. or } 0.54 \text{ cm.}$$

in which 0.000006 is the modulus of elasticity of concrete and 30° being the average change in temperature (this last value is taken from the table of variation of temperatures given in chapter II Part B).

But in order to have a bigger safety against the occurence of cracks, the distance between two blocks must be one centimeter filled either simply with Asphalt or with tarred paper which do not present resistance to the movement of the blocks.

To make the passage of water more tortuous through the contraction joints and to provide shearing resistance between adjacent sections it is desirable to recess one section and to provide a key on the adjacent one. An illustration of the typical contraction joint is shown on plate 2. Moreover, to add to the difficulty of water percolating through the contraction joints, copper stops are placed in them near and parallel to the upstream face of the dam at a distance of 1.3m (4.25ft approximately) from the face. These water stops are placed in the structure in such a manner as to prevent water from passing around their ends and as to provide flexibility in expansion and contraction without rupturing the metal or disturbing its bond with the concrete.

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CHAPTER VI

A- SPILLWAYS.

Spillways are required to act as safety valves, in case the inflow becomes so great as to endanger safety of the dam, by causing water to flow over the top or to prevent the reservoir from filling to a level sufficient to cause damage to adjacent property. In the design of the spillway, the problem is to provide a structure capable of passing this flow without exceeding the permissible maximum flow elevation and without damage to the dam.

The type of spillway, which is most suitable to be used for the case of the Euphrates, is the overflow dam. The shape of the crest may be one of several forms which will be discussed later and the general equation for computing the overflow capacity is given in the "LowDams" text book as:

$$Q = CLe\ (H+h_o)^{3/2}$$

in which Q = Total discharge capacity in C.F.S.

C = A constant depending on the shape of the crest and the depth of overflow.

Le= Effective length of required crest in ft.

H = Head measured from top of crest to reservoir level.

But for large reservoirs such as the present case, the effecive length L_e is the same as the overall length L of the required crest, and the effect of velocity of approach is usually slight and may be neglected. The value of the constant C may be taken, according to the recommendation of the "(Subcommittee on Small Water Storage Projects)" as 3.33

From the table given in chapter II, the maximum value of discharge, which was observed, was that of April 1931 in which 2080 m³/sec were registered. The height over the crest which is allowed for the overflow of this amount of water is usually taken as 2m; (6.56 ft); and therefore the above mentioned formula is used to find the length of overflow without endangering the dam.

Should have been re-typed

Hence we have that:

$Q = 2080\ m^3/see \quad = \quad 73600\ ft^3/see$

$H = 2m. \quad = \quad 6.56\ ft$

$C \quad = \quad 3.33$

therefore $73600 = 3.33 \times L \times 6.56^{3/2} = 56 \times L$

from which $L = 1310\ ft = 400$ meters.

Thus, the dam will be divided into three portions : One portion on each side, having a length of 200M. and a height of 25 m., over which the water will flow in case of ordinary floods (See plate 6), and a middle portion whose length is 300 m. and its height is lower than the other part by the amount necessary for the construction of gates which, in case of extraordinary floods, might be opened to permit the water to flow more easily and without endangering the safety of the dam. A cross-section of a typical gate is shown on plate 2. This gate will be put in the reversed position as shown on plate 7, in order to save in the space which it requires.

Openings of emergency must be left when the dam is being constructed so as to provide a way to get rid of silt and dirty water when necessary. The dimensions of these openings are calculated according to the following formula given by Dr. Weyranch in his book : "Hydraulisches Rechnen".

$$Q = 2/3\ rb\ \sqrt{2g}\ \left[(h_1 + h'_{Vo})^{3/2} - (h_2 + h'_{Vo})^{3/2}\right]$$

in which Q= minimum discharge per second in m^3/sec.

r= a constant equal to 0.62

b= the total width of the openings in m.

h_1= the depth from the crest of the dam to the lower base of the opening in m.

h_2= the depth from the crest to the higher base of the opening in m.

h_{Vo} and h'_{Vo} are heights of water due to the velocity of approach, and they are assumed to be equal to zero in this case. Therefore if the opennings are 2m. high, and the lower base is 5m. from the base of the dam we can write that:

$$200 = 2/3 \times 0.62 \times b \times \sqrt{2 \times 9.8}\ (20^{3/2} - 18^{3/2})$$

from which : b = 8m.

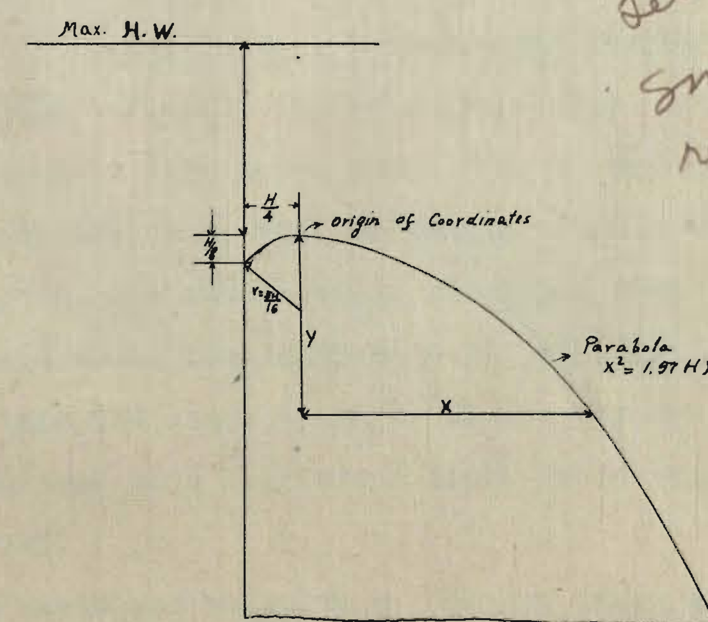
Hence we must provide 4openings each one having 2x2m., so that two of them are on each side of the middle portion of the dam as shown on plate I in the downstream view of the whole dam.

In order to provide a way for the passage from one end of the dam to the other, a continuous bridge will be constructed. Each portion of it will have to span the distance between two piers which is selected to be 15m. or half the distance between two contraction joints.

B- MODIFICATION OF THE SHAPE OF THE CREST.

Modification of the shape of the crest is desirable for two reasons : 1) To improve the flow coefficient by rounding the upstream corner and to keep the underside of the overflow sheet in contact with the downstream face of the dam for all conditions of overflow and at all stages.

There are numerous formula~~s~~e and equations for defining these modifications which accomplish much the same result. A scheme given by the subcommittee on small Water Storage Projects which is found acceptable for most dams is presented below :



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The curve on the downstream side is defined by a parabola which can be approximated by a compound curve of two radii for simplified construction.

The shape of the overflow section near the downstream toe of the dam will be governed by the requirements for dissipating the energy of the overfalling water

C- SPILLWAYS AND OUTLET PROTECTION

Spillways and outlet works usually develop high velocities and it is therefore necessary to provide a devicë which will dessipate the energy and reduce the velocity to a value which will avoid erosion in the flowing away channel. This velocity V_1 is equal to $\sqrt{2gH}$ in which $H = 25 + 2 = 27$ m. = 88.5 ft. Therefore $V_1 = \sqrt{64.4 x 88.5} =$ 75.5 ft./see = 23m./sec.

The protective measure which must be used for this energy dissipation is the adjustment of the flow channel so that a hydraulic jump will occur, when the stream reaches the base of the spillway, it is usually flowing at a depth less than critical. The relatively flat slope of the tailrace channel will not support flow at the shallow depth, and hence the depth will be increased and the velocity diminished. The water passes therefore from the high depth stage above the dam through the critical depth at the crest into low stage depth as it reaches the foot of the dam and back to a high stage depth after passing through the jump.

If the depth and velocity of the low stage are respectively D_1 and V_1 ; the depth D_2 of the high stage below the jump is :

$$D_2 = -\frac{D}{2} + \sqrt{\frac{D_1^2}{4} + \frac{2V_1^2 D_1}{g}}$$

since $D_1 = \frac{Q(ft^3/ft/\ sec)}{V_1(ft/sec.)} = \frac{56}{75.5} = 0.74$ ft.

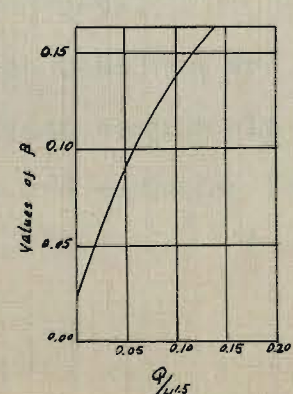
therefore $D_2 = -\frac{0.74}{2} + \sqrt{\frac{0.74^2}{4} + \frac{2x75.5^2x0.74}{32.2}} = 15.8$ ft.

and the velocity V_2 of the high stage is equal to :

$$V_2 = \frac{56}{15.8} = 3.54 \text{ ft/sec.}$$

Because of the high velocity above the jump and the turbulence in the jump itself, destructive erosion will occur unless properly designed stilling pool is provided.

In designing the stilling pool, it is important that the jump be formed in the pool for all stages of flow. From the various experiments made by A. Schoklisch, it was found that the length of this pool must be equal to tow-thrids of the drop H from spillway crest to pool bottom, and therefore it is 18m. Its depth which is proportional to $\frac{Q}{H^{1.5}}$, where Q is the spillway discharge per linear meter of spillway crest in cu.m. per Sec., is found on the chart given on page 919 of his book "Hydraulic Structures" and which is reproduced below:



After Computing the value of $\frac{Q}{H^{1.5}}$ which is $\frac{5.2}{27^{1.5}} = 0.037$ a value of $B = 0.075$ is taken from this chart, and the correct heiht of the sill for satisfactory energy dissipation is then $BH = 0.075 \times 27 = 2m.$

D - CONSTRUCTION OF THE DAM

It was shown before, that this dam is to be of the concrete overflowing type, and usually its construction requires the skill of Specialists who have practiced for a long time in this kind of work. But generally speaking the following method is used.

Each block having a length of 30m. the distance between two contraction joints, is to be built apart from the others, leaving the construction of the two blocks which are on both sides untill the first one will get dry and have its walls asphalted . Every block is to be constructed of successive layers 50cm. each. The upstream face of the dam being directly subjected to the action of water must be of a rich concrete mixture having 500Kg. of cement per cu.m. This rich layer 80cm. thick, will prevent the percolation of water through the remaining part of the dam which is of a mixture having 250Kg. of cement per cu.m. of concrete.

Even if such measures are taken, some drains are used in order to prevent completely the precolation of water through the dam and endanger its safety by taking with it the particles of cament and leaving the other ingredients exposed. The row of drains, to be used, has 9 pipes, each one having a diameter of 10 cms. and is put at 1m. from the upstream face of the dam.

In order to control the occurence of cracks and to get into the dam when necessary, two inspection galleries are to be left during the construction, each one having 2m. height by 1m. width allowing for a man to get through easily. The descent from the higher gallery to the lower one is possible by means of a vertical hole 1m. by 1m. and which is shown in the contraction joint plan on plate 2. The downstream view of the whole dam is represented on plate 1-

E- FOUNDATION

The purposes of a foundation under a dam are (1) to insure stable support for the structure under all conditions of loading (2) to provide the necessary resistance to the passage of water, so that the purposes of the dam may be fully attained. therefore, if the soil is permeable, good results cannot be expected, since the uplift pressure will help in the overturning or sliding of the dam. On the other hand, even if the soil is impermeable as in the case of the basaltic rock on which this dam is to be constructed, some water might find its way through the cracks, joints and faults and therefore in order to obtain a watertight foundation, and assist in eliminating uplift, it is usually necessary to grout the foundation.

Several methods are generally proposed, but the one to be used here, is the blanket grouting or general grouting over almost the entire area of the foundation, Three rows of holes should be drilled : one row neab the upstream face of the dam having 10m. depth and 15m. center to center of holes, a second row at the middle having 8m. depth and 2m. center to center of holes, a third row near the downstream face of the dam having 7m. depth and 3m. center to center of holes. All the holes are to be 25cm. in diameter in each one of them a steel pipe of smaller diameter is driven, and through this pipe, the ciment Cement grouting is pressed under a pressure corresponding to there times the height of water in the dam or 7.5 atmospheres. In this way all cracks, faults and joints will be filled with grouting and prevent therefore the passage by water.

not clear

CHAPTER V11

A- DESIGN OF THE BRIDGE

It is required to design a T-beam bridge for the following conditions:-

Clear Span = 134 m. = 44ft.0 in.

Clear width = 3.50m. = 11ft.6 in.

Live loading = 15 ton truck.

Axles spaced 10 ft center to center.

Wheels spaced 6 ft center to center.

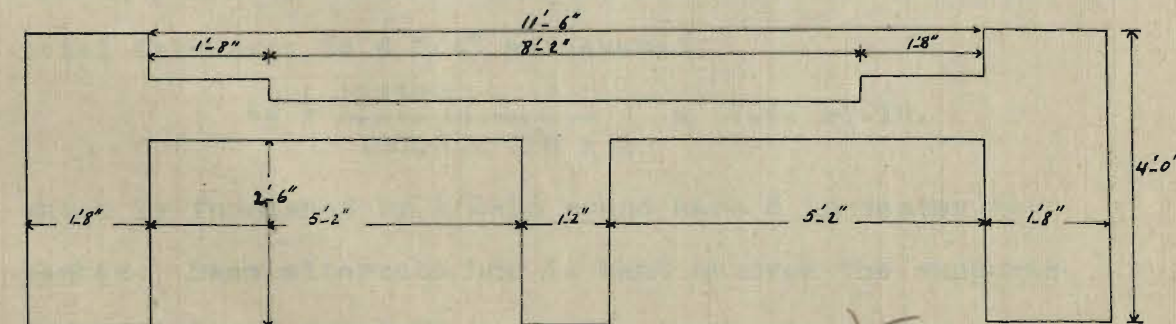
Two thirds of the total load is carried on the rear axle.

Impact allowance 30%

Allowable tension in steel 18000 P.S.i.

Design of Slab

The bridge is to consist of one intermediate beam and two outside beams supporting a floor slab. Assuming that the intermediate beam will be 14" in width, the clear span of the slab will be 5'.2"



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Assuming a total thickness of slab (including a 3/4 in wearing surface) of 6 3/4" and allowing 15 lb per sq.ft for protective covering, the total dead load per sq.ft is: $\frac{6.75 \times 150}{12} = 85$ lbs. + 15 = 100 lbs

The dead load moment is: $0.8(\frac{1}{8} \times 100 \times 5.16^2 \times 12) = 3200$ in-lb

and $D = \frac{6.58}{2} = 3.29$ ft

$T = 1.25$ ft

$E = 0.7(2 \times 3.29 + 1.25) = 5.48$ ft (See Urquhart & O'rourke Concrete Structures Art 263)

The concentrated load at the center of a 1-ft. strip of slab is: $\frac{10000}{5.48} = 1820$ lb

The live load moment is : $0.8(\frac{1820}{2} \times \frac{5.16}{2} \times 12) = 22600$ in-lbs

The impact moment is : $0.30 \times 22600 = 6780$ in-lbs

The total moment is : $3200 + 22600 + 6780 = 32580$ in-lbs

Then :-

$$d = \sqrt{\frac{32580}{12 \times 108}} = 5 \text{ in}$$

Taking 1 in. of insulation below the center of the bars, the total thickness is 6 3/4" as assumed.

$$As = \frac{32580}{18000 \times 7/8 \times 5} = 0.42 \text{ sq.in.}$$

which is furnished by 1/2-in round bars 5 in center to center. Each alternate bar is bent up over the supports and additional straight bars 10in center to center are placed in the top of the slab continuous from outside beam to outside beam to complete the negative moment reinforcement. Temperature and distribution stresses in the direction of the span are provided for by placing four $\frac{1}{2}$ - in round bars in the top and bottom of each slab panel parallel to the beams at about 12- in centers.

DESIGN OF INTERMEDIATE BEAM

The intermediate beam is a T-beam with an effective span length of 49 ft center to center of piers.

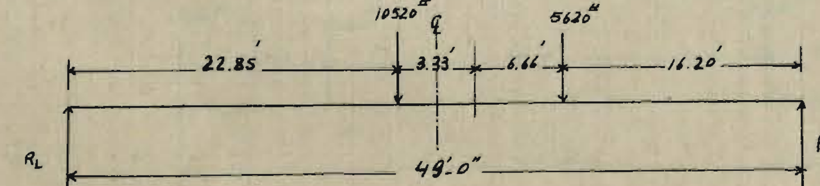
DEAD - LOAD MOMENT

The weight from the slab per foot of beam is 100 x 6.33 = 633 lbs. The cross-section of the beam below the slab is assumed as 14" x 30" which adds an additional weight of $\frac{14 \times 30}{144} \times 1 \times 150 = 437$ lb per foot making the total weight 1070 lb per ft. Then the dead-load moment at the center of the span is :

$$M_D = \frac{1}{8} \times 1070 \times 49^2 \times 12 = 3,850,000 \text{ in lbs}$$

LIVE - LOAD MOMENT

The maximum live-load moment (See theory of simple structures by Shedd and Vawter page 125) will occur with the 15-ton truck on the bridge in the position shown in the following figure:-



With distribution of loads as specified by O'rourke and Uruquhart, the intermediate beam must support $1 + \frac{0.33}{6.33} = 1.052$ wheel loads per wheel.

Therefore the load from the rear wheel is 1.052 x 10000 = 10520 lb.

and from the front wheel 1.052 x 5000 = 5260 lb.

$$R_L = \frac{15780 \times 22.86}{49} = 7400 \text{ lb.}$$

and

$$M_L = 7360 \times 22.86 \times 12 = 2020,000 \text{ in lb.}$$

IMPACT MOMENT

$$M_I = 0.30 \times 2,020,000 = 606,000 \text{ in lb.}$$

MAXIMUM TOTAL MOMENT

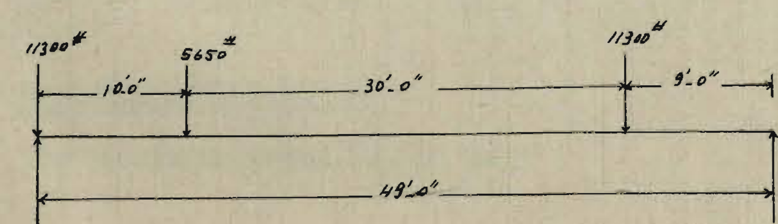
The sum of the maximum dead-load, live load, and impact moments is: 3,850,000 + 2,020,000 + 606000 = 6,476,000 in-lb

DEAD-LOAD SHEAR

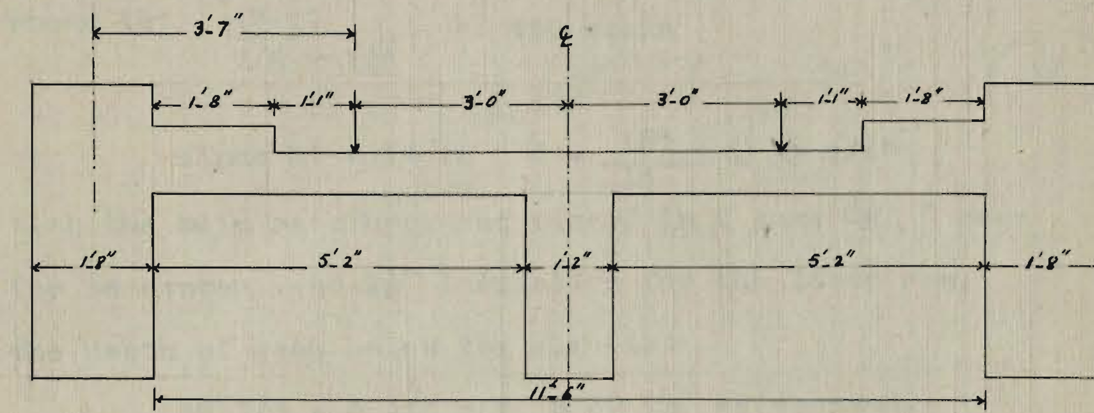
The maximum dead-load shear at the end of the beam is 1070 x 24.5 = 26200 lbs.

LIVE-LOAD SHEAR

The maximum shear in the center beam occurs with the truck train on the span in the position shown in the following figure:-



The design arrangement of the truck wheels is shown below:-



With the loads so placed, the center beam sustains

$\frac{3.58 \times 2}{6.333} = 1.13$ times the value of each wheel load.

Therefore 10,000 x 1.13 = 11300 lbs from the rear wheel.

5000 x 1.13 = 5650 lbs from the front wheel. Then the maximum live load shear is:

$$11300 + \frac{5650 \times 39}{49} + \frac{11300 \times 9}{49} = 17880 \text{ lbs}$$

IMPACT SHEAR

The impact shear at the end is 0.30 x 17880 = 5360 lbs.

MAXIMUM TOTAL SHEAR

The maximum total shear is:

26200 + 17880 + 5360 = 49440 lbs

DETERMINATION OF CROSS-SECTION AND STEEL AREA

The area b'd required to sustain the maximum shear is:

$$\frac{49440}{7/8 \times 125} = 450 \text{ sq.in}$$

15" ? 30" ? OK.

$$\text{since } b' = 14 \text{ in} \quad d = \frac{450}{14} = 32\ 1/4"$$

With the main reinforcement placed in 2 rows $2\frac{1}{2}$ " center to center, and $2\frac{1}{2}$" insulation for the lower row, the depth of beam below the slab is:

$$32\ 1/4 + 3\ 3/4 - 6 = 30 \text{ in as assumed.}$$

$$As = \frac{6,476,000}{18000(32\ 1/4 - 6/2)} = 12.3 \text{ sq in}$$

which is furnished by ten 1 1/8 - in square bars

$$Eo = \frac{49440}{125 \times 7/8 \times 32\ 1/4} = 14 \text{ in.}$$

This is furnished by 4 - of the 1 1/8-in bars allowing the remaining 6 to be bent up to assist in resisting the diagonal tension stresses.

Stirrups are used in order to increase the safeguard against diagonal tension cracks; they are arranged as follows: with the first stirrup 4 in.from the edge of the support, 6 at 16 in., 4 at 15 in. and 6 at 16 in.

DESIGN OF EXTERIOR BEAM

The exterior beam is a rectangular beam with an effective span length of 49ft center of pier

DEAD-LEAD MOMENT

The weight from the slab per foot of beam is 100 x 2.58 = 260 lbs. The weight of the railings, details, etc... is assumed as 300 lb per ft. The cross-section of the beam is assumed as 20 x 48 in., which weighs $\frac{20 \times 48}{144} \times 150 = 1000$ lb per ft. The total dead load is $1060 + 300 + 260 = 1560$ lb per ft and the maximum dead-load moment at the center of the span is: $M_D = \frac{1}{8} \times 1560 \times 49^2 \times 12 = 5{,}620{,}000$ in.lbs.

LIVE-LOAD AND IMPACT MOMENTS

The portion of each wheel load which rests on the exterior slab panel and which is supported by the exterior beam is equal to $\frac{3}{6.58} = 0.455$. The longitudinal position of the load which will produce the absolute maximum bending moment is the same as for the intermediate beam. The moments are directly proportional and the absolute maximum moment in the exterior beam is: $M_L = \frac{0.455}{1.052} \times 2{,}020{,}000 = 874000$ in-lbs

The impact moment is $0.3 \times 874000 = 262000$ in-lbs

TOTAL MAXIMUM MOMENT

The total maximum moment is:

$5{,}620{,}000 + 874000 + 262000 = 6{,}756{,}000$ in-lbs

SHEARS

The maximum dead-load shear at the end of the beam is : $V_D = 1560 \times \frac{49}{2} = 38200$ lbs.

The maximum live-load shear is proportional to the maximum live-load shear in the intermediate beam and is: $V_L = \dfrac{0.445}{1.13} \times 17880 = 7040$ lb

The impact shear is:

$76040 \times 0.30 = 2110$ lb

The total shear is:

$38200 + 7040 + 2110 = 47,350$ lb.

DETERMINATION OF CROSS-SECTION AND STEEL AREA

For the assumed width of 20in. the required depth is:

$$d = \sqrt{\frac{6,756,000}{173 \times 20}} = 44.2 \text{ in.}$$

Taking d = 44.5 in. and assuming the same arrangement of steel and insulation as in the intermediate beam, the total height of the cross-section is 44.5 + 2 1/4 +1 1/4=48 in. as assumed.

$$As = \frac{6756000}{18000 \times 7/8 \times 44.5} = 9.65 \text{ sq. in.}$$

which is furnished by eight 1 1/8-in square bars. Four bars are placed in the lower row and four in the upper row. Four bars are sufficient to develop the bond stress so that the remaining four bars can be bent up to assist in resisting diagonal tension.

In order to reinforce the beam against shrinkage and to tie it together, ½-in. round U stirrups will be placed about 2 ft. center to center throughout its length.

Full details of the whole bridge are shown on Plate 5.

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CHAPTER V111

C O N C L U S I O N

It is necessary at the end of this work to add a chapter dealing with the results which may be obtained, if such a project would be performed.

No one can deny the importance of irrigation in the life and development of our country, and as it was shown before, Syria was and will remain an agricultural state; and therefore, because of the lack of rainfall precipitation in the eastern part of the country, it is necessary to search any suitable way to get water from and use it for irrigation.

Previously, I marked this for a "re-write" - Still N.G.

Since the construction of a dam on the most abundant source of water, the Euphrates, was found possible, and the irrigation of wider plains will, therefore, be secured, it is very important to perform this project, even if some losses are to be experienced for the first ten years, but this project will be very fruitful, when its valley will be much more populated and new villages will, therefore, be created.

Another advantage, which results from the construction of the dam, is that it will help in the construction of turbines which generate an electrical power of at least 50,000 hp.(Estimation of Dr. Mazloum) and which may be used for two purposes: first for running factories and second for running pumps, in

more

order to raise the water of the Euphrates and convey it to Aleppo, to be used for water supply and for irrigation.

The question of generating electrical power and using it in factories is of the utmost importance. Because, once this dam is constructed and irrigation is provided, cotton, wheat, and sugar canes, can be planted to a wide extent, and therefore, the presence of textile factories, mills and sugar factories are necessary for the working of these raw materials, to fulfill the needs of the country. It is not necessary here to show their importance, since they are a vital factor in the economical development of our country.

Another important factor, which is to be considered, is the use of this electrical energy in pumping the water of the river and conveying it to Aleppo. This city has felt a big necessity for water in the last few years, due to the fact that the Turkish Government has diverted the flow of the Queik river, which has been providing the water supply of Aleppo, directly from its source; another reason is the insufficiency of the underground water which has been feeding the local wells.

Therefore, after many investigations carried on by specialists on the possibility of getting either the underground water from different sources like Ein-El-Tell, Heilan, El-Bab, and Djouboul, or the surface water from the rivers like Spin, Orontes and Roui, it was found that the Euphrates is the most suitable source of water for future water supply development.

The project is planned to consist of a pumping installation in a place called Wardi, at a distance of 80 Kms. to the East of the city. The water is pumped from this point, which is at 275 m. above sea level, over a hill 420 m. high above sea level and 8 Km distant from the Wardi place. At this last point, the water flows down by gravity to the filtration and sedimentation basins. It is, then, pumped 20m. higher to the distribution reservoir from which it is, therefore, conveyed to the different parts of the city. The estimated power, necessary to pump this water at the installation station on the river, is about 2700 hp. and at the distribution reservoir about 600 hp.

From all this, it may be concluded that this dam must be constructed, in order to impound water and use it in irrigation and in the generation of electrical energy, both of them being the basic conditions for the economical development of Syria, and any other country.

not this particular Dam

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I WOULD LIKE AT THE END OF THIS THESIS TO EXPRESS MY HEARTIEST THANKS TO PROFESSOR EDWARD ROMANSKY WHOSE KNOWLEDGE AND EXPERIENCE HAVE BEEN OF THE UTMOST HELP TO ME IN THE PERFORMANCE OF SUCH WORK.

MAY 17, 1948

G E O R G E N. K A N A W A T I

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